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No figures were included in these sections.

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3 DESIGN OF STRUCTURES, SYSTEMS, COMPONENTS, AND EQUIPMENT

3.3 Wind and Tornado Loadings

3.3.1 Wind Loadings

3.3.1.1 *Introduction*

Safety-related structures need to meet General Design Criterion (GDC) 2 which requires they be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety functions. This section documents the findings from the staff's review and evaluation of the information provided by the applicant that describes the basis for the U.S. EPR design wind speed and the procedures used to transform this wind speed into an equivalent design wind load acting on plant structures.

3.3.1.2 *Summary of Application*

Final Safety Analysis Report (FSAR) Tier 1: The FSAR Tier 1 information associated with the section is found in FSAR Tier 1, Chapter 5, "Site Parameters."

FSAR Tier 2: In FSAR Tier 2, Section 3.3.1, "Wind Loadings," the applicant has provided a description for determining wind loads on structures, which is summarized, in part, as follows:

Wind loads are determined from a design basis wind speed which represents a 3 second gust 10.0 m (33 ft) above the ground. The design wind speed is 233 km/hr (145 mph). The resultant wind load on the structure is a function of the wind velocity, building height, topographic factors, wind directionality factor, and importance factor. The importance factor is 1.15. When combined with the design wind speed, the recurrence interval for the design wind speed is approximately 100 years.

ITAAC: The inspections, tests, analyses, and acceptance criteria (ITAAC) for structures including external loads are addressed in FSAR Tier 1, Section 2.1, "Structures."

3.3.1.3 *Regulatory Basis*

The relevant requirements of NRC regulations for this area of review, and the associated acceptance criteria, are given in NUREG-0800, "Standard Review Plan (SRP) for the Review of Safety Analysis Reports for Nuclear Power Plants – LWR Edition," (hereafter referred to as NUREG-0800 or the SRP), Section 3.3.1, "Wind Loadings," and are summarized below. Review interfaces with other SRP sections also can be found in NUREG-0800, Section 3.3.1.

1. Title 10 of the *Code of Federal Regulation*s (10 CFR) Part 50, Appendix A, "General Design Criteria for Nuclear Power Plants," General Design Criteria (GDC) 2, "Design Bases for Protection Against Natural Phenomena," as it relates to the requirement that structures, systems, and components (SSCs) important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes, tornados, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety functions. The design bases for these SSCs shall reflect consideration of the most severe of

natural phenomena that have been historically reported for a site and appropriate combinations of the effects of normal and accident conditions with the effects of the natural phenomena.

2. 10 CFR 52.47, “Contents of applications; technical information,” as it relates to the requirement that a design certification application contain the proposed ITAAC that are necessary and sufficient to provide reasonable assurance that if the inspections, tests, and analyses are performed and the acceptance criteria met, a plant that incorporates the design certification is built and will operate in accordance with the design certification, the provisions of the Atomic Energy Act of 1954, and Nuclear Regulatory Commission (NRC) regulations.

Acceptance criteria adequate to meet the above requirements include:

- The acceptance criteria specifically listed in NUREG-0800, Section 3.3.1

3.3.1.4 *Technical Evaluation*

In this section the staff evaluated the design wind speed and development of design-basis wind loads that are applied to Seismic Category I structures.

3.3.1.4.1 Applicable Wind Design Parameters

Safety-related structures need to meet GDC 2 which requires they be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without losing the capability to perform their safety functions including consideration of the most severe of natural phenomena that have been historically reported for a site and appropriate combinations of the effects of normal and accident conditions with the effects of the natural phenomena. Information provided by the applicant in FSAR Tier 2, Section 3.3.1, Revision 2 states that wind and related parameters used as a basis for the applicable wind design load are based on a design wind speed representing a 3 second gust measured at 10 m (33 ft) above the ground. The design wind speed is 233 km/h (145 mph) in open terrain with an importance factor, “I”, of 1.15. The basis for the wind speed is American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI) Standard 7-05, “Minimum Design Loads for Buildings and Other Structures,” Figure 6-1 and represents the highest hurricane wind speed for the U.S., with the exception of the tip of Florida and the eastern tip of Louisiana. Since the design wind speed represents the highest hurricane wind speed over most of the U.S. (with only two exceptions), and because the COL applicant will need to confirm this design maximum wind speed for its specific site, the staff finds the use of a 233 km/h (145 mph) wind speed acceptable.

The design wind speed represents a recurrence interval of 50 years. In the ASCE/SEI Standard 7-05, it states that the use of the importance factor equal to 1.15 with the wind speed from ASCE/SEI Standard 7-05, Figure 6-1 is approximately equivalent to a wind speed with a recurrence interval of 100 years. As this meets the return period stated in SRP Acceptance Criteria 2.3.1.II.4 for establishing wind load on plant structures, the staff finds this acceptable. This design wind speed agrees with the information presented in FSAR Tier 1, Chapter 5, Table 5.0-1, “Site Parameters for the U.S. EPR Design,” and in FSAR Tier 2, Section 2.3.1, “Regional Climatology,” which references FSAR Tier 2, Table 2.1-1, “U.S. EPR Site Design Envelope.” However, FSAR Tier 2, Section 3.3.1.1, “Design Wind Velocity,” did not provide the basis for the wind load design of structures that are not Seismic Category I

(non-Seismic Category I structures) provided with the U.S. EPR design. Additionally, if the wind loading for these structures is less than that used for Seismic Category I structures, then the impact and consequences of failure of these other structures on Seismic Category I structures need to be addressed. Therefore, in Request for Additional Information (RAI) 94, Question 03.03.01-1(a) the staff requested that the applicant provide the basis for the wind load design of other structures covered in the FSAR, and if the wind loading for these other structures is less than that for safety-related structures, to address the consequences of their failure.

In a December 8, 2008, response to RAI 94, Question 03.03.01-1(a), the applicant clarified that all standard plant structures, given in FSAR Tier 2, Section 1.8, "Interfaces with Standard Designs and Early Site Permits," use the wind load design value used for Seismic Category I structures, eliminating the need to address the consequences of failure for these structures due to wind loading. As identified above in this technical evaluation, the wind speed used for Seismic Category I structures is 233 km/hr (145 mph). Therefore, the staff finds the applicant's December 8, 2008, response to RAI 94, Question 03.03.01-1(a) addresses the staff concerns and is acceptable. The staff considers RAI 94, Question 03.03.01-1(a) resolved.

3.3.1.4.2 Procedures to Transform Wind Parameters into Equivalent Loads on Structures

In FSAR Tier 2, Section 3.3.1.2, "Determination of Applied Wind Forces," wind velocity is converted into effective pressure loads using the formula from ASCE/SEI Standard 7-05, Section 6.5.10. This meets SRP Acceptance Criteria 3.3.1.II.3.A, and thus the staff finds the use of this formula acceptable. This formula has a number of parameters that are used to calculate the resulting wind pressure load. These parameters are K_z, which is a velocity pressure exposure coefficient that varies with height z, K_{zt} which is a function of topography, K_d which is a wind directionality factor, and I which is an importance factor for the structure. The values specified in the acceptance criteria of SRP Section 3.3.1 and those used in the FSAR are identical and as such the staff finds the FSAR values acceptable.

SRP Acceptance Criteria 3.3.1.II.3.B states that the coefficients for K_z should be based on exposure level C which is applicable for flat open country, grasslands, and all water surfaces in hurricane prone regions. The selection of exposure Level D provides higher K_z coefficients. According to the standard, exposure Level D includes smooth mud flats, salt flats, and unbroken ice outside of hurricane prone regions. Continental U.S. shorelines in exposure Level D include inland waterways; the Great Lakes; and coastal areas of California, Oregon, and Washington. Since the wind pressure load is a function of the velocity squared, and the velocity in exposure Level D regions provided in ASCE/SEI Standard 7-05, Figure 6-1 is lower and, in most cases, significantly lower than the hurricane wind speed selected, the staff finds the use of K_z values for exposure Level C acceptable as the higher hurricane wind speed more than compensates for the higher K_z value associated with exposure Level D. In addition, as shown in Combined License (COL) Information Item 3.3-1 in Table 3.3.1-2 of this report, a COL applicant that references the U.S. EPR design certification will determine site-specific wind and tornado characteristics and compare these to the standard plant criteria. If the site-specific wind and tornado characteristics are not bounded, then the COL applicant will analyze the design for site specific wind and tornado events and demonstrate that these loadings will not adversely affect the ability of safety-related structures to perform their safety functions during or after such events. The COL applicant's analysis will need to be sufficient to justify the departure from the associated site parameters in FSAR Tier 1, as required under 10 CFR 52.79(d) and the change procedures in the design certification rule. In FSAR Tier 2, Section 3.3.1.2, the applicant states

that effective pressure loads on structural elements and members are determined according to the applicable requirements of the ASCE/SEI Standard 7-05, Sections 6.5.12 and 6.5.13. As this meets SRP Acceptance Criteria 3.3.1.II.3.C, the staff finds the use of these sections of the standard acceptable. However, no mention is made of the use of ASCE/SEI Standard 7-05, Sections 6.5.14 and 6.5.15, which are applicable to the design wind loads on solid freestanding walls and solid signs, and to the design wind loads on other structures not covered by the other standard sections. Therefore, in RAI 108, Question 03.03.01-2(1), the staff requested that, if applicable, the applicant add the use of these additional sections to the FSAR.

In a December 8, 2008, response to RAI 108, Question 03.03.01-2(1), the applicant stated that FSAR Tier 2, Section 3.3.1.2 would be revised to indicate that the effective pressure loads on structural elements and members will conform to the applicable requirements of ASCE/SEI Standard 7-05, Reference 1, Sections 6.5.12 through 6.5.15. Since all four of the applicable sections of ASCE/SEI Standard 7-05 for determining effective pressure loads on structural elements have been included in FSAR Revision 1, dated May 29, 2009, the staff finds the response acceptable. Therefore, the staff considers RAI 108, Question 03.03.01-2(1) resolved.

FSAR Tier 2, Section 3.3.1.2, indicates that the shape coefficients for distribution of wind pressures around the circumference of the Reactor Shield Building (RSB) and the vent stack are taken from ASCE paper No. 3269, "Comparison of Analytical Methods for Calculation of Wind Loads," but the basis for the use of this reference is not given. Therefore, in RAI 108, Question 03.03.01-2(2), the staff requested that the applicant provide the shape factors and justification for the shape factors used for the RSB and vent stack be provided and compared to those of ASCE/SEI Standard 7-05.

In a December 8, 2008, response to RAI 108, Question 03.03.01-2(2), the applicant stated: "ASCE paper No. 3269, 'Wind Forces on Structures' (Reference 2) is used to determine the external pressure coefficients for distribution of wind pressures around the circumferences of the Reactor Shield Building and the vent stack." The external pressure coefficients (C_p) used for the Reactor Shield Building and the vent stack vary with the angle (α), where α = 0 corresponds to a vector contacting the windward face and parallel to the wind direction. Since C_p also depends on the ratio of structure height to structure diameter (h/d), different sets of C_p values are used for the RSB cylinder wall and vent stack. Both sets of C_p values are shown below in Table 3.3.1-1:

Table 3.3.1-1 Normal Wind Pressure Variation on U.S. EPR Structures

	α (degrees)										
	0	15	30	45	60	75	90	105	120	135	150-180
C_p for RSB	1.0	0.8	0.1	-0.7	-1.2	-1.6	-1.7	-1.2	-0.7	-0.5	-0.4
C_p for Vent Stack	1.0	0.8	0.1	-0.81	-1.71	-2.22	-2.22	-1.71	-0.81	-0.61	-0.51

ASCE/SEI Standard 7-05 does not provide a method to calculate C_p values for round structures. ASCE/SEI Standard 7-05, Section 6.5.12.2.1, states "C_p= external pressure coefficient from Figure 6-6 or 6-8." Figure 6-6 provides C_p values for square and rectangular buildings, while Figure 6-8 provides C_p values for arched roofs. This shortcoming of ASCE/SEI Standard 7-05 (and its predecessors ANSI/ASCE Standard 7 and ANSI A58.1) was acknowledged in previous

versions of NUREG-0800, Section 3.3.1 (Revision 2 and DRAFT Revision 3) by specifically stating: "ASCE Paper No. 3269, *Wind Forces on Structures*, may be used to obtain the effective wind pressures for cases which ANSI A58.1 does not cover." Therefore, since the current revision of SRP Section 3.3.1 references ASCE/SEI Standard 7-05 which does not contain a manner for calculation of C_p values for round structures, and since previous revisions of SRP Section 3.3.1 specifically acknowledged the adequacy of ASCE Paper No. 3269 for calculation of C_p values, use of this paper is justified. Accordingly, the staff finds the response acceptable. In addition to the reasons cited above, in the review of the AP 1000 design, the staff was provided justification by the AP1000 design certification applicant for the circumferential pressure distribution indicated in ASCE Paper 3269, which was based on a comparison to the pressure distribution obtained from AP600 shield building wind tunnel tests as documented in WCAP-13294-P, Appendix C. The ASCE/SEI Standard 7-05 does provide C_p values for domed roofs in ASCE/SEI Standard 7-05, Figure 6-7. These C_p values were used by the applicant to determine wind loads on the Reactor Shield Building roof. Therefore, since the applicant used an adequate, technical method to derive the shape factors for the RSB and the vent stack, the staff considers RAI 108, Question 03.03.01-2(2) resolved.

The vent stack is a tall cylindrical structure and such structures can fail under wind load due to vortex shedding. If vortex shedding was considered in design, then the staff would need to review the analytical methodology and related wind parameters that were used need to be provided. If the analytical methodology and related wind parameters were not provided, the basis for not including vortex shedding in the design and the consequences of a vent stack failure on safety-related structures would need to be reviewed. Therefore, in RAI 94, Question 03.03.01-1(c), the staff requested that the applicant provide additional information on vortex shedding considerations in the design of the vent stack.

In a December 8, 2008, response to RAI 94, Question 03.03.01-1(c), the applicant stated that the effects of vortex shedding with respect to the wind parameters defined in FSAR Tier 2, Section 3.3.1.1, were investigated, and the applicant stated that a tuned mass damper will be utilized to minimize vortex shedding-induced load effects. With a tuned mass damper in place, vortex shedding is not the limiting design load; rather, the safe-shutdown earthquake (SSE) produces the limiting vent stack design load. The staff finds the December 8, 2008, response to RAI 94, Question 03.03.01-1(c) acceptable in view of the following two considerations. First, tuned mass dampers are commonly used on tall cylindrical structures to limit the structural resonance and possible structural failure due to vortex shedding. Second, with a tuned mass damper in place, vortex shedding is not the limiting design load, the safe shutdown earthquake produces the limiting vent stack design load. Since the vent stack is a Seismic Category I structure and must be designed for the SSE, the staff finds the design basis for this structure acceptable. The staff finds that the issue has been resolved based on the RAI response and audit discussion and, therefore, considers RAI 94, Question 03.03.01-1(c) resolved.

3.3.1.4.3 Tier 1 Information

The FSAR Tier 1 information associated with this section is found in FSAR Tier 1, Chapter 5, "Site Parameters," where the maximum wind speed excluding tornado wind is 233km/hr (145 mph). The staff notes that this agrees with the wind speed cited in FSAR Tier 2, Section 3.3.1, which the staff finds acceptable as summarized in its review above. Design loads for structures are identified in FSAR Tier 1, Section 2.1 design descriptions and their associated ITAAC. Design loads for external events identified in design commitment summary in the ITAAC tables do not include wind loads. In accordance with GDC 2, SSCs important to safety should be designed to withstand the effects of natural phenomena including hurricane wind

loads. Therefore, in RAI 508, Question 03.03.01-5, the staff requested that the applicant add wind to the list of external event design basis loads for Seismic Category I structures to the ITAAC tables of FSAR Tier 1, Section 2.1. **RAI 508, Question 03.03.01-5 is being tracked as an open item**.

3.3.1.5 *Combined License Information Items*

Table 3.3.1-2 provides a list of wind loading related COL information item numbers and descriptions from FSAR Tier 2, Table 1.8-2:

Table 3.3.1-2 U.S. EPR Combined License Information Items

Item No.	Description	FSAR Tier 2 Section
3.3-1	A COL applicant that references the U.S. EPR design certification will determine site-specific wind and tornado characteristics and compare these to the standard plant criteria. If the site-specific wind and tornado characteristics are not bounded by the site parameters, postulated for the certified design, then the COL applicant will evaluate the design for site-specific wind and tornado events and demonstrate that these loadings will not adversely affect the ability of safety-related structures to perform their safety functions during or after such events.	3.3
3.3-2	A COL applicant that references the U.S. EPR design certification will demonstrate that failure of site-specific structures or components not included in the U.S. EPR standard plant design, and not designed for wind loads, will not affect the ability of other structures to perform their intended safety functions.	3.3.1

The staff finds the above listing to be complete. Also, the list adequately describes actions necessary for the COL applicant. No additional COL information items need to be included in FSAR Tier 2, Table 1.8-2 for wind loading

3.3.1.6 *Conclusions*

Except for the open item specified above, the staff concludes that sufficient information has been provided by the applicant with respect to the development of wind loads and their application to the structures of the U.S. EPR design to satisfy applicable NRC regulations. Based upon its review, the staff has made the following conclusions.

The guidance in the acceptance criteria of NUREG- 0800, Section 3.3.1 as it relates to the development of design-basis wind loads on structures has been met, as described above. The applicant has met the requirements of 10 CFR Part 50, Appendix A, “General Design Criteria for Nuclear Power Plants,” GDC 2, and 10 CFR 52.47 with respect to the capability of the

structures to withstand design wind loading such that the design reflects appropriate consideration of severe hurricane wind, appropriate combination of the effects of normal and accident conditions with the design wind load, and the importance of the safety function to be performed by safety-related structures.

The staff notes that the applicant has designed the plant structures with sufficient margin to prevent structural damage during a severe hurricane wind and has used methods provided in ASCE/SEI Standard 7-05 to transform wind speed into equivalent pressures on structures which the staff reviewed and found acceptable, as described above. The design of Seismic Category I structures has included wind load and the loads resulting from normal and accident conditions.

The use of these methods provides reasonable assurance that in the event of design basis winds, the structural integrity of the plant structures that must be designed to resist the effects of the design wind speed will not be impaired and, in consequence, safety-related systems and components located within these structures are adequately protected and will perform their intended safety functions.

3.3.2 Tornado Loadings

3.3.2.1 *Introduction*

Seismic Category I structures must withstand design basis tornado loads and maintain their safety-related functions during and following a tornado event. This section documents the findings from the staff's review and evaluation of the description of the U.S. EPR design parameters applicable to tornadoes and the procedures used to transform tornado wind and associated atmospheric pressure drop into equivalent loads on structures. Also provided is the evaluation of Non-Seismic Category I structures that have the potential for interacting with Seismic Category I structures. The response of Non-Seismic Category I structures must not affect the safety-related functions of Seismic Category I structures under tornado load conditions.

3.3.2.2 *Summary of Application*

FSAR Tier 1: FSAR Tier 1 information providing tornado design parameters is found in FSAR Tier 1, Chapter 5, Table 5.0-1, "Site Parameters for the U.S. EPR Design." Other related FSAR Tier 1 information associated with this section is found in FSAR Tier 1, Section 2.1, "Structures."

FSAR Tier 2: In FSAR Tier 2, Section 3.3.2, "Tornado Loadings," the applicant has provided a description for determining tornado loadings on structures which is summarized, in part, as follows.

Tornado loads are determined from a design-basis tornado which has a probability of exceedance equal to 1×10^{-7} per year. Tornado loads include loads caused by tornado wind pressure, atmospheric pressure change, and tornado-generated missile impact. As specified in FSAR Tier 2, Table 2.1-1, "U.S. EPR Site Design Envelope," the maximum tornado wind speed is 370 km/hr (230 mph), maximum tornado rotational speed is 296 km/hr (184 mph), maximum translational wind speed is 74 km/hr (46 mph), maximum tornado pressure drop is 8.3 kPa (1.2 psi) at 3.45 kPa/sec (0.5 psi/sec), and the radius of maximum rotational speed is 45.7 m (150 ft). The description of wind-generated missile loads and missile protection design criteria is provided in FSAR Tier 2, Section 3.5, "Missile Protection." In addition to the description of the

tornado wind load applied to Seismic Category I structures, this section addresses tornado wind load on Non-Seismic Category I structures and the measures used to prevent their interaction with Seismic Category I structures.

ITAAC: The inspections, tests, analyses, and acceptance criteria items for this area of review relate to structures being designed for external events including tornado and tornado generated missiles. The ITAAC are provided in FSAR Tier 1, Section 2.1.

3.3.2.3 *Regulatory Basis*

The relevant requirements of NRC regulations for this area of review, and the associated acceptance criteria, are given in NUREG-0800, Section 3.3.2, "Tornado Loadings," as summarized below. Review interfaces with other SRP sections also can be found in NUREG-0800, Section 3.3.2.

1. 10 CFR Part 50, Appendix A, GDC 2, "Design Bases for Protection Against Natural Phenomena," as it relates to the requirement that structures, systems, and components that are important to safety be designed to withstand the effects of natural phenomena such as earthquakes, tornados, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety functions. The design bases for these SSCs must reflect appropriate combinations of the effects of normal and accident conditions with the effects of the natural phenomena including consideration of the most severe of natural phenomena that have been historically reported for a site and appropriate combinations of the effects of normal and accident conditions with the effects of the natural phenomena.

2. 10 CFR 52.47, as it relates to the requirement that a design certification application contain the proposed ITAAC that are necessary and sufficient to provide reasonable assurance that if the inspections, tests, and analyses are performed and the acceptance criteria met, a plant that incorporates the design certification is built and will operate in accordance with the design certification, the provisions of the Atomic Energy Act of 1954, and NRC regulations.

Acceptance criteria adequate to meet the above requirements include:

1. Regulatory Guide (RG) 1.76, "Design-Basis Tornado and Tornado Missiles for Nuclear Power Plants," as it relates to the maximum tornado wind speed, rate of pressure drop, and tornado missile characteristics.

2. RG 1.143, "Design Guidance for Radioactive Waste Management Systems, Structures, and Components Installed in Light-Water-Cooled Nuclear Power Plants," as it relates to design requirements for tornado wind and tornado missiles for radioactive waste structures.

3.3.2.4 *Technical Evaluation*

Below is the staff's evaluation of the applicant's development of tornado wind loads used in the design of Seismic Category I structures. This review ensures compliance with 10 CFR Part 50, Appendix A, GDC 2 for that part which requires structures be designed to withstand the effects of a tornado without loss of the capability to perform their safety functions. The specific loads considered are those associated with design-basis tornado winds and atmospheric pressure changes. The staff also reviewed the requirements for Non-Seismic Category I structures or

components not designed for tornado loads to insure they will not affect the capability of safety-related structures or components to perform their necessary safety functions. The staff also reviewed the application in accordance with NUREG-0800, Section 3.3.2. Missiles generated from tornadoes and extreme winds are addressed in FSAR Tier 2, Section 3.5.1.4, “Missiles Generated by Tornadoes and Extreme Winds.” Protection from tornado generated missiles is discussed in FSAR Tier 2, Section 3.5.3, “Barrier Design Procedures.”

3.3.2.4.1 Tornado Loadings

In FSAR Tier 2, Section 3.3.2, the applicant states that 25 percent of the design live load is considered with tornado load combinations. SRP Acceptance Criteria (AC) 3, “Loads and Load Combinations,” states that all load combinations are to be in accordance with American Concrete Institute (ACI) 349, “Code Requirements for Nuclear Safety-Related Concrete Structures.” The reduction to 25 percent live load with tornado load as stated in FSAR Tier 2, Section 3.3.2 and in FSAR Tier 2, Section 3.8.4.3.2, “Loading Combinations,” appeared to contradict ACI 349 and RG 1.143, November 2011, Table 3, “Design Load Combinations,” which requires the use of full live load instead of the 25 percent live load described above. Therefore, in RAI 126, Question 03.03.02-1(5), the staff requested that the applicant provide a basis for this live load reduction in combination with tornado load be provided along with a justification for this deviation from the pertinent provisions of SRP Section 3.8.3. In a November 24, 2008, response to RAI 126, Question 03.03.02-1(5), the applicant stated that 100 percent of the live load is used for structural design activities and that FSAR Tier 2, Section 3.3.2, will be updated to state: “One hundred percent of the design live load is considered with tornado load combinations.” The reference to use 25 percent of the live load in FSAR Tier 2, Section 3.8.1.3.2 (2nd bullet), Section 3.8.4.4.1 (9th bullet), and Section 3.8.5.4.1 (4th bullet) will be updated to state: “Twenty-five percent of the design live load is considered during static analysis with seismic load combinations. The full potential live load is used for local analysis of structural members.” However, neither the original nor the revised statement agreed with ACI 349-97 or with the load combinations in FSAR Tier 2, Section 3.8.1.3.2, “Design Load Combinations,” or FSAR Tier 2, Section 3.8.4.3.2 in which load combinations that contain either tornado load or seismic load include the full live load. The FSAR text should be made consistent with the load combination equations where the full live load is added with either the tornado load or seismic load. Therefore, in follow-up RAI 211, Question 03.03.02-3(c), the staff requested that in addition to correcting the text in FSAR Tier 2, Section 3.3.2, the applicant update the text in FSAR Tier 2, Sections 3.8.1.3.2, 3.8.4.4.1, and 3.8.5.4.1 to state the full live load is added to either the tornado load or seismic load for the appropriate load combination. In an August 26, 2009, response to RAI 211, Question 03.03.02-3(c), the applicant stated that FSAR Tier 2, Sections 3.8.1.3.2, 3.8.4.4.1, and Section 3.8.5.4.1 would be revised to clarify that the full potential live load is used in the structural load combinations including those containing seismic or tornado load. The staff has confirmed these changes have been made to the text of the FSAR. Since analyses include consideration of the full live load, the staff finds the August 26, 2009, response to follow-up RAI 211, Question 03.03.02-3(c) acceptable. Therefore, the staff considers RAI 126, Question 03.03.02-1(5) and follow up RAI 211, Question 03.02.02-3(c) resolved.

3.3.2.4.2 Applicable Tornado Design Parameters

Information provided in FSAR Tier 2, Section 3.3.2 states that tornado wind and related parameters used as a basis for the applicable tornado design load parameters are chosen according to the Characteristic Design Basis Tornado for Tornado Region I. This tornado region provides the most severe tornado conditions of the three regions presented in RG 1.76, Table 1,

"Design - Basis Tornado Characteristics." These conditions correspond to a translational wind speed of 21 m/s (46 mph), a rotational wind speed of 82 m/s (184 mph), and a total wind speed of 103 m/s (230 mph). In addition, the radius of maximum rotational speed (45.7 m (150 ft)), maximum pressure drop (8.3kPa (1.2 psi)), and rate of pressure drop (3.45 kPa/sec (0.5 psi/sec)) all meet the criteria of RG 1.76. Since these conditions represent the worst case design basis in the contiguous U.S., the staff finds these parameters acceptable.

3.3.2.4.3 Procedures to Transform Tornado Parameters into Equivalent Loads on Structures

Tornado wind velocity is converted into effective wind pressure using the formula from ASCE/SEI 7-05, Section 6.5.10, which is in agreement with SRP Section 3.3.2 Acceptance Criteria 3.B.i for calculating wind pressure loads. This formula has a number of parameters the values of which must be selected to calculate the resulting pressure load. The staff noted differences between the values provided in SRP Section 3.3.2 AC 3.B.i and those used in the FSAR. In FSAR Tier 2, Section 3.3.2.2, "Determination of Tornado Forces on Structures," the equation for wind pressure velocity used a value of K_z and a value of I both equal to 1.0, while in the SRP the value of K_z is equal to .87 and the value of I is equal to 1.15. The use of the FSAR values results in a pressure load that is slightly lower than that computed based on the SRP provisions. Therefore, in RAI 126, Question 03.03.02-1, the staff requested that the applicant provide the basis for using different parameter values (i.e., those for K_z and I) than those specified in SRP Section 3.3.2 and to quantify the resultant impact on the design of Seismic Category I structures. In a November 24, 2008, response to RAI 126, Question 03.03.02-1, the applicant stated that FSAR Tier 2, Section 3.3.2.2.1, "Notes on Values Used," was intended to reconcile the differences between the FSAR specified values of velocity pressure exposure coefficient (K_z) and the importance factor (I) and those in the SRP. As stated in FSAR Tier 2, Section 3.3.2.2.1, using the values of the factors K_z and I from FSAR Tier 2, Section 3.3.2.2, the procedure results in a conversion of tornado wind speed into an equivalent tornado wind pressure with no significant differences from the results generated using the factors as given in the SRP. The staff agrees that there is not a significant difference between using the FSAR values for the exposure coefficient (K_z) and the importance factor (I) and using the values stated in the acceptance criteria of the SRP. The difference in the calculated tornado wind load is only .05 percent lower using the applicant's coefficients. However, the bases for the applicant's values were not provided in the response. Therefore, in follow-up RAI 283, Question 03.03.02-4 the staff requested again that the applicant provide the basis for the values of K_z and I used in the FSAR. In a November 12, 2009, response to RAI 283, Question 03.03.02-4, the applicant stated it would change the values of K_z and I to agree with those in SRP Section 3.3.2. The staff has confirmed these changes have been made to the text of the FSAR. Since the values in the FSAR now correspond to those in the SRP, the staff considers RAI 126, Question 03.03.02-1(1) and RAI 283, Question 03.03.02-4 resolved.

RG 1.76 provides guidance for determining the pressure drop and the rate of pressure drop caused by the passage of a tornado. In FSAR Tier 2, Section 3.3.2.2, the applicant states that the walls and roofs of Seismic Category I structures are designed for the maximum differential pressure of 8.3 kPa (1.2 psi). For design purposes, this assumes that the structures are totally enclosed (unvented) and are in agreement with the applicable SRP provision for the pressure drop of enclosed structures. The FSAR further states that when the pressure boundary is not established by exterior walls or roofs, the pressure drop is taken as zero. This would be the case for the vent stack which is an open structure and this agrees with the SRP provision which states that for completely open structures the pressure drop approaches zero. For a partially enclosed (vented) structure, the SRP indicates that the assumed pressure drop should be

reviewed on a case-by-case basis. It was not clear if any structures in the application's scope were treated as partially enclosed structures and if so, what assumption was used for the tornado pressure drop for building design. Therefore, in RAI 126, Question 03.03.02-2(1), the staff requested that the applicant identify any partially enclosed structures in the U.S. EPR design and provide the basis for the pressure drop used in the analysis of those structures. In a November 24, 2008, response to RAI 126, Question 03.03.02-2(1), the applicant stated that no Seismic Category I structures have been identified which fall into the partially enclosed category. As all structures are either open or closed with respect to tornado wind differential pressure, and the methods cited agree with the acceptance criteria of SRP Section 3.3.2, the application conforms to the guidance on pressure drop. The staff finds the November 24, 2008, response to RAI 126, Question 03.03.02-2(1) acceptable and, therefore, considers RAI 126, Question 03.03.02-2(1) resolved.

FSAR Tier 2, Section 3.3.2.2, states that the tornado-generated missile impact parameters conform to the guidance in RG 1.76. In FSAR Tier 2, Section 3.5.1.4, the tornado generated missile parameters are presented in Table 3.5-1, "Spectrum of Design Basis Tornado Missiles." The parameters correspond to the values given in RG 1.76, Table 2, "Design Basis Tornado Missile Spectrum and Maximum Horizontal Speeds," for Region I. Vertical velocity is taken as 67 percent of the maximum horizontal speed which also agrees with RG 1.76. Therefore, the staff finds the parameters that are used are acceptable.

Tornado-generated wind effects, atmospheric pressure change effects, and missile impact effects should be combined in a conservative manner and then added to other loads in the appropriate load combinations for the structure. The staff finds that the equations provided in the FSAR for combining tornado generated wind effects agree with those given in SRP Acceptance Criteria 3.3.2.II.3.E, and are therefore acceptable.

3.3.2.4.4 Effect of Failure of Structures not Designed for Tornado Loads

SRP Acceptance Criteria 3.3.2.II.4 indicates that information should be provided to demonstrate that failure of any structure or component not designed for tornado loads will not affect the capability of safety-related structures or components to perform their necessary safety functions.

According to FSAR Tier 2, Section 3.3.2.3, "Effect of Failure of Structures not Designed for Tornado Loads," Non-Seismic Category I structures are not designed for tornado loads unless their failure during a tornado could adversely affect nearby Seismic Category I SSCs. Seismic Category I structures are protected from failure of adjacent Non-Seismic Category I structures during a tornado by one of the following methods:

- The adjacent Non-Seismic Category I structure is designed to resist applicable tornado loadings.

- The integrity of a Seismic Category I structure is evaluated for failure of an adjacent Non-seismic Category I structure during a design basis tornado to verify the functionality and continued operation of Seismic Category I structure during and after the tornado.

- A structural barrier is provided to protect the Seismic Category I structure from failure of the adjacent Non-Seismic Category I structure as a result of a tornado.

These criteria are similar to the acceptance criteria of SRP Section 3.3.2 which state that the information provided to demonstrate that failure of any SSC to perform its safety function is acceptable if:

- The postulated failure or collapse of structures and components not designed for tornado loads, including missiles can be shown not to result in any structural or other damage to safety-related structures, systems, or components.

- Safety-related structures are designed to resist the effects of the postulated structural failure, collapse, or generation of missiles from structures and components not designed for tornado loads.

Structures close or adjacent to Seismic Category I structures and whose possible failure could impact Seismic Category I structures are the Vent Stack, Nuclear Auxiliary Building (NAB), Access Building (ACB), and Turbine Building (TB).

FSAR Tier 2, Section 3.3.2.3, states that the acceptance criteria for the design of the vent stack includes the use of ASCE Standard 43 (Limit State A) for overall stability, and the use of ACI Standard 349 for anchorages. However, it was unclear to the staff how these methods were applied and if these methods provide an acceptable level of protection for Seismic Category I structures. Therefore, in RAI 126, Question 03.03.02-2(2) the staff requested that the applicant provide the factors used for developing the tornado load on the vent stack, the basis for the loads used to design the anchorages, and the margin of safety for the vent stack against its collapse on adjacent Seismic Category I structures. In a November 24, 2008, response to RAI 126, Question 03.03.02-2(2), the applicant stated that although the vent stack is classified as a Seismic Category II structure, it is globally designed to the same requirements as a Seismic Category I structure due to its proximity to other Seismic Category I structures as stated in FSAR Tier 2, Section 3.7.2.8. Accordingly, the factors used for development of tornado wind loads on the vent stack are as specified in FSAR Tier 2, Section 3.3.2.2. These loadings are used for the design of the vent stack structure, as well as the anchorage of the vent stack structure to the stair tower roof slab. Since this response was received, the vent stack in FSAR Revision 2 has been re-categorized as a Seismic Category I structure. As the vent stack analysis and design for tornado load will now conform to that of a Seismic Category I structure, the staff finds its tornado design basis acceptable. Therefore, the staff considers RAI 126, Question 03.03.02-2(2) resolved.

Although the vent stack is designed to not fail under tornado loads, the possibility of vortex shedding on the vent stack under tornado load was not discussed in the FSAR. Therefore, in RAI 126, Question 03.03.02-1(6), the staff requested that the applicant clarify the effects of vortex shedding on the vent stack's structural integrity. In a November 24, 2008, response to RAI 126, Question 03.03.02-1(6), the applicant stated that the effects of vortex shedding induced loads will be minimized by the use of a tuned mass damper. With a tuned mass damper in place, vortex shedding is not the controlling load case for design; rather, the safe shutdown earthquake controls the design of the vent stack. As tuned mass dampers are one of several devices that are used on tall cylindrical structures to prevent the structural resonance and possible structural failure due to vortex shedding, the staff finds the response acceptable and, therefore, considers RAI 126, Question 03.03.02-1(6) resolved.

In FSAR Tier 2, Section 3.3.2.3, the applicant states that for the NAB the methodology of ASCE Standard 58, "Structural Analysis and Design of Nuclear Plant Facilities," was used to show the structure will not collapse under tornado loads. However, the implementation of this

methodology was not described and a conclusion as to the adequacy of the NAB under tornado loads could not be reached until additional information was provided. Therefore, in RAI 126, Question 03.03.02-1(2), the staff requested that the applicant provide the methodology from ASCE Standard 58 that was used to determine that the NAB will not collapse under a tornado loading. In a November 24, 2008, response to RAI 126, Question 03.03.02-1(2), the applicant stated the methodology of ASCE 43-05 (Limit State A) would be utilized to ensure that the NAB would not collapse under tornado loads and affect Seismic Category I Nuclear Island Common Basemat structures. Additionally, the applicant stated that the NAB would be evaluated for tornado loadings in accordance with RG 1.143, due to its classification as a radioactive waste (RW)-IIa (high hazard) structure.

The staff's assessment was that ASCE 43-05 provides seismic design criteria for SSCs in nuclear facilities, but has not been accepted by the staff for the seismic design of nuclear power plant structures. Furthermore, the Limit States in ASCE 43-05 were developed for seismic loads and not for wind loads. Limit States define acceptable damage to a structure with Limit State A providing the least margin of safety, as structures designed to Limit State A can be expected to have significant damage. In addition, RG 1.143 provides design tornado wind loads that are three-fifths of the tornado wind load of RG 1.76. These design loads may be appropriate for a Non-Seismic Category I stand-alone structure, but in this instance the NAB is situated adjacent to Seismic Category I structures. SRP Acceptance Criteria 3.7.2.II.08.C, which addresses the interaction of a Non-Seismic Category I structure with a Seismic Category I structure, states that a Non-Seismic Category I structure analyzed and designed to prevent its failure under SSE conditions, should have a margin of safety that is equivalent to that of a Seismic Category I structure. The same design approach is applicable to the design of Non-Seismic Category I structures for tornado loads as the tornado wind represents an extreme environmental load and the level of protection provided the Seismic Category I structure for a tornado wind load should be no less than that provided under a seismic load. Therefore, in RAI 211, Question 03.03.02-3(a), the staff requested that the applicant provide the design tornado wind velocity, tornado loads, loading combinations, and the design code for the NAB structure such that its design under the design tornado loading conditions would provide a margin of safety that was equivalent to that of a Seismic Category I structure. If the design provided a margin of safety that was not equivalent to that of a Seismic Category I structure, the staff requested that the applicant provide to the staff the methods of analysis, acceptance criteria, critical sections, and calculated displacements of the NAB which demonstrated that the NAB would not collapse on adjacent Seismic Category I structures or prevent them from performing their intended safety functions. In an August 26, 2009, response to RAI 211, Question 03.03.02-3(a), the applicant stated that FSAR Tier 2, Section 3.3.2.3 would be revised to remove reference to ASCE Standard 43. Tornado wind effects would be considered in NAB design because of the NAB's potential to interact with adjacent Seismic Category I structures. The applicant's response also stated that "RG 1.76 tornado wind characteristics above the requirements of RG 1.143 tornado wind loading would be considered in NAB design so that no unanalyzed loads are transferred to adjacent Seismic Category I SSC [sic]." The staff's assessment was that because the phrase, "no unanalyzed loads are transferred to the protected Category I SSC," was ambiguous and did not preclude an analyzed load from being transferred to the protected Seismic Category I structure, the applicant needed to clearly state in the FSAR that no load will be transferred from the NAB to a Seismic Category I structure due to a tornado having RG 1.76 tornado wind characteristics. Therefore, in follow-up RAI 384, Question 03.03.02-5, the staff requested that the applicant provide additional information regarding the capability of the NAB to withstand loads from the design basis tornado. The requested information included:

1. the methodology for converting the tornado wind velocity to a load on the structure

2. the design code for the structure

3. the design allowable for the load combination that includes the tornado wind load

For the Access Building and the Turbine Building, FSAR Tier 2, Section 3.3.2.3 stated that "one of the methods" will be utilized for determining that the ACB and TB will not collapse. It was unclear to the staff what method was being referenced or how it was being implemented. A conclusion that the ACB and TB would not impact adjacent Nuclear Island basemat structures under a tornado load could not be reached from the information provided. Therefore, in RAI 126, Question 03.03.02-1(3) the staff requested that the applicant specify and provide the method(s) used to assure protection of Seismic Category I structures from ACB or TB failure under tornado loading. In a November 24, 2008, response to RAI 126, Question 03.03.02-1(3), the applicant stated that the methods referred to were the three bulleted items in FSAR Tier 2, Section 3.3.2.3. Specifically, the three methods were:

- The adjacent Non-Seismic Category I structure is designed to resist applicable tornado loadings.

- The integrity of a Seismic Category I structure is evaluated for failure of an adjacent Non-Seismic Category I structure during a design basis tornado to verify the functionality and continued operation of the Seismic Category I structure during and after the tornado.

- A structural barrier(s) is provided to protect the Seismic Category I structure from failure of the adjacent Non-Seismic Category I as a result of a tornado.

For both the ACB and the TB, the applicant stated that the FSAR would be changed to state that one of the methodologies identified in the three bullets would be utilized to provide reasonable assurance that the ACB (or TB) would not collapse under tornado loads and affect Seismic Category I structures. However, only the first bullet dealt with designing these structures to resist applicable tornado loads. The second and third bullet assumed the building would collapse and other measures would be implemented to assure the collapse did not affect the safety function of a Seismic Category I structure. As it was still unclear to the staff what design approach was being followed, in RAI 211, Question 03.03.02-3(b), the staff requested that the applicant revise the RAI response to state that the first bullet in FSAR Tier 2, Section 3.3.2.3 would be followed in the design of the TB and ACB and to provide the design tornado wind, tornado loads, loading combinations, and design codes for the TB and ACB structures such that the design under the design tornado loading conditions provided a margin of safety that was equivalent to that of a Seismic Category I structure. If the margin of safety was not equivalent to that of the Seismic Category I structure, the staff requested that the applicant provide and clarify the detailed methods of analysis, acceptance criteria, expected damage, critical sections and calculated displacements of these structures for the staff's review.

In an August 26, 2009, response to RAI 211, Question 03.03.02-3(b), the applicant stated that the ACB and TB are site-specific structures that are not part of the U.S. EPR certified design, and the codes and standards employed in their design are the responsibility of the COL applicant as described in FSAR Tier 2, Table 1.8-2, COL Information Item 3.3-3. However, due to the proximity of these Non-Seismic Category I structures to safety-related structures, there exists the potential for structural interaction. The applicant further stated that RG 1.76 tornado wind characteristics would be considered in TB and ACB design so that safety functions of adjacent Seismic Category I structures were not impaired. The staff' concluded that the

response did not provide sufficient information to determine the design requirements for either the ACB or the TB. Furthermore, it was unclear to the staff how Seismic Category I structures are protected from the effects of tornado load on these Non-Seismic Category I structures. In follow-up RAI 384, Question 03.03.02-5, the staff requested that the applicant provide information similar to what had been requested for the design of the NAB.

In a July 29, 2010, response to RAI 384, Question 03.03.02-5, the applicant stated that the NAB, ACB, and TB would be analyzed and designed using RG 1.76 tornado wind characteristics and would be designed to the codes and standards associated with Seismic Category I structures so that the margin of safety is equivalent to that of a Seismic Category I structure. The applicant also stated that because the NAB, ACB, and TB do not have a safety function, it was possible to allow sliding or uplift provided the gap between the Seismic Category II structure and the Seismic Category I structure was sufficient to prevent structure-to-structure interaction. The applicant stated that it would remove the phrase regarding unanalyzed loads from FSAR Tier 2, Section 3.3.2.3 to clarify that no transfer would be allowed from the NAB, ACB, and TB to an adjacent Seismic Category I structure. Since the NAB will be designed with a safety margin that is equivalent to that of a Seismic Category I structure, the staff finds the applicant's July 29, 2010, response to RAI 384, Question 03.03.02-5 acceptable. With regard to the ACB and TB, the information in the FSAR is conceptual, and the design approach will need to be verified by the COL applicant as these are site-specific structures and are not included in the U.S. EPR design certification. A COL applicant will need to take exception to the conceptual design information in the FSAR and demonstrate the manner in which adjacent Seismic Category I structures are protected from the response of the TB and ACB under a tornado wind load. As a result, the staff finds the response regarding the TB and ACB also acceptable. Therefore, the staff considers RAI 384, Question 03.03.02-5 resolved. Potential sliding or uplift of the NAB under a tornado load demonstrating there is no interaction with an adjacent Seismic Category I structure is covered in the review of FSAR Tier 2, Section 3.7.2, "Seismic System Analysis." Interface requirements for the the ACB and TB are included in FSAR Tier 2, Table 1.8-1, Summary of U.S. EPR FSAR Plan interfaces with Remainder of Plant," under Items 1-2 and 1-3, repectively. The FSAR section identified with Items 1-2 and 1-3 which is relevant to the design basis for the ACB and TB is FSAR Tier 2, Section 3.7.2. This FSAR section provides the conceptual design information which indicates that the ACB and TB will be designed with a safety margin that is equivalent to that of a Seismic Category I structure, The staff finds the information provided in the interface requirements of FSAR Tier2, Table 1.8.-1 acceptable,

The Radioactive Waste Building (RWB) is classified as RW-IIa (high hazard) structure and will be designed to conform to the guidance of RG 1.143. As such the structure will be designed for wind tornado loads that are three-fifth of the RG 1.76 tornado loads. In addition, the RWB is a reinforced concrete structure designed to meet the standards of ACI 349 which is the concrete code applicable to nuclear power plant structures. Although unlikely under the load of a RG 1.76 tornado, the collapse of the RWB will not impact Seismic Category I structures, as its height above grade is less than the distance to the nearest Seismic Category I structure. In addition, if portions of the RWB were to be damaged and carried by tornado wind, the design, of a Seismic Category I structure, with respect to external missiles would ensure that the Seimic Category I structure's safety function would be maintained. Accordingly, the staff finds the tornado design basis for this structure acceptable.

Tier 1 Information

The FSAR Tier 1 requires that Seismic Category I structures be constructed to withstand design basis loads without loss of structural integrity and safety-related functions. The design basis loads are those loads associated with normal plant operation, as well as external events (including tornadoes). In addition, FSAR Tier 1, Table 5.0-1 provides tornado design parameters which are in agreement with design parameters found in NUREG-0800. As such, the staff finds the FSAR Tier 1 information for this area of review acceptable and conforms to the acceptance criteria described above.

ITAAC items associated with design loads including those from tornado are covered in the review of FSAR Tier 2, Section 14.3.2, "Tier 1, Chapter 2, System Based Design Descriptions and ITAAC."

3.3.2.5 *Combined License Information Items*

Table 3.3.2-1 provides a list of tornado loading related COL information item numbers and descriptions from FSAR Tier 2, Table 1.8-2:

Table 3.3.2-1 U.S. EPR Combined License Information Items

Item No.	Description	FSAR Tier 2 Section
3.3-1	A COL applicant that references the U.S. EPR design certification will determine site-specific wind and tornado characteristics and compare these to the standard plant criteria. If the site-specific wind and tornado characteristics are not bounded by the site parameters, postulated for the certified design, then the COL applicant will evaluate the design for site-specific wind and tornado events and demonstrate that these loadings will not adversely affect the ability of safety-related structures to perform their safety functions during or after such events.	3.3
3.3-3	A COL applicant that references the U.S. EPR design certification will demonstrate that failure of site-specific structures or components not included in the U.S. EPR standard plant design and not designed for tornado loads, will not affect the ability of other structures to perform their intended safety functions.	3.3.2

The staff finds the above listing to be complete. Also, the list adequately describes actions necessary for the COL applicant. No additional COL information items need to be included in FSAR Tier 2, Table 1.8-2 for tornado loading.

3.3.2.6 *Conclusions*

The staff concludes that sufficient information has been provided by the applicant with respect to the development of tornado loads and their application to the structures of the U.S. EPR

design. Based on its review of the information provided in FSAR Tier 2, Section 3.3.2, as set forth above, the staff concludes that the requirements of 10 CFR Part 50, Appendix A, "General Design Criteria for Nuclear Power Plants," GDC 2, and 10 CFR 52.47, as they relate to the design basis tornado and the development of tornado loads on structures, have been met. Specifically, in regard to the design of non-safety-related structures that are adjacent to safety-related structures, the staff concludes that the application of a Region I tornado load to a non-safety-related structure combined with a safety margin in its design equivalent to that of a Seismic Category I structure is an acceptable approach for preventing interaction of a Non-Seismic Category I structure with a Seismic Category I structure during a design basis tornado event.

Accordingly, the staff concludes that the applicant has met the requirements of GDC 2 with respect to the capability of structures to withstand design-basis tornado wind effects and tornado-generated atmospheric pressure change effects so that their design reflects:

1. appropriate consideration for the postulated most severe tornado with an appropriate margin

2. appropriate combinations of the effects of normal and accident conditions with the effects of the natural phenomena

3. the importance of the safety function to be performed

The applicant has designed the plant structures with sufficient margin to prevent structural damage during the postulated most severe tornado loadings so that the requirements in Item 1 given above are met. In addition, the design of Seismic Category I structures, as required in Item 2, includes load combinations reflecting the most severe tornado load with loads resulting from normal and tornado missile loads.

The use of these methods provides reasonable assurance that in the event of a design-basis tornado, the structural integrity of the plant structures that have to be designed for tornadoes will not be impaired and, in consequence, safety-related systems and components located within these structures will be adequately protected and may be expected to perform necessary safety functions as credited, thus satisfying the requirement in Item 3 listed above.

3.4 Water Level (Flood) Design

3.4.1 (To Be Inserted)

3.4.2 External Flood Protection

3.4.2.1 *Introduction*

To meet the requirements of GDC 2, Seismic Category I structures must be designed to withstand the effects of the highest probable maximum flood (PMF) and maximum groundwater levels. Consideration of external flooding must also include potential dynamic effects of the PMF such as currents and flood waves. This section documents the staff's review, evaluation, and findings regarding the U.S. EPR PMF, maximum groundwater elevation, and the measures implemented for Seismic Category I structures that provide protection from the effects of these natural phenomena.

3.4.2.2 *Summary of Application*

FSAR Tier 1: The FSAR Tier 1 information associated with this section is found in FSAR Tier 1, Section 2.1.1, “Nuclear Island”; Section 2.1.2, “Emergency Power Generating Buildings”; Section 2.1.5, “Essential Service Water Building”; and Section 5.0, “Site Parameters.”

FSAR Tier 2: The applicant has provided an FSAR Tier 2, external flood protection description in Section 3.4.2, “External Flood Protection,” summarized here in part, as follows:

Seismic Category I structures, are designed to be protected from external floods and groundwater. Protective measures are in part provided by the following:

- The PMF elevation of the U.S. EPR design is 0.30 m (1 ft) below finished grade.
- The maximum groundwater elevation for the U.S. EPR generic design is 1.0 m (3.3 ft) below finished grade.
- The finished yard grade slopes away from Seismic Category I structures, so that external flood waters flow away from these structures.
- No access openings or tunnels penetrate the exterior walls of U.S. EPR Seismic Category I structures below grade.
- The maximum rainfall rate for roof design is .49m (19.4 in.) per hour, and the maximum static roof load because of snow and ice is 4.8kPa (100 lb per square foot).
- Portions of Seismic Category I structures located below grade elevation incorporate the use of waterstops and waterproofing to mitigate environmental deterioration of exposed surfaces and thereby minimize long term maintenance.
- Exterior wall or floor penetrations of Seismic Category I structures below grade have watertight seals.
- Seismic Category I structures can withstand hydrostatic loads resulting from groundwater pressure and external flooding.

ITAAC: ITAAC items for this area of review are provided in FSAR Tier 1, Table 2.1.1-8, “Reactor Building ITAAC”; Table 2.1.1-10, “Safeguard Buildings ITAAC”; Table 2.1.1-11, “Fuel Building ITAAC”; Table2.1.2-3, “Emergency Power Generating Building ITAAC”; and Table 2.1.5-3, “Essential Service Water Building ITAAC.”

3.4.2.3 *Regulatory Basis*

The relevant requirements of NRC regulations for this area of review, and the associated acceptance criteria, are given in NUREG-0800, Section 3.4.2, “Analysis Procedures,” and are summarized below. Review interfaces with other SRP sections also can be found in NUREG-0800, Section 3.4.2.

1. 10 CFR Part 50, Appendix A, GDC 2, as it relates to the structures, systems, and components important to safety being designed to withstand the effects of natural phenomena such as earthquakes, tornados, hurricanes, floods, tsunami, and seiches

without loss of capability to perform their safety functions. The design bases for these SSCs shall reflect appropriate combinations of the effects of normal and accident conditions with the effects of the natural phenomena including consideration of the most severe of natural phenomena that have been historically reported for a site and appropriate combinations of the effects of normal and accident conditions with the effects of the natural phenomena.

2. 10 CFR 52.47, as it relates to the requirement that a U.S. EPR application contain the proposed ITAAC that are necessary and sufficient to provide reasonable assurance that if the inspections, tests, and analyses are performed and the acceptance criteria met, a plant that incorporates the design certification is built and will operate in accordance with the design certification, the provisions of the Atomic Energy Act of 1954 and NRC regulations.

Acceptance criteria adequate to meet the above requirements include:

- the acceptance criteria specifically listed in NUREG-0800, Section 3.4.2

3.4.2.4 *Technical Evaluation*

In this section of the report, the staff reviews the procedures used in the design of Seismic Category I structures to withstand the effects of external flooding and groundwater. The effects of flooding may include dynamic effects caused by currents and flood waves. This review was performed following the review procedures and acceptance criteria of NUREG-0800, Section 3.4.2.

3.4.2.4.1 Highest Flood and Groundwater Levels

Safety-related structures need to meet GDC 2 which requires they be designed to withstand the effects of natural phenomena such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without losing the capability to perform their safety functions. For the U.S. EPR design, the yard grade elevation is assumed to be .305 m (1 ft) above the elevation of the PMF. As such, wave or current actions are not considered to be necessary considerations for structural design.

The maximum groundwater table is 1 m (3.3 ft) below the finished grade elevation. FSAR Tier 2, Section 3.4.2, “External Flood Protection,” Page 3.4-3 states that flood protection measures taken protect against flooding from the postulated failure of onsite storage tanks. However, it was unclear to the staff that failure of a tank will not cause temporary flooding and wave action on nearby structures. Therefore, in RAI 162, Question 03.04.02-5, the staff requested that the applicant addresses flooding and wave action due to the consequences of a ruptured tank. In a February 27, 2009, response to RAI 162, Question 03.04.02-5, the applicant indicated that the finished floor is at an elevation of zero meters. The finished grade is set at -0.30 m (-1.0 ft) elevation. Large volume fluid storage tanks are located on the site, but are not in close proximity to safety-related structures. FSAR Tier 2, Section 3.4.2, states that the site has a positive grade which causes fluids at the site to flow away from safety-related structures. In addition, the applicant stated that distance, hydraulic energy dissipation, and site grading mitigate effects of a postulated tank rupture and its ability to generate wave action or hydraulic pressure on safety-related structures. Since the tanks are located away from the safety-related structures and at lower elevation, the staff concludes that postulated tank ruptures will not

generate an adverse hydraulic pressure effect on those structures, the staff finds the response acceptable and, therefore, considers RAI 162, Question 03.04.02-5 resolved.

Lateral earth pressure loads including the effects of groundwater and flood loads from the PMF are considered in the design of the foundations and embedded portions of structures as described in FSAR Tier 2, Section 3.8, "Design of Category I Structures." Since the loads and loading combinations in FSAR Tier 2, Section 3.8 appropriately include these effects which the staff finds acceptable. However, in RAI 162, Question 03.04.02-1, the staff requested that the applicant provide the methods used for determining buoyancy effects and loads from hydrostatic pressure as these were not described. Since lateral soil loads including the effect of hydrostatic pressure are included in the calculation of structural stability, the staff requested that the applicant provide the coefficient of friction used in stability determinations.

In a February 27, 2009, response to RAI 162, Question 03.04.02-1, the applicant described the buoyancy effect as being determined from the hydrostatic pressure at the base of a foundation. This pressure is calculated by taking the depth of the PMF at the foundation base and multiplying this value by the unit weight of water. The maximum hydrostatic pressure is conservatively based on a flood level at grade. Hydrostatic pressure is calculated as $h*\gamma$, where h is the height of the column above, and γ is the unit weight of water. Static lateral soil loads are calculated as the product of $(K_o)(h)(\gamma)$, where K_o is the at-rest lateral soil coefficient, and γ is the unit weight of soil accounting for the effect of the water. The applicant stated that lateral loads due to surcharge are calculated according to the Boussinesq method. The stability calculation is carried out for a coefficient of friction (μ) of 0.7 and 0.5. The value 0.7 is the static coefficient of friction between concrete and soil based on a slip plane occurring in the soil and is calculated as tan φ, where $\varphi = 35^\circ$ is the angle of internal friction. To check, the limit of sliding the angle of internal friction is reduced to 27˚, which corresponds to a coefficient of friction of 0.5.

The staff finds the applicant's February 27, 2009, response to RAI 162, Question 03.04.02-1, regarding the calculation of buoyancy effects, hydrostatic pressure, and static lateral soil pressure conforms to standard engineering practice and is acceptable. As the values of the selected soil parameters have an impact on the design and stability of Seismic Category I structures and are a function of the soil conditions found at a specific site, in RAI 222, Question 03.04.02-8, the staff requested that the applicant add the soil parameters assumed in the design for the coefficients of friction to FSAR Tier 2, Table 2.1-1 "U.S. EPR Site Design Envelope." In addition, because the Boussinesq method was developed to calculate the stress distribution in an elastic half space due to a point load, the staff also requested that the applicant comment on the suitability of this method for determining lateral pressures acting on an embedded structural wall.

In a September 11, 2009, response to RAI 222, Question 03.04.02-8, the applicant stated that although lateral earth pressures were initially determined using the Boussinesq method, upon further review, the applicant determined that it was more appropriate to calculate lateral pressures due to surcharge in accordance with the method developed for rigid walls described in the U.S. Army Corps of Engineers Engineer Manual 1110-2-2502, "Engineering and Design: Retaining and Flood Walls." The U.S. Army Corps of Engineers Manual 1110-2-2502 provides methods for developing surcharge loads on rigid walls based on soil mechanics principles and is acceptable to the staff as the applicant proposes here. Regarding the coefficients of static friction under the basemat, FSAR Tier 2, Section 2.5.4.2 identifies the coefficient of friction between concrete and dry soil as 0.7. However, in a February 27, 2009, response to RAI 162, Question 03.04.02-1, the applicant stated that to check the upper limit of sliding and uplift

(including the effects of maximum water table and dynamic versus static coefficient of friction), the angle of internal friction is reduced to 27 degrees, which corresponds to a coefficient of friction of 0.5. Since this is a key parameter used in determining the stability of Seismic Category I structures, in RAI 384, Question 03.04.02-13, the staff requested that the applicant revise FSAR Tier 2, Section 2.5.4.2 to include this value. In a March 30, 2011, response to RAI 384, Question 03.04.02-13, the applicant stated that the FSAR would be revised to identify that a coefficient of friction equal to 0.5 would be applicable to all interfaces between building foundations and the soil. In addition, FSAR Tier 2, Table 1.8-2 COL Information Item 2.0-1 calls for a COL applicant that references the U.S. EPR design to compare the site characteristics with the site parameters in FSAR Tier 2, Table 2.1-1. In FSAR Tier 2, Interim Revision 3, Table 2.1-1, the applicant identified the coefficient of friction at all interfaces between the mat and soil has a value of 0.5. The staff finds the applicant's March 30, 2011, response to RAI 384, Question 03.04.02-13, acceptable since it removes any inconsistency between what is being used in the U.S. EPR design certification and what the COL applicant will use to demonstrate that the U.S. EPR design is suitable for site-specific conditions. The staff finds the applicant's responses to RAI 162, Question 03.04.02-1, RAI 222, Question 03.04.02-8, and RAI 384, Question 03.04.02-13 acceptable and considers RAI 162, Question 03.04.02-1 resolved. **However, RAI 384, Question 03.04.02-13 is being tracked as a confirmatory item** to ensure the FSAR is revised accordingly.

3.4.2.4.2 Protective Measures Implemented for the Effects of Flooding and Groundwater

The FSAR states that no access openings or tunnels penetrate the external walls of the Nuclear Island below grade. This is an acceptable protective measure to the staff, but the FSAR did not address possible access openings or tunnel penetrations in other Seismic Category I structures. Therefore, in RAI 162, Question 03.04.02-4 the staff requested that the applicant provide similar information for other Seismic Category I structures. In an April 30, 2009, response to RAI 162, Question 03.04.02-4, the applicant stated that the Emergency Power Generating Buildings (EPGBs) and Essential Service Water Buildings (ESWBs) which are the other Seismic Category I structures have no access openings or tunnels penetrating exterior walls below grade. The applicant has revised FSAR Tier 2, Section 3.4.2 (Bullet 4) to state that no access openings or tunnels penetrate the exterior walls of the Nuclear Island or any other Seismic Category 1 structure below grade. The staff finds this eliminates a potential path for water ingress, and as such, finds this part of the response acceptable. However, as shown in FSAR Tier 2, Figure 1.2-50, "[[Access Building General Arrangement Drawing Plan View -31 Feet]]," there was a below grade access into the Tendon Gallery from the Access Building. As this interface can create a potential path for water ingress, in RAI 248, Question 03.04.02-12, the staff requested that the applicant address how water is prevented from entering into the Tendon Gallery at the interface between the two structures.

In a November 18, 2009, response to RAI 248, Question 03.04.02-12, the applicant stated that a flexible elastomeric seal will bridge the separation gap between the Access Building and the Tendon Gallery. This seal is intended to allow differential movement between the two structures while providing protection against water ingress. However, there was no combined license information item which calls for the COL applicant to design such a seal to allow differential movement between the Tendon Gallery and the Access Building. Therefore, in RAI 384, Question 03.04.02-14, the staff requested that the applicant add such an item to FSAR Tier 2, Table 1.8-2. In an August 12, 2010, response to RAI 384, Question 03.04.02-14, the applicant stated that COL Information Item 3.4-7 will be added to the FSAR Tier 2, Table 1.8-2. This item states, "a COL applicant that references the U.S. EPR design certification will design the

watertight seal between the Access Building and the adjacent Category I access path to the Reactor Building Tendon Gallery. The watertight seal design will account for hydrostatic loads, lateral earth pressure loads, and other applicable loads." In an August 10, 2010, response to RAI 370, Question 03.07.02-65, the applicant stated that COL Information Item 3.7-7 has been added to FSAR Tier 2, Table 1.8-2, which calls for a COL applicant that references the U.S. EPR design certification to demonstrate that the response of the Access Building to an SSE event will not impair the ability of Seismic Category I systems, structures, or components to perform their design basis safety functions. Interface requirements for the Access Building are included in FSAR Tier 2, Table 1.8-1, "Summary of U.S. EPR Plant Interfaces with Remainder of Plant," under Item 1-2. The FSAR section identified Item 1-2 which is relevant to the design basis for the Access Building is FSAR Tier 2, Section 3.7.2. This FSAR section provides conceptual design information which indicates that the Access Building will be designed with a safety margin that is equivalent to that of a Seismic Category I structure. As this meets the staff's concern regarding the possible interaction of the Access Building with the Reactor Building Tendon Gallery, the staff finds the information provided in the interface requirement of FSAR Tier 2, Table 1.8-1 acceptable.

COL Information Item 3.4-7 has been added to FSAR Tier 2, Table 1.8-2 and designates design responsibility and design criteria for the elastomer seal and COL Information Item 3.7-7 has been added to FSAR Tier 2, Table 1.8-2 which calls for the COL applicant to demonstrate that the Access Building will not impair the safety function of Seismic Category I SSCs. Therefore, the staff finds the applicant's responses to RAI 162, Question 03.04.02-4, RAI 248, Question 03.04.02-12, and RAI 384, Question 03.04.02-14 acceptable and considers RAI 162, Question 03.04.02-4, RAI 248, Question 03.04.02-12, and RAI 384, Question 03.04.02-14 resolved.

FSAR Tier 2, Section 3.4.2 states that portions of Seismic Category I structures below grade elevation are protected from external flooding by waterstops and waterproofing. Since waterproofing measures below grade are not readily repaired and are difficult to maintain, in RAI 162, Question 03.04.02-2, the staff requested that the applicant provide the specified design life for waterstops, water seals, and waterproofing utilized in the foundation designs. In an April 30, 2009, response to RAI 162, Question 03.04.02-2, the applicant stated that protection from external flooding is provided by engineered structures designed to withstand hydrostatic loads associated with the bounding external flooding event. Waterstops and waterproofing are weather resistant sacrificial barriers or coatings added to reduce direct exposure of external structural surfaces to environmental influences and mitigate resultant deterioration of these surfaces that may result from such exposure. Waterstops and waterproofing also reduce moisture seepage that can occur with concrete structures. Waterstops and waterproofing also perform no structural function and are not credited in either design or analysis with preventing or mitigating external flooding. Due to their inaccessibility, waterproofing and waterstops are made of materials with significant inherent durability and will not be replaced or maintained during the design life of the structure.

The staff determined that additional information was needed as it is unclear from the April 30, 2009, response to RAI 162, Question 03.04.02-2, what the functions of the waterproofing and waterstops are in providing protection to Seismic Category I structures from groundwater or flood. If the waterproofing or the waterstops are necessary features that enable these structures to perform their required safety functions, they need to be classified as Seismic Category I materials. Therefore, in RAI 239, Question 03.04.02-10, the staff requested that the applicant provide the Quality Assurance (QA) Category under which the waterproofing or waterstops are procured and, if Seismic Category I, provide the codes and standards that these

materials must meet in carrying out their safety-related function. If not, then the applicant should describe how Seismic Category I structures are otherwise protected below grade from the effects of flood or from the long term effects of groundwater including in-seepage of water and deterioration of the concrete.

In a July 8, 2009, response to RAI 239, Question 03.04.02-10, the applicant stated that U.S. EPR Seismic Category I structures do not depend on the use or effectiveness of waterstops or waterproofing membranes to perform their safety-related design basis functions. Protection of safety-related SSCs from groundwater or flooding effects is provided by engineered Seismic Category I concrete structures. When waterstops or waterproofing membranes are installed, they act as sacrificial barriers to mitigate potential erosion of below grade external structural surfaces that may be caused by unbuffered contact with aggressive groundwater. Waterstops and waterproofing membranes are not installed to protect SSCs against external flooding. Marginal protection against external flooding that may be afforded by waterstops or waterproofing membranes is coincidental to their purpose. Waterproofing membrane and waterstop leakage is expected to occur during the life of the plant. This long-term degradation does not significantly impair or adversely affect the structural integrity of Seismic Category I structures or their safety-related design basis function. Consequently, waterproofing membranes and waterstops that may be installed to mitigate erosion of external below grade structural surfaces due to exposure to aggressive groundwater are not safety related and are not classified as Seismic Category I materials. Seismic Category I structural concrete is designed, mixed, placed, and inspected in accordance with codes and standards listed in FSAR Tier 2, Sections 3.8.1.2.1, “Codes”; 3.8.3.2.1, “Codes and Standards”; and 3.8.4.2.1, “Codes and Standards.” Adherence to the specifications of these codes and standards produces concrete that is resistant to deterioration and is suitable for use in non-aggressive groundwater without installing waterproofing membranes or waterstops. American Concrete Institute 349-01, “Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary,” Chapter 4, “Durability Requirements,” and American Society of Mechanical Engineers (ASME), Section III, Division 2, Article CC-2231.7, “Durability,” standards include provisions for protection against aggressive soil and groundwater. In aggressive soil and groundwater conditions, various protective measures may be taken in accordance with site-specific conditions, including use of epoxy coated reinforcing steel as described in FSAR Tier 2, Section 3.8.5.6.1, “Materials.”

In a July 8, 2009, response to RAI 239, Question 03.04.02-10, the applicant stated that protection from flood and groundwater does not depend on waterstops, nor does it depend on the use of waterproof membranes. The applicant also stated that using the waterstops and waterproof membranes will mitigate the effects of groundwater or flood, but not assure protection throughout the life of the plant. Rather, protection is provided by proper design of the concrete mix and the use of epoxy coated reinforcing steel. The staff finds the applicant’s July 8, 2009, response to RAI 239, Question 03.04.02-10 an appropriate approach and finds the response acceptable. Therefore, the staff considers RAI 162, Question 03.04.02-2 and follow-up RAI 239, Question 03.04.02-10 resolved.

Although a description of protective measures is provided for Seismic Category I structures, there is no description of protective measures from the effects of the PMF or groundwater for buried structures or utilities. Therefore, in RAI 162, Question 03.04.02-3, the staff requested that the applicant provide protective measures for safety-related underground pipe, conduit, and duct banks that are in-scope of the U.S. EPR design. In a February 27, 2009, response to RAI 162, Question 03.04.02-3, the applicant stated that a list of Seismic Category I SSCs for the U.S. EPR, including buried items, is provided in FSAR Tier 2, Table 3.2.2-1, “Classification

Summary." Protection measures/criteria are listed in FSAR Tier 2, Section 3.4.2. After reviewing this information, the staff concluded that the referenced section of the FSAR did not provide the information requested. Therefore, in follow-up RAI 222, Question 03.04.02-9, the staff requested that the applicant clarify each of these items individually and describe how they are protected. In a September 11, 2009, response to follow-up RAI 222, Question 03.04.02-9, the applicant stated that protection against the effects of groundwater and PMF for Seismic Category I embedded or buried structures is provided by concrete that meets the ACI Code criteria for durability and quality. Adherence to applicable codes and standards, as given in FSAR Tier 2, Sections 3.8.1.2.1, 3.8.3.2.1, and 3.8.4.2.1, produces concrete that is adequately resistant to deterioration. FSAR Tier 2, Section 3.8.5.6.1, states that water-proofing membranes and epoxy-coated reinforcing steel may also be used on a site-specific basis to counteract long-term effects of aggressive environments. The applicant notes that the protection of buried pipes, conduits, and duct banks is the responsibility of the COL applicant as provided in FSAR Tier 2, Table 1.8-2, COL Information Items 3.8-4, 3.8-5, 3.8-7, 3.8-14, 3.8-15, and 3.8-16. Insofar as the protection against the effects of groundwater and PMF for Seismic Category I embedded or buried structures will meet ACI Code criteria for durability and quality, the staff finds this portion of the applicant's response acceptable. In addition, the applicant has identified the responsibility for the design and analysis for buried pipes, conduits, and duct banks as belonging to the COL applicant as identified in the COL information items in FSAR Tier 2, Table 1.8-2. Therefore, it will be the responsibility of the COL applicant to specify protective measures against the long term effects of groundwater and the PMF. Since the design responsibility for buried pipes, conduits and duct banks has been clearly assigned to the COL applicant, the staff also finds this portion of the response acceptable. The staff finds the applicant's September 11, 2009, response to follow-up RAI 222, Question 03.04.02-9 acceptable. Therefore, the staff considers RAI 162, Question 03.04.02-3 and follow up RAI 222, Question 03.04.02-9 resolved.

The FSAR states that maximum rainfall rate for roof designs is 0.49 m (19.4 in.) per hour, and the maximum static roof load because of snow and ice is 4.8 kPa (100 lb per square foot). These agree with the values in FSAR Tier 2, Table 2.1-1 and in FSAR Tier 2, Section 2.4 "Hydrologic Engineering." FSAR Tier 2, Section 2.4 states that the hydrologic information in FSAR Tier 2, Section 2.4 is site-specific and will be provided by the COL applicant that references the U.S. EPR design certification. This action is COL Information Item 2.4-2 in FSAR Tier 2, Table 1.8-2. The staff finds the information provided in the FSAR for rainfall rates, and snow and ice loading provide reasonable design parameters to be verified for a specific site by a COL applicant. Accordingly, the staff finds the information provided acceptable.

The staff notes that the title of FSAR Tier 2, Section 3.4.2, "External Flood Protection," is not consistent with the title of SRP Section 3.4.2, "Analysis Procedures." Also, FSAR Tier 2, Section 3.4.3, "Analysis of Flooding Events," does not correspond to an SRP section. Those portions of FSAR Tier 2, Section 3.4.3 which relate to internal flooding should be included in FSAR Tier 2, Section 3.4.1, "Internal Flood Protection" and address the acceptance criteria of SRP Section 3.4.1. Those portions of FSAR Tier 2, Section 3.4.3 that relate to external flooding should be included in FSAR Tier 2, Section 3.4.2 and address the acceptance criteria of SRP Section 3.4.2. In addition, COL Information Items 3.4-1 to 3.4-3 given in FSAR Tier 2, Table 1.8-2, should reference FSAR Tier 2, Section 3.4.2. Therefore, in RAI 162, Question 03.04.02-6, the staff requested that the applicant add these changes into the FSAR.

In an April 22, 2009, response to RAI 162, Question 03.04.02-6, the applicant stated that FSAR Tier 2, Section 3.4, conforms to RG 1.206, "Combined License Applications for Nuclear Power Plants." FSAR Tier 2, Section 3.4.1 addresses internal flooding, and FSAR Tier 2,

Section 3.4.2 addresses external flooding. FSAR Tier 2, Section 3.4.3 provides analyses of the internal and external flooding events, and FSAR Tier 2, Section 3.4.4 addresses respective analysis procedures. The applicant concluded that FSAR Tier 2, Section 3.4 contents satisfy technical requirements set forth by regulatory guidance. The applicant also stated that reformatting FSAR Tier 2, Section 3.4 to precisely conform to RG 1.206 as requested by the staff would result in significant changes throughout both the FSAR and COL applications that reference the U.S. EPR design and that the reformatting would produce no significant health or safety benefit. After reviewing the areas covered by SRP Section 3.4.1, "Internal Flood Protection for Onsite Equipment Failures," and SRP Section 3.4.2, "Analysis Procedures," the staff concluded these areas have been covered in FSAR Tier 2, Sections 3.4.1 through 3.4.4. The staff agrees that reformatting would produce no significant health or safety benefit. The staff finds the applicant's response to RAI 162, Question 03.04.02-6 acceptable and consider RAI 162, Question 03.04.02-6 resolved.

In the FSAR it was not apparent to the staff that the depth of embedment defined by the distance between the finished grade and the bottom of the foundation mat is explicitly defined for each of the structures. This dimension is important as it could impact earth pressure loads, as well as a structure's seismic analysis if a different embedment depth is used by the COL applicant. Therefore, in RAI 211, Question 03.04.02-7, the staff requested that the applicant provide this information in a table or figure in the FSAR for each in-scope U.S. EPR structure and include a COL item in the FSAR for the applicant to verify the depths of embedment for U.S. EPR structures (i.e., the foundation depth from finished grade to the bottom of each foundation mat), so that it meets the design basis of the U.S. EPR standard design.

In a July 13, 2009, response to RAI 211, Question 03.04.02-7, the applicant stated that elevation 0 (zero) is the elevation of the threshold of the lowest personnel access penetration through which flood water might gain access. "Plant grade" is the finished top of the supporting soil. Elevation 0 is approximately 1 foot above plant grade where the soil meets the structure. FSAR Tier 2, Section 3.8.5.1.2 states that the EPGB is embedded approximately 5 feet into the supporting soil, or that the bottom of its foundation is 5 feet below plant grade. FSAR Tier 2, Section 3.8.5.1.3 states that the ESWB is embedded approximately 22 feet into the supporting soil. FSAR Tier 2, Section 3.8.5.1.1 will be revised to clarify that the Nuclear Island common basemat structure is "embedded approximately 40 ft into the supporting soil." There is approximately 40 feet of soil between the bottom of the basemat and plant grade. A COL applicant who references the certified U.S. EPR design must construct U.S. EPR structures in accordance with U.S. EPR FSAR design specifications or take departures in accordance with the procedures set forth in NRC regulations. Thus, it is unnecessary to provide a new COL item for verifying embedment depths of U.S. EPR structures. The applicant has revised FSAR Tier 2, Section 3.8.5.1.1, Revision 2 to identify the embedment depth of the NI basemat foundation. Furthermore, the depth of embedment for other seismic Category I structures has been identified in the FSAR Tier 2, Section 3.8.5, "Foundations." The applicant has also added verification of embedment depth to FSAR Tier 1, Section 2.1 ITAAC. The staff finds the action by the applicant acceptable and considers RAI 211, Question 03.04.02-7 resolved.

3.4.2.4.3 Tier 1 Information

FSAR Tier 1 requires that Seismic Category I structures be constructed to withstand design basis loads without loss of structural integrity and safety-related functions. The design basis loads are those loads associated with normal plant operation (hydrostatic loads), as well as external events (flood). As such, the staff considers the FSAR Tier 1 information for this area of review acceptable and conforms to the acceptance criteria described above. In addition the

depth of embedment for Seismic Category I structures has been included as a key dimension, requiring ITAAC verification.

ITAAC items associated with this area of review are evaluated in the review of FSAR Tier 2, Section 14.3.2 for Structures

3.4.2.5 *Combined License Information Items*

Table 3.4.2-1 provides a list of water level (flood) design related COL information item numbers and descriptions from FSAR Tier 2, Table 1.8-2:

Table 3.4.2-1 U.S. EPR Combined License Information Items

Item No.	Description	FSAR Tier 2 Section
3.4-7	A COL applicant that references the U.S. EPR design certification will design the watertight seal between the Access Building and the adjacent Category I access path to the Reactor Building Tendon Gallery. Watertight seal design will account for hydrostatic loads, lateral earth pressure loads, and other applicable loads.	3.4.2

The staff finds the above listing to be complete. Also, the list adequately describes actions necessary for the COL applicant. No additional COL information items need to be included in FSAR Tier 2, Table 1.8-2 for water level (flood) design.

3.4.2.6 *Conclusions*

Based on the foregoing, the staff concludes that sufficient information has been provided by the applicant with respect to analysis procedures for external flood protection and their application to the structures of the U.S. EPR design. Based on its review of the information provided in FSAR Tier 2, Section 3.4.2, as set forth above, the staff concludes that, in general, the requirements of 10 CFR Part 50, Appendix A, “General Design Criteria for Nuclear Power Plants,” GDC 2, 10 CFR 52.47, and the specific acceptance criteria of SRP Section 3.4.2 for the consideration of the effects of flood and groundwater on the design of safety-related structures have been met.

As described above, the design of the U.S. EPR meets the specific acceptance criteria in NUREG-0800, Section 3.4.2 as it relates to the consideration of flood effects and hydrostatic loads on structures. The staff finds that the applicant also meets the requirements of GDC 2 with respect to the ability of the safety-related structures of the certified design to withstand the effects of the flood and groundwater through consideration of the following:

1. Appropriate consideration for the effects of flood and groundwater

2. Appropriate combination of the effects of normal and accident conditions with the hydrostatic and buoyancy effects of flood and groundwater

3. The importance of the safety functions to be performed

3.5 Missile Protection

3.5.1 Missile Selection and Descriptiont

3.5.2 Structures, Systems, and Components to be Protected from Externally Generated Missiles

3.5.3 Barrier Design Procedures

3.5.3.1 *Introduction*

Safety-related structures must be designed to satisfy the requirements of GDC 2 and GDC 4, “Environmental and Dynamic Effects Design Bases,” to provide protection against the effects of externally and internally generated missiles, as well as from whipping pipes and impulsive loads from discharging fluids. This section documents findings from the staff’s review and evaluation of the U.S. EPR procedures used in design of missile barriers to withstand the local and overall effects of missile impact and impulsive loads.

3.5.3.2 *Summary of Application*

FSAR Tier 1: There is no FSAR Tier 1 information related to missile barrier design procedures. Related FSAR Tier 1 information associated with missile protection barriers is found in FSAR Tier 1, Sections 2.1.1, “Nuclear Island”; 2.1.2, “Emergency Power Generating Buildings”; and 2.1.5, “Essential Service Water Building.”

FSAR Tier 2: In FSAR Tier 2, Section 3.5.3, “Barrier Design Procedures,” the applicant has provided a description of missile barrier design summarized here, in part, as follows:

Seismic Category I structures are designed to provide protection from external missiles caused by natural phenomena, and internal missiles caused by equipment and piping failures. In this design, the barrier must have sufficient strength to prevent perforation by the missile and be able to withstand, in combination with other design loads for the structure, the overall effects of the missile impact. Secondary effects from the impact such as back-face scabbing and front-face spalling must also be considered, as well as ductility limits on the overall response of the barrier when an elasto-plastic analysis has been used.

ITAAC: There are no inspections, tests, analyses, and acceptance criteria related to missile barrier design procedures. Related ITAAC associated with missile protection barriers are found in FSAR Tier 1, Tables 2.1.1-4, “Nuclear Island ITAAC”; 2.1.1-8, “Reactor Building ITAAC”; 2.1.1-10, “Safeguards Building ITAAC”; 2.1.1-11, “Fuel Building ITAAC”; 2.1.2-3, “Emergency Power Generating Building ITAAC”; and 2.1.5-3, “Essential Service Water Building ITAAC,” Regulatory Basis.

3.5.3.3 *Regulatory Basis*

The relevant requirements of NRC regulations associated with this area of review, and the associated acceptance criteria, are given in NUREG-0800, Section 3.5.3, “Barrier Design Procedures,” and are summarized below. Review interfaces with other SRP sections can also be found in NUREG-0800, Section 3.5.3.

1. GDC 2, as it relates to the structures, systems, and components important to safety being designed to withstand the effects of natural phenomena such as earthquakes, tornados, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety functions. The design bases for these SSCs shall reflect appropriate combinations of the effects of normal and accident conditions with the effects of the natural phenomena including consideration of the most severe of natural phenomena that have been historically reported for a site and appropriate combinations of the effects of normal and accident conditions with the effects of natural phenomena.

2. GDC 4, as it relates to the protection of SSCs that are important to safety against dynamic effects of missiles, pipe whip, and discharging fluids that may result from equipment failures and from events and conditions outside the nuclear power unit.

Acceptance criteria adequate to meet the above requirements include:

1. RG 1.76, "Design-Basis Tornado and Tornado Missiles for Nuclear Power Plants," as it relates to the minimum acceptable barrier thickness for tornado missiles.

2. RG 1.142, "Safety-Related Concrete Structures for Nuclear Power Plants (Other than Reactor Vessels and Containments)," as it relates to the design of safety-related concrete structures for nuclear power plants.

3. RG 1.136, "Design Limits, Loading Combinations, Materials, Construction, and Testing of Concrete Containments," as it relates to the design limits, loading combinations, materials, construction, and testing of concrete containments.

3.5.3.4 *Technical Evaluation*

In this section of the report, the staff discusses the procedures used in design of seismic Category I structures, shields, and barriers to withstand the effects of missile impact. For the prediction of local damage from missiles, the applicant provided information on the procedures used in the design of concrete, steel, and composite structures. For overall damage assessment, the applicant provided information on the elasto-plastic analysis methods that are utilized and structural ductility considerations. In FSAR Tier 2, Section 3.8, "Design of Category I Structures," the applicant discusses loads and loading combinations for Seismic Category I structures which include pipe whip and jet impingement loads. FSAR Tier 2, Section 3.8 states that the design for these loads conforms to the procedures of FSAR Tier 2, Section 3.5, "Missile Protection."

3.5.3.4.1 Local Damage Prediction

Reviews of this portion of the application were performed in accordance with NUREG-0800, Section 3.5.3 and RG 1.76.

<u>Concrete Barriers</u>

Information provided in FSAR Tier 2, Section 3.5.3.1.1, "Concrete Barrier Analysis," presents several empirical equations including the modified National Defense Research Committee (NDRC) formula for determining missile protection afforded by concrete barriers. These formulas are presented in, "A Review of Procedures for the Analysis and Design of Concrete Structures to Resist Missile Impact Effects," by R.P. Kennedy, which is referenced in the acceptance criteria of SRP Section 3.5.3. The staff finds the use of these formulas, which

include methods for calculating missile penetration, perforation, and scabbing acceptable for missile barrier design. One of the factors used in the calculation of penetration or perforation of a barrier is the missile shape factor. A number of typical missile shape factors are presented in this FSAR section. However, how the shape factors are determined for the specific missiles considered in the design was not addressed. Therefore, in RAI 162, Question 03.05.03-2, the staff requested that the applicant address this issue. In an April 30, 2009, response to RAI 162, Question 03.05.03-2, the applicant stated that the U.S. EPR missile penetration depth calculations assume a 0.15 m (6 in.) schedule 40 pipes as the controlling case for barrier penetration depth, as described by RG 1.76. The calculation checks this missile for two cases: One using a shape factor of 0.72 (flat nose missile) and the other using a shape factor of 1.0 (average bullet nosed) which conservatively assumes the end of the pipe is deformed. The staff's assessment of this response is that the use of the 0.15 m (6 in.) schedule 40 pipes is appropriate for calculating barrier penetration for the tornado missile. It is not clear why a barrier penetration calculation using the shape factor of 0.72 is used, because the shape factor of 1.0 will always result in a larger value when calculating the missile penetration or perforation. Since the applicant states that both cases are checked, the staff finds this acceptable. In addition to external missiles, the applicant assesses internal missiles but did not specify a shape factor to determine the effectiveness of a barrier in providing missile protection. Therefore, in RAI 239, Question 03.04.02-11, the staff requested that the applicant provide the missiles and the basis of their shape factors used in the design of internal missile barriers. In a July 8, 2009, response to RAI 239, Question 03.04.02-11, the applicant stated that the U.S. EPR missile barrier design employs the same methodology for both internally and externally generated missiles. Internally generated missiles from equipment like pressurized components, high-energy piping, and rotating equipment that possess a statistically significant probability of damaging a safety-related target are evaluated. The methodology for determining the shape factor for use in missile barrier design calculations considers the particular internal missile under consideration and its corresponding geometry (i.e., flat nosed, blunt, sharp nosed, etc.).,. Accordingly, this additional clarification regarding the selection of shape factors for use in the design of internal missile barriers is acceptable to the staff. Therefore, the staff considers RAI 162, Question 03.05.03-2 and RAI 239, Question 03.04.02-11 resolved.

The last paragraph of FSAR Tier 2, Section 3.5.3.1.1 states that FSAR Tier 2, Table 3.5-2, "Minimum Concrete Barrier Thickness Requirements for Local Damage Prediction Against Tornado Generated Missiles," shows minimum concrete barrier thicknesses relied upon for local damage prediction against tornado generated missiles. These are based on the Region I guidelines in NUREG-0800. The staff finds the values in FSAR Tier 2, Table 3.5-2 acceptable, because they conform to the values in SRP Section 3.5.3, Table 1, "Minimum Acceptable Barrier Thickness Requirements for Local Damage Prediction Against Tornado Generated Missiles," and the SRP acceptance criteria. FSAR Tier 2, Sections 3.5.3.1.1.1, "Penetration"; 3.5.3.1.1.2, "Perforation"; and 3.5.3.1.1.3, "Scabbing," provide equations for calculating missile penetration, perforation, and scabbing which come from the R.P. Kennedy paper mentioned above. SRP Acceptance Criteria 3.5.3.II.1.A states that these equations are acceptable for determining the required thicknesses of missile barriers. Thicknesses resulting from such calculations should not be less than those given in SRP Section 3.5.3, Table 1, which specifies the minimum thicknesses necessary to protect against tornado missiles. Therefore, in RAI 162, Question 03.05.03-1, the staff requested that the applicant confirm that when the equations shown in FSAR Tier 2, Sections 3.5.3.1.1.1 through 3.5.3.1.1.3 are used to compute concrete barrier thicknesses and the computed barrier thicknesses are greater than those of FSAR Tier 2, Table 3.5-2, the larger of the two thicknesses will be used for the U.S. EPR barrier design. In an April 22, 2009, response to RAI 162, Question 03.05.03-1, the applicant stated that calculated thicknesses using the equations in FSAR Tier 2, Sections 3.5.3.1.1.1

through 3.5.3.1.1.3 will be compared to those thicknesses set forth in FSAR Tier 2, Table 3.5-2 with the larger thickness used in the design of the barrier. FSAR Tier 2, Section 3.5.3.1.1 will be revised to state: "The thicknesses determined by these equations shall be compared to those in Table 3.5-2 with the larger thickness used in the design of the barrier." As the applicant has stated it will make a change to the FSAR to state that the larger of the barrier thicknesses calculated from the equations in FSAR Tier 2, Section 3.5.3.1.1 or set forth in FSAR Tier 2, Table 3.5-2 will be used in the design, the staff finds the applicant's April 22, 2009, response to RAI 162, Question 03.05.03-1 acceptable. The staff confirmed that Revision 1 of the FSAR, dated May 29, 2009, contains the changes committed to in the RAI response. Accordingly, the staff finds that the applicant has adequately addressed this issue and, therefore, the staff considers RAI 162, Question 3.05.03-1 resolved.

Steel Barriers

FSAR Tier 2, Section 3.5.3.1.2, "Steel Barrier Analysis," states that barrier penetration will be calculated by either the Ballistic Research Laboratory (BRL) formula or by the Stanford Research Institute (SRI) equation. The SRI formula meets the acceptance criteria of the SRP and is acceptable to the staff. If the BRL formula is used, the SRP Acceptance Criteria 1.B states the applicant should verify that the results are comparable to the SRI formula or else be validated by penetration tests. This stipulation was not provided by the applicant and additional clarification was requested by the staff. Therefore, in RAI 162, Question 03.05.03-3, the staff requested that the applicant describe the circumstances under which the BRL formula might be used and, if used, provide comparative calculations using the SRI equation and confirm that the larger of the two thicknesses thus calculated will be used in the design. In an April 22, 2009, response to RAI 162, Question 03.05.03-3, the applicant stated that if the BRL equation is used to calculate steel penetration thicknesses, the result will be compared to that thickness obtained using the SRI equation with the larger thickness used in the design of the barrier. For clarification, the last paragraph of FSAR Tier 2, Section 3.5.3.1.2 has been revised to state: "The perforation thicknesses (e), obtained from the BRL formula and the SRI equation, shall be compared to each other with the larger value selected for use in the barrier design. This design value shall be increased by 25 percent to prevent perforation." Since the greater thickness from either the SRI or BRL calculation will be used and subsequently increased by 25 percent, the staff finds the applicant's April 22, 2009, response to RAI 162, Question 03.05.03-3 acceptable. The staff confirmed that Revision 1 of the U.S. EPR FSAR, dated May 29, 2009, contains the changes committed to in the RAI response. Accordingly, the staff finds that the applicant has adequately addressed this issue and, therefore, the staff considers RAI 162, Question 03.05.03-3 resolved.

Composite Sections

FSAR Tier 2, Section 3.5.1.3, "Composite Section Barrier Analysis," states that composite sections will be evaluated for local damage from missile impact by assuming the residual velocity of the missile perforation of the first element in the composite section is the striking velocity for the impact to the next element in the composite section. The striking velocity for the next element will be calculated using the following reference: R. F. Recht and T. W. Ipson, "Ballistic Perforation Dynamics," ASME Journal of Applied Mechanics, Volume 30, Series E, Number 3, September 1963, which the staff finds acceptable. However, in situations where the first barrier in a composite section is concrete, the SRP indicates this type of design should be reviewed on a case-by-case basis. Therefore, in RAI 162, Question 03.05.03-4, the staff requested that the applicant specify how and where composite barrier designs will be utilized and provide the details of its use to the staff. In an April 22, 2009, response to RAI 162,

Question 03.05.03-4, the applicant stated, “no areas in the U.S. EPR design have been identified where the use of composite barrier design will be required. Should the need to use composite barrier design be identified later in the design process, the composite barrier will consist of an outer steel layer and designed following the guidance provided in Reference 6 of SRP Section 3.5.3.” Since the use of composite sections conforms to the guidance in SRP Section 3.5.3, Reference 6 summarized above, the staff finds the applicant’s April 22, 2009, response to RAI 162, Question 03.05.03-4 acceptable. Accordingly, the staff finds that the applicant has adequately addressed this issue and, therefore, considers RAI 162, Question 03.05.03-4 resolved.

3.5.3.4.2 Overall Damage Prediction

Overall damage prediction as described in FSAR Tier 2, Section 3.5.3.2 is to be carried out using SRP Acceptance Criteria 3.5.3.II.2 and RG 1.142. As set forth in RG 1.142 the use of ACI 349-97, “Code Requirements for Nuclear Safety Related Concrete Structures,” is generally acceptable for the design of concrete structures. The exceptions to the ACI 349-97 Code relative to impact or impulsive loads are provided in RG 1.142, Regulatory Positions 10 and 11found in Appendix C of the RG. However, the applicant indicates that concrete missile barriers are designed in accordance with ACI 349-2001 and also in compliance with the Regulatory positions of the RG. Since the code referenced in RG 1.142 is ACI 349-97, the applicant needed to identify any differences between the 2001 and 1997 versions of ACI 349 Code and demonstrate the acceptability of using the ACI-349-01 Code. This issue was addressed by the staff in its review of FSAR Tier 2, Section 3.8.3 under RAI 155, Question 03.08.03-3 and RAI 376, Question 03.08.03-21. The resolution of these RAIs was that ACI 349-01 was acceptable for design of Seismic Category I concrete structures (other than the Reactor Containment Building which is designed using ASME Code, Section III, Division 2 Code) with the only exception being anchorage designs which will follow ACI 349-06, Appendix D. Based on the successful resolution of RAI 155, Question 03.08.03-3, and RAI 376, Question 03.08.03-21 as discussed in Section 3.8.3 of this report, the staff finds the use of the later ACI Code edition together with RG 1.142, Regulatory Positions 10 and 11 an acceptable basis for impulsive and impact load design.

In FSAR Tier 2, Section 3.5.3, the applicant states that steel barriers are designed for impact loads according to American National Standards Institute/American Institute of Steel (ANSI/AISC) N690-1994 (R2004) S2, “Specification for Structural Steel Buildings.” Use of this code meets the SRP acceptance criteria for determining the maximum allowable ductility ratios for steel barriers and is acceptable for this use. However, the applicant also states that ASCE 58, “Structural Analysis and Design of Nuclear Plant Facilities,” is used for the evaluation of steel barriers. The staff determined the applicant needed to provide additional information on the methods used for the evaluation of steel barriers from ASCE 58 and how these methods meet Acceptance Criteria 2 of SRP Section 3.5.3. Therefore, in RAI 162, Question 03.05.03-6, the staff requested that the applicant describe its use of ASCE 58 and clarify how that document satisfies the acceptance criteria specified in SRP Section 3.5.3. In an April 30, 2009, response to RAI 162, Question 03.05.03-6, the applicant stated that the evaluation of steel missile barriers will be in accordance with the methodology described in FSAR Tier 2, Section 3.5.3.1.2 and that FSAR Tier 2, Section 3.5.3.2, Paragraph 4 will be revised to remove reference to ASCE 58 and state, “Steel missile barriers will be evaluated using the equations as defined in Section 3.5.3.1.2.” Since the method and equations of FSAR Tier 2, Section 3.5.3.1.2 will be used to evaluate steel missile barriers and these meet the acceptance criteria of SRP Section 3.5.3, the staff finds the response acceptable. The staff has confirmed that these

changes have been made to the text of the FSAR. Therefore, the staff considers RAI 162, Question 03.05.03-6 resolved.

3.5.3.4.3 Ductility Requirements for Missile Barriers

FSAR Tier 2, Section 3.5.3 lists the allowable ductility ratios for safety-related concrete and steel structures. For structures other than the Reactor Containment Building these are in agreement with ACI 349-01, and RG 1.142, including the exceptions noted in Regulatory Positions 10 and 11, and with ANSI/AISC N690/1994. Based on the successful resolution of the question regarding the applicable ACI Code edition described above, the staff finds these ductility ratios acceptable.

In FSAR Tier 2, Section 3.5.3.3, the applicant states that the Reactor Containment Building (RCB) is designed according to the requirements of ASME Boiler and Pressure Vessel (B&PV) Code, Section III, Division 2. However, there was no discussion of ductility factors for the RCB in this section of the FSAR. Therefore, in RAI 162, Question 03.05.03-8, the staff requested that the applicant describe the method used for determining the ductility ratios for the RCB and to specify the limits for these ductility ratios as was done for barriers designed to the ACI 349 Code and the ANSI/AISC N690 Code. In an April 30, 2009, response to RAI 162, Question 03.05.03-8, the applicant stated that for the Reactor Containment Building, the ductility limits used in the design for impulse loads are not to exceed one-third the ductility at failure, and the ductility limits for impact loads are not to exceed two-thirds the ductility determined at failure. The applicant has revised the FSAR to state that the design the Reactor Containment Building. for impact and impulse loads will be based on the ASME B&PV Code, Section III, Division 2. "Code for Concrete Containments." The staff finds the applicant's response acceptable and, therefore, considers RAI 162, Question 03.05.03-8 resolved.

FSAR Tier 2, Section 3.5.2, "Structures, Systems, and Components to be Protected from Externally Generated Missiles," states that safety-related pipes and cables routed outside of missile protected structures are buried at sufficient depth to provide protection for these items from missile impact, or concrete or steel enclosures are provided that are designed to withstand missile impact. No criteria are provided to define how the sufficient depth of burial is determined. FSAR Tier 2, Section 3.8.4.1.8, "Buried Conduit and Duct Banks," states that the design of buried conduit and duct banks is site-specific. FSAR Tier 2, Section 3.8.4.1.8 also states that the duct bank depth and encasement methods will also consider effects from external hazards (e.g., tornado missile). SRP Section 3.5.3, Acceptance Criteria Requirements states that the design of structures, shields, and barriers must meet GDC 2 which requires that structures, systems, and components important to safety shall be designed to withstand the effects of natural phenomena such as tornados, and GDC 4 which requires that SSCs important to safety shall be appropriately protected against the dynamic effects of missiles. Therefore, in RAI 162, Question 03.05.03-9, the staff requested that the applicant to provide for buried safety-related SSCs the criteria and basis of protection including minimum depth of burial for protection from missile impact. In a February 27, 2009, response to RAI 162, Question 03.05.03-9, the applicant replied that a COL information item exists for the analysis and design of duct bank, buried conduits, buried pipes and pipe ducts (Refer to FSAR Tier 2, Table 1.8-2, COL Information Item 3.8-14). This COL Information Item states, "A COL applicant that references the U.S. EPR design certification will describe the design and analysis procedures for buried conduit and duct banks, and buried pipe and pipe ducts." Since there is a COL information item stating the design of buried utilities is the responsibility of the COL applicant, and FSAR Tier 2, Section 3.5.2 states that safety-related pipes and cables routed outside of missile protected structures are buried at sufficient depth to provide protection for

these items from missile impact, or concrete or steel enclosures must be provided to withstand missile impact, the staff finds the applicant's January 27, 2009, response to RAI 162, Question 03.05.03-9 acceptable. Therefore, the staff considers RAI 162, Question 03.05.03-9 resolved.

FSAR Tier 2, Section 3.5.2 states that missile protection for openings and penetrations in structures is provided by enclosures, missile-resistant doors and covers, labyrinth structures, or physical protection features. For U.S. EPR Seismic Category I structures, In RAI 211, Question 03.05.03-10, the staff requested that the applicant provide specific examples of each of these protective features, including the method of analysis used for the missile barrier and the external missile for which the barrier is being designed. In a July 13, 2009, response to RAI 211, Question 03.05.03-10, the applicant stated, in part, that FSAR Tier 2, Section 3.5.3 discusses design procedures for concrete and steel missile barriers. Specific examples of U.S. EPR Seismic Category I structure missile barriers are:

- Physical Protection Features: Four trains of safeguards equipment are located in separate buildings, which protect their design basis safety functions from postulated missiles.

- Missile Resistant Doors: See FSAR Tier 2, Appendix 3B, Figure 3B-43, "Safeguard Buildings 2 and 3 Dimensional Plan Elevation 0 m (0 ft)." The opening on the west side of the building along grid line 15, between grid lines V and W is protected from postulated missiles by a concrete door-barrier.

- Missile Resistant Covers: See FSAR Tier 2, Appendix 3B, Figure 3B-76, "Essential Service Water Building Dimensional Section B-B." Grid Line 3, Elevation 24.99 m (81.99 ft), shows that the cooling tower fans are protected from postulated missiles by a concrete cover or slab.

- Labyrinth structures: See FSAR Tier 2, Appendix 3B, Figure 3B-43. The opening on the west side of the building along grid line 15, between grid lines X and Y is protected from postulated missiles by a concrete labyrinth structure.

- Enclosures: The Reactor Containment Building, Safeguard Buildings 2 and 3, and the Fuel Building are protected from postulated missiles by hardened concrete enclosures designed to withstand beyond design basis aircraft hazards.

The applicant has identified the protective measures used in the U.S. EPR design for missile impact and has provide specific examples of each. The staff has determined that these protective measures when used with the design methods of FSAR Tier 2 ,Section 3.5.3 provide adequate missile protection for Seismic Category I structures. Therefore, the staff finds the response acceptable and considers RAI 211, Question 03.05.03-10 resolved.

3.5.3.5 *Combined License Information Items*

The staff determined that no COL information items need to be included in FSAR Tier 2, Table 1.8-2, "U.S EPR Combined License Information Items," for barrier design procedures consideration.

3.5.3.6 *Conclusions*

In view of the foregoing, the staff concludes that sufficient information has been provided by the applicant with respect to the design of missile barriers and their capacity to protect the SSCs of the U.S. EPR design from both internal and external missiles and from other types of impact loads.

As discussed above, the staff finds that, in general, the applicant used acceptable methods in barrier design. The staff also finds that the barrier design methods meet the SRP Section 3.5.3 Acceptance Criteria guidelines with respect to the capabilities of the structures, shields, and barriers to provide sufficient protection to the safety-related SSCs. The use of these methods provides reasonable assurance that if a design-basis missile should strike a Seismic Category I structure or other missile shields and barriers, the structures, shields, and barriers will not be impaired or degraded to an extent that will result in a loss of required protection and that structures, missile shields or barriers designed with these methods, meet the requirements of 10 CFR Part 50, Appendix A, “General Design Criteria for Nuclear Power Plants,” GDC 2 and GDC 4. The use of these methods provides reasonable assurance that in the event of a design basis missile or other impact load striking Seismic Category I structures, missile shields or other barriers, the structural integrity of these structures, shields, and barriers will not be impaired or degraded to an extent that will result in a loss of required protection. Seismic Category I SSCs are, therefore, adequately protected against the effects of missiles and other impact loads and will perform their intended safety functions. The staff finds that conformance with these methods is an acceptable basis for satisfying the requirements of GDC 2 and GDC 4 as they relate to providing missile and impact protection for safety-related structures systems and components.

3.6 Protection Against Dynamic Effects Associated with Postulated Piping Ruptures

This section describes the design bases and measures needed to protect essential systems against the effects of postulated rupture of piping located in either inside or outside of containment. FSAR Tier 2, Section 3.6.1, “Plant Design for Protection Against Postulated Piping Failures in Fluid Systems Outside of Containment,” provides the design bases and criteria required to demonstrate that SSCs are protected against piping failures outside containment; FSAR Tier 2, Section 3.6.2, “Determination of Rupture Locations and Dynamic Effects Associated with the Postulated Rupture of Piping,” describes the criteria for determining the location and configuration of postulated breaks and cracks; and FSAR Tier 2, Section 3.6.3, “Leak-Before-Break Evaluation Procedures,” describes the application of LBB criteria used to eliminate from the design basis the dynamic effects of certain pipe ruptures.

3.6.1 Plant Design for Protection Against Postulated Piping Failures in Fluid Systems Outside of Containment

3.6.1.1 *Introduction*

This section evaluates the U.S. EPR design bases and criteria relied upon to demonstrate essential systems and components are protected against postulated piping failures outside of containment. High- and moderate-energy systems representing potential sources of dynamic effects associated with pipe rupture are identified, and the criteria for separation and the evaluation of adverse consequences are defined.

3.6.1.2 *Summary of Application*

The FSAR describes design bases and criteria relied upon for protection against postulated piping failures in fluid systems outside of containment as follows:

FSAR Tier 1: There is no FSAR Tier 1 information directly related to protection against postulated piping failures in fluid systems outside of containment. However, FSAR Tier 1, Section 2.1.1, “Nuclear Island”; FSAR Tier 1, Section 2.1.2, “Emergency Power Generating Buildings”; and FSAR Tier 1, Section 2.1.5, “Essential Service Water Building,” describe the design of the Nuclear Island structures, Emergency Power Generating Buildings (EPGBs), and Essential Service Water Buildings, respectively. Also, in response to the staff RAIs, the applicant created FSAR Tier 1 Section 3.8, “Pipe Break Hazards,” to address the completion of the pipe break hazards analyses reports.

FSAR Tier 2: FSAR Tier 2, Section 3.6.1, describes the methodology used in designing the protection of essential systems and components from the consequences of postulated piping failures outside containment. The methodology includes: (1) The criteria for identification of the system and components that are located near high- or moderate-energy pipe systems that need to be protected; (2) identification of the failures for which protection is being provided and assumptions used; and (3) identification of protection considerations in the design. Separation and redundancy of essential systems, methods of analyzing piping failures, and habitability of the main control room are also addressed.

FSAR Tier 2, Section 3.6.1, also identifies that a COL applicant that references the U.S. EPR design certification will complete the pipe break hazards analysis and reconciles deviations in the as-built configuration to what is described in the design certification application.

The design bases (including criteria and assumptions), protective considerations, and failure modes and effects analyses performed for protection of essential systems and components against postulated pipe ruptures outside of containment are as given in FSAR Tier 2, Sections 3.6.1.1, “Design Basis,” through 3.6.1.3, “Failure Mode and Effects Analysis.”

FSAR Tier 2, Table 3.6.1-1, “High-Energy and Moderate-Energy Fluid Systems Considered for Protection of Essential Systems,” lists the fluid systems that contain high- or moderate-energy lines. FSAR Tier 2, Table 3.6.1-2, “Building, Room, and Postulated Pipe Ruptures,” lists the terminal end breaks for the high-energy systems by building and room.

ITAAC: FSAR Tier 1, Table 2.1.1-4, “Nuclear Island ITAAC,” FSAR Tier 1, Table 2.1.2-3, “Emergency Power Generating Buildings,” and FSAR Tier 1, Table 2.1.5-3, “Essential Service Water Building Inspections, Tests, Analyses, and Acceptance Criteria,” provide ITAAC for the Nuclear Island structures, EPGBs, and Essential Service Water Buildings, respectively. FSAR Tier 2, Section 14.2, “Initial Plant Test Program,” and FSAR Tier 2, Section 14.3, “Inspection, Test, Analysis, and Acceptance Criteria,” do not provide any initial testing or ITAAC associated with this review item. However, in response to the staff’s RAI, the applicant created FSAR Tier 1, Section 3.8, to address completion of the pipe break hazards analysis report. FSAR Tier 2, Section 14.3 was revised to reflect the added FSAR Tier 1, Section 3.8.

3.6.1.3 *Regulatory Basis*

The relevant requirements of NRC regulations for this area, and the associated Acceptance Criteria, are given in NUREG-0800, Section 3.6.1, “Plant Design for Protection Against

Postulated Piping Failures in Fluid Systems Outside Containment," and are summarized below. NUREG-0800, Section 3.6.1 also identifies review interfaces with other SRP sections.

1. 10 CFR Part 50, Appendix A, GDC 2, "Design Bases for Protection Against Natural Phenomena," as it relates to the protection of SSCs important to safety to withstand the effects of natural phenomena, such as earthquakes, The application of GDC 2 to this section is to incorporate environmental effects of full-circumferential ruptures of non-seismic moderate-energy piping in areas where effects are not already bounded by failures of high-energy piping.

2. GDC 4, "Environmental and Dynamic Effects Design Bases," as it relates to SSCs important to safety being designed to accommodate the effects of and to be compatible with the environmental conditions associated with postulated pipe rupture.

3. 10 CFR 52.47(b)(1), "Contents of applications, technical information," as it relates to the requirement that a U.S. EPR application contain the proposed ITAAC that are necessary and sufficient to provide reasonable assurance that, if the inspections, tests, and analyses are performed and the acceptance criteria met, a plant that incorporates the design certification is built and will operate in accordance with the design certification, the provisions of the Atomic Energy Act of 1954, and NRC regulations.

Acceptance Criteria adequate to meet the above requirements include:

- Branch Technical Position (BTP) 3-3, "Protection Against Postulated Piping Failures in Fluid Systems Outside Containment," as it relates to postulated piping failure analyses.

3.6.1.4 *Technical Evaluation*

In FSAR Tier 2, Section 3.6.1, the applicant describes the methodology used in designing the U.S. EPR to protect essential systems and components from the consequences of postulated piping failures outside containment. The steps include identification of the essential systems and components that are located near high- or moderate-energy piping systems, identification of the failures for which protection is being provided, and identification of protection considerations that are utilized in the design to safeguard essential SSCs.

FSAR Tier 2, Section 3.6.1 defines essential SSCs as safety-related Seismic Category 1 SSCs, as given in FSAR Tier 2, Section 3.2, "Classification of Structures, Systems, and Components," that are necessary to shut down the reactor and mitigate the consequences of a postulated pipe rupture without offsite power. The staff evaluated this definition and finds that it complies with the guidance provided in SRP Section 3.6.1 and BTP 3-4, "Postulated Rupture Locations In Fluid System Piping Inside /and Outside Containment."

The fluid systems that contain high- and moderate-energy piping are identified in FSAR Tier 2, Table 3.6.1-1. Postulated terminal end breaks for high-energy systems by building and by room are presented in FSAR Tier 2, Table 3.6.1-2, and FSAR Tier 2, Table 3.6.1-3, "Building, Room, Break, Target, and Protection Required." However, as noted in FSAR Tier 2, Section 3.6.1, performance of the actual pipe break hazards analysis will the responsibility of the COL applicant.

The staff reviewed the U.S. EPR design regarding protection of essential SSCs against postulated piping failures in fluid systems outside the containment in accordance with 10 CFR Part 50, Appendix A, GDC 2, GDC 4, and the guidance provided by SRP Section 3.6.1.

GDC 2, "Design Bases for Protection Against Natural Phenomena"

The requirements of 10 CFR Part 50, Appendix A, GDC 2 states that SSCs important to safety shall be designed to withstand the effects of natural phenomena such as earthquakes. During a seismic event, it is postulated that non-seismic SSCs could fail. This section evaluates the impact of full-circumferential ruptures of non-seismic moderate-energy piping in areas close to SSCs important to safety. Acceptance criteria are based on conformance to BTP 3-3.

With respect to GDC 2 requirements, FSAR Tier 2, Section 3.6.1.2.2, "Design Features," states that essential systems and components are designed to meet the guidance regarding seismic design of Regulatory Guide (RG) 1.29, "Seismic Design Classification," as described in FSAR Tier 2, Section 3.2. FSAR Tier 2, Section 3.6.1.2.2 states that protective structural walls and compartments are designed to protect essential systems and components from the effects of piping failures. The protective structures are Seismic Category I, and are required to withstand an SSE, along with the effects of postulated piping failures such as jet impingement, pipe whip, compartment pressurization, and flooding. The staff finds the above acceptable to meet the requirements of GDC 2; however, based on the information provided in the application, the staff was unable to conclude that the environmental effects of full-circumferential ruptures of non-seismic moderate-energy piping in areas containing SSCs are addressed. No design description or tables were identified addressing such failures. Therefore, in RAI 80, Question 03.06.01-7, the staff requested that the applicant update the FSAR to address the environmental effects of full-circumferential ruptures of non-seismic moderate-energy piping in areas containing essential SSCs.

In a November 21, 2008, response to RAI 80, Question 03.06.01-7, the applicant stated that an evaluation of pipe rupture environmental effects is discussed in FSAR Tier 2, Section 3.6.2. In areas where there are both high-energy and moderate-energy pipes, the effects of the high-energy pipe ruptures are the dominant technical concern. In areas where there are only moderate-energy pipes, the most severe effect will be from flooding. The flooding analyses described in FSAR Tier 2, Section 3.4, "Water Level (Flood) Design," already considered full breaks of moderate-energy lines where the discharge of such lines might challenge safe shutdown equipment. Environmental effects due to temperature, pressure, and humidity are considered in the essential system environmental equipment qualification profiles, as stated in FSAR Tier 2, Section 3.11, "Environmental Qualification of Mechanical and Electrical Equipment." The applicant also stated in the November 21, 2008, response to RAI 80, Question 03.06.01-7, that redundant safety trains are located in each of four separate Safeguard Buildings. This four train separation and redundancy precludes the need to specifically evaluate rupture consequences and protection between trains, since rupture events are assumed to occur as separate events. Floor drains are assumed to be not available at all times; therefore, the floor drains are not relied upon for removal of internal flood water. Floor drains are not considered a path for bypassing divisional separation, since they are not interconnected between safety-related divisions or between safety-related buildings.

The staff finds the applicant's November 21, 2008, response to RAI 80, Question 03.06.01-7 acceptable because the applicant sufficiently described an acceptable methodology for evaluating breaks due to a seismic event. The applicant's response clarifies that the environmental conditions resulting from a pipe failure were already taken into consideration of the equipment qualifications profiles. The flooding protections are evaluated in Section 3.4 of this report. Specific consideration of such breaks will be contained in the pipe break hazards analysis which is to be completed as part of ITAAC 3.8, and COL Information Items 3.6-1 and 3.6-2. Therefore, the staff considers RAI 80, Question 03.06.01-7 resolved.

The U.S. EPR utilizes a Shield Building that encloses the Containment Building with an annulus between these two structures. FSAR Tier 2, Section 3.6.1.2.2 states that high-energy piping penetrating these two structures is enclosed within guard pipes such that the annulus is not affected by pipe ruptures inside or outside containment. This topic is evaluated further in Section 3.6.2 of this report. However, the applicant did not specify whether the moderate-energy lines are also enclosed with guard pipes and whether leakage cracks, as defined in BTP 3-3, are postulated in these areas. Therefore, in RAI 80, Question 03.06.01-8, the staff requested that the applicant specify whether the moderate-energy lines are enclosed with guard pipes and whether leakage cracks are postulated in these areas. In a November 21, 2008, response to RAI 80, Question 03.06.01-8, the applicant stated that the moderate-energy lines are not enclosed in guard pipes inside the annulus. The limiting line break in the annulus is the fire water piping break as described in FSAR Tier 2, Section 3.4.3.3, "Reactor Building Flooding Analysis." In addition, the applicant stated that the piping associated with containment penetrations were designed to Seismic Category I with RG 1.26, "Quality Group Classifications and Standards for Water-, Steam-, and Radioactive-Waste-Containing Components of Nuclear Power Plants," March 2007, Revision 4 and BTP 3-3, Section B1. The staff finds this acceptable because the impact of a moderate energy leakage crack in the annulus will be bounded by the fire water piping break. Therefore, the staff finds the applicant's response to RAI 80, Question 03.06.01-8 acceptable, and considers this RAI resolved.

Main steam (MS) and main feedwater (MFW) lines penetrate Safeguard Buildings 1 and 4. The lines run outside of Safeguard Buildings 2 and 3, where both the main control room (MCR) and remote shutdown stations are located. The MS and MFW lines are physically separated from the MCR by the building walls, which meets the criteria established in BTP 3-3, Section B.1 and which the staff finds acceptable. Therefore, a postulated pipe rupture in these lines will not adversely affect MCR habitability.

<u>GDC 4, "Environmental and Dynamic Effects Design Bases"</u>

The plant design for protection against postulated piping failure in fluid systems outside containment must meet the requirements of GDC 4, as it relates to accommodating the dynamic effects of postulated pipe ruptures, including the effects of pipe whipping and discharging fluids. The design is considered to comply with GDC 4 if it conforms to BTP 3-3.

The U.S. EPR defines the postulated pipe failure type based on whether the piping system is a high- or moderate-energy system. FSAR Tier 2, Section 3.6.1.1.1, "Criteria and Assumptions," states that a system is considered to be high energy when the maximum operating temperature exceeds 95 °C (200 °F) and/or the maximum operating pressure exceeds 1,900 kPa (275 psig) during normal conditions. The staff finds this criterion conforms to the criteria provided in BTP 3-3, Appendix A, and is therefore acceptable. FSAR Tier 2, Table 3.6.1-1 lists those systems that contain high- and moderate-energy lines that are considered when determining the need for protection of essential systems. While the systems given appeared to the staff to be consistent with the typical pressurized water reactor (PWR) classification of systems presented in SRP Section 3.6.1, Table 3.6.1-2, "Typical High Energy Systems Outside Containment," the staff was unable to confirm that U.S. EPR systems are properly classified, since maximum and normal operating pressures and temperatures were not specified in the FSAR.

Therefore, in RAI 80, Question 03.06.01-1, the staff requested that the applicant provide the maximum and normal operating pressures and temperatures for the systems given in FSAR Tier 2, Table 3.6.1-1. Additionally, the staff requested that the applicant clarify what systems provide heating to the Nuclear Island and identify any auxiliary steam or heating steam systems

used in the U.S. EPR. In a November 21, 2008, response to RAI 80, Question 03.06.01-1, the applicant stated that the systems listed in FSAR Tier 2, Table 3.6.1-1 meet BTP 3-3 criteria, and that specific information about piping maximum operating pressures and temperatures are contained in the applicant's internal documents and drawings which are available for NRC inspection. Additionally, the applicant clarified that the Nuclear Island uses electric heating and that there are no auxiliary steam systems used in areas containing safety-related SSCs. The staff finds the applicant's November 21, 2008, response to RAI 80, Question 03.06.01-1 acceptable because FSAR Tier 2, Table 3.6.1-1 was complete, and the systems were properly classified as high- or moderate-energy. Therefore, the staff considers RAI 80, Question 03.06.01-1 resolved. (As discussed below, the specific location of the piping will be covered by COL Information Item 3.6-1.)

FSAR Tier 2, Section 3.6.1 states that performance of the pipe break hazards analysis is the responsibility of the COL applicant. The staff finds this partially inconsistent with information contained in FSAR Tier 2, Section 3.4.3.1, "Internal Flooding Events," which states that the internal flooding analysis including consideration of high- and moderate-energy line ruptures is complete. Additionally, the results shown in FSAR Tier 2, Table 3.6.1-2 indicated that other portions of the hazard analysis (high-energy break locations and identification of protection requirements) may be complete.

Therefore, in RAI 80, Question 03.06.01-2, the staff requested that the applicant justify why the pipe break hazards analysis cannot be completed at the design certification stage, indicate what portions have been completed, and specify which portions are the responsibilities of the COL applicant. In a November 21, 2008, response to RAI 80, Question 03.06.01-2, the applicant stated that the pipe hazards analysis is the responsibility of the COL applicant, since its completion depends on ASME Code stress and fatigue analysis results which will be available later in the design process. The applicant also proposed to revise FSAR Tier 1, Table 2.1.1-7, "RBA Penetrations that Contain High Energy Pipelines," to add a new ITAAC to ensure the pipe break analysis is completed and meets GDC requirements and criteria. The staff determined that the applicant's response did not address all the staff's concerns.

Therefore, in responses to RAI 107, Question 03.06.02-17; RAI 222, Question 03.06.02-31; and RAI 354, Question 03.06.02-42, all of which are discussed in Section 3.6.2 of this report, the applicant proposed to add a new FSAR Tier 1, Section 3.8, "Design of Category I Structures." This section included a new ITAAC that requires the completion of the as-designed pipe break hazards analyses summary. The applicant also proposed to modify COL Information Items 3.6-1 and 3.6-2 to instruct the COL applicant to specify reconciliation of the as-designed pipe break hazards analysis. The staff reviewed the applicant's August 31, 2010, response to RAI 354, Question 03.06.02-42, which is discussed in Section 3.6.2 of this report. The staff determined that the resulting COL information item cannot be addressed by the COL applicant within the COL application review phase because it requires inspection of the as-built site. Therefore, in RAI 533, Question 03.06.01-13, the staff requested that the applicant revise FSAR Tier 2, Table 1.8-2, COL Information Items 3.6-1 and 3.6-2 to remove the references to the reconciliation of deviations between the as-built configuration and the as-designed analysis. **RAI 533, Question 03.06.01-13 is being tracked as an open item**.

In the proposed wording for FSAR Tier 1, Section 3.8, Table 3.8-1, "Piping Hazard Analysis ITAAC," the applicant referenced the completion of the pipe break hazards analyses summary. The staff determined the proposed wording unacceptable. In order to properly demonstrate that all SSCs that are relied upon to be functional are protected against or qualified to withstand the dynamic and environmental effects associated with postulated pipe breaks, the applicant needs

to complete the pipe break hazards analyses report, as described in FSAR Tier 2, Section 3.6.1 and Section 3.6.2, rather than a summary. Therefore, in RAI 503, Question 03.06.01-11, the staff requested that the applicant modify FSAR Tier 1 Section 3.8, Table 3.8-1 to require the completion of a pipe break hazards analyses report.

In an August 16, 2011, response to RAI 503, Question 03.06.01-11, the applicant proposed to revise FSAR Tier 1, Table 3.8-1, Item 2.1, to make reference to the pipe hazards analysis report. The staff reviewed the applicant's proposed modification to FSAR Tier 1, Table 3.8-1, Item 2.1 and finds that the new wording is consistent with the report description located in FSAR Tier 2, Section 3.6.2. Therefore, the staff considers RAI 503, Question 03.06.01-11 acceptable. **RAI 503, Question 03.06.01-11 is being tracked as a confirmatory item** pending incorporation of the proposed changes into the next FSAR revision.

Regarding high-energy line breaks, FSAR Tier 2, Section 3.6.1.1.1 discusses the criteria and analysis methodology for evaluating the effects of postulated pipe breaks in high-energy fluid systems. High-energy systems may experience through-wall leakage cracks or circumferential or longitudinal breaks, as discussed in FSAR Tier 2, Section 3.6.1.1, and such cracks and breaks are analyzed for their environmental effects. Dynamic effects include jet impingement and pipe whip, while the environmental effects include flooding, spray wetting, and increased temperature, pressure, and humidity inside the rooms affected by the postulated failure. FSAR Tier 2, Section 3.6.1 considerations, in conjunction with the postulated pipe failures, will include loss of offsite power and single active component failures. Therefore, the staff finds that the criteria in BTP 3-3, Section B.3.b(1) and (2) are met with respect to these assumptions.

Regarding the main steam and feedwater systems, FSAR Tier 2, Section 3.6.1.1.5, "Leak-Before-Break," states that no LBB methodology is applied to piping systems outside of containment. In addition, FSAR Tier 2, Section 3.6.1.1.6, "Containment Penetration Exclusion Zones," states that an assumed non-mechanistic longitudinal pipe break of one square foot cross-sectional area is postulated for these systems, as recommended in BTP 3-3. The staff finds this conforms to the criteria contained in BTP 3-3, Section B.1.a (1) for these systems, and is therefore acceptable. As discussed above for GDC 2, the main steam and feedwater lines are not routed around or near the vicinity of the control room; therefore, the staff finds the guidance contained in BTP 3-3, Section B.1.a (2) is met.

FSAR Tier 2, Table 3.6.1-2 provides a listing of terminal end breaks for high-energy systems, and provides the location for these breaks by building and room number. FSAR Tier 2, Table 3.6.1-3 provides a summary of the evaluation of a subset of the terminal end breaks where there are nearby essential systems and components requiring protection, as well as the essential system targets and type of protection to be designed. While the staff concluded that FSAR Tier 2, Table 3.6.1-2 includes all high-energy systems given in FSAR Tier 2, Table 3.6.1-1, it was unclear to the staff as to why safety-related portions of the component cooling water (CCW) system were considered high-energy break locations. Therefore, in RAI 80, Question 03.06.01-3, the staff requested that the applicant provide additional information as to why only portions of this system are considered high-energy. In a November 21, 2008, response to RAI 80, Question 03.06.01-3, the applicant clarified that only the non-safety-related portions of the CCW system, dedicated specifically to the severe accident heat removal system (SAHRS), are high-energy and that the safety-related portions are not. The applicant also proposed to revise FSAR Tier 2, Table 3.6.1-1 to correct this deficiency, to make several enhancements, and to rename the two subsystems of the component cooling water system for clarification as the component cooling water supply train and component cooling water common header. The staff finds the proposed changes acceptable because the

portions of the CCW system which are high- or moderate-energy, are now clearly and properly defined. The staff confirmed that the proposed FSAR changes were included in FSAR Revision 2, dated August 30, 2010. Accordingly, the staff finds that the applicant has adequately addressed this issue and, therefore, considers RAI 80, Question 03.06.01-3 resolved.

Additionally, the staff was unable to independently confirm that high-energy fluid systems are separated from essential systems and components, since individual system descriptions of the piping runs or routing and system arrangement drawings were not provided in the application. Therefore, in RAI 80, Question 03.06.01-4, the staff requested that the applicant provide additional information in this area. In a November 21, 2008, response to RAI 80, Question 03.06.01-4, the applicant confirmed that the detailed information on the systems in FSAR Tier 2, Table 3.6.3-1, “Main Coolant System Piping Dimensions and Operating Condition,” and arrangement drawings are provided in the applicant’s internal documents which are available for NRC inspection. The applicant further explained the methodology for determination of the terminal end locations for high-energy lines provided in FSAR Tier 2, Tables 3.6.1-1, 3.6.1-2, and 3.6.1-3, as follows:

- The starting point for the identification of high-energy lines in buildings containing safe shutdown equipment was an electronic manipulation of piping data (line number, design temperature, design pressure, and building/room location) from the 3-D plant models, This data was used to identify potential high-energy systems, supplemented by reviews if piping and instrumentation diagrams (P&IDs) and system descriptions.

- A calculation was performed to document and refine this list of high-energy systems based on the use of exclusion criteria, such as lines evaluated for leak-before-break, or known to be considered moderate-energy based on limited system operability above high-energy limits. This information was then used to develop Table 3.6.1-1, where moderate-energy systems were categorized as the remainder of the liquid systems not already identified as high-energy.

- The determination and location of terminal end breaks for the high-energy systems were identified based on the terminal end definitions in U.S. EPR FSAR Tier2, Section 3.6.2. These terminal end breaks, along with their building and room locations are provided in U.S. EPR FSAR Tier 2, Table 3.6.1-2. Actual locations were determined from equipment arrangement drawings or the 3-D plant models.

- The effects of each terminal end break were evaluated based on the affected SSC. The evaluation of affected SSE items was based on the safe shutdown equipment list, which identified equipment items by building and room location. For evaluations of nearby piping electrical distribution targets (conduits and cable trays), HVAC duct targets, and safety related building structural elements, the 3-D plant models were utilized. The results of these evaluations identified the need for protection devices listed in U.S. EPR FSAR Tier 2, Table 3.6.1-3.

Additionally, the applicant proposed to revise FSAR Tier 2, Table 3.6-2 to include several additional terminal end breaks. The staff evaluation of the terminal end definition is presented in

Section 3.6.2 of this report. The staff finds that the methodology described above is adequate to identify all the high and moderate energy piping systems and their relative location to SSCs that are important to safety because it is systematic, comprehensive, and envelopes all plant piping systems. Therefore, based on the description of the methodology use by the applicant, the staff finds the applicant's November 21, 2008, response to RAI 80, Question 03.06.01-4 acceptable. The staff confirmed that Revision 2 of the U.S. EPR FSAR, dated August 30, 2010, contains the changes committed to in the RAI response. Accordingly, the staff finds that the applicant has adequately addressed this issue and, therefore, considers RAI 80, Question 03.06.01-4 resolved.

FSAR Tier 2, Section 3.6.1 also discussed pipe leakage crack events involving moderate-energy fluid systems for flooding, and other environmental effects, and addressed the methods for protecting the essential systems against the effects of piping failures. FSAR Tier 2, Section 3.6.1 states that a combination of redundancy and separation will be utilized such that the reactor can be safely shut down after a postulated piping failure. The plant has four redundant safety trains for many of its essential systems. This configuration is designed to safely shut down the plant in the event that one safety train is out for maintenance, and one train is out due to single failure. In accordance with FSAR Tier 2, Section 3.6.1.2.1, "Criterion 10 – Reactor Design," other methods to achieve protection of essential systems include physically separating the essential SSCs from pipe ruptures by distance or through use of an intervening structure or barrier, providing protection enclosures, or by enclosing the pipes themselves.

FSAR Tier 2, Table 3.6.1-1 provides a list of all of the U.S. EPR moderate- or high-energy fluid systems. Protection will be provided to essential systems that could be adversely impacted by postulated pipe failures of these systems. The staff concludes that the table complies with the guidance contained in SRP Section 3.6.1. However, unlike high-energy line breaks, no information was provided in FSAR Tier 2, Table 3.6.1-2 to demonstrate that moderate-energy line failure locations have been identified, and that the resulting environmental effects on essential SSCs were specifically considered. Therefore, in RAI 80, Question 03.06.01-5 , the staff requested that the applicant update the FSAR in order to identify the moderate-energy line failure locations, the primary means of protection for each of these line ruptures (e.g., separation, enclosure, special means, etc.,) and to provide a listing of essential SSCs in each area requiring protection. In a November 21, 2008, response to RAI 80, Question 03.06.01-5, the applicant stated that the evaluation and protection for moderate-energy line failures is provided in FSAR Tier 2, Section 3.6.2, which is consistent with the "Review Interfaces" section of SRP Section 3.6.1 and in accordance with SRP Section 3.6.2. The staff also evaluated the methodology described in FSAR Tier 2, Sections 3.6.1 and 3.6.2, and concluded that if the applicant conforms to this methodology, there is high confidence that the applicant will properly identify all the moderate-energy line failure locations and the essential SSCs that will require protection, therefore, the staff considers RAI 80, Question 03.06.01-5 resolved.

As stated in FSAR Tier 2, Section 3.6.1.1.2, "Postulated Piping Failures and Ruptures," pipe failure evaluations are made based on circumferential or longitudinal pipe breaks, through-wall cracks, or leakage cracks. Breaks in high-energy lines are either circumferential breaks, where a through-wall crack extends around the entire circumference of the pipe; or longitudinal breaks (splits), where a through-wall crack runs parallel to the longitudinal axis of the pipe. Breaks in high-energy lines need an evaluation of jet discharge forces (thrust), evaluation of jet impingement, evaluation of the development of pipe whip hinges, and evaluation for the location of pipe whip restraints. These considerations, as well as the postulated break, through-wall

crack, and leakage crack locations are determined in accordance with FSAR Tier 2, Section 3.6.2. In accordance with FSAR Tier 2, Section 3.6.1.1.6, through-wall cracks are not postulated in containment penetration exclusion zones, with the exception of a non-mechanistic longitudinal pipe break of one square foot cross-sectional area, postulated in the main feed and main steam lines, at a location that has the greatest effect on essential equipment.

FSAR Tier 2, Section 3.6.2 describes the content of the pipe break hazards analyses report. However, this section did not explicitly include the evaluation of a non-mechanistic longitudinal pipe break of one square foot cross-sectional area within the pipe break exclusion zone, as recommended in SRP Section 3.6.1, and as discussed in FSAR Tier 2, Section 3.6.1.1.6. Therefore, in RAI 503, Question 03.06.01-12, the staff requested that the applicant update FSAR Tier 2, Section 3.6.2.1, "Criteria Used to Define Break and Crack Location and Configuration," to include the evaluation of the impact of a one square foot break in the main steam and main feed lines, within the pipe break exclusion zone.

In an August 16, 2011, response to RAI 503, Question 03.06.01-12, the applicant proposed to revise FSAR Tier 2, Section 3.6.2.1 to incorporate the changes described in the response. The staff reviewed the applicant's proposed modification to FSAR Tier 2, Section 3.6.2.1 and finds that the new wording explicitly includes the evaluation non-mechanistic longitudinal pipe break of one square foot cross-sectional area within the pipe break exclusion zone. Therefore, the staff considers RAI 503, Question 03.06.01-12 acceptable. **RAI 503, Question 03.06.01-12 is being tracked as a confirmatory item** pending the incorporation of the proposed changes into the next FSAR revision. A COL applicant will complete this report, pursuant to COL Information Item 3.6-1.

For postulated longitudinal or circumferential breaks in high-energy lines, pressurization loads on components and structures are also evaluated. The staff's evaluation of pressurization loading on structures is described in Section 3.8 of this report.

As discussed in detail below, the staff reviewed the assumptions used in pipe failure evaluations, as described in FSAR Tier 2, Section 3.6.1, and determined that the assumptions conform to the criteria contained in either BTP 3-3 or SRP Section 3.6.1; therefore, the staff finds the above criteria acceptable. FSAR Tier 2, Section 3.6.1.2 states that the protection considerations include separation and redundancy, barriers, and shields, and in some cases special protection considerations, as recommended in BTP 3-3. Where it is not practical to separate or shield essential equipment from postulated pipe failures, special measures will be taken to protect the operability of safety-related features. Examples of these types of features are given in FSAR Tier 2, Table 3.6.1-3. In RAI 80, Question 03.06.01-6, the staff requested that the applicant provide additional discussion of any design details regarding the application of special protective features which were not specifically given in FSAR Tier 2, Table 3.6.1-3. In a November 21, 2008, response to RAI 80, Question 03.06.01-6, the applicant confirmed that FSAR Tier 2, Table 3.6.1-3 contains the complete listing of all special protection devices used in the U.S. EPR design. Therefore, the staff finds the applicant's November 21, 2008, response to RAI 80, Question 03.06.01-6 acceptable, and considers this RAI resolved. The adequacy of these special protective measures is evaluated in Section 3.6.2 of this report.

Based on the above, the staff concludes that the applicant has provided sufficient design information to ensure that the guidelines of BTP 3-3 and SRP Section 3.6.1 will be met. Specifically, the U.S. EPR design provides protection of essential SSCs from the effects of high- and moderate-energy pipe failures by separation of redundant divisions of these systems, by separating (distance) essential SSCs from piping failures, or by locating essential SSCs such

that necessary protection is provided. Furthermore, all essential SSCs meet the guidelines of RG 1.29 and are designed to Seismic Category I standards. Piping failures are postulated in accordance with BTP 3-3, pipe restraints will be provided in accordance with the guidelines in SRP Section 3.6.2 (addressed in Section 3.6.2 of this report), and the effects of the postulated piping failures, including those from non-seismic systems, are considered. Based on this information, the staff concludes that the guidelines of both BTP 3-3 and SRP Section 3.6.1 will be met. Therefore, the staff finds that the requirements of both GDC 2 and GDC 4 are met with respect to the protection of SSCs important to safety from the impact of moderate and high energy piping systems.

ITAAC

There were no ITAAC explicitly directed to protection against postulated piping failures addressed in the FSAR Revision 1. However, there are ITAAC with respect to the inspection for installation of piping systems according to the design. Protection against piping failures design bases and criteria are defined and addressed in FSAR Tier 2, Section 3.6.2.

The applicant has proposed to create a COL information item that would instruct the COL applicant to complete the pipe hazards analysis. The completion and submittal of this report would provide the staff with sufficient time to evaluate the report before the start of the construction phase. However, the final pipe design may not be completed at the time of the staff reviews the COL application. Accordingly, the staff believed that the pipe hazards analysis should be addressed by an ITAAC that will require the COL applicant to complete the pipe hazards analysis based on the design piping configuration. This ITAAC should have sufficient details about the assumptions and the method of developing the analysis so that the staff will have confidence all the staff concerns related to the pipe hazards analysis will be properly addressed. In RAI 80, Question 03.06.01-9, the staff requested that the applicant justify why an ITAAC has not been created that requires the completion of the pipe hazards analysis before the start of the construction of the plant. The applicant's November 21, 2008, response to RAI 80, Question 03.06.01-9 was superseded by the March 16, 2010, response to RAI 354, Question 03.06.02-42, in which the applicant proposed the creation of a new ITAAC in FSAR Tier 1, Section 3.8 to address the completion of the pipe break hazards report. The evaluation of the applicant's response to RAI 354, Question 03.06.02-42 is discussed above in this section of the report, and in Section 3.6.2 of this report.

During the construction phase of the plant, it is expected that some essential SSCs may be located or routed in areas other than originally planned. Accordingly, the staff requested information in this regard in RAI 80, Question 03.06.01-10. Subsequently, however, in RAI 354, Question 03.06.02-42, the staff requested that the applicant provide an ITAAC item to confirm that all essential SSCs are protected from the effects of the postulated pipe failure according to the as-built piping configuration. Therefore, the staff considers RAI 80, Question 03.06.01-10 resolved.since these concerns were addressed by the applicant in the non-system based ITAAC Table 3.8-1, "Piping Hazard Analysis ITAAC," item created in response to RAI 354, Question 03.06.02-42. The disposition of RAI 354, Question 03.06.02-42 is discussed earlier in this section of the report.

Technical Specifications and Initial Plant Test Program

There are no Technical Specification sections for protection against postulated piping failures in fluid systems outside of containment. No specific initial plant testing program requirements were identified associated with this topic. Based on the staff evaluation of FSAR Tier 2, Section 3.6.1, the staff concluded that no specific initial plant testing program is required in

relation to the protection against postulated piping failures in fluid systems outside of containment.

3.6.1.5 *Combined License Information Items*

Table 3.6.1-1 provides a list of plant design for protection against postulated piping failures in fluid systems outside of containment related COL information item numbers and descriptions from FSAR Tier 2, Table 1.8-2:

Table 3.6.1-1 U.S. EPR Combined License Information Items

Item No.	Description	FSAR Tier 2 Section
3.6-1	A COL applicant that references the U.S. EPR design certification will perform the pipe break hazards analysis and reconcile deviations in the as-built configuration to the as-designed analysis.	3.6.1

The staff finds the above listing to be complete; however, the adequacy of this COL Information Item is still under review as disussed in Section 3.6.1.4 of this report. Also, the list adequately describes actions necessary for the COL applicant. No additional COL information items need to be included in FSAR Tier 2, Table 1.8-2 for plant design for protection against postulated piping failures in fluid systems outside of containment consideration.

3.6.1.6 *Conclusions*

In view of the foregoing, the staff concludes that the U.S. EPR design, as it relates to the protection of safety-related SSCs from the effects of piping failures outside containment, meets the guidelines of SRP Section 3.6.1 and, therefore, satisfies the requirements of GDC 2 and GDC 4 with respect to accommodating the effects of postulated pipe failures, except for the open item item discussed in RAI 533, Question 03.06.01-13 Therefore, except for the confirmatory items described above, the staff concludes that the U.S. EPR design is acceptable. It is noted that the detailed pipe break hazards analysis has not been completed and is the responsibility of the COL applicant and will require staff review at a later date.

3.6.2 Determination of Rupture Locations and Dynamic Effects Associated with the Postulated Rupture of Piping

3.6.2.1 *Introduction*

NRC regulations require, in part, that SSCs important to safety be designed to accommodate the effects of postulated accidents, including protection against the dynamic effects of postulated pipe ruptures. Dynamic effects of postulated pipe ruptures include pipe-whip and the jet impingement loads on proximate SSCs important to safety. Pipe whip is caused by the reactive thrust loads produced by the fluid jet exiting the break location. The objective of the staff's review is to verify and ensure that adequate protection has been provided such that the effects of the postulated pipe breaks do not adversely affect the functionality of SSCs relied upon for safe reactor shutdown, and for mitigating the consequences of the postulated pipe rupture.

3.6.2.2 *Summary of Application*

FSAR Tier 1: The applicant provided nonsystem based design description and ITAAC in FSAR Tier 1, Section 3.0, "Nonsystem Based Design Descriptions and ITAAC," Revision 2. Specifically, in FSAR Tier 1, Section 3.8, Table 3.8-1, "Piping Hazard Analysis ITAAC," the applicant provided the piping hazards analyses ITAAC pertaining to as-designed pipe break hazards analysis and the as-built pipe break hazards analysis reconciliation.

FSAR Tier 2: In FSAR Tier 2, Section 3.6.2, "Determination of Rupture Locations and Dynamic Effects Associated with the Postulated Rupture of Piping," Revision 2, the applicant provided the following information to address GDC 4, "Environmental and Dynamic Effects Design Bases," requirements:

- Criteria used to define break and crack location and configuration (FSAR Tier 2, Section 3.6.2.1, "Criteria Used to Define Break and Crack Location and Configuration")

- Guard pipe assembly design criteria (FSAR Tier 2, Section 3.6.2.2, "Guard Pipe Assembly Design Criteria")

- Analytical methods to define forcing functions and response models (FSAR Tier 2, Section 3.6.2.3, "Analytical Methods to Define Forcing Functions and Response Models")

- Dynamic analysis methods to verify integrity and operability (FSAR Tier 2, Section 3.6.2.4, "Dynamic Analysis Methods to Verify Integrity and Operability")

- Implementation of criteria dealing with special features (FSAR Tier 2, Section 3.6.2.5, "Implementation of Criteria Dealing with Special Features")

AREVA NP INC. (AREVA) provided the safety-related high- and moderate-energy fluid systems in FSAR Tier 2, Table 3.6.1-1, "High-Energy and Moderate-Energy Fluid Systems Considered for Protection of Essential Systems," Revision 2 for fluid systems located both inside and outside the containment.

FSAR Tier 2, Section 3.6.2.1 provides the criteria for the location and configuration of the postulated pipe breaks and cracks, except for those portions of piping that satisfy the requirements for LBB analysis. In FSAR Tier 2, Section 3.6.2.2, the applicant discusses the design criteria for guard pipe assembly as part of the containment penetration design for high-energy piping. FSAR Tier 2, Section 3.6.2.3 provides analytical methods to define forcing functions and pipe response models to determine the forcing functions and reaction forces that can dynamically excite the piping systems as a result of pipe breaks. FSAR Tier 2, Section 3.6.2.4 discusses the dynamic analysis methods used to evaluate the effects of pipe breaks or cracks on safety-related SSCs in the vicinity of the pipe rupture location. AREVA Technical Report, ANP-10318P, "Pipe Rupture External Loading Effects on U.S. EPR Essential Structures, Systems, and Components," Revision 0, March 2011, and its associated draft Revision 1, provide a detailed description of AREVA's new approaches for assessing the effects of all types of loads induced on structures from loss-of-coolant accident (LOCA) events. These effects include: blast waves, static jet impingement loads, and dynamic jet impingement loads, including the effects of resonance within jets impinging upon nearby structures. The applicant conservatively models all loading types, as well as the response of the target structures to the various loads. In FSAR Tier 2, Section 3.6.2, the applicant lists the technical report as

Reference 5. Finally, FSAR Tier 2, Section 3.6.2.5 discusses criteria for the location of pipe whip restraints, the design of pipe whip supports and structural barriers, and the evaluation of pipe rupture environmental effects.

FSAR Tier 2, Section 1.8, Table 1.8-2, "U.S. EPR Combined License Information Items," gives COL information items that pertain to FSAR Tier 2, Section 3.6.2. COL Information Items 3.6-2 and 3.6-4 are addressed associated with FSAR Tier 2, Section 3.6.2. FSAR Tier 2, Appendix 3C, "Reactor Coolant System Structural Analysis Methods," Section 3C.3, "Hydraulic Loading Analyses," addresses hydraulic loading analyses for thrust and jet impingement loadings associated with the RCS primary piping.

3.6.2.3 *Regulatory Basis*

The relevant requirements of the NRC regulations for this area of review, and the associated acceptance criteria, are given in NUREG-0800, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition," (hereafter referred to as NUREG-0800 or SRP), Section 3.6.2, "Determination of Rupture Locations and Dynamic Effects Associated with the Postulated Rupture of Piping," Revision 2, March 2007. NUREG-0800, Section 3.6.2 also identifies review interfaces with other SRP sections.

- 10 CFR Part 50, Appendix A, "General Design Criteria for Nuclear Power Plants," GDC 4, "Environmental and Dynamic Effects Design Bases," requires that nuclear power plant SSCs important to safety be designed to accommodate the effects of, and be compatible with, environmental conditions associated with normal operation, maintenance, testing, and postulated accidents, including loss-of-coolant accidents. These SSCs are to be protected against the effects of pipe whip and discharging fluids resulting from pipe breaks.

Acceptance Criteria adequate to meet the above requirements include:

1. BTP 3-3, "Protection Against Postulated Piping Failures in Fluid Systems Outside Containment," Revision 3, March 2007, which contains criteria for protection against postulated piping failures in fluid systems outside the containment.

2. BTP 3-4, "Postulated Rupture Locations in Fluid System Piping Inside and Outside Containment," Revision 2, March 2007, which contains the criteria for defining postulated rupture locations in fluid system piping inside and outside the containment.

3.6.2.4 *Technical Evaluation*

To meet the requirements of GDC 4, the NRC requires, in part, that SSCs important to safety be designed to be compatible with, and to accommodate, the effects of the environmental conditions resulting from postulated pipe rupture accidents, including LOCAs. The NRC also requires that SSCs important to safety are adequately protected against dynamic effects (including the effects of pipe whipping and discharging fluids) that may result from postulated pipe rupture events.

In accordance with SRP Section 3.6.2 and the BTP 3-4, the staff reviewed the proposed criteria and methodology presented by the applicant in FSAR Tier 2, Section 3.6.2, and other associated sections and appendices.

3.6.2.4.1 Criteria Used to Define Pipe Break and Crack Locations and Configurations

FSAR Tier 2, Section 3.6.2.1.1, "Locations of High-Energy Line Breaks and Leakage Cracks," provides the criteria for defining break and crack location and its configuration in high-energy fluid systems piping. For high-energy fluid system piping in containment penetration break exclusion areas, the applicant states that for the portions of fluid systems in these areas, breaks and cracks are not postulated from the containment wall up to and including the inboard and outboard containment isolation valves, when the systems meet the requirements of ASME, B&PV Code, Section III, Subarticle NE-1120, and where there are additional requirements given in Items 1 through 3 included in FSAR Tier 2, Section 3.6.2.1.1.1, "Break Locations in Containment Penetration Areas." The applicant also states that the U.S. EPR has no ASME Code, Section III, Class 1 piping in these break exclusion areas. Postulation of break locations in these areas for Class 2 piping is included, with special considerations involving criteria for welded attachments, piping welds, inservice examinations of welds, and design of guard pipes within containment penetrations. The staff reviewed these criteria and determined that these criteria are consistent with BTP 3-4, Part B, Item A(ii). Therefore, the staff finds the criteria acceptable.

FSAR Tier 2, Section 3.6.2.1.1.2, "Break Locations in Areas other Than Containment Penetration Areas," provides criteria for postulating pipe breaks in ASME Class 1, 2, and 3, and non-ASME Class piping for high-energy systems in areas other than containment penetrations (i.e., outside the break exclusion areas). Since these criteria are consistent with BTP 3-4, Part B, Item A(iii), the staff finds the criteria acceptable.

BTP 3-4, Part B, Item A(iv) states that in complex systems such as those containing arrangements of headers and parallel piping running between headers, the designer should identify and include all such piping within the designated run in order to postulate the number of breaks required by the criteria in Item A(iii). In RAI 222, Question 03.06.02-19, the staff requested the applicant to clarify whether the criterion BTP 3-4 Part B, Item A(iv) is applicable to the U.S. EPR piping design.

In a July 6, 2009, response to RAI 222, Question 03.06.02-19, the applicant stated that the pipe break hazards analysis will identify each piping run considered for break postulation. For complex systems, such as those containing arrangements of headers and parallel piping running between headers, all such piping will be included within a designated run for the purposes of break postulation. The staff confirmed that FSAR Tier 2, Section 3.6.2.1, Revision 2 addresses the concern stated in the original RAI and finds it acceptable, since it is in conformance with BTP 3-4, Part B, Item A(iv). Therefore, the staff considers RAI 222, Question 03.06.02-19 resolved.

In FSAR Tier 2, Section 3.6.2.1.1.3, "Leakage Crack Locations in High-Energy Piping Systems," the applicant provides criteria for postulating pipe leakage crack locations in ASME Class 1, 2, and 3, and non-ASME Class piping for high-energy systems. The staff reviewed these criteria and concluded that they are consistent with BTP 3-4, Part B, Item A(v). Therefore, the staff finds the criteria acceptable.

In FSAR Tier 2, Section 3.6.2.1.1.4, "High-Energy Fluid Systems Separated From Essential Systems and Components," the applicant provided the separation criteria for postulating break and crack locations for high-energy systems separated from essential systems and components. FSAR Tier 2, Section 3.6.2.1.1.4, states that the criteria in FSAR Tier 2, Sections 3.6.2.1.1.2 and 3.6.2.1.1.3 are only postulated if the consequences of the rupture can be shown to have an environmental effect on the essential equipment, such as an increased

temperature in a room containing essential equipment that results from a high-energy line break in a nearby, separate room. In addition, rupture and target interactions need not be evaluated in cases where the essential system targets are in systems with complete train separation and redundancy. Similar criteria are also included in FSAR Tier 2, Section 3.6.2.1.2.4, "Moderate-Energy Fluid Systems Separated from Essential Systems and Components," for postulating pipe failures in moderate energy systems. The staff noted that many essential systems in U.S. EPR are designed with four redundant trains, with each train capable of performing the system's safety function. Outside of containment, each of these trains is contained in a separate Safeguard Building to accomplish the separation criteria, and inside containment this is generally accomplished by separate compartments/rooms used for individual trains. The staff also noted that BTP 3-4, Part B, Items A(i) and B(i), do not allow such exclusion from pipe break postulation and evaluation based on systems with complete train separation and redundancy for both high- and moderate-energy systems located both inside and outside the containment. Therefore, in RAI 354, Question 03.06.02-32, the staff requested that the applicant further clarify the redundancy and separation method to be used inside the containment for the U.S. EPR design.

In a July 29, 2010, response to RAI 354, Question 03.06.02-32, the applicant stated that the U.S. EPR piping design will postulate pipe failures and will evaluate their associated pipe failure effects irrespective of whether the system is designed with four redundant and separate trains or not. The staff concluded that the U.S. EPR's criteria are consistent with BTP 3-4, Part B, Items A(i) and B(i) for high- and moderate-energy systems, respectively. In addition, the applicant accordingly revised FSAR Tier 2, Sections 3.6.2.1.1.4 and 3.6.2.1.2.4. The staff verified the revised texts in FSAR Tier 2, Revision 2 and finds them acceptable. Therefore, the staff considers RAI 354, Question 03.06.02-32 resolved.

FSAR Tier 2, Section 3.6.2.1.2, "Locations of Leakage Cracks in Moderate Energy Lines," provides criteria for postulating leakage crack locations in moderate-energy piping. FSAR Tier 2, Section 3.6.2.1.2.1, "Leakage Crack Locations in Fluid Systems in Containment Penetration Areas," stated that leakage cracks are not postulated in those portions of moderate-energy lines in the break exclusion area near containment penetrations, where the Level A or Level B stress calculated by the sum of Equations 9 and 10 from ASME B&PV Code, Section 3, Paragraph NC-3653 does not exceed 0.4 times the sum of the stress limits given in NC-3653. This criterion is consistent with BTP 3-4, Part B, Item B(ii), and therefore the staff finds this acceptable.

FSAR Tier 2, Section 3.6.2.1.2.2, "Leakage Crack Locations in Fluid Systems in Areas other than Containment Penetration Areas," provides criteria for postulating leakage cracks in areas other than the break exclusion areas and is consistent with the requirements presented in BTP 3-4, Part B, Item B(iii). Also, FSAR Tier 2, Section 3.6.2.1.2.3, "Moderate-Energy Fluid Systems in Close Proximity to High-Energy Fluid Systems," states that leakage cracks are not postulated in moderate-energy lines where the crack is in close proximity to a high-energy line break, as long as the crack does not result in more limiting environmental conditions than the high-energy line break. When a leakage crack in a moderate-energy line causes more limiting environmental conditions than a proximate high-energy line break, the provisions from FSAR Tier 2, Section 3.6.2.1.2.2 are used. The staff finds this is consistent with BTP 3-4, Part B, Item B(iv). Further, in FSAR Tier 2, Section 3.6.2.1.2.4, the applicant provided the separation criteria dealing with moderate-energy piping from essential systems and components. These criteria are already discussed in the resolution of RAI 354, Question 03.06.02-32 earlier in this section and are consistent with BTP 3-4, Part B, Item B(i). Therefore, the staff finds these criteria for moderate-energy piping acceptable.

FSAR Tier 2, Section 3.6.2.1.3, “Types of Breaks and Leakage Cracks,” provides the criteria for defining break and crack types in both high- and moderate-energy fluid systems piping. FSAR Tier 2, Section 3.6.2.1.3.1, “Circumferential Pipe Breaks,” Section 3.6.2.1.3.2, “Longitudinal Pipe Breaks,” and Section 3.6.2.1.3.3, “Leakage Cracks,” provide the respective criteria for the circumferential pipe breaks, the longitudinal pipe breaks, and the leakage cracks. Based on its review of these criteria, the staff concluded that the criteria included in FSAR Tier 2, Sections 3.6.2.1.3.1 through 3 are consistent with BTP 3-4, Part B, Items C(i), (ii), and (iii), respectively and, therefore, the staff finds the criteria acceptable.

The staff concludes that the U.S. EPR design as it relates to the criteria for locating and configuring pipe breaks and leakage cracks in high- and moderate-energy piping systems to protect the safety-related SSCs from the effects of pipe ruptures meets the guidelines of BTP 3-4 and is therefore acceptable.

3.6.2.4.2 Guard Pipe Assembly Design Criteria

A guard pipe, an integral part of certain containment penetrations, is a protective device to limit pressurization of the space between dual barriers of the containments to acceptable levels.

FSAR Tier 2, Section 3.6.2.2 provides criteria for the design of guard pipes in containment penetrations which are consistent with BTP 3-4, Part B, Items A(ii)(3) and (6). Therefore, the staff finds these criteria acceptable.

3.6.2.4.3 Analytical Methods to Define Forcing Functions and Response Models

FSAR Tier 2, Section 3.6.2.3 discusses criteria for the analytical methods to be used to calculate the structural loads resulting from a pipe rupture, including pressure waves caused by the initial blast emanated by the pipe rupture and fluid jet discharge forces, both static and dynamic. The fluid jet discharge forces acting on the ruptured pipe are dependent on the state of the fluid in the pipe at the time and location of the rupture, pipe flow area at the break, system fluid inventory between the source and the break, piping system frictional losses, and piping system geometry. The applicant states that movement of pipe, due to pipe breaks and cracks, is analyzed to show that the motion does not result in overstress of any SSCs important to safety. This section addresses the criteria for dynamic or pseudo-dynamic analysis of piping systems, targets, and protection devices.

The applicant also states that no response models or forcing functions are applied for those piping selected for LBB analysis. In addition, the applicant states that thrust and jet loads for the types of breaks, wave forces and dynamic force of the fluid jet, potential pipe whipping, and jet impingement effects are evaluated for piping not analyzed for LBB analysis. Furthermore, the applicant states that jet impingement forces and pipe whip impacts are calculated only if they can potentially cause damage to an essential system.

In FSAR Tier 2, Section 3.6.2.3.2.1, “Development of Jet Discharge Forces,” the applicant referred to American National Standards Institute / American Nuclear Society, (ANSI/ANS)-58.2-1988, (also referred to as ANS 58.2), Section 6.2, “Design Basis for Protection of Light Water Nuclear Power Plants Against the Effects of Postulated Pipe Rupture,” for the development of the dynamic jet forces. The applicant also referred to simplified methodology provided in Appendix B of that standard and stated that the thermal hydraulic problem is solved using standard computer programs, or the analysis is simplified using the methodology from Appendix B of ANS 58.2 Standard. The staff evaluated the methods presented in ANSI/ANS-58.2-1988, Section 6.2 and ANSI/ANS-58.2-1988, Appendices A and B. In the

method presented in the standard to solve the mathematical equations for the reaction thrust force, computer programs are used to predict the transient thermodynamic state properties of the fluid in a piping system following pipe rupture. The program requires inputs related to break area characteristics and pipe fluid transient conditions. Also, the standard suggests simplified methods that may be used when demonstrated to be conservative.

However, the staff noted that the applicant did not provide any sample calculations to illustrate the adequacy of the analytical methods that will be used in the pipe break hazard analysis. Also, the staff noted there did not appear to be any consideration in the analysis of how potential feedback between the jet and any nearby reflecting surface, which can increase substantially the dynamic jet forces impinging on the nearby target component and the dynamic thrust blowdown forces on the ruptured pipe through resonance, is considered. In addition, the precise physics, numerical methods, and level of validation of the referenced computational modeling were not provided. Therefore, in RAI 107, Question 03.06.02-6, the staff requested that the applicant provide some combination of references and written description yielding a complete picture of the physics, numerics, and validation status of the intended modeling. The staff also requested that the applicant clarify the criteria used to determine when the simplified modeling is to be applied and when the more detailed modeling is needed, including the staff's concerns that are related to the use of methodologies listed in ANS 58.2 Standard.

In a December 1, 2008, response to RAI 107, Question 03.06.02-6, the applicant stated that the simplified methodology provided in ANS 58.2 Standard, Appendix B and FSAR Tier 2, Section 3.6.2.3.2.1, for developing jet discharge forces, is the preferred method for jet impingement calculations. The applicant provided a description of the thrust and jet impingement evaluation for the RCS components using this methodology in FSAR Tier 2, Appendix C, Sections 3C.3.1, "Thrust Loading," and 3C.3.2, "Jet Impingement Loading." For cases where the stress qualification of the component would not be possible by using the simplified methodology in ANS 58.2 Standard, either a more detailed jet thrust evaluation would be performed using a computational modeling approach or jet shields would be used to protect the component from the jet force. Therefore, in RAI 107, Questions 03.06.02-7 through 03.06.02-13, the staff requested that the applicant address numerous concerns described by the staff regarding the use of methods given in the ANS 58.2 Standard for determining the jet impingement dynamic loading. Based on its evaluation of the applicant's December 1, 2008, response to RAI 107, Question 03.06.02-6, the staff determined that the applicant did not adequately address the staff's concerns pertaining to the use of ANS 58.2 methodology. Therefore, the staff closed RAI 107, Question 03.06.02-6 and in follow-up RAI 222, Question 03.06.02-21, the staff requested that the applicant provide the information requested in the original RAI 107, Question 03.06.02-6. In a September 11, 2009, response to RAI 222, Question 03.06.02-21, the applicant continued to refute the importance of dynamic jet forces. Therefore, the staff closed RAI 222, Question 03.06.02-21 and in follow-up RAI 354, Question 03.06.02-33, the staff once again requested that the applicant provide a conservative methodology which addresses oscillatory jet loads throughout the blowdown process.

In an August 30, 2011, response to RAI 354, Question 03.06.02-33, the applicant proposed changes to the Technical Report ANP-10318P, "Pipe Rupture External Loading Effects on U.S. EPR Essential Structures, Systems, and Components," Revision 0, March 2011. The applicant also included proposed changes to FSAR Tier 2, Section 3.6.2. In the August 30, 2011, response to RAI 354, Question 03.06.02-33, and Technical Report ANP-10318P, the applicant provided a detailed, conservative procedure for assessing the dynamic response of structures to oscillatory jet loads. ANP-10318P, Figure 3-13 summarizes their procedure. Once break locations are identified, source flow parameters are determined for jets throughout

the blowdown process using thermal-hydraulics codes such as RELAP and CRAFT2, which are listed in FSAR Tier 2, Section 3.9.1.2, "Computer Programs Used in Analyses." The staff's evaluations of the acceptability of the computer codes are addressed in Section 3.9.1 of this report.

The applicant also provided a table (ANP-10318P, Table 3-2) of typical exit planes and jet conditions for U.S. EPR plant design, along with calculation procedures for determining thrust coefficients at the postulated break locations using source flow parameters and dimensions. The applicant then determined which SSCs are impacted by the jet. In some cases, if less than one-half or less than one-fourth of an SSC is impacted by a jet, the forces are reduced accordingly. Static initial impulse forces are applied to target SSCs for all jet types, and SSCs within 10 pipe diameters of 2-phase jets and 25 pipe diameters of steam jets are assumed to be loaded with dynamic jet forces. For non-resonant dynamic jet forces, the applicant determined that the broad-band jet oscillations are sufficiently small such that their effects on a rugged impinged structure are smaller than those of the initial pulse loading analysis (when the jet first impacts on the structure). However, lightly constructed SSCs, such as valve operators and instrumentation, will be assumed to fail when impacted with any dynamic jet loading, and will, therefore, be protected with jet shields, which will be analyzed using the methodologies described below.

Subcooled flashing jets and single phase steam jets are evaluated to determine if resonance can occur, where feedback between acoustic waves reflected from the target and the vortices emanating from the jet source amplify significantly the pressure oscillations on the target. The applicant cited previous references which state that resonance can occur only for pressure ratios (source vs. ambient) less than 3.38, but conservatively assumes jets are resonant for pressure ratios much higher than 3.38. The references also state that jets can only be resonant when impinging on surfaces within 5 pipe diameters of the source, but the applicant conservatively assumed resonant jets up to 10 pipe diameters from the source. For resonant jets, the applicant conservatively assumed the oscillatory target force amplitudes are significantly higher than values cited to date in the open literature. Since the applicant assumed a conservatively high oscillating force, no additional jet analyses, such as those based on computational fluid dynamics (CFD) methods, are required.

Next, the applicant computed a minimum possible jet resonance frequency for each break condition based on studies described in referenced literature, and compares that frequency to resonance frequencies in the impacted SSCs. The applicant provided the minimum jet resonance frequencies for pipes in an U.S. EPR plant in ANP-10318P, Table 3-3. Simple structures, such as pipe supports, whip restraints, and jet shields, may be modeled as simple one degree of freedom systems, analyzed statically, and responses adjusted using dynamic load factors (DLFs). For modes with resonance frequencies 40 percent below the jet frequency, simplified static structural response calculations are conducted, since there is insignificant dynamic amplification.

All other structural modes are evaluated for potential strong interaction with jet resonances. The modes of both plate/shell and beam structures are examined to determine their potential response to jet loading locations and frequencies. For plate/shell structures, modes with responses less than 75 percent of that of the most responsive mode are ignored. For beam structures, all modes contributing to the initial 95 percent of the effective target static mass are included, and higher frequency modes are ignored. Within these mode sets, any structural resonance within a wide range of a jet resonance frequency is analyzed dynamically using finite element modeling and either modal superposition or direct integration techniques, and

assuming the jet loading and structural resonance frequencies are coincident, which is the worst-case condition. The loads are applied over an area approximately equal to that of the jet source. Although the loading is, in fact, spread over a larger area, applying it over a smaller area produces concentrated, conservative stresses in the loading region, and does not affect stresses computed away from the loaded region, since the stresses depend on the overall applied force, and not its spatial distribution. Damping is assumed to be one percent of critical, which is conservative, and consistent with previously accepted applications for new power plants, with several recent extended power uprate (EPU) boiling-water reactor (BWR) applications, and with RG 1.20, "Comprehensive Vibration Assessment Program for Reactor Internals during Preoperational and Initial Startup Testing," Section 2.1, Item 2. The damping is applied using the Rayleigh Damping model, with anchor points chosen to ensure that one percent damping is applied near the modal frequencies of interest. To ensure conservatism of computed stresses, the applicant will perform mesh convergence studies.

The stresses are then evaluated against applicable standards from ASME, American Institute of Steel Construction (AISC), and American Concrete Institute (ACI) for various steel and concrete SSCs, including piping and pressure vessel standards. Finally, the applicant provides a sample calculation of a steam generator using their methodology, which confirms the adequacy and conservatism of their approach.

Based on its evaluation of the above information, the staff concluded that the applicant's approach is conservative, assesses static as well as dynamic jet loads, applies conservative oscillatory jet loads to SSCs, assumes the worst-case condition of coincident jet loading and structural response frequencies, applies conservative one percent damping to structures, confirms convergence of computed stresses, and ensures that the stresses in all SSCs impinged upon are below requirements in applicable standards. Any SSCs which are determined to fail due to jet impingement loading will be protected by jet shields, which will be evaluated using the same conservative procedures described above. Therefore, staff finds the applicant's responses including the proposed changes to FSAR Tier 2, Section 3.6.2 and Technical Report ANP-10318P, Revision 0, March 2011, adequately addressed the staff's concerns and are acceptable. Therefore, **RAI 354, Question 03.06.02-33 is being tracked as a confirmatory item** until the applicant formally updates the FSAR and the technical report accordingly.

3.6.2.4.4 Dynamic Analysis Methods to Verify Integrity and Operability

FSAR Tier 2, Section 3.6.2.4 provides the methods to perform dynamic analyses of a ruptured pipe and its jet impingement and whipping effects on the surrounding safety-related SSCs. Time dependent and steady state thrust reaction and jet impingement loads caused by saturated or superheated steam, saturated or sub-cooled water, and cold water (non-flashing) fluid from a ruptured pipe are used in the design against the dynamic effects of a pipe break. The effects of blast loads emanating from the initial postulated rupture on nearby SSCs are also considered. The staff's evaluations of the applicant's dynamic analysis methods are described in the following subsections.

3.6.2.4.4.1 *Evaluation of Jet Impingement*

Note: Since the jet impingement evaluation procedure is integrally related to the thrust loads applied to the pipe with the postulated break, much of the discussion of the applicant's procedures appears above in Section 3.6.2.4.3 of this report. The staff notes that several of the RAIs that appear in this section were eventually resolved as part of the resolution of RAI 354, Question 03.06.02-33.

FSAR Tier 2, Section 3.6.2.4.1, “Evaluation for Jet Impingement,” discusses the potential for jet impingement on an essential component or structure, or on a designated barrier between the jet and an essential target in a high-energy line break event. Such jet impingements depend on the source fluid conditions, placement of the target relative to the break, jet energy discharged, jet shape, and also target characteristics (shape, structural, and dynamic characteristics). In addition, there is the probability of increased temperature and wetting of the target resulting from a direct impingement from the jet. Although these last two effects are also considered for environmental effects in the general area, there may be a more significant effect on the target itself from direct impingement.

The methods used to evaluate the jet effects resulting from the postulated breaks in high energy piping were originally based on Appendices C and D of ANSI/ANS 58.2-1988. The ANS 58.2 jet forces consist of an initial transient jet force, followed by a steady state thrust force condition, which continues until the fluid inventory is exhausted. This thermal hydraulics problem is solved using standard computer programs, or the analysis is simplified using the methodology from ANSI/ANS 58.2-1988, Appendix B. Industry commonly uses the ANSI/ANS 58.2-1988 (Appendices C and D) for estimating jet plume geometries and loads based on the fluid conditions internal and external to the piping, and to date the ANSI/ANS 58.2-1988 standard has been accepted by NRC.

The staff has considered the recent scrutiny of the ANS 58.2 expanding jet models by the Advisory Committee on Reactor Safeguards (ACRS) which has revealed several inaccuracies that may lead to nonconservative assessments of the strength, zone of influence, and space and time-varying nature of the loading effects of supersonic expanding jets on neighboring structures. The ACRS also stated that initial blast waves are unaccounted for in the standard.

The ACRS review of the ANS 58.2 jet models was motivated by Generic Safety Issue (GSI)-191, “Assessment of Debris Accumulation on PWR Sump Performance,” which addresses the blockage of strainers upstream of emergency sump pumps by particulate. The particulate is formed by fibrous ceramic insulation, which can be broken loose by blast waves and/or jets emanating from nearby pipe ruptures. ACRS Safety Evaluation letters to the Chairman of the NRC are cited in SRP Section 3.6.2, Revision 3, Section III.3. Also, examples of inconsistencies between existing standards for simulating the effects of LOCAs on neighboring structures are given in the Knowledge Base for Emergency Core Cooling System Recirculation Reliability, February 1996, issued by the Nuclear Energy Agency/Committee on the Safety of Nuclear Installations (NEA/CSNI). Although the focuses of the ACRS and the NEA/CSNI report was on debris generation and sump blockage, their comments directly impact the assessments of postulated pipe breaks on neighboring SSCs. The following RAIs summarize the ACRS criticisms that relate specifically to possible non-conservatisms in the ANS 58.2 Standard. The applicant was advised by the staff that, as stated in SRP Section 3.6.2 Section III.3, the ANS 58.2 Standard is no longer universally acceptable for modeling jet expansion in nuclear power plants.

The applicant presented methods to be used to evaluate the jet effects resulting from the postulated breaks in high energy piping as described in ANSI/ANS 58.2-1988, Appendices C and D. The staff notes there does not appear to be any consideration of oscillatory jet loads, and of how potential feedback between the jet and any nearby reflecting surface, which can substantially increase the dynamic jet forces impinging on the nearby target component and the dynamic thrust blowdown forces on the ruptured pipe through resonance in the applicant’s analysis. The staff also notes that no discussion on the structural integrity of safety-related SSCs, other than piping, and operability assurance of all safety-related SSCs were provided in

the FSAR Revision 0. Therefore, in RAI 222, Question 03.06.02-22, the staff requested that the applicant provide a conservative methodology that addressed ACRS criticisms of ANS 58.2, as well as dynamic jet effects. The staff also requested that the applicant provide an updated list of high-energy breaks for all conditions, including subcooled water.

In a September 11, 2009, response to RAI 222, Question 03.06.02-22, the staff determined that the applicant did not adequately address how the jet dynamic oscillatory loads will be accounted for and how the amplifying effects of structural resonances will be accounted for. Therefore, the staff closed RAI 222, Question 03.06.02-22, and in follow-up RAI 354, Question 03.06.02-34 requested that the applicant address the staff's concerns discussed above.

In a July 22, 2011, response to RAI 354, Question 03.06.02-34, the applicant referenced their response to RAI 354, Question 03.06.02-33. Based on its review, the staff finds the response to RAI 354, Question 03.06-02-33 acceptable and addresses adequately the staff's concern in RAI 354, Question 03.06.02-34. Therefore, the staff considers RAI 354, Question 03.06.02-34 resolved.

The staff noted that blast waves were not considered in AREVA's initial application. Therefore, In RAI 222, Question 03.06.02-23, the staff requested that the applicant clarify how the effects of blast wave loads on near-by SSCs will be assessed. Based on ACRS concerns, the staff stated that the information in the Knowledge Base for Emergency Core Cooling System Recirculation Reliability, February 1996, issued by the NEA/CSNI, all high pressure and temperature pipes should be considered as sources of blast waves with initial energy and mass roughly equal to the exposed volume from a hypothesized break. The subsequent damage from such waves has been well documented and is not properly accounted for by the isolated analysis of a pure spherically expanding wave. Therefore, in RAI 222, Question 03.06.02-23, the staff requested that the applicant provide a rigorous and thorough explanation of the procedures to be used for estimating the effects of blast waves on nearby SSCs. Also, the staff noted that blast wave load analyses should be based on three-dimensional (or asymmetric) unsteady analysis of the flow field, with appropriate representation of the surrounding structures, subsequent to the initial blast.

In a September 11, 2009, response to RAI 222, Question 03.06.02-23, the applicant stated that the information in Army TM 5-1300/Navy NAVFAC P-397/Air Force AFR 88-22, "Joint Departments of the Army, the Navy, and the Air Force, Structures to Resist the Effects of Accidental Explosions," November 1990, would be used to account for the dynamic loads caused by blast waves. The staff notes that the applicant's approach appears to be appropriate, since the U.S. Military Services endorse this document. However, the applicant did not clearly describe how this manual will be used in their analysis. Therefore, the staff closed RAI 222, Question 03.06.02-23 and in follow-up RAI 354, Question 03.06.02-35, the staff requested that the applicant provide a summary of the planned evaluation approach, citing specific chapters, tables, and figures in the Army/Navy/Air Force manual, along with a demonstration calculation representative of a blast wave in the U.S. EPR design. The staff also requested that the applicant clarify how the plan to be used that will account for the different fluid properties in an U.S. EPR blast (steam and/or water) compared to those of air considered in the Army/Navy/Air Force manual. **RAI 354, Question 03.06.02-35 is being tracked as an open item.**

In RAI 222, Question 03.06.02-24, the staff stated to the applicant that ANS 58.2 was no longer considered universally acceptable for modeling at impingement loading in nuclear power plants. Therefore, in this RAI, the staff requested that the applicant substantiate this rejection in light of

evidence from NEA/CSNI/R (95) 11, “Knowledge Base for Emergency Core Cooling System Recirculation Reliability” February 1996, and to

- Clarify the analysis and/or testing used to substantiate the use of the ANS 58.2, Appendix C for defining conservatively which SSCs are in jet paths and the subsequent loading areas on the SSCs.

- Substantiate this assumption in light of the findings in NEA/CSNI/R (95)11 that steam jets can cause significant damage at distances up to 25 pipe diameters. In FSAR Tier 2, Section 3.6.1.1.1, the applicant stated: ”Components that are in the path of steam or subcooled liquid that can flash at the break are assumed to fail if they are within a distance of ten (10) pipe diameters (broken pipe outside diameter) from the break.”

In a September 11, 2009, response to RAI 222, Question 03.06.02-24, the applicant referred to their response to RAI 222, Question 3.6.2-21 (which was later amended into the response to RAI 354, Question 03.06.02-33) to address ANS 58.2 issues. The applicant also agreed to consider jet loading on SSCs less than 25 pipe diameters away from any postulated breaks. Since the applicant has extended their damage assessments to 25 pipe diameters, and has also updated the jet loading procedures to be conservative (as stated in the applicant’s response to RAI 354, Question 03.06.02-33, which the staff considered acceptable), the staff finds this RAI response acceptable and considers RAI 222, Question 03.06.02-24 resolved.

In RAI 107, Question 03.06.02-10, the staff requested that the applicant:

- Clarify the analysis and/or testing used to substantiate use of ANS 58.2, Appendix D for defining conservatively the net jet impingement loading on SSCs in light of the information presented by the ACRS as discussed earlier in this section of the report, which challenges the accuracy of the pressure distribution models presented in ANS 58.2.

- Expand the table of all postulated break types (FSAR Tier 2, Table 3.6.1-1) to include the properties of the fluid internal and external to the ruptured pipe. The table should specify the type of jet the applicant assumes will emanate from each pipe break – incompressible nonexpanding jet, or compressible supersonic expanding jet - along with how impingement forces will be calculated for each jet. Specific examples of jet impingement loading calculations made using the ANS 58.2 Standard for the postulated piping breaks in an U.S. EPR should be given, along with proof that the calculations lead to conservative impingement loads in spite of the cited inaccuracies and omissions in the ANS 58.2 models stated by the ACRS.

In a February 20, 2009, response to RAI 107, Question 03.06.02-10, the applicant stated that pressure distribution models from ANS 58.2, Appendix D were not used in the jet force calculations, and that the worst-case static pressure at the pipe break location is used to compute jet force, which is then applied to the target area. The applicant cited NUREG/CR-2913, “Two-Phase Jet Loads,” to support this approach, which is acceptable for static forces. The staff determined that the dynamic oscillatory jet forces were not adequately addresses in the applicant’s approach. Therefore, the staff closed RAI 107, Question 03.06.02-10, and in follow-up RAI 222, Question 03.06.02-25, the staff requested that the applicant clarify its position on dynamic jet loads in the analysis approach or provide an updated approach which considers them. The staff also requested that the applicant expand

the table of postulated pipe break types to include properties of the fluid internal and external to the ruptured pipes.

In a September 11, 2009, response to RAI 222, Question 03.06.02-25, the applicant stated that the spatial pressure distribution throughout a jet cross section is not used in the jet impingement loading calculations. The applicant noted in several of its positions, that either (1) dynamic pressure loading may be disregarded or (2) that the calculation approach somehow bounds dynamic pressure loading. However, it is still unclear to the staff how the applicant's current approach bounds the dynamic loads. Therefore, the staff closed RAI 222, Question 03.06.02-25, Part (a), and in follow-up RAI 354, Question 03.06.02-36(a), the staff requested that the applicant prove that the current approach bounds dynamic loads, or submit a revised approach which computes the dynamic loads conservatively. For RAI 222, Question 03.06.02-25, Part (b), the applicant provided a partial table of jet initial conditions and postulated jet loads, but did not (1) associate the jet types with a calculation procedure, as requested, and (2) did not associate the jet types with the postulated ruptures in FSAR Tier 2, Table 3.6.1-2. Therefore, the staff closed RAI 222, Question 03.06.02-25(b), and in follow-up RAI 354, Question 03.06.02-36(b) the staff requested that the applicant include this information in the FSAR.

In a July 22, 2011, response to RAI 222, Question 03.06.02-36, Part (a), the applicant referred to the response to RAI 354, Question 03.06.02-33. Since the staff considers RAI 354, Question 03.06.02-33 resolved, and since the applicant now conservatively accounts for jet dynamic pressure loading, the staff considers RAI 354, Question 03.06.02-36(a) resolved. In the applicant's response to RAI 222, Question 03.06.02-36, Part (b), the applicant provided a list of design pressure and temperature conditions for some terminal end breaks from FSAR Tier 2, Table 3.6.1-3, "Building, Room, Break, Target, and Protection Required." The applicant also provided the jet types with the analysis procedures which will be used for respective postulated breaks. Furthermore, the applicant stated that a complete list of break locations and conditions will be determined during the design process. The staff finds the response adequately addressed the staff's concern, and is therefore acceptable. The staff considers RAI 354, Question 03.06.02-36(b) resolved.

In RAI 107, Question 03.06.02-11, and follow-up RAI 222, Question 03.06.02-26, the staff requested that the applicant:

- Provide information that establishes that their interpretation of the jet impingement force as static is conservative.
- Clarify whether any postulated pipe break locations are within 10 diameters of a neighboring SSC (or barrier/shield), and if so, the process of accounting for jet feedback/resonance and resulting dynamic load amplification.
- Clarify whether dynamic jet loads are to be considered, and if so, using what methods. Also, should the dynamic loading include strong excitation at discrete frequencies corresponding to resonance frequencies of the SSC impinged upon, provide the basis for assuming a static analysis with a dynamic load factor of two is conservative.

In a February 20, 2009, response to RAI 107, Question 03.06.02-11, and in a September 11, 2009, response to RAI 222, Question 03.06.02-26, the staff determined that the applicant did not adequately address the staff's concerns regarding how they plan to evaluate SSCs at resonance due to jet impingement and how they plan to perform hand calculations or computer

analyses to determine whether resonance with time varying jet pressure is possible. Therefore, the staff closed RAI 107, Question 03.06.02-11 and RAI 222, Question 03.06.02-26, and in follow-up RAI 354, Question 03.06.02-37, the staff requested that the applicant address the staff's concerns discussed above.

In a July 22, 2011, response to RAI 354, Question 03.06.02-37, the applicant referred to the response to RAI 354, Question 03.06.02-33. Based on its review, the staff finds the response to RAI 354, Question 03.06-02-33 acceptable, and addresses the staff's concerns included in RAI 354, Question 03.06.02-37. Therefore, the staff considers RAI 354, Question 03.06.02-37 resolved.

In RAI 107, Question 03.06.02-12, the staff requested that the applicant clarify quantitatively how reflections of jets by neighboring structures will be considered. In a December 1, 2008, response to RAI 107, Question 03.06.02-12, the applicant stated that the jet reflections are disregarded due to energy losses, and that reflected jet amplitudes are negligible compared to other loads, like safe shutdown earthquake and pipe break loads. The applicant is basing this assessment on the ANS 58.2 Standard, which the staff does not consider to be universally applicable. Therefore, the staff closed RAI 107, Question 03.06.02-12 and in follow-up RAI 222, Question 03.06.02-27, the staff requested that the applicant summarize the quantitative approach used for modeling jet reflections. If the applicant planned to maintain the assertion that the reflected jet loads are negligible compared to other design loads, the staff requested that the applicant submit analyses confirming this assertion.

In a September 11, 2009, response to RAI 222, Question 03.06.02-27, the applicant stated that the jet reflections would be addressed as follows:

- No reflections will be considered for liquid jets, which should propagate parallel to the surface impinged upon.
- Compressible supersonic jets are assumed to lose significant energy due to normal shocks within the jet. This loss of energy is assumed to be strong enough to make the loads of any reflected jets negligible.
- Incompressible subsonic jet deflections will be considered.

However, the staff identified that liquid jets may propagate onto other structures, and their impact should still be assessed. Also, the applicant's view of compressible supersonic jets assumes a pure reflection and ignores deflections of less than 180 degrees. Smaller deflections would only involve oblique shocks and could still contain significant momentum. Therefore, the staff closed RAI 222, Question 03.06.02.27, and in follow-up RAI 354, Question 03.06.02-38, the staff requested that the applicant address the staff's concerns about liquid and compressible supersonic jets, along with providing further details on how incompressible subsonic jet deflections will be considered.

In a July 22, 2011, response to RAI 354, Question 03.06.02-38, the applicant stated that any target impacted by a deflected jet would be treated as a primary target, which the staff considers to be conservative. To determine targets impacted by subsonic incompressible reflected jets, the applicant assumes the deflected jet travels parallel to the deflecting surface. For supersonic jets, the applicant assumes reflections within allowable oblique shock angles, computed according to Ascher H. Shapiro, "The dynamics and thermodynamics of compressible fluid flow," Wiley and Sons, 1953. Since the applicant is conservatively computing reflections, and

then conservatively treating a secondary target as a primary target, the staff finds the response acceptable and, therefore, considers RAI 354, Question 03.06.02-38 resolved.

In RAI 354, Question 03.06.02-39, the staff requested that the applicant clarify how barriers or shields will be designed so as to not be damaged or destroyed by dynamic jet resonant loading.

In a July 22, 2011, response to RAI 354, Question 03.06.02-39, the applicant referred to their response to RAI 354, Question 03.06.02-33. Based on its review, the staff finds the response to RAI 354, Question 03.06-02-33 acceptable, and addresses adequately the staff's concerns included in RAI 354, Question 03.06.02-39. Therefore, the staff considers RAI 354, Question 03.06.02-39 resolved.

3.6.2.4.4.2 *Analysis of Essential System Piping Due to a Break in Attached Piping*

FSAR Tier 2, Section 3.6.2.4.2, "Analysis of Essential System Piping Due to a Break in Attached Piping," provides methods to analyze piping systems to evaluate the dynamic aspects of the jet discharge from a broken high-energy line. This includes significant dynamic loads and displacements in essential system piping attached to the broken pipe, such as, wave forces arising from rapidly traveling waves due to the depressurization of the broken pipe in attached essential system piping. There might be dynamic displacements of in-line components due to such dynamic events caused by the line break, which might be significant enough to be considered as terminal end (anchor) motions. Such dynamic problems may exist depending on the location of check valves in the remaining unbroken piping. With closing check valves, the faster the closing time for the valve the higher the dynamic loading in the remaining piping.

All ASME Code Class 1, 2, and 3 piping systems that are essential for safe shutdown must retain their functional capability for all Service Level D loading conditions as required by GDC 2. Designs meeting the recommendations in NUREG-1367, "Functional Capability of Piping Systems," are accepted by the staff as satisfying the functional capability requirements. In RAI 354, Question 03.06.02-40, the staff requested that the applicant address the functional capability of the essential system piping and operability of the in-line components due to a break in attached piping.

In a January 5, 2011, response to RAI 354, Question 03.06.02-40, the applicant referred to Topical Report ANP-10264NP-A, "U.S. EPR Piping Analysis and Pipe Support Design Topical Report," Section 3.5. As discussed in Section 3.12.4.4.12 of this report, the staff finds that the topical report meets the guidance and code requirements in NUREG-1367, and therefore is acceptable. In addition, as discussed in Section 3.9.3.4.3 of this report, the staff finds the U.S. EPR's method for pump and valve operability assurance acceptable. Based on the information as described, the staff concluded that the applicant's response to RAI 354, Question 03.06.02-40 adequately addressed the staff's concern and is acceptable. Therefore, the staff considers RAI 354, Question 03.06.02-40 resolved.

3.6.2.4.4.3 *Development of Pipe Whip Hinges*

FSAR Tier 2, Section 3.6.2.4.3, "Development of Pipe Whip Hinges," provides criteria for the development of pipe whip hinges and, therefore, the whipping of the broken pipe. The criteria described in developing rupture response models concern the fluid discharge of a broken pipe, neglecting the potential of a plastic hinge to develop in the pipe, and thus allowing whipping of the broken pipe. For protection purposes, the following are considered:

- The broken pipe is shown to have insufficient energy to develop a plastic hinge.

- A whip restraint is located and designed to preclude the whip from taking place.

- The whipping pipe is shown to not contact an essential system target, or to cause a jet to impact an essential system target.

- The essential system target impacted by the whipping pipe, or a jet due to the whipping motion, is shown to be structurally adequate to withstand the effects of the impact.

The applicant also states that if a pipe whip from a high-energy line break can be postulated to occur and to have an adverse effect on an essential system target, the location of a plastic hinge, is established. These hinge locations will determine the potential whipping motion and travel of the broken pipe. In addition, if a whip restraint is to be designed, but cannot be placed close to the first elbow upstream of the postulated circumferential break point (or close to the longitudinal break point), a hinge may develop downstream of the whip restraint. Using these methods, to comply with SRP Section 3.6.2, Item III.2.B, the whipping pipe problem is characterized to determine the appropriate pipe movements, pipe impact loads, and pipe whip restraint design forces. Since this complies with the staff position in SRP Section 3.6.2, the staff finds this acceptable.

3.6.2.4.5 Implementation of Criteria Dealing with Special Features

FSAR Tier 2, Section 3.6.2.5 discusses the criteria for locating and designing the pipe whip restraints and structural barriers. Also, this section includes the evaluation of pipe rupture environmental effects on essential components and systems.

FSAR Tier 2, Section 3.6.2.5.1, “Pipe Whip Restraints,” discusses energy absorbing whip restraints. Each restraint is a combination of an energy absorbing element using a crushable material and a restraining structure suitable for the geometry required to transfer the load from the whipping pipe to the main building structure. The design allows normal movement of the piping and yields plastically at rupture absorbing the pipe’s kinetic energy. For a whip restraint near the first elbow upstream of a circumferential break, or near a longitudinal break, a static analysis calculation is performed using the maximum jet discharge force multiplied by a factor of 1.1 for rebound effects, and a factor of 2.0 for a dynamic load factor. The allowable capacity of such a crushable material is limited to 80 percent of its rated energy dissipating capacity, as determined by dynamic testing, at loading rates within plus or minus 50 percent of the specified design loading rate. This complies with SRP Section 3.6.2, Item III.2.A and, therefore, the staff finds this acceptable.

The applicant further states that pipe whip restraints are designed for a one-time accident event, so they are designed to undergo deformation as long as the whipping pipe is fully restrained for the entire time of the blowdown event. However, the applicant did not discuss which loads will be evaluated in combination with pipe break forces. Therefore, in RAI 354, Question 03.06.02-41, the staff requested that the applicant provide the design and analysis of whip restraints, loads and load combinations, and the codes and standards to be used for maintaining whip restraint structural integrity prior to a pipe break event. In a July 29, 2010, response to RAI 354, Question 03.06.02-41, the applicant stated that pipe whip restraints will be designed as Seismic Category I miscellaneous structures, in accordance with FSAR Tier 2, Section 3.8.4, “Other Seismic Category I Structures.” The applicant also stated that the load combinations, codes and standards, and acceptance criteria described in FSAR Tier 2, Section 3.8.4 are consistent with SRP Section 3.8.4 for Seismic Category I miscellaneous structures. The staff’s evaluation of FSAR Tier 2, Section 3.8.4 is included in Section 3.8.4 of

this report. The staff confirmed that the applicant has revised FSAR Tier 2, Section 3.6.2.5.1.2, “Pipe Whip Support Design,” in FSAR Revision 2, stating that the pipe whip restraints will be designed as Seismic Category I miscellaneous structures and therefore finds it acceptable. Therefore, the staff considers RAI 354, Question 03.06.02-41 resolved.

In FSAR Tier 2, Section 3.6.2.5.2, “Structural Barrier Design,” the applicant states that the methodology used for barrier structural analysis for pipe rupture design loads is based on the criteria described in FSAR Tier 2, Section 3.8.4.4.1, “General Procedures Applicable to Other Seismic Category I Structures,” which comply with SRP Section 3.8.4. The staff’s evaluation of FSAR Tier 2, Section 3.8.4 is discussed in Section 3.8.4 of this report.

In FSAR Tier 2, Section 3.6.2.5.3, “Evaluation of Pipe Rupture Environmental Effects,” the applicant states that the environmental effects include increased temperature, increased pressure, increased humidity, spray wetting, and flooding. Temperature, pressure, and humidity are considered in the essential system environmental equipment qualification. The staff notes that other environmental effects, such as humidity and temperature, are included in FSAR Tier 2, Section 3.11, “Environmental Qualification of Mechanical and Electrical Equipment,” for equipment qualification. The staff also notes that the effects of increased pressure in a compartment are described further in FSAR Tier 2, Section 3.8, “Design of Category I Structures,” as part of the containment design. The staff’s evaluations of FSAR Tier 2, Sections 3.11 and 3.8 are addressed in Sections 3.11 and 3.8 of this report, respectively.

3.6.2.4.6 Combined License Information Items

FSAR Tier 2, Section 1.8, “Interfaces with Standard Designs and Early Site Permits,” identifies items that will be provided by the COL applicant. FSAR Tier 2, Table 1.8-2, COL Information Item 3.6-2 states that a COL applicant that references the U.S. EPR design certification will perform the pipe break hazards analysis and reconcile deviations in the as-built configuration to this analysis. COL Information Item 3.6-4 states that a COL applicant will provide diagrams showing the final as-designed configurations, locations, and orientations of the pipe whip restraints in relation to break locations in each piping system. The staff notes that in this same table, the applicant identified these two COL information items as actions required by COL applicant. The staff also notes that the pipe break hazards analysis report will be completed by the COL applicant.

10 CFR 52.47(b)(1), “Contents of applications, technical information,” requires that a design certificate application contain the proposed ITAAC that are necessary and sufficient to provide reasonable assurance that, if the inspections, tests, and analyses are performed and the acceptance criteria met, a plant that incorporates the design certification is built and will operate in compliance with the design certification, the provisions of the Atomic Energy Act of 1954, and NRC regulations. As described in SRP Section 14.3.3, “Piping Systems and Components – ITAAC,” one ITAAC item should be included to complete a pipe break evaluation report that documents that SSCs that are required to be functional during and following an SSE have adequate high-energy pipe break mitigation features. The design description should discuss the criteria used to postulate pipe breaks, the analytical methods used to analyze pipe breaks, and the methods to confirm the adequacy of the results of the pipe break analyses. The design description should be verified in a pipe break hazard analysis report that provides assurance that the postulated pipe break analyses have been completed. The pipe break hazard analysis report shall conclude that, for each postulated pipe failure, the reactor can be shut down safely and maintained in a safe, cold shutdown condition without offsite power. Detailed information that supports this ITAAC should be contained in FSAR Tier 2, Chapter 3, “Design of Structures,

Components, Equipment and Systems." Furthermore, an as-built reconciliation review of this pipe break hazard analysis report should be included in the ITAAC.

Based on its review of the FSAR, the staff determined that the applicant has not provided sufficient information related to the pipe break analysis hazard report as described above. Specifically, FSAR Tier 1, ITAAC Table 3.8-1 does not contain any ITAAC item related to pipe break hazard analysis report. In addition, FSAR Tier 2, Section 3.6.2 does not contain a section that lists/summarizes the specific information that will be included in the pipe break hazard analysis report. Moreover, the applicant did not provide any justification and closure milestone for the COL applicant items identified in FSAR Tier 2, Section 3.6.2 as described in RG 1.206, "Combined License Applications for Nuclear Power Plants," Section C.III.4.3, June 2007. Therefore, in RAI 107, Question 03.06.02-17, the staff requested that the applicant provide additional information related to the COL information items and the pipe break hazard analysis report.

In a December 1, 2008, response to RAI 107, Question 03.06.02-17, the applicant did not completely address the staff's concern related to the pipe break hazard analysis report. In this response, the applicant proposed a system based ITAAC in FSAR Tier 1, Table 2.1.1-7, "RBA Penetrations that Contain High Energy Pipelines," of the Nuclear Island for pipe break evaluation. The staff notes that system-based ITAAC includes only ASME Code Class 1, 2, and 3 piping, but not non-safety class piping that is within the scope of SRP Section 3.6.2. Therefore, the staff closed RAI 107, Question 03.06.02-17, and in follow-up RAI 222, Question 03.06.02-31, the staff requested that the applicant address how the system-based ITAAC, as proposed, will include all the piping systems within the scope of SRP Section 3.6.2, to include both as-designed aspect and as-built reconciliation in the ITAAC table for the pipe break hazards analysis report, to include a description in FSAR Tier 2, Section 3.6.2 that clearly outlines the information that will be included in the as-designed pipe break hazards analysis report, and to address the closure milestone of the as-designed pipe break hazards analysis report.

In a September 11, 2009, response to RAI 222, Question 03.06.02-31, the applicant further clarified its positions. First, the applicant included the pipe break hazards analysis ITAAC as part of the structure ITAAC, which the staff determined unacceptable. Second, the applicant stated that the inspection of the as-installed configuration of the pipe break analysis protection features will be performed against the construction drawings such that they agree with the construction drawings. The staff determined that this was not acceptable, since the as-built reconciliation should be performed using the as-built information against the as-designed drawings as opposed to construction drawings. Finally, the staff notes that the applicant needs to clarify whether the COL applicant may either follow the standard ITAAC closure schedule as set forth in 10 CFR 52.99, "Inspection during construction," or propose a plant-specific closure schedule. Therefore, the staff closed RAI 222, Question 03.06.02-31, and in follow-up RAI 354, Question 03.06.02-42, the staff requested that the applicant provide further clarification on the COL applicant's responsibility.

In an August 31, 2010, response to RAI 354, Question 03.06.02-42, the applicant added a non-system based ITAAC Table 3.8-1, "Piping Hazard Analysis ITAAC," which includes both as-designed aspect and as-built reconciliation of pipe break hazards analysis. In addition, the applicant revised both COL Information Items 3.6-1 and 3.6-2 in FSAR Tier 2, Table 1.8-2 to clarify that the as-built pipe break reconciliation will be performed against the as-designed pipe break hazard analysis. Furthermore, in FSAR Tier 2, Section 3.6.2.1, the applicant gave the information which will be provided in the pipe break hazards analysis report.

Based on its review of the above information, the staff finds that the applicant has addressed many of the staff's concerns described in RAI 354, Question 03.06.02-42. However, in FSAR Tier 1, Section 3.8, Table 3.8-1, "Piping Hazard Analysis ITAAC," the applicant makes reference to the completion of the pipe break hazards analyses summary. The staff determined the proposed wording unacceptable. In order to demonstrate that all SSCs, that are needed to perform a safety-related function or are needed to safely shutdown the plant, are protected against or qualified to withstand the dynamic and environmental effects associated with postulated pipe breaks, the applicant needs to complete the pipe break hazards analyses report, as described in FSAR Tier 2, Section 3.6.1, "Plant Design for Protection Against Postulated Piping Failures in Fluid Systems Outside of Containment," and Section 3.6.2, not a summary. In addition, the applicant's proposed content of the pipe break hazards analysis report does not explicitly include the evaluation of a non-mechanistic longitudinal pipe break of a one square foot cross-sectional area within the pipe break exclusion zone, as recommended in SRP Section 3.6.1, and as discussed in FSAR Tier 2, Section 3.6.1.1.6, "Containment Penetration Exclusion Zones."

Therefore, the staff closed RAI 354, Question 03.06.02-42, and in RAI 503, Questions 03.06.01-11 and 03.06.01-12, the staff requested that the applicant address the above two staff concerns. The staff addressed its evaluation of these two issues in Section 3.6.1 of this report.

With respect to the closure milestone of the as-designed pipe break hazards analysis report, the staff finds that the applicant's response to RAI 307, Question 14.03.03-45 has adequately addressed the issue. Specifically, the applicant added COL Information Item 14.3-3 in FSAR Tier 2, Table 1.8-2, which requires that a COL applicant that references the U.S. EPR design certification will identify a plan for implementing design acceptance criteria (DAC). The plan will identify (1) the evaluations that will be performed for DAC, (2) the schedule for performing these evaluations, and (3) the associated design processes and information that will be available to the staff for audit. The staff's evaluation of COL Information Item 14.3-3 is addressed in Section 14.3.3 of this report.

3.6.2.5 *Combined License Information Items*

Table 3.6.2-1 provides a list of determination of rupture locations and dynamic effects associated with the postulated rupture of piping related COL information item numbers and descriptions from FSAR Tier 2, Table 1.8-2:

Table 3.6.2-1 U.S. EPR Combined License Information Items

Item No.	Description	FSAR Tier 2 Section
3.6-2	A COL applicant that references the U.S. EPR design certification will perform the pipe break hazards analysis and reconcile deviations in the as-built configuration to the as-designed analysis.	3.6.2.1
3.6-4	A COL applicant that references the U.S. EPR design certification will provide diagrams showing the final as-designed configurations, locations, and orientations of the pipe whip restraints in relation to break locations in each piping system.	3.6.2.5.1

The staff finds the above listing to be complete and adequately describes actions necessary for the COL applicant. However, based on its review of COL Information Item 3.6-2, the staff issued RAI 533, Question 03.06.01-13, concerning the as-built reconciliation aspect of this COL information item. The resolution of this COL information item will be addressed in Section 3.6.1 of this report. No additional COL information items need to be included in FSAR Tier 2, Table 1.8-2 for determination of rupture locations and dynamic effects associated with the postulated rupture of piping consideration.

3.6.2.6 *Conclusions*

Based on its review, and except for confirmatory item in RAI 354, Question 03.06.02-33 and the open item RAI 354, Question 03.06.02-35, the staff finds that the criteria for postulating pipe rupture and crack locations, and the methodology for evaluating the subsequent environmental and dynamic effects on safety-related SSCs resulting from these ruptures comply with the relevant requirements of 10 CFR Part 50, Appendix A, GDC 4, "Environmental and Dynamic Effects Design Bases." Elements of the submission that are acceptable include the following:

The proposed pipe rupture locations will be adequately determined using the staff-approved criteria and guidelines given in BTP 3-4. The applicant has sufficiently and adequately defined the design methods for high-energy mitigation devices and the measures to deal with the subsequent dynamic effects of pipe whip and jet impingement to provide adequate assurance that, upon completion of the high-energy line break analyses as part of the ITAAC process, the ability of safety-related SSCs to perform their safety functions will not be impaired by the postulated pipe ruptures.

The arrangement of piping and restraints and the final design considerations for high- and moderate-energy fluid systems inside and outside the containment, including the reactor coolant pressure boundary (RCPB), will be the responsibility of the COL applicant. These staff-approved high-energy and moderate-energy line break criteria and guidelines will be used to assure that the SSCs important to safety that are in close proximity to the postulated pipe ruptures will be adequately protected. Using these criteria and guidelines will ensure that the consequences of pipe ruptures will be adequately mitigated so that the reactor can safely be shut down and maintained in a safe-shutdown condition in the event of a postulated rupture of a high- or moderate-energy piping system inside or outside the containment.

3.6.3 Leak-Before-Break Evaluation Procedures

3.7 Seismic DesignSeismic Design Parameters

3.7.1.1 *Introduction*

For the seismic design of nuclear power plants, 10 CFR Part 50, Appendix A, "General Design Criteria for Nuclear Power Plants," GDC 2, "Design Bases for Protection Against Natural Phenomena," requires that SSCs important to safety be designed to withstand the effects of natural phenomena such as earthquakes, tornados, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety functions. This section of the report describes the certified design ground motion response spectra, design ground motion time histories, percentage of critical damping values, and the supporting subgrade media used in the earthquake analysis of Seismic Category I structures.

3.7.1.2 *Summary of Application*

FSAR Tier 1: The FSAR Tier 1 information associated with this section is found in FSAR Tier 1, Section 5.0, "Site Parameters." FSAR Tier 1, Table 5.0-1, "Site Parameters for the U.S. EPR Design," specifies seismic and soil related parameters that were used in the U.S. EPR Certified Design.

FSAR Tier 2: The applicant has provided in FSAR Tier 2, Section 3.7.1, "Seismic Design Parameters," a description of the seismic design parameters summarized here, in part, as follows:

The SSE design basis for the U.S. EPR consists of three sets of ground input motion response spectra based on the European Utility Requirements (EUR) Document, and a fourth set of spectra which represents a site with high frequency (HF) ground input motion. The set of EUR spectra (EURH, EURM, EURS) correspond to hard, medium, and soft soil conditions, respectively. Each is anchored to 0.30 g peak ground acceleration (PGA). The vertical spectra are identical to the horizontal spectra. The PGA for the high frequency horizontal (HFH) spectra is 0.21 g, and for the high frequency vertical (HFV) spectra the PGA is 0.18 g. Collectively, the EUR and HF spectra make up the certified seismic design response spectra (CSDRS) for the U.S. EPR certified design. The CSDRS seismic input are considered to be outcrop motions located at the foundation level of the NI Common Basemat which is 12.6 m (41.33 ft) below grade. For Seismic Category I structures not on the NI Common Basemat, namely the EPGBs and the Essential Service Water Buildings (ESWBs), the seismic input at the foundation of these structures is based on a modified CSDRS which accounts for the effects of structure-soil-structure interaction (SSSI) due to the seismic response of the NI. The operating-basis earthquake (OBE) is set at one-third of the SSE. There is no specific design for the OBE as its PGA is set at one-third of the PGA for the SSE. The artificial time histories used in the seismic analyses have been developed to envelope each of the spectra that make up the CSDRS and the modified CSDRS. The time history properties such as the cross-correlation coefficients among time history components, the response spectra of the time histories, Arias intensity functions, and maximum values of integrated ground velocities and displacements are computed to show compliance with the requirements for the development of artificial time histories.

The damping values used in seismic analysis of SSCs are in general agreement with the values identified in RG 1.61, "Damping Values for Seismic Design of Nuclear Power Plants,"

Revision 1, March 2007. The exceptions are the damping values for cable trays and for fuel assemblies. The supporting media for Seismic Category I structures consist of soil profiles that are either uniform half-space profiles or layered profiles. Site velocity profiles considered represent a range from those corresponding to a soft soil site to those of a hard or high frequency rock site.

ITAAC: There are no ITAAC items for this section.

3.7.1.3 *Regulatory Basis*

The relevant requirements of NRC regulations for this area, and the associated Acceptance Criteria, are given in NUREG-0800, Section 3.7.1, "Seismic Design Parameters," and are summarized below. NUREG-0800, Section 3.7.1 also identifies review interfaces with other SRP sections.

1. 10 CFR Part 50, Appendix A, GDC 2 - The design basis shall reflect appropriate consideration of the most severe earthquakes that have been historically reported for the site and surrounding area with sufficient margin for the limited accuracy, quantity, and period of time in which historical data have been accumulated.

2. 10 CFR Part 50, Appendix S is applicable to applications for a design certification or combined license pursuant to 10 CFR Part 52, "Licenses, Certifications, And Approvals For Nuclear Power Plants," or a construction permit or operating license pursuant to 10 CFR Part 50 on or after January 10, 1997. For SSE ground motions, SSCs will remain functional and not exceed applicable stress, strain, and deformation limits. The required safety functions of SSCs must be assured during and after the vibratory ground motion through design, testing, or qualification methods. The evaluation must take into account soil-structure interaction effects and the expected duration of the vibratory motion. If the OBE is set at one-third or less of the SSE, an explicit analysis or design is not required. If the OBE is set at a value greater than one-third of the SSE, an analysis and design must be performed to demonstrate that the applicable stress, strain, and deformation limits are satisfied. 10 CFR Part 50, Appendix S also requires that the horizontal component of the SSE ground motion in the free-field at the foundation level of the structures must be an appropriate response spectrum with a peak ground acceleration of at least 0.1 g.

3. 10 CFR 52.47(a)(1), "Contents of applications, technical information," requires a design certification applicant provide site parameters postulated for the design.

Acceptance Criteria adequate to meet the above requirements include:

1. RG 1.60, "Design Response Spectra for Seismic Design of Nuclear Power Plants," Revision 1, December 1973, as it relates to defining the seismic response spectra for a nuclear power plant site.

2. RG 1.61, "Damping Values for Seismic Design of Nuclear Power Plants," March 2007, as it relates to damping values assigned to Seismic Category I SSCs, including material damping and system damping.

3.7.1.4 *Technical Evaluation*

In this section of the report the staff discusses the procedures used to develop the seismic design parameters which are the basic inputs for the seismic analysis of safety-related Seismic Category I structures. The information provided by the applicant includes a description of the design ground motion, percentage of critical damping values, and supporting media for Seismic Category I structures. These procedures are reviewed against the acceptance criteria of SRP Section 3.7.1 and the regulatory guides referenced above.

3.7.1.4.1 Design Ground Motion

In this section of the report the staff reviewed the ground motion inputs used as the basis for the seismic analysis of the building structures that make up the EPR certified design. In FSAR Tier 2, Section 3.7.1.1, "Design Ground Motion," three design ground motion inputs were specified. Each was represented by a ground motion response spectrum. These were based on the EUR document.

In a June 22, 2011, response to RAI 320, Question 03.07.02-63, the applicant added in FSAR Tier 2, Section 3.7.1.1, "Design Ground Motion," Revision 3-interim, an additional ground motion input representing the ground acceleration for a high frequency rock site. The description of the EUR and HF design ground motions includes the design ground motion response spectra and the development of the design ground motion time histories.

The applicant has set the OBE ground acceleration at one-third of the SSE value. As a result, an explicit design for the OBE is not performed; this complies with 10 CFR Part 50, Appendix S when the OBE is set at one-third or less of the SSE.

3.7.1.4.2 Design Ground Motion Response Spectra

The CSDRS shown in FSAR Tier 2, Section 3.7.1, Figure 3.7.1-1, "Design Response Spectra for EUR Control Motions (hard, medium and soft sites)," Revision 3 interim response, provided in the response to RAI 320, Question 03.07.02-63 consists of three sets of EUR ground motion response spectra corresponding to hard, medium, and soft soil sites plus an additional set of control motion response spectra to represent a HF Central Eastern U.S. (CEUS) rock site. This latter set of spectra is representative of the design spectra for the Bell Bend Nuclear Plant site in Luzerne County, PA. The EUR control motions are anchored at 0.3 g peak ground acceleration in both the horizontal and vertical directions, while the HF control motions are anchored at 0.21 g in the horizontal direction and 0.18 g in the vertical direction. The CSDRS provide the basis for the time histories used in the seismic analysis of the NI Common Basemat Structures. The CSDRS also provide the basis for the modified CSDRS used in the seismic analysis of the EPGB and ESWB.

The staff reviewed the comparisons of the three EUR spectra with the RG 1.60 spectra as shown in FSAR Tier 2, Figure 3.7.1-2, "Comparison of CSDRS to RG 1.60 and the Minimum Required Spectrum, Horizontal Motion," and FSAR Tier 2 Figure 3.7.1-3, "Comparison of CSDRS to RG 1.60, Vertical Motion," for a damping value of five percent. Although each of the EUR control-motion spectra is similar to the RG 1.60 spectra, there are some differences in that the RG 1.60 spectra exceeds the envelope of the EUR horizontal spectra at frequencies below about 3 hertz (Hz) and exceed the envelope of the EUR vertical spectra at frequencies below 0.65 Hz. However, the accelerations at these frequencies are comparatively low. In addition, these frequencies are below the fundamental frequency for most safety-related SSCs. Accordingly, the CSDRS exceedence of the RG 1.60 spectra at these low frequencies will not

have a significant effect on SSCs important to safety in the event of an SSE. Due to the similarity of the EUR spectra to the RG 1.60 spectra and the fact that HF spectra have been added to represent earthquake input motion for a CEUS rock site, the certified design is based on a broad frequency range of seismic input. Therefore, the staff finds the use of the three EUR spectra and HF spectra as the basis for the ground motion input to the U.S. EPR certified design acceptable. **RAI 320, Question 03.07.02-63 is being tracked as a confirmatory item,** with respect to the control motions, pending revision of the FSAR to incorporate the proposed changes.

The CSDRS were not included in FSAR Tier 1, Section 5.0. Since the CSDRS constitute the earthquake design basis for the U.S. EPR certified design, in RAI 130, Question 03.07.01-2, the staff requested that the applicant include figures depicting the CSDRS for five percent damping in FSAR Tier 1, Chapter 5.0, "Site Parameters."

In a February 20, 2009, response to RAI 130, Question 03.07.01-2, the applicant indicated that the control motions will be added to FSAR Tier 1, Figure 5.0-1, "Design Response Spectra for EUR Control Motions (Hard, Medium and Soft Sites)." The staff confirmed that FSAR Tier 1, Section 5.0, Revision 1, dated May 29, 2009, contained the changes committed to in the RAI response, including EUR design response spectra (EURS, EURM, and EURH). Accordingly, the staff finds that the applicant has adequately addressed this issue and, therefore, the staff considers RAI 130, Question 03.07.01-2 resolved. Later in the response to RAI 320, Question 03.07.02-63, the applicant updated FSAR Tier 1, Figure 5.0-1, to include the HFH and HFV design spectra which as discussed previously, the staff also finds acceptable.

In the staff's review of FSAR Tier 2, Section 3.7.1.1.1, "Design Ground Motion Response Spectra," it was noted that the NI was analyzed as a surface founded structure even though the common basemat is embedded to a depth of 12.60 m (41.3 ft) below grade. According to SRP Acceptance Criteria 3.7.2.II.4.B, the effect of embedment of the structure should be accounted for in its soil-structure interaction (SSI) analysis. Therefore, in RAI 201, Question 03.07.01-19, the staff requested that the applicant provide justification for neglecting the depth of embedment on SSI results and quantify the impact on structural design loads, as well as on the computation of in-structure response spectra (ISRS).

In an August 18, 2009, response to RAI 201, Question 03.07.01-19, the applicant included a statement that a parametric analysis had been performed to determine the embedment effect on the NI Common Basemat Structures seismic analysis. The applicant stated that the embedded model produced ISRS that were not significantly different from those generated using the surface founded model. In follow-up RAI 291, Question 03.07.01-26, the staff requested that the applicant provide a detailed description of the parametric analyses that were performed including the definition of the input motion and how it was derived for the embedded model. The staff also requested that the applicant provide the results from the parametric studies and compare these with the results computed for the surface founded NI Common Basemat Structures.

In a February 11, 2010, response to RAI 291, Question 03.07.01-26, the applicant stated that the NI Common Basemat Structures surface founded stick model had been replaced by an embedded finite element model. Therefore, the difference in results between a surface founded and an embedded stick model were no longer applicable. The applicant described the changes in the SSI methodology in the June 22, 2011, response to RAI 320, Question 03.07.02-63. These changes were made to address both the effects of embedment and to assure the structural model was sufficiently detailed to account for the effects of the HF input motion on its

dynamic response. The NI is a complex structure which consists of a number of buildings supported on a common basemat. A finite element model provides a more accurate representation of both the geometry and stiffness of the NI and is more appropriate for analysis when treated as an embedded structure in an SSI analysis. Therefore, the staff finds the change to a finite element model acceptable. The finite element model of the NI is discussed further in Section 3.7.2 of this report. Based on the change in methodology which includes embedment effects, the staff considers RAI 201, Question 03.07.01-19 and RAI 291, Question 03.07.01-26 resolved. As described in Section 3.7.2 of this report, the staff reviewed the revised seismic analysis of the NI Common Basemat Structures as described in the response to RAI 320, Question 03.07.02-63, and finds it acceptable. **RAI 320, Question 03.07.02-63 is being tracked as a confirmatory item**, with respect to the revised seismic analysis, pending revision of the FSAR to incorporate the proposed changes.

To determine the seismic input for the EPGB and ESWB, the CSDRS are modified to account for the SSSI effect created by the seismic response of the NI Common Basemat Structures. As discussed above, the original seismic analysis described in FSAR Tier 2, Section 3.7.1.1.1, treated the NI as a surface-founded structure. In determining the SSSI effect, the basemats of both the EPGB and the ESWB were also assumed to be surface founded. However, the actual elevation for the EPGB is at grade, and the ESWB is embedded at 6.70 m (22 ft) below grade. Both of these elevations are higher than the actual NI common basemat foundation elevation. Consequently, the SSSI effect for the EPGB and ESWB used to determine the modified CSDRS did not include the effect of the differences in foundation elevations between these structures and the NI. Therefore, in RAI 130, Question 03.07.01-1, the staff requested that the applicant provide the basis for not accounting for the differences in elevations of these structures and that the impact on the development of the modified CSDRS be addressed.

In a February 20, 2009, response to RAI 130, Question 03.07.01-1, the applicant indicated that the SSSI effect on the input motion to the EPGB and ESWB would be addressed by the COL applicant, as required by COL Information Item 2.5-3 and as described in FSAR Tier 2, Section 2.5.2.6, “Ground Motion Response Spectrum.” Subsequent to this response, in a June 22, 2011, response to RAI 320, Question 03.07.02-63, the applicant presented a revised analysis of the NI described in FSAR Tier 2, Section 3.7.1.1, Revision 3-interim. In this revised analysis, the NI is modeled as an embedded structure. The SSSI effect on the EPGB and ESWB is determined by calculating the difference in surface ground response at the locations of the EPGB and ESWB over that of the surface input motion associated with the CSDRS. This modification of the CSDRS for the EPGB and ESWB is described in the applicant’s July 22, 2011, interim response to RAI 376, Question 03.08.05-31, which mainly deals with procedures to determine static and dynamic bearing pressures for the EPGB and ESWB and procedures used to calculate the minimum factors of safety for these structures (discussed in Section 3.8.5 of this report), but includes the modification of the CSDRS for SSSI effects as described FSAR Tier 2, Revision 3-interim, Section 3.7.1.1.1 attached to the response. For the EUR spectra, the SSSI effect is accounted for by determining in the seismic analysis of embedded NI Common Basemat Structures, the modified response spectra of the ground surface at footprint locations of the EPGB and ESWB for each of the NI EUR analysis cases. The modified EUR CSDRS for the EPGB and ESWB analysis is an envelope of the individual envelopes for each of these structures determined from the NI results for all of the EUR analysis cases. In the revised analysis, the SSSI effect for the EPGB and ESWB is now determined from an analysis in which the NI is modeled as an embedded structure. Thus, any effects due to elevation differences between the NI basemat and ground surface are accounted for in the analysis. In addition, the modified CSDRS for the EPGB and ESWB is an envelope of the SSSI effect for each EUR

CSDRS and for each structure which is conservative. Therefore, the staff finds the development of the modified CSDRS of the EPGB and ESWB for the EUR spectra acceptable.

The modified CSDRS for the HF input accounts for the difference in elevations directly in that foundation input response spectra (FIRS) at the basemat elevations for both the EPGB and ESWB are determined probabilistically as part of the Bell Bend site response analysis without the effect of the NI. The SSSI effect from the response of the NI is determined by SSSI amplification factors which are determined by dividing the response spectra on the surface at the footprint locations of the EPGB and ESWB by the input surface response spectra of the NI. The SSSI amplification factors are then applied to the FIRS for the EPGB and ESWB. In turn, these are then enveloped and define the modified CSDRS for the HF input motion.With the exception of determining the FIRS for the EPGB and ESWB directly for the HF input motion case, the process used to determine the SSSI effect is similar to that used for the EUR input motion cases. Since the modified CSDRS for both the EUR and HF input cases are determined directly from the SSI SASSI analysis of the NI which accounts for the difference in elevation between the NI and that of the EPGB and ESWB, and because the modified CSDRS conservatively represents an envelope of the SSSI effect for both the EPGB and ESWB, the staff finds the process described in FSAR Tier 2, Revision 3-interim, Section 3.7.1.1.1 acceptable. The staff considers RAI 130, Questions 03.07.01-1 resolved in that the method of determining the SSSI effect of the NI on the EPGB and ESWB has been revised as provided in the interim response to RAI 376, Question 03.08.05-31. Nonetheless, **RAI 376, Question 03.08.05-31 is being tracked as an open item** with respect to other issues raised in that question as discussed in Section 3.8.5 of this report, and to confirm that the FSAR is revised to incorporate the proposed change discussed above. Since the method described provides an acceptable way to determine the modified CSDRS, and since the modified CSDRS represents an envelope of the SSSI effect from each of the NI analyses, the staff finds them acceptable. Therefore, the staff considers RAI 130, Question 03.07.01-1 resolved**. RAI 376, Question 03.08.05-31 is being tracked as an open item** pending responses to other portions of this question and revision of the FSAR to incorporate the proposed change discussed above.

10 CFR Part 50, Appendix S requires 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 0.1 g. For the U.S. EPR standard plant, the NI minimum design basis spectra is defined as an envelope of the three EUR design response spectra scaled to a peak ground acceleration of 0.1 g. The staff reviewed the applicant's comparison between the CSDRS used as input for the NI Common Basemat Structures with the minimum required spectra anchored at 0.1 g in FSAR Tier 2, Figure 3.7.1-2, Revision 3-interim provided with the response to RAI 320, Question 03.07.02-63. Although the 10 CFR Part 50, Appendix S minimum required response spectra as defined by the applicant is not bounded by the HF horizontal spectra at frequencies below 8 Hz, the staff concludes this is acceptable, because the U.S. EPR certified design is based on an envelope of spectra (EUR soft, medium, and hard plus HF) which together bound the 10 CFR Part 50, Appendix S minimum spectra. Since the input for the EPGB and ESWB are amplified by SSSI effects over that of the NI input, the envelope of the ground motion input spectra for the EPGB and ESWB have a minimum horizontal ground acceleration of at least 0.10 g and thus also envelope the 10 CFR Part 50, Appendix S minimum horizontal spectra. Therefore, the staff finds the design response spectra for the EPGB and ESWB also meet the requirement of 10 CFR Part 50, Appendix S regarding minimum horizontal spectra, and are therefore acceptable.

3.7.1.4.3 Design Ground Motion Time History

In this section, the staff reviewed the methods for the development of artificial time histories which are used in the SSI analysis of the Seismic Category I and Seismic Category II structures of the EPR certified design. FSAR Tier 2, Section 3.7.1.1.2, "Design Ground Motion Time History," described three statistically independent sets of artificial time histories that are generated to envelope the three EUR control motions of the CSDRS. The three components of each set are designated according to their respective control motion. For example, EURH1 and EURH2 are the orthogonal EUR horizontal control motions for a hard soil site, while EURH3 designates the vertical control motion. In FSAR Tier 2, Revision 3-interim markup provided with the response to RAI 320, Question 03.07.02-63, an additional set of time histories is described and represents, the HF input motion for a rock site. These time histories are based on the EUR and HF response spectra are used as the basis for the input motion to the SSI analysis of the NI. For the EPGBs and ESWBs, there are two sets of modified CSDRS. The first set is an envelope of EUR soft, EUR medium, and EUR hard response spectra modified to account for basemat elevation and SSSI effects; and the second set represents the HF response spectra also modified to account for basemat elevation and SSSI effects. The applicant developed one set of statistically independent artificial time histories to envelope the modified CSDRS based on the EUR response spectra and a second set to envelope the modified CSDRS based on the HF input response spectra.

The development of the artificial time histories used in the U.S. EPR analysis was reviewed by the staff against SRP Acceptance Criteria 3.7.1.II.B for design time histories summarized below. The time histories used comply with SRP Acceptance Criteria 3.7.1. II.B, Option 1, Approach 2. These acceptance criteria are as summarized below and comply with these criteria. In addition, the artificial ground motion time histories meet the SRP Acceptance Criteria 3.7.1.II.B Option 1, Approach 2. These acceptance criteria are:

- The time history shall have a sufficiently small time increment and sufficiently long duration with a minimum Nyquist frequency of at least 50 Hz and a total duration of 20 seconds. The U.S. EPR time histories have a Nyquist frequency of 100 Hz and time history duration in excess of 20 seconds and meet the acceptance criteria.

- Spectral acceleration at five percent damping shall be computed at a minimum of 100 points per frequency decade uniformly spaced over the log frequency scale from 1 to 50 Hz. FSAR Tier 2, Section 3.7.1.1.2, states that response spectra (for five percent damping) are computed at a minimum of 100 points per frequency decade and, therefore, meet the SRP acceptance criteria.

- The computed five percent damped response spectra shall not fall more than 10 percent below the target response spectrum at any one frequency. Response spectra at no more than nine adjacent frequency points may fall below the target response spectrum. The spectra meet the acceptance criteria.

- The computed five percent damped response spectrum of the artificial ground motion time history shall not exceed the target response spectrum at any frequency range by more than 30 percent in the frequency range of interest. The spectra meet the acceptance criteria.

Since the generation of artificial time histories meets SRP Acceptance Criteria 3.7.1.II.B, Option 1, Approach 2, the staff finds them acceptable. With respect to the development of

ground motion time histories, **RAI 320, Question 03.07.02-63 is being tracked as a confirmatory item,** pending revision of the FSAR to incorporate the proposed changes.

According to SRP Acceptance Criteria 3.7.1.II.1.B, the ratios of V/A and AD/V^2 (V, A and D represent the maximum ground velocity, the maximum ground acceleration, and the maximum ground displacement, respectively) should be consistent with characteristic values for the magnitude and distance of the appropriate controlling events defining the uniform hazard response spectra. In FSAR Tier 2, Section 3.7.1.1.2, the characteristics (V/A and AD/V^2) of artificial time histories are described as generally consistent with the characteristic values for the magnitude and distance of "the appropriate controlling" events defined for the uniform hazard response spectra (UHRS). In RAI 130, Question 03.07.01-9, the staff requested that the applicant provide further clarification with regard to the magnitude and distance of the controlling event or add to the FSAR a COL information item which identifies that the COL applicant will provide this information as part of the site-specific analysis.

In a February 20, 2009, response to RAI 130, Question 03.07.01-9, the applicant indicated that for generic site conditions in certified standard plants, the properties (design input ground motion) are assumed values. These assumed properties will be reconciled by the COL applicant stated in COL Information Item 2.5-3 and 3.7-1 and further described in FSAR Tier 2, Section 2.5.2.6 and Section 3.7.2, "Seismic System Analysis," respectively. Furthermore, the 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 the site-specific ground motion can be calculated, the controlling event must be identified and its characteristics defined. The applicant concluded that no additional COL information items are required. The staff concludes that since it is the responsibility of the COL applicant to establish the seismic hazard for the site in accordance with 10 CFR 100.23 and reconcile site-specific seismic response with the seismic response as defined in the reconciliation guidelines of FSAR Tier 2, Section 2.5.2.6, the staff finds the applicant's response acceptable. Therefore, the staff considers RAI 130, Question 03.07.01-9 resolved.

Artificial time histories used for dynamic earthquake response analyses are typically developed from a seed record which is used to define the phase content of the artificial time history. (A seed record is an actual earthquake time history or representative earthquake time history record.) SRP Acceptance Criteria 3.7.1.II.1.B, states that artificial time histories should not be used unless the artificial time histories are based on a seed record time history. This provision is to ensure that the artificial time histories contain the characteristics of actual earthquakes. As the basis for the artificial time histories wasn't provided in the FSAR, in RAI 130, Question 03.07.01-8, the staff requested that the applicant confirm the artificial time histories used in the U.S. EPR design are based on seed recorded time histories.

In an April 17, 2009, response to RAI 130, Question 03.07.01-8, the applicant stated that the artificial time histories described in FSAR Tier 2, Section 3.7.1.1.2 are based on seed recorded time histories and comply with the guidance in SRP Acceptance Criteria 3.7.1.II.1.B. The staff finds this response acceptable and, therefore, considers RAI 130, Question 03.07.01-8 resolved.

3.7.1.4.4 Percentage of Critical Damping

The staff reviewed the specific percentages of critical damping values used in the analyses of Category I SSCs in accordance with RG 1.61. FSAR Tier 2, Table 3.7.1-1, "Damping Values for Safe Shutdown Earthquake," provides damping values used for the SSE analyses. In general,

the values in the table agree with those in RG 1.61. The exceptions are discussed in this section of the report.

FSAR Tier 2, Table 3.7.1-1 lists the maximum damping value to be used for fuel assemblies as 30 percent. This value is taken from Framatome Topical Report BAW 10133PA-01 including Addendum 1 and 2 which has been accepted by the NRC. Therefore, the staff finds the maximum damping value of 30 percent acceptable.

RG 1.61 states that the use of SSE damping values is acceptable for developing SSE seismic structural loads regardless of the actual level of stress during a seismic event. However, for generating the ISRS the RG recommends the use of OBE damping values. The ISRS for the NI Common Basemat Structures are generated using SSE damping values rather than the OBE damping values as suggested in RG 1.61, Table 2. The applicant states in FSAR Tier 2, Section 3.7.1.2 that the reason this was done is that the NI seismic input coupled with the broad range of soil cases has resulted in high structural loads on both the walls and floors of the NI structures. Based on this outcome, the FSAR indicates that it is appropriate to use SSE structural damping for the NI Common Basemat Structures to generate the ISRS. SRP Acceptance Criteria 3.7.1.II.2, states that damping values different from those given in RG 1.61 may be used if supported by test data and a demonstration that there is a correlation between the stress levels and the damping values selected. Therefore, in RAI 130, Question 03.07.01-11, the staff requested that the applicant provide the computed stress levels (attributed to load combination using the SSE) for the major load carrying members of the NI Common Basemat Structures to substantiate the justification for using SSE structural damping values to generate the ISRS.

In an April 17, 2009, response to RAI 130, Question 03.07.01-11, the applicant stated that the calculated stress for critical sections shows that stress levels for combinations using the SSE are consistent with the use of SSE damping values and meet RG 1.61, Revision 1 guidelines. However, the applicant did not provide the stress levels that support the use of these damping values which the staff had requested in RAI 130, Question 03.07.01-11. Since the requested information was not provided, in follow-up RAI 248, Question 03.07.01-25, the staff requested that the applicant provide a table of computed stresses for each of the NI structures, including representative examples of stresses in both walls and floors and a comparison of these stresses to respective code allowable stresses. In addition, the staff requested that the applicant provide a technical justification for the damping value selected for each of the structures based on the comparison of actual stress levels to code allowable stresses.

In a December 18, 2009, response to RAI 248, Question 03.07.01-25 the applicant presented comparisons of critical section strength demands and corresponding section capacities for the controlling in-plane shear, out-of-plane shear, and combined axial and bending stresses. The section capacities were based on ACI 349, “Code Requirements for Nuclear Safety-Related Concrete Structures.” These comparisons considered load combinations containing SSE dynamic loads and the range of EUR soil cases used for the U.S. EPR design.

In NUREG/CR-6919, “Recommendations for Revision of Seismic Damping Values in Regulatory Guide 1.61,” Page 6 states, “If significant stresses due to load combinations that include SSE are less than 80 percent of the applicable code stress limits, then using SSE damping values may under-predict the structure’s response to seismic loads. In this case structural evaluation and development of ISRS should be based on a seismic analysis utilizing the OBE damping values specified in Table 2.” The OBE damping value in RG 1.61, Table 2 recommended for reinforced concrete is four percent. In the tables provided in the applicant’s response, there are

only 19 instances out of the 102 examples where one of the load components results in a structural demand that exceeds 80 percent of the section's design capacity. In addition, the examples provided are at the critical sections of the NI Common Basemat Structures, suggesting that demands at other locations are lower than those presented in the applicant's response. Due to the low number of instances where the structural demand exceeded 80 percent of design capacity, in RAI 370, Question 03.07.01-27, the staff requested that the applicant provide additional information on the state of stresses within the NI Common Basemat Structures to support its position on the use of SSE structural damping values for generating the ISRS. In addition, the staff requested that the applicant include the results from the HF input motion, as appropriate Based on the above discussion, the staff concluded that RAI 130, Question 03.07.01-11 and RAI 248, Question 03.07.01-25 are closed and superseded by **RAI 370, Question 03.07.01-27 which is being tracked as an open item** pending the completion of the applicant's analysis and review of the applicant's final response.

FSAR Tier 2, Section 3.7.1.2, "Percentage of Critical Damping Values," describes the ISRS generated for the EPGB and the ESWB. These are based on OBE structural damping values. The staff notes that this conforms to RG 1.61 and finds this acceptable. However, in RAI 130, Question 03.07.01-12, the staff requested that the applicant incorporate into the FSAR an appropriate table showing the specific OBE level structural damping values used for the analysis. The staff confirmed that FSAR Tier 2, Section 3.7.1.2, Revision 2, dated August 31, 2010, provided the requested table. Accordingly, the staff finds that the applicant has adequately addressed this issue and, therefore, considers RAI 130, Question 03.07.01-12 resolved.

FSAR Tier 2, Table 3.7.1-1 lists the damping value of piping systems in uniform support motion response spectrum analysis as five percent. The five percent damping value is different from the value accepted by the staff in RG 1.61, Revision 1, which lists acceptable damping values of four percent for the SSE and three percent for the OBE. Technical justification for the use of a higher damping value was provided in the applicant's November 20, 2007, response to RAI-EPR-26 regarding the AREVA Piping Analysis Topical Report ANP-10264NP. The applicant provided an April 18, 2008, revised response to RAI-EPR-26 for ANP-10264NP, in which AREVA committed to use damping values given in RG 1.61, Revision 1 for uniform support motion response spectrum analysis. As such, the staff determined that FSAR Tier 2, Table 3.7.1-1 should be revised to confirm the commitment of using the damping value given in RG 1.61, Revision 1. In addition, in FSAR Tier 2, Table 1.9-2, "U.S. EPR Conformance with Regulatory Guides," the EXCEPTION in the row, "1.61, R1," should also be revised to reflect this commitment. As a result, in RAI 130, Question 03.07.01-10, the staff requested that the applicant resolve these issues.

In an April 17, 2009, response to RAI 130, Question 03.07.01-10, the applicant indicated that FSAR Tier 2, Table 3.7.1-1, "Damping Values for Safe Shutdown Earthquake," had been revised to confirm the commitment of using the damping value given in RG 1.61, Revision 1. In addition, FSAR Tier 2, Table 1.9-2 had been revised to delete the exception to RG 1.61, Revision 1. Since Revision 2 of the FSAR reflects damping values in conformance with RG 1.61, and the exception in FSAR Tier 2, Table 1.9-2 has been deleted, the staff finds the response acceptable and, therefore, considers RAI 130, Question 03.07.01-10 resolved.

SRP Acceptance Criteria 3.7.1.II.2, states that the material soil damping values for foundation soils should 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 values should be limited to 15 percent. Since this information was not provided in the FSAR, in RAI 130,

Question 03.07.01-6, the staff requested that FSAR Tier 2, Table 3.7.1-6, "Generic Soil Profiles for the U.S. EPR Standard Plant," be revised to specify the damping values used in the SSI analysis for the corresponding generic soil profiles and indicate whether these values are strain compatible values and do not exceed 15 percent.

In an April 17, 2009, response to RAI 130, Question 03.07.01-6, the applicant states that FSAR Tier 2, Table 3.7.2-9, "Soil Properties Associated with Different Generic Shear Wave Velocities," shows damping values used in the soil-structure interaction analysis for the corresponding generic profiles. As a clarification, the applicant stated that FSAR Tier 2, Table 3.7.1-6 will be revised to include a note that references FSAR Tier 2, Table 3.7.2-9 for applied damping values. The applicant also stated that FSAR Tier 2, 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. Based on the clarifications made in Revision 1 to the FSAR regarding the fact that the soil shear wave velocities are strain compatible and the fact that the soil damping values do not exceed 15 percent, the staff finds the response acceptable and, therefore, considers RAI 130, Question 03.07.01-6 resolved.

FSAR Tier 2, Table 3.7.1- lists damping values for the reactor coolant system. A damping value of seven percent is listed for the reactor pressure vessel (RPV) closure head equipment tie rods which is appropriate for a bearing connection, but not for a friction connection. Therefore, in RAI 130, Question 03.07.01-13, the staff requested that the applicant verify that the tie rod connection represents a bearing connection as opposed to a friction connection.

In a February 20, 2009, response to RAI 130, Question 03.07.01-13, the applicant indicated that closure head equipment tie rod ends are connected to supporting elements with pins and clevises using bearing connections. Based on this response, the staff concluded that the use of a damping value of seven percent is appropriate. Therefore, the staff considers RAI 130, Question 03.07.01-13 resolved.

The seismic analysis of the RCS consists of a linear modal analysis supported by a nonlinear analysis to determine the effect of gaps in the support system. The nonlinear analysis uses Rayleigh mass and stiffness damping coefficients. FSAR Tier 2, Section 3.7.1.2 indicates that the Rayleigh mass and stiffness damping coefficients are selected to provide generally conservative damping across the frequency range of interest relative to the damping values provided in FSAR Tier 2, Table 3.7.1-1. Since Rayleigh damping coefficients are affected by the specified frequency range of interest, in RAI 130, Question 03.07.01-14, the staff requested that the applicant specify the frequency range of interest for the calculations to assure that the frequency range is appropriate and the damping used in the nonlinear analysis is conservative.

In a February 20, 2009, response to RAI 130, Question 03.07.01-14, the applicant stated that, because a nonlinear model was used with a step-by-step time history integration solution, natural frequencies of the system are automatically incorporated 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. The staff's assessment was that the computation of Rayleigh mass and stiffness weighted damping coefficients requires specifying an effective damping value for a given frequency range. In the absence of selecting a frequency range of interest, it was unclear to the staff whether the Raleigh mass and stiffness weighted damping matrix was conservatively developed and appropriate for the nonlinear analysis. Therefore, in follow-up RAI 215, Question 03.07.01-22, the staff requested that the applicant describe the computation method, equation, damping

values, and assumptions used to determine the Rayleigh mass and stiffness weighting damping coefficients.

In an October 20, 2009, response to RAI 215, Question 03.07.01-22, the applicant stated that the computation methods, equations, damping values, and assumptions used to determine Rayleigh mass and stiffness damping coefficients, specific to the RCS, are provided in FSAR Tier 2, Appendix 3C, "Reactor Coolant System Structural Analysis Methods." In FSAR Tier 2, Figure 3C-9, "Rayleigh Damping Curve," two curves of damping value as a function of frequency are presented. One curve is a Rayleigh damping curve and the other is a modal damping curve. The modal curve is produced (in accordance with RG 1.61, Revision 1) with a linear model to obtain a set of natural frequencies with associated damping values. In the response to RAI 215, Question 03.07.01-22, Figure 3.7.1-22-1, "Modal Mass vs. Rayleigh Damping," the applicant shows modal curve numerical values for the frequency and damping scales. The applicant believes the range of frequencies plotted is sufficient to address significant modal contributions for RCS components. The Rayleigh curve is produced using the equation in FSAR Tier 2, Section 3C.4.2.1.1, "Reactor Coolant System Four Loop Structural Model High Energy Line Break Analysis," with alpha and beta values (see below) that result in conservatively lower values of damping than the corresponding modal values for most of the lower frequencies within the range.

The Rayleigh damping curve is calculated according to the following:

$$\xi_i = \frac{\alpha}{2}\frac{1}{\omega_i} + \frac{\beta}{2}\omega_i$$

In this equation α is the Rayleigh damping mass constant, β is the Rayleigh damping stiffness constant and ω is the modal frequency for the i^{th} mode. The alpha and beta values used to develop the Rayleigh damping curve in Figure 3.7.1-22-1 of the response to RAI 215, Question 03.07.01-22 are 0.9 and 0.00045, respectively. However, in FSAR Tier 2, Appendix 3C.4.2.1.1, these values are not the values used for seismic analysis as was requested by staff, but are the values used for the RCS four-loop high energy line break analysis. In FSAR Tier 2, Appendix 3C.4.2.2.1, a different set of values of alpha and beta are given for the RCS seismic analysis. These values are 1.7 for alpha and 0.00055 for beta. As such, the staff noted there was an inconsistency between the applicant's response to RAI 215, Question 03.07.01-22 and what was provided in FSAR Tier 2, Appendix 3C.4.2.2.1. The use of higher values of alpha and beta can result in damping values that may not be conservative. Therefore, in follow-up RAI 371, Question 03.07.01-29, the staff requested that the applicant clarify why two different sets of alpha and beta values were used and provide justification for the use of the higher damping values in the RCS seismic analysis. In addition, since the HF spectral input is included in the CSDRS for the design certification and since these spectral curves contain significant seismic input above 35 Hz, the applicant was requested to provide a basis for using this value as a cutoff frequency in the RCS seismic analysis.

In an October 5, 2010, response to RAI 371, Question 03.07.01-29, the applicant stated that the high energy line break (HELB) analysis used 0.9 and 0.00045 for alpha and beta values. Due to the severity of the seismic ground accelerations, a set of higher alpha and beta damping values was used for the seismic analysis of the RCS to yield accurate, yet still conservative results. The alpha and beta values used in the seismic analysis of the RCS are 1.7 and 0.00055, respectively. In an October 20, 2009, response to RAI 215, Question 03.07.01-22, which includes RAI 215, Question 03.07.01-22, Figure 03.07.01-29-1, "Comparison of Alpha-Beta /

RG 1.61 Damping-HELB/Seismic,” the applicant compares the alpha-beta damping used for HELB and seismic analyses with the modal damping values computed using RG 1.61.

Recognizing that the alpha-beta damping curves increase significantly as frequencies increase beyond 30 Hz, the applicant indicates that it had evaluated the impact of the modes in this frequency range on the dynamic response of the RCS. For the RCS, approximately 99.5 percent of the effective mass is accounted for by considering the frequencies below 28 Hz. This indicates that the impact on the response of any modes above 28 Hz is negligible. In addition, 98 percent of the lateral effective mass (global X and Z directions) is accounted for below 18 Hz. The vertical response (global Y direction), is at a higher frequency due to the rigidity of the vertical supports and component shells in the axial direction.

The linear regression trend in RAI 215, Question 03.07.01-22 Figure 03.07.01-29-1 represents a curve fit for the applied modal damping values. RAI 215, Question 03.07.01-22 Figure 03.07.01-29-1 shows that the seismic alpha-beta damping curve falls below the curve fit up to 20 Hz where more than 98 percent of the lateral effective mass of the RCS components is accounted for. As explained in FSAR Tier 2, Appendix 3C.4.2.2.1, nonlinear and linear seismic time-history analyses of the four loop RCS are performed using the alpha-beta damping values. A load comparison between the linear and nonlinear analyses is performed to arrive at bounding factors. These factors (always greater than 1.0) are applied to the results of a linear modal superposition time history analysis (base runs) of the RCS to bound the effects of nonlinearities in the RCS model. The alpha-beta damping parameters are only used for comparison between the linear and nonlinear analyses for arriving at appropriate bounding factors. The base linear runs are performed using modal damping properties per RG 1.61 and as indicated in FSAR Tier 2, Revision 2, Appendix 3C.4.2.2.1.

In view of the foregoing, the applicant concludes that, the alpha-beta values used in the seismic analysis are conservative. The Rayleigh damping is used to determine bounding factors on the effect on nonlinearities, and these factors are then applied to the results of the linear modal analysis which used RG 1.61 damping. In addition, the Rayleigh damping curve is, for the most part, enveloped by RG 1.61 modal damping values up to frequencies of about 20 Hz, below which 98 percent of the effective lateral mass has been included in the response. Accordingly, the staff finds the use of the alpha-beta damping values used in the seismic analysis to determine bounding factors acceptable.

In addressing the issue of the 35 Hz cutoff frequency, the applicant describes that the impact of these high frequency accelerations on the seismic response of the RCS was evaluated by performing analyses for the Bell Bend lower bound, best estimate and upper bound site profiles. However, the applicant did not provide the results of this evaluation as part of the response to RAI 371, Question 03.07.01-29. The Bell Bend HF spectra exceed that of the EUR hard soil spectra at frequency values between approximately 22 Hz and 70 Hz. Although much of the structural response of the RCS appears to take place at frequencies below 22 Hz, the applicant had failed to demonstrate that the Bell Bend HF spectra have no impact on the results of the RCS structural analysis. Therefore, the staff could not conclude that the applicant had adequately addressed issues concerning the seismic analysis for the RCS. As a result, in RAI 463, Question 03.07.01-30, the staff requested that the applicant provide a comparison of the RCS structural response using the EUR governing cases with that of the Bell Bend lower bound, best estimate and upper bound cases and to include a comparison of RCS amplified response spectra to demonstrate that the EUR input motions control the seismic design of the RCS. In addition, the staff also requested that the applicant update FSAR Tier 2, Appendix 3C.4.2.2.1, to include the Bell Bend cases in the seismic design of the RCS.

In a July 28, 2011, response to RAI 463, Question 03.07.01-30, the applicant described that the Bell Bend cases are included in the seismic design of the RCS. FSAR Tier 2, Revision 3, interim markup, Section 3C.4.2.2, was revised to identify the soil cases in FSAR Tier 2, Table 3.7.1-6, Revision 3 interim markup as providing the input motions for the RCS four loop structural model. FSAR Tier 2, Table 3.7.1-6 has been revised in the response to RAI 320, Question 03.07.02-63 to add the Bell Bend soil cases which are now included in the seismic design basis of the U.S. EPR certified design. To ensure the effect of the high frequency content of Bell Bend input motion is included in the RCS response, a cutoff frequency of 70 Hz is used and a missing mass component is added for frequencies above 70 Hz. Since 70 Hz is well above the significant frequency response of the RCS, the staff finds this acceptable. In a July 30, 2011, response to RAI 201, Question 03.07.02-35, the applicant demonstrated that the Bell Bend input motions do not control the response of the RCS. This is discussed in Section 3.7.2 of this report. For the EUR soil cases, the applicant uses a cutoff frequency of 35 Hz with the missing mass component included for frequencies above 35 Hz. Based on the response spectra for the EUR input motions, the staff finds 35 Hz is an acceptable cutoff frequency and adding the missing mass component accounts for any response of the RCS above 35 Hz. Therefore, on the basis of using acceptable cutoff frequencies and the addition of the missing mass component for both the HF and EUR analysis cases, the staff finds the response to RAI 201, Question 03.07.01-30 acceptable, and therefore, RAI 130, Question 03.07.01-14; RAI 215, Question 3.7.1-22; and RAI 371, Question 03.07.01-29 resolved. **RAI 463, Question 03.07.01-30 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed changes.

Also included in FSAR Tier 2, Table 3.7.1-1 are the damping values for cable trays. This table indicates that for cable tray systems that are represented by Reference 3, a damping value of 15 percent may be used in SSE analysis. Reference 3 is a report 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). While RG 1.61 lists a 10 percent damping value for loaded cable tray systems, RG 1.61 permits the use of higher damping values subject to review and acceptance on a case-by-case basis. Therefore, in RAI 130, Question 03.07.01-15, the staff requested that the applicant provide the technical basis for using a cable tray damping value of greater than 10 percent.

In a February 20, 2009, response to RAI 130, Question 03.07.01-15, the applicant briefly described the results of the test program and indicated that a damping value of 20 percent and higher for cable trays was justified by the Bechtel-ANCO test results. In the staff's review of a summary of the Bechtel-ANCO report, the staff notes that for those systems achieving higher damping the cable tray response was nonlinear and, in some cases, included yielding of the cable tray support system. The report also indicates that the resonant frequency of the cable tray system was dependent on the input level of the support excitation. Higher input levels resulted in lower resonant frequencies. In addition to the use of higher damping values, the staff was concerned about the apparent nonlinear behavior of the cable tray system tested under dynamic load. Therefore, in follow-up RAI 215, Question 03.07.01-23, the staff requested that the applicant provide additional justification for the use of damping values higher than those provided in RG 1.61 and describe how the ANCO test results are applied to the analysis and design of cable tray systems.

In an October 20, 2009, response to RAI 215, Question 03.07.01-23, the applicant stated that the nonlinear cable tray system dynamic behavior is predominantly attributed to amplitude dependent energy dissipation caused by friction between cables, and movement and bouncing

of cables within the tray. The cable tray and its support system components behaved elastically and only cable behavior contributed to the observed nonlinear response. Localized yielding of support connection fittings was observed in a few test cases where the connections experienced large amplitude loading. This is accounted for in analysis and design by modeling connections as flexible joints with appropriately degraded rotational stiffness obtained from experimental tests of representative strut and joint configurations. The response to RAI 215, Question 03.07.01-23, Figure 03.07.01-23 -4, "Comparison of Proposed Damping Ratio Curve with Transverse Lowest Mode Trends in Damping Ratios Using Data from Bechtel-ANCO Report for Rigidly Mounted Cable Tray from Various Cable Tray Manufacturers," presents damping ratios experimentally identified and documented in the Bechtel-ANCO report for rigidly mounted cable tray systems (support fundamental frequency greater than approximately 50Hz) fabricated by different manufacturers that were loaded from near zero of cable fill to 100 percent of full cable loading. From this figure it can be inferred that above an input acceleration of 0.25 g, damping ratios of at least 15 percent are developed in the cable tray system regardless of the tray manufacturer. Since the tray was rigidly mounted during these tests, it was further concluded by the applicant that all of the damping is attributed to the damping of the tray and the damping due to friction between cables and movement and bouncing of cables within the tray. For design purposes, damping ratios that are in conformance with RG 1.61 (up to 10 percent for greater than 50 percent loaded cable trays and 7 percent for empty cable trays) will be used for general cable tray configurations that are different from the configurations that were experimentally tested and reported in the Bechtel-ANCO report. For cable tray systems similar to the configurations tested in the Bechtel-ANCO report, and which are greater than 50 percent loaded, damping ratios of up to 15 percent will be used in accordance with curve 2 in RAI 215, Question 03.07.01-23 response Figure 03.07.01-23 -3, "Damping Ratio Experimental Data from Bechtel-ANCO Report, Damping Curve with Maximum Damping ratio of 20%, Proposed Damping Ratio Curve with Maximum Damping Ratio of 15%, and Damping Ratios Based on RG1.61." However, because of the limited test data available in the longitudinal and vertical directions, the damping ratio of 15 percent is used only for the transverse direction. The applicant states that the damping ratio values in RG 1.61, Revision 1 will be used for developing seismic responses in the longitudinal and vertical directions. Methods in accordance with the guidance in SRP Section 3.7.3 acceptance criteria will be used to analyze the cable tray systems. The cable tray system and anchor system, including their components, are designed to remain within the code allowables.

The staff's assessment of the response to RAI 215, Question 03.07.01-23 is that it was unclear in the FSAR markup that accompanied the response what the specific cable tray configurations were to which the proposed higher damping values would apply and, in follow-up RAI 371, Question 03.07.01-28, the staff requested that the applicant provide suitable cable tray configuration criteria.

In a July 6, 2011, response to RAI 371, Question 03.07.01-28, the applicant provided cable tray system configuration criteria to be used for evaluating the similarity of U.S. EPR cable tray system configurations with the cable tray systems tested in the Bechtel-ANCO test program. The criteria provided included support member type, length, and spacing; fitting types; tray types, widths and spans; and bracing requirements. These criteria were included in FSAR Tier 2, Revision 3-interim, Table 3.7.1-7. The staff concludes these are adequate to define the configurations to which the higher damping would apply.

The staff concludes that if the U.S. EPR cable tray systems satisfy the above criteria, then the damping value of 15 percent for determining the tray response in the transverse direction is adequate because it is supported by the test date in the Bechtel-ANCO report. For the other

two directions (longitudinal and vertical), the applicant will use RG 1.61 damping values. Criteria have been provided in FSAR Tier 2, Revision 3-interim, Table 3.7.1-7, which define the configurations to which the higher damping value applies. These criteria are based on the configurations used in the Bechtel ANCO tests. Therefore, the staff finds the damping values to be used for seismic analysis of cable trays acceptable. The staff considers RAI 130, Question 03.07.02-15 and RAI 215, Question 03.07.01-23 resolved**. RAI 371, Question 03.07.01-28 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed change.

3.7.1.4.5 Supporting Media for Seismic Category I Structures

The staff reviewed the supporting media for Seismic Category I structures used in the seismic analysis for the U.S. EPR standard plant in FSAR Tier 2, Section 3.7.1.3, "Supporting Media for Seismic Category I Structures." The supporting media consisted of ten soil profiles which were identified in FSAR Tier 2, Table 3.7.1-6. These soil profiles were associated with one or more of the three EUR design ground motions used in the U.S. EPR certified design. In a June 22, 2011, response to RAI 320, Question 03.07.02-63, the applicant included three new profiles to represent the HF soil cases. These profiles represented lower bound, best estimate, and upper bound soil properties. In addition, the number of EUR soil profiles was reduced to five, because the applicant determined that a number of the profiles did not govern the response of the Seismic Category I structures. Each soil profile is associated with one of the three EUR generic control motions (i.e., hard, medium, and soft) or with the HF control motion. For the EPGB, the soil profiles were provided in a July 22, 2011, interim response to RAI 376, Question 03.08.05-31. Soil profiles for the ESWB will be provided in the final response to RAI 376, Question 03.08.05-31. The soil profiles for the NI and EPGB represent a wide range of soil properties and include uniform half space profiles and layered soil profiles for the NI. As such, the seismic design for these structures will encompass a wide range of soil the conditions which the staff finds acceptable for the certified design. With respect to soil the profiles for the NI and EPGB, **RAI 320, Question 03.07.02-63 is being tracked as confirmatory items** pending revision of the FSAR to incorporate the proposed changes. **RAI 376, Question 03.08.05-31 is being tracked as an open item** pending responses to other portions of this question as well as for the receipt and review of the soil profiles for the ESWB.

In a review of FSAR Tier 2, Section 3.7.2.4.1, "Step 1 - Selection of Generic Soil Profiles," the staff noted that the assumed generic shear wave velocities are taken to be the final strain-compatible values consistent with the seismic events (i.e., the calculation of strain-dependent properties from low strain values using assumed relationships to depict the strain-dependent modulus-reduction is not performed). 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, when determining the site acceptability of a particular site for the construction of a facility incorporating the U.S. EPR certified design, a COL applicant needs to compare final iterated site soil parameters with the standard plant generic soil parameters. Since this wasn't identified in the FSAR, in RAI 130, Question 03.07.01-4 and follow-up RAI 215, Question 03.07.01-21, the staff requested that the applicant add this instruction to COL Information Item 2.5-3 and to include reference to FSAR Tier 2, Section 2.5.4.7 in addition to the existing reference to FSAR Tier 2, Section 2.5.2.6. The staff also requested that the applicant add to Guideline 5 of FSAR Tier 2, Section 2.5.2.6, instructions that the site-specific soil profile that is compared to the U.S. EPR soil profile be a strain-compatible soil profile.

In a September 17, 2009, response to RAI 215, Question 03.07.01-21, the applicant described that FSAR Tier 2, Table 1.8-2, “U.S. EPR Combined License Information Items,”and FSAR Tier 2, Section 2.5.2.6 would be revised to clarify that the site-specific soil profile must be strain-compatible. The staff confirmed that Revision 2 of the U.S. EPR FSAR, dated August 31, 2010, contains the changes committed to in the RAI response. Accordingly, the staff finds that the applicant has adequately addressed this issue and, therefore, the staff considers RAI 130, Question 03.07.01-4 and RAI 215, Question 03.07.01-21 resolved.

FSAR Tier 2, Section 2.5.2.6 states 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 Structures is 304 m/s (1,000 ft/s) or greater. However, a similar COL Information Item requirement for the other Seismic Category I structures was not provided. SRP Acceptance Criteria 3.7.1.II.3, states that potential impact on soil-structure interaction and settlement must be addressed if the minimum shear wave velocity is less than 304 m/s (1,000 ft/s). Therefore, in RAI 130, Question 03.07.01-17, the staff requested that the applicant develop an appropriate COL information item that instructs the COL applicant to address the minimum shear wave velocity for other Seismic Category I structures not located on the NI common basemat.

In a February 20, 2009, response to RAI 130, Question 03.07.01-17, the applicant stated that to clarify this issue, FSAR Tier 2, Section 2.5.2.6, Guideline 2 would be revised to state that COL applicants will confirm that shear wave velocity for NI basemat and for other Seismic Category I structures is 1,000 ft/s or greater, to comply with the guidance in SRP Acceptance Criteria 3.7.1.II.3. The staff confirmed that Revision 2 of the U.S. EPR FSAR, dated August 31, 2010, contains the changes committed to in the RAI response. Accordingly, the staff finds that the applicant has adequately addressed this issue and, therefore, the staff considers RAI 130, Question 03.07.01-17 resolved.

3.7.1.4.6 FSAR Tier 1 Information

Information in FSAR Tier 1, Revision 3-interim, Table 5.0-1, “Site Parameters,” reflects the peak ground acceleration of 0.3 g for the EUR spectra and 0.21 g horizontal and 0.18 g vertical for the HF spectra. FSAR Tier 1, Section 5.0, Revision 3-interim, Figure 5.0-1, which accompanied the applicant’s June 22, 2011, response to RAI 320, Question 03.07.02-63, provides the CSDRS at five percent damping for the EUR and HF control motions. Since this reflects the basis for the U.S. EPR standard plant seismic analysis, the staff finds the information in FSAR Tier 1 acceptable and consistent with the FSAR Tier 2 information discussed earlier in this section of the report. **RAI 320, Question 03.07.02-63 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate these Tier I table changes.

3.7.1.5 *Combined License Information Items*

No applicable COL items were identified by the staff in FSAR Tier 2, Table 1.8-2. However, COL Information Item 2.5-3 in FSAR Tier 2, Revision 3-interim markup, Table 1.8-2 provided with the response to RAI 320, Question 03.07.02-63 requires that a COL applicant that references the U.S. EPR certified design confirm the site is within the seismic parameters and soil profiles described in FSAR Tier 2, Sections 2.5.2.6, 2.5.4.7, and 3.7.1 and summarized in FSAR Tier 2, Tables 3.7.1-6, 3.7.1-8, and 3.7.1-9. The staff finds the list in FSAR Tier 2, Table 1.8-2 with respect to seismic parameters complete with the addition of this COL information item. No additional COL information items need to be included in FSAR Tier 2, Table 1.8-2 for seismic design parameters consideration as discussed in Section 3.7.2 of this

report, **RAI 320, Question 03.07.02-63 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed changes.

3.7.1.6 *Conclusions*

Except for the open item specified above, the staff concludes that the seismic design parameters used in the design of plant SSCs are adequate and meet applicable requirements of 10 CFR Part 50, Appendix A, GDC 2; 10 CFR 52.47(a)(1); and 10 CFR Part 50, Appendix S. This conclusion is based on the following considerations:

The CSDRS used in the certified design of Seismic Category I SSCs meet the minimum requirements of 10 CFR Part 50, Appendix S in that the horizontal component of the SSE ground motion in the free field at the foundation level of Seismic Category I structures is an appropriate response spectrum with a peak ground acceleration of at least 0.1 g. The critical damping values used in the seismic analysis of Seismic Category I SSCs conform to RG 1.61, or exceptions have been justified to the staff's satisfaction as discussed above. The artificial time histories used for seismic design have been adjusted in amplitude and frequency content to obtain response spectra that envelope the design response spectra specified for the certified design and also exhibit sufficient energy in the frequency range of interest. Meeting these criteria provides assurance that adequate seismic design bases have been established such that Seismic Category I SSCs will be adequately designed to withstand the effects of earthquakes, and will be able to perform their intended safety function and thus meet the requirements of GDC 2.

3.7.2 Seismic System Analysis

3.7.2.1 *Introduction*

For the seismic design of nuclear power plants, 10 CFR Part 50, Appendix A, GDC 2, "Design Bases for Protection against Natural Phenomena," requires that the design basis reflect appropriate consideration of the most severe earthquakes that have been historically reported for a site and the surrounding area. Two levels of design earthquake ground motions are considered: (1) Operating basis earthquake (OBE); and (2) safe-shutdown earthquake. The provisions of 10 CFR Part 50, Appendix S, "Earthquake Engineering Criteria for Nuclear Power Plants," require that for SSE ground motion, SSCs will remain functional and within applicable stress, strain, and deformation limits. For the U.S. EPR design, the OBE is set at one-third of the SSE, and in accordance with the requirements of 10 CFR Part 50, Appendix S, an explicit OBE analysis is not required. For the SSE, the analysis must take into account soil-structure interaction effects and the expected duration of the vibratory motion. This section of the report provides evaluations of the methods used to perform seismic analyses and their results for Seismic Category I structures and other structures of the U.S. EPR certified design.

3.7.2.2 *Summary of Application*

FSAR Tier 1: The FSAR Tier 1 information associated with this section is found in Section 4.0, "Interface Requirements," and Section 5.0, "Site Parameters." FSAR Tier 1, Section 4.1, "Site Specific Structures," addresses site-specific structures not within the scope of the U.S. EPR certified design. FSAR Tier 1, Sections 4.2, "Fire Protection Storage Tanks and Building," 4.4, "Turbine Building," and 4.12, "Access Building," address the Fire Protection Storage Tanks and Pump Building, Turbine Building, and Access Building (AB), respectively. FSAR Tier 1,

Table 5.0-1, “Site Parameters for the U.S. EPR Design,” specifies seismic and soil-related parameters used in the U.S. EPR certified design.

FSAR Tier 2: The applicant has provided an FSAR Tier 2 description of the seismic system analysis in FSAR Tier 2, Section 3.7.2, “Seismic System Analysis,” which is summarized as follows:

The Nuclear Island Common Basemat Structures are designated as Seismic Category I and consist of 10 structures that share a common basemat. These structures are the Reactor Containment Building, Reactor Building Internal Structure (RBIS), Safeguard Buildings 1 thru 4, Fuel Building, Safeguard Buildings 2 and 3 shield structures, FB shield structure, Reactor Shield Building, main steam valve stations (MSVSs), and the Vent Stack (VS). Additional Seismic Category I structures are the two EPGBs and the four Essential Service Water Buildings (ESWBs) each having their own foundations.

The seismic analysis of Seismic Category I structures including soil-structure interaction (SSI) effects is performed using the complex frequency response analysis method. Seismic Category I structures are modeled as three-dimensional (3-D) finite element models (FEMs). The SSI analysis results include the maximum accelerations at each node point in the FEM, as well as nodal acceleration time histories from which ISRS are developed.

The seismic analysis is performed in each of three orthogonal directions of earthquake ground motion (two horizontal and one vertical). SSI analyses for the NI, EPGBs, and ESWBs cover a range of subgrade properties from soft soil to hard rock. The seismic inputs for the NI are based on the CSDRS described in FSAR Tier 2, Section 3.7.1, “Seismic Design Parameters.” The seismic input motion for the EPGBs and ESWBs is based on a modified CSDRS. The modified CSDRS accounts for the structure-soil-structure interaction (SSSI) effect that the NI Common Basemat Structure has on the free-field motion of the earthquake at the EPGB and ESWB basemat foundations and for the differences in elevation between the basemats of the EPGB, ESWB and that of the NI.

The NAB and the Radioactive Waste Processing Building (RWPB), which are non-safety-related structures, are also included as part of the U.S. EPR certified design. Due to the potential interaction with Seismic Category I structures, the applicant classifies the NAB as Seismic Category II. As such, it is designed and analyzed to withstand the SSE without adversely impacting adjacent Seismic Category I structures. The RWPB is designed to conform to the guidance of RG 1.143, “Design Guidance for Radioactive Waste Management Systems, Structures, and Components Installed in Light-Water-Cooled Nuclear Power Plants,” Revision 2, November 2001, and is classified as Radwaste Seismic. Accordingly, the RWPB is designed for one-half of the SSE.

Also included in this section is a description of the development of the ISRS, the method for combining modal responses, the interaction of non-Seismic Category I structures with Seismic Category I structures, and the methods for determining building seismic stability.

ITAAC: There are no ITAAC items for this area of review.

Interface Requirements: This section of the FSAR contains information related to the following plant interfaces that will be addressed in the COL designs: ACB (FSAR Tier 2, Table 1.8-1, “Summary of U.S. EPR Plant Interfaces with Remainder of Plant,” Item No. 1-2) TB (FSAR Tier 2, Table 1.8-1, Item No. 1-3), and Fire Protection Storage Tanks and Building (FSAR Tier 2, Table 1.8-1, Item No. 1-4).

COL Information Items: This section of the FSAR contains COL information items which the COL applicant must address. In FSAR Tier 2, Table 1.8-2, these include COL Information Items 3.7-1, 3.7-2, 3.7-6, 3.7-7, and 3.7-8.

3.7.2.3 *Regulatory Basis*

The relevant requirements of NRC regulations for this area, and the associated acceptance criteria, are given in NUREG-0800, Section 3.7.2, "Seismic System Analysis," and are summarized below. Review interfaces with other SRP sections are also identified in this SRP section.

1. 10 CFR Part 50, Appendix A, GDC 2, "Design Bases for Protection against Natural Phenomena." The design basis shall reflect appropriate consideration of the most severe earthquakes that have been historically reported for the site and surrounding area with sufficient margin for the limited accuracy, quantity, and period of time in which historical data have been accumulated.

2. 10 CFR Part 50, Appendix S is applicable to applications for a design certification or combined license pursuant to 10 CFR Part 52, "Licenses, Certifications, and Approvals for Nuclear Power Plants," or a construction permit or operating license pursuant to 10 CFR Part 50, "Domestic Licensing of Production and Utilization Facilities," on or after January 10, 1997. If an SSE ground motion occurs, certain SSCs will remain functional and within applicable stress, strain, and deformation limits. The required safety functions of SSCs must be assured during and after the vibratory ground motion associated with the SSE Ground Motion through design, testing, or qualification methods. The evaluation must take into account soil-structure interaction effects and the expected duration of the vibratory motion. If the OBE is set at one-third or less of the SSE, an explicit analysis or design is not required. If the OBE is set at a value greater than one-third of the SSE, an analysis and design must be performed to demonstrate that the applicable stress, strain, and deformation limits are satisfied. 10 CFR Part 50, Appendix S also requires that the horizontal component of the SSE ground motion in the free-field at the foundation level of the structures must be an appropriate response spectrum with a peak ground acceleration of at least 0.1 g.

Acceptance criteria adequate to meet the above requirements include:

1. RG 1.92, "Combining Modal Responses and Spatial Components in Seismic Response Analysis," July 2006, Revision 2, as it relates to combining modal responses and spatial components in seismic response analysis.

2. RG 1.122, "Development of Floor Design Response Spectra for Seismic Design of Floor-Supported Equipment or Components," February 1978, Revision 1, as it relates to developing ISRS for the seismic design of building-supported equipment and components.

3.7.2.4 *Technical Evaluation*

In this section, the staff describes the review of the seismic analyses methods and results for the structures of the U.S. EPR standard plant. This review includes the procedures used for seismic modeling, SSI analysis methods, development of the ISRS, how modal responses are combined, modeling for torsion effects, and analysis for building stability. The review also

covers the methods used to prevent or mitigate the interaction of non-Seismic Category I structures with Seismic Category I structures. The staff reviewed the information provided in FSAR Tier 2, Section 3.7.2 against the acceptance criteria of SRP Section 3.7.2 and the RGs referenced above.

The original seismic analysis of the NI Common Basemat Structures described in FSAR Tier 2, Section 3.7.2, was based on the approach to incorporating various nonlinear effects (uplift, sliding) in the seismic design. In RAI 215, Question 03.07.02-39, the staff requested that the applicant justify various modeling assumptions used in the nonlinear analysis. In a September 27, 2009, response to RAI 215, Question 03.07.02-39, the applicant stated that AREVA no longer uses the nonlinear analysis; instead, a linear analysis approach would be employed for the seismic analysis. However, the staff noted that the applicant did not make any changes to FSAR Tier 2, Revision 1, Section 3.7.1.1, which provided the formulation for the nonlinear analysis. Therefore, in follow-up RAI 320, Question 03.07.02-63, the staff requested that the applicant revise the FSAR to reflect the revised linear analysis approach. On this basis, since the applicant has adopted a linear analysis approach, the staff considers RAI 215, Question 03.07.02-39 resolved. The staff's review and evaluations of technical issues associated with the revised linear analyses are discussed below.

In addition, the original seismic analysis of the NI Common Basemat Structures described in FSAR Tier 2, Section 3.7.2, used lumped-mass stick models to represent the mass and stiffness of each of the buildings. The common basemat foundation which supports the NI structures was assumed to be rigid. During its review of FSAR Tier 2, Section 3.7.2, the staff generated a number of questions regarding the basis of the NI seismic analysis. These included RAI 130, Question 03.07.02-3 which dealt with the adequacy of the lumped-mass stick models to represent the frequency response of the NI, and RAI 130, Question 03.7.02-4 which dealt with the effect of the rigid mat assumption on calculated seismic design loads and ISRS. The applicant addressed the staff questions in a June 22, 2011, response to RAI 320, Question 03.07.02-63, which described a revised seismic analysis of the NI Common Basemat Structures. These are described in FSAR Tier 2, Revision 3-interim markup, Sections 3.7.1 and 3.7.2, which accompanied the response.

In the revised analysis, the NI structures including the common basemat are represented by a linear elastic 3-D FEM model in place of the previously used stick models. To properly account for the SSI effect, the NI 3-D model reflects that the NI is embedded into the subgrade. In addition, the revised analysis includes the certified seismic design response spectra for the high frequency (HF) input motion of a rock site which was not previously included in FSAR Tier 2, Section 3.7.1. The changes identified in the response to RAI 320, Question 03.07.02-63 are discussed in more detail below.

3.7.2.4.1 Seismic Analysis Methods

In FSAR Tier 2, Section 3.7.2.1, "Seismic Analysis Methods," the applicant identified four methods used in seismic analysis. These are:

1. Time History Analysis Method: The time history method is used to solve the equations of motion used to represent the dynamic equilibrium of a structure or system acted upon by a seismic input at its supports. The equations of motion are solved by direct integration or modal superposition techniques.

2. Response Spectrum Method: The response spectrum method is used on flexible long spans of the NAB to obtain the amplified vertical accelerations of the floors. This

assures that the analyzed floors are not under-designed for vertical seismic loads. The response spectrum method is also used to analyze the vent stack.

3. Complex Frequency Response Analysis Method: The complex frequency response method is used for the SSI analysis of all Seismic Category I structures. This analysis is performed using the System for Analysis of Soil-Strusture Interaction (SASSI) computer code. The equations of motion are solved in the frequency domain with the time history input transformed from the time domain to the frequency domain by the Fast Fourier Transform method. The seismic responses calculated in the frequency domain are transformed back to the time domain as time histories for use in subsequent design of building structures and for the development of the ISRS.

4. Equivalent Static Load Method: The equivalent static load method is used to determine seismic forces and moments in the NI Common Basemat Structures, EPGBs, ESWBs, and NAB by taking the maximum acceleration nodal results from the SSI analysis and applying these as equivalent static loads to static 3-D FEMs developed in either the ANSYS or GT STRUDL computer codes.

All of the above analysis techniques are accepted for use in accordance with SRP Acceptance Criteria 3.7.2.II.1.A and B and are appropriate for the applications identified in FSAR Tier 2, Section 3.7.2. Accordingly, the staff finds the methods described acceptable.

3.7.2.4.2 Natural Frequencies and Responses

According to Interim Staff Guidance (ISG) DC/COL-ISG-1, “Interim Staff Guidance on Seismic Issues of High Frequency Ground Motion in Design Certification and Combined License Applications,” structural models used for SSI calculations should be modeled in sufficient detail such that they contain frequencies of up to at least 50 Hz. This is intended to address the fact that many Eastern U.S. sites are characterized by HF input, and the models should be capable of capturing a response that would occur at these higher frequency sites. In the original analysis described in FSAR Tier 2, Section 3.7.2, the Seismic Category I buildings of the NI Common Basemat Structures were modeled as lumped-mass stick models. Since FSAR Tier 2, did not address Eastern U.S. HF site characteristics, it was unclear to the staff that the building stick models were appropriate for these types of sites. Therefore, in RAI 130, Question 03.07.02-3, the staff requested that the applicant address Eastern U.S. sites and provide the highest natural frequencies which the seismic models could adequately capture.

In a June 22, 2011, response to RAI 320, Question 03.07.02-63, the applicant provided the natural frequencies for the building 3-D FEMs used in the NI Common Basemat Structures seismic analysis. The highest mode for the NI Common Basemat Structures has a frequency of 48.48 Hz. As this frequency is approximately equal to 50 Hz, the staff finds the response acceptable, because it is in conformance with the guidance provided in the DC/COL‑ISG‑1. In the SSI analysis of the NI Common Basemat Structures, the RBIS is modeled as a 3-D FEM. A second model of the RBIS is used in the detailed analysis of the RCS. This second model is a stick model using lumped masses connected by linear elastic structural elements. The highest natural frequency of this stick model is 50 Hz with the predominate frequencies found below 28 Hz. The staff finds that this model also conforms to the guidance of the DC/COL‑ISG‑1, and is acceptable. On this basis, the staff considers RAI 130, Question 03.07.02-3 resolved. **RAI 320, Question 03.07.02-63 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed changes.

The seismic response of the EPGB is determined using a 3-D FEM. In a July 22, 2011, interim response to RAI 376, Question 03.08.05-31, the applicant provided a finite element model for this structure. The model was modified by adding shear keys to the foundation to improve the building's dynamic stability. Shear keys are discussed further in Section 3.8.5.4 of this report in connection with the RAI response. FSAR Tier 2, Revision 3-interim, Table 3.7.2-7, "Modal Frequencies of 3-D FEM of Emergency Power Generating Building," accompanying the applicant's response shows the highest frequency for the finite element model to be 61.94 Hz. Since this meets the guidance of the DC/COL-ISG-1, the staff finds the level of detail in the model acceptable. On this basis, the staff concluded that the interim response to RAI 376, Question 03.08.05-31 regarding the frequency content of the EPGB seismic finite element model is acceptable.

The seismic response for the ESWB is also determined using a 3-D FEM. The details of this FEM were not provided in the interim response RAI 376, Question 03.08.05-31, but will be provided in the final response. **RAI 376, Question 03.08.05-31 is being tracked as an open item** with respect to the ESWB seismic model and is confirmatory for the EPGB seismic model pending revision of the FSAR.

3.7.2.4.3 Procedures Used for Analytic Modeling

FSAR Tier 2, Section 3.7.2.3, "Procedures Used for Analytical Modeling," presents the procedures used for analytic modeling. In the original seismic analysis of the NI, the common basemat structures were represented by lumped-mass stick models supported by a rigid mat. Based on a number of staff questions including RAI 130, Question 03.07.02-3 and RAI 130, Question 03.07.02-4, which are referenced earlier in this section of this report, and the fact that the applicant had added a HF input motion to the seismic design, the applicant in its June 22, 2011, response to RAI 320, Question 03.07.02-63, revised the NI dynamic model from a stick model to a 3-D FEM. These changes are described in FSAR Tier 2, Revision 3-interim, Section 3.7.2.3, which was included as part of the response.

In its review of the response, the staff noted that dynamic FEMs for the NI Common Basemat Structures are first developed as static FEMs in the ANSYS program. Due to the limitations in running large dynamic FEMs using the SASSI Code, the SASSI dynamic model was modified to contain fewer structural elements than does the detailed static FEM on which the dynamic model is based. If the two models are not dynamically equivalent and the SASSI condensed model is not an accurate representation of the more refined static model, non-conservative errors could be introduced in the results of the dynamic analysis. In addition, according to SRP Acceptance Criteria 3.7.2.II.3.C.ii, a seismic finite element model should demonstrate that further refinement of the model has only a negligible effect on the results of the analysis. Therefore, in RAI 371, Question 03.07.02-67, the staff requested that the applicant demonstrate the dynamic SASSI model of the NI Common Basemat Structures was dynamically equivalent to the static FEM developed in ANSYS and demonstrate that the model in SASSI meets SRP Acceptance Criteria 3.7.2.II.3.C.ii by providing a comparison of seismic results for both models. **RAI 371, Question 03.07.02-67 is being tracked as an open item** pending the completion of a comparison of the results between the two models.

During the June 6-10, 2011, audit, the staff reviewed a comparison of results for two common basemat structure models. However, both models were based on ANSYS. One was a simplified ANSYS model used as the basis for the SASSI model, and the other was the full ANSYS static model. This was not the comparison the staff had sought, because it did not compare output from the SASSI dynamic model with output from the full ANSYS static model

which was needed to demonstrate the SASSI dynamic model met SRP Acceptance Criteria 3.7.2.II.3.C.ii. Therefore, in RAI 371, Question 03.07.02-67, the staff requested that the applicant run a fixed base SASSI analysis and a fixed base ANSYS analysis, using the full static model, and provide a comparison of results at selected points. On this basis, the staff considers RAI 130, Question 03.07.02-4 resolved. **RAI 371, Question 03.07.02-67 is being tracked as an open item** pending the completion of a comparison of the results between the two models.

DC/COL-ISG-1, states that information should be provided to demonstrate that the SSI and structural models are adequately refined to sufficiently capture the HF content of the horizontal and vertical ground motion response spectra/foundation input response spectra in the structural response. This is done to assure that higher frequency inputs are not filtered out by the lack of modeling detail in the combined soil-structure model used in the SSI analysis. In RAI 371, Question 03.07.02-68, the staff requested that the applicant describe how the problems used for NI SSI analyses were modeled such that the subgrade is capable of transmitting the highest frequency of interest for each of the CSDRS time histories and to present the results of any sensitivity studies that were performed to assure that the seismic models meet the frequency criterion of DC/COL-ISG-1.

In a July 11, 2011, response to RAI 371, Question 03.07.02-68, the sensitivity studies which accompanied the applicant's response indicate that the passing frequency for some of the models was less than the highest frequency where the spectra for the input motion return to the non-amplified region of the response spectra curve (the cutoff frequency). However, the staff's review of the material presented indicates that, in general, the passing frequency exceeds the frequency range of interest (the frequency range where a particular soil case controls the design) by a factor of roughly 2 to 3. Since the design consists of the envelope of all of the soil cases considered, and the stiffer soil cases that control the higher frequency range have passing frequencies exceeding the cutoff frequency, the staff concluded that taken as a group the passing frequencies inherent in the finite element models were generally adequate to capture the seismic demands for which the structure must be designed. As part of its response, the applicant provided an additional analysis in which the NI basement mesh was refined to demonstrate the adequacy of the HF transmission capability of the existing NI dynamic FEM. The HF ground motion input considered in this comparison was the Bell Bend site response. The staff reviewed a comparison of results of ISRS from both the refined and existing models at selected locations within the NI. In general, this comparison demonstrated that the results from the existing NI dynamic model when compared to the results of the refined NI dynamic model produced similar results. The one exception was at node 40425 where for the Y direction spectral acceleration of the refined model produced a result which was approximately 28 percent higher than the existing model. This occurred at a frequency of approximately 38 Hz. However, the HF ISRS for the refined model is enveloped by the certified design ISRS. This is due to the fact the ISRS for the certified design is a peak broadened envelope of all cases and is a conservative representation of actual ISRS for any one particular case. With the additional comparison to the certified design ISRS demonstrating that it envelopes the exception at node 40425 and with the demonstration that the passing frequencies exceed the frequency of interest for each of the analysis models as described above, the staff finds the applicant's response acceptable. **RAI 371, Question 03.07.02-68 is being tracked as a confirmatory item** pending revision to the FSAR to incorporate the proposed change.

The applicant's dynamic 3-D FEM for the NI is composed of plate and shell elements representing the superstructure and the mat. The plate elements not only model out-of-plane bending and shear, but also model in-plane tension, compression, and shear forces. The plate elements representing the basemat have effectively zero thickness; whereas, the mat is at least

3.05m (10 ft) thick. Since the plate elements have been located at the bottom of the basemat, the wall elements of the superstructure must be extended an additional 3.05m (10 ft). Since this model of the design could lead to errors in either the frequency response of the superstructure or design loads at the mat superstructure interface, in RAI 371, Question 03.07.02-66(b) the staff requested that the applicant describe the detail of the 3-D FEM at the foundation wall interface and assess its effect on the accuracy of the analysis results.

In a June 24, 2011, interim response to RAI 371, Question 03.07.02-66(b), the applicant stated that to compensate for longer walls in the SASSI model, the wall elements from the top of the mat through the basemat thickness are modeled with an increased stiffness that is less than 10 times the actual stiffness from the top of the mat through the basemat thickness. The stiffness used will be verified by comparing the ISRS at the top and bottom of the basemat to demonstrate that the accelerations are in reasonable agreement using the increased stiffness at the extension of the vertical walls. The staff finds the proposed approach technically acceptable because it represents the increased stiffness provided by the thickness of the basemat but will need to review the results when they become available. **RAI 371, Question 03.07.02-66(b) is being tracked as an open item** pending receipt and review of the applicant's response and confirmation of the revision to the FSAR to incorporate the proposed change.

As noted above, the 3-D FEM of the NI basemat is composed entirely of plate and shell elements. These elements have effectively zero thickness in the plane of the element leading the staff to the following concern: When a plate element of a section of the basemat of one thickness is connected to an adjacent element of a different thickness, the centerlines of the adjacent elements are connected continuously in a single plane. However, the actual basemat has a varying thickness. The actual centerline does not lie in a single plane, but its location changes as the thickness of the mat changes. Since this has the potential to introduce non-conservative errors in the design loads for the basemat, in RAI 371, Question 03.07.02-66(a), the staff requested that the applicant evaluate the impact of the analysis simplification regarding the location of the basemat centerline on the bending evaluation of the basemat and whether the SASSI analysis results provided accurate mat design loads.

In a June 24, 2011, interim response to RAI 371, Question 03.07.02-66(a), the applicant described a new basemat foundation seismic analysis which is used to determine the design loads for the basemat. This analysis is based on a 3-D FEM.

The foundation is represented by five layers of solid brick elements developed through the thickness of the basemat. These elements replace the zero thickness plate elements used to represent the basemat in the SSI model. The superstructure is represented by the ANSYS superstructure FEM used in the static analysis. This combined mat and superstructure FEM is used to calculate the seismic moments and shears developed in the basemat using the ANSYS computer program. Time history inputs used are consistent with the in-column motions at the level of the bottom of the soil profiles and are consistent with those motions used in the corresponding SASSI SSI calculations. Compression-only vertical soil springs model the connection of the bottom of the basemat to the subgrade, while contact/sliding elements are used to treat potential sliding of the model. The spring parameters were obtained from the Gazetas formulation, which were found by the applicant to produce displacements and base reactions similar to the SASSI results. Lateral soil pressures along the embedded portion of the vertical walls were represented by springs following a standard geotechnical approach in that the parameters of these springs were developed to yield maximum and minimum pressures defined by the Rankine passive ($K_p = 3.0$) and active ($K_a = 0.333$) pressure states with an

at-rest pressure (Ko) value of 0.50. The stiffness of these springs was selected to yield a displacement for full passive pressure based on recommendations for loose cohesionless soil of 0.006H and for minimum active pressure to yield a displacement of .002H, where H is the embedded depth of the NI. These values were selected to provide lower bound estimates of lateral soil stiffness and corresponding upper bound estimates of the potential for sliding. The staff considers the model and the analysis an acceptable methodology to estimate the seismic demands on the basemat. This acceptability is based on the following:

- The plate elements previously used have been replaced by solid brick elements which will provide a more accurate predictionof the response of the mat to the design basis loads including seismic loads.

- Consistency with the SASSI model has been maintained by the use of the seismic input motions used in the SASSI analysis and by the selection of soil springs that produce displacements and base reactions similar to those of the SASSI results.

- Assumptions regarding the location of the basemat centerline and where to locate the centerline that were raised in the SASSI analysis are addressed through the use of a model that represents the actual thickness of the mat. Concerns regarding the interface loads between the mat and superstructure due to the use of thin plate elements are also addressed directly through the use of the thick mat.

The applicant has committed to include in its final response to RAI 371, Question 03.07.02-66(a), a check of the seismic loads on the mat shear key when embedded in hard rock due to the higher stiffness of the side wall springs adjacent to the key and to verify that the shear key is adequate for the loads determined in the analysis. **RAI 371, Question 03.07.02-66(a) is being tracked as an open item** pending completion of the analysis to justify the seismic adequacy of the shear key for the hard rock site condition and for the proposed changes to the FSAR describing the basemat analysis.

In FSAR Tier 2, Section 3.7.2.3, the dynamic models of the EPGB and ESWB were FEMs. Since the information was not provided in the FSAR, in RAI 248, Question 03.07.02-47, the staff requested that the applicant demonstrate that the seismic models for each of these structures meet the SRP Acceptance Criteria 3.7.2.3.II.C.ii, in that the dynamic models are sufficiently detailed such that further refinement of the model has a negligible effect on the dynamic results.

In a December 18, 2009, response to RAI 248, Question 03.07.02-47, the applicant demonstrated this acceptance criteria had been met at one elevation in one direction for one of the structures (the EPGB). Therefore, in follow-up RAI 371, Question 03.07.02-71, the staff requested that the applicant provide additional examples for the EPGB and ESWB which demonstrated that the SRP acceptance criteria had been met.

In an August 31, 2010, response to RAI 371, Question 03.07.02-71, the applicant provided additional comparisons for the EPGB and indicated that the ESWB used a finer mesh and, therefore, the results should be applicable to both structures. The comparison consisted of ISRS developed from the EPGB FEM used for the structure's seismic analysis with the ISRS developed from a second model in which the FE mesh was twice as fine as that used in the first model. The results, which included HF input motion were nearly identical for each model. Therefore, the staff concluded that the mesh sizes used in the seismic analysis of the EPGB and ESWB were adequate to meet the SRP acceptance criteria. Therefore, the staff considers RAI 248, Question 03.07.02-47 and RAI 371, Question 03.07.02-71 resolved. Since these

original questions were generated, the designs of both the ESWB and EPGB have been changed to address dynamic stability issues. This has also resulted in a change to the dynamic FEMs for these structures.

In a July 22, 2011, interim response to RAI 376, Question 03.08.05-31, the applicant identified the highest frequency for the revised EPGB FEM as 61.94 Hz. In FSAR Tier 2, Table 3.7.2-7, the highest frequency for the EPGB was 51.82 Hz. Since the staff concludes that the previous EPGB mesh size was acceptable, and since the revised model has a finer mesh than did the original, the staff concludes that the new dynamic model meets SRP Acceptance Criteria 3.7.2.3.II.C.ii, and is, therefore, acceptable regarding the EPGB mesh size pending receipt of the final response to RAI 376, Question 03.08.05-31 and the revision to the to the FSAR to incorporate the proposed change. Since the seismic analysis for the ESWB is not yet complete, the modal frequencies corresponding to the revised FEM were not provided in the interim response to RAI 376, Question 03.08.05-31. Therefore, the staff does not have a basis at this time to conclude the revised mesh size is adequate to meet SRP Acceptance Criteria 3.7.2.3.II.C.ii. **RAI 376, Question 03.08.05-31 is being tracked as an open item** pending completion of the seismic analysis for the ESWB and is confirmatory with respect to the highest modal frequency for the EPGB associated with the EPGB refined FEM mesh.

Although the NI Common Basemat Structures seismic model has been revised from a stick model to a 3-D FEM, two lumped mass stick models are still used in the seismic analysis of the U.S. EPR certified design. The first is a lumped mass stick model of the NAB which is a Seismic Category II structure made of reinforced concrete. The second is a lumped mass stick model of the RBIS which is used in the detailed analysis of the RCS described in FSAR Tier 2, Appendix 3C, “Reactor Coolant System Structural Analysis Methods.” The staff reviewed the development of these lumped mass stick models in FSAR Tier 2, Section 3.7.2.3.1, “Seismic Category I Structures – Nuclear Island Common Basemat.” The stick models lump the building mass at building floor elevations and the structural linear elements which connect the lumped masses represent the building stiffness. The structural elements take into account the eccentricities between the cross section centroid, center of rigidity, and center of mass. These modeling methods used for the stick models comply with SRP Acceptance Criteria 3.7.2.II.3.C.i and are acceptable to the staff.

The original seismic analysis of the NI was based on using stick models for each of the NI structures. Accordingly, in RAI 130, Question 03.07.02-5 and Question 03.07.02-9, the staff requested additional information regarding the modeling of the out-of-plane stiffness for walls and floors and also requested that the applicant clarify whether these were modeled as cracked or un-cracked structural elements. The staff raised this issue because the dynamic response of the wall or slab is frequency dependent and the frequency in turn is a function of the assumed stiffness.

The applicant provided a response to these RAIs in a June 22, 2011, response to RAI 320, Question 03.07.02-63, which revised the methodology for the seismic analysis of the NI structures. In FSAR Tier 2, Revision 3-interim, Section 3.7.2.3.2 which accompanied the response, the applicant indicated that two models are used for each soil case. In one case, the walls and floors are assumed to have cracked properties, and in the other case the walls and floors are assumed to have uncracked properties. The out-of-plane cracked bending stiffness is assumed to be approximately one-half that of the uncracked bending stiffness. The staff finds this is an acceptable value, because for typically reinforced concrete floor and wall slabs the cracked moment of inertia is approximately one-half of the uncracked moment of inertia. For structural design and the development of ISRS, the applicant described that the worst case

acceleration is used for floor or wall design, and an envelope of the two cases (cracked and uncracked) is used to represent the ISRS. Since the ISRS are an envelope of both cases (cracked and uncracked), the staff finds this approach appropriate, and therefore acceptable. The staff considers RAI 130, Question 03.07.02-5, and RAI 130, Question 03.07.02-9 resolved. **w**ith respect to addressing NI seismic analysis considering the concrete cracking effect, **RAI 320, Question 03.07.02-63 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed change.

In the original dynamic models of the EPBG and ESWB, and in the February 20, 2009, response to RAI 130, Question 3.07.02-17, the applicant described that all walls and elevated slabs used cracked concrete properties in the out-of-plane direction. For the uncracked case, single-degree-of-freedom (SDOF) oscillators in the EPGB and ESWB finite element models were used to determine the out-of-plane responses. The worst case acceleration was used for structural design, and an envelope of both cases was used to develop ISRS. Since this a conservative approach, the staff finds the response acceptable, and considers RAI 130, Question 03.07.02-17 resolved.

In a July 22, 2011, interim response to RAI 376, Question 03.08.05-31, the applicant modified the above approach such that consideration of whether the walls and slabs are cracked or uncracked is treated in a manner identical to that of the NI Common Basemat Structures (i.e., analyses are run with the out-of-plane stiffness of walls and slabs modeled as both cracked and uncracked structural elements). Since the methodology is the same as that used for the NI Common Basemat Structures, which the staff considers acceptable, the staff finds the treatment of cracked and uncracked walls in the dynamic analysis of the EPGB and ESWB also acceptable pending confirmation of a revision of the FSAR to incorporate the proposed change with respect to addressing the concrete cracking effect in the seismic analyses for the EPGB and ESWB. **RAI 376, Question 03.08.05-31 is being tracked as an open item** with respect to other portions of the question and confirmatory with respect to proposed changes to the FSAR to address the treatment of cracked and uncracked walls in the EPGB and ESWB seismic analysis.

FSAR Tier 2, Section 3.7.2.3.1.2, “Development of Stick Models for NI Common Basemat Structures and NAB,” described how the effect of hydrodynamic loads was determined in building seismic analysis. This was accomplished by adding the tributary water mass to the pool’s walls and bottom slabs. For the spent fuel racks in the spent fuel pool, these were included by lumping 100 percent of the spent fuel load at the base of the pool. Since this adequately accounts for the effect of the mass of the water and spent fuel racks on the NI seismic analysis, the staff finds this acceptable. FSAR Tier 2, Section 3.7.2.3.1.2 also indicates that the frequency of the water sloshing is typically low compared to the first horizontal mode frequency of the structure housing the pool. Therefore, the applicant concluded that the water sloshing has a negligible effect on the response of the structure and can be ignored in the development of the seismic model. This effect is considered in the local analysis and detailed design of the pool. Since the details regarding how the sloshing effect was taken into consideration are not provided in the FSAR and could have an effect on the design of the pools, in RAI 412, Question 03.07.02-74, the staff requested that the applicant provide examples of the sloshing frequency and compare them with the fundamental mode of the structural frequency. In addition, the staff requested that the applicant describe how sloshing is taken into account in the local analysis of the pool and to describe how the convective loads are determined from the sloshing effect. The staff additionally requested that the applicant determine if the free board, (i.e. the distance between the top of the water level and the pool) was sufficient to

accommodate sloshing in each of the pools and describe whether or not an impact load on the bottom of any pool needs to be considered due to the earthquake acting in the vertical direction.

In a May 12, 2011, response to RAI 412, Question 03.07.02-74, the applicant stated that 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. 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 in the static FEM as an applied pressure to the pool walls and is combined with the pressure due to the impulsive loads which are also calculated from TID-7024. The staff finds the applicant's response acceptable, because the method of analysis complies with SRP Acceptance Criteria 3.7.3.II.11. In addition, the applicant confirmed that the amount of freeboard in each pool exceeded the height of the sloshing effect and that the vertical acceleration at the bottom of each of the pools was less than 1.0 g. Based on this last point, the staff concludes that it is unnecessary to consider a water impact load in the vertical direction as part of the pool design. **RAI 412, Question 03.07.02-74 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed change.

To account for its effect on the seismic response of the RBIS, the NI seismic model contains a simplified model of the RCS. For the seismic analysis of the RCS described in FSAR Tier 2, Appendix 3C, the seismic model consists of a detailed model of the RCS and a lumped mass model of the RBIS which supports the RCS. The seismic input for the RCS coupled model consists of the time histories at the foundation mat determined from the NI SSI seismic analysis. The NI is analyzed using the SASSI computer code, and the RCS is analyzed using the BWSPAN computer code. To assure the results from each of the analyses are in reasonable agreement, which would verify that the models used and the methods of analysis were providing acceptable results, in RAI 201, Question 03.07.02-35, the staff requested that the applicant provide load comparisons at key interface points between the RCS and RBIS from each of the analyses.

In a May 26, 2009, response to RAI 201, Question 03.07.02-35, the applicant provided comparisons of loads at various support locations in the RBIS from each of the analyses. This comparison indicated the two models were providing similar results, therefore confirming the structural consistency between the two models. Since that comparison, the applicant has replaced the original NI seismic lumped-mass stick model with a 3-D embedded FEM for seismic design of the NI Common Basemat Structures. Also, the latest NI analysis includes HF input motion. During the April 26-30, 2010, onsite audit, the applicant committed to update the comparison of results for the two models to provide assurance that the previous conclusions were still applicable.

An update to the comparison of the BWSPAN results to the SASSI results that included the change in the NI Common Basemat Structures to a 3-D FEM and the addition of the HF input motion was provided in a June 30, 2011, response to RAI 201, Question 03.07.02-35. With one exception, the BWSPAN load results were higher by 3 percent to 40 percent over that of the SASSI results. The only exception was the RPV horizontal load result where the BWSPAN result was 14 percent lower than the SASSI result. The applicant could not make a load comparison between the two models at this location, because in the SASSI model the RPV support is represented by only a horizontal and vertical support; whereas, in the BWSPAN

model the RPV support consists of sixteen supports (eight nozzles and a horizontal and vertical support at each nozzle). Since the BWSPAN load results were generally conservative, the staff finds the comparison of results acceptable. The applicant provided a second comparison which consisted of ISRS obtained from both the SASSI and BWSPAN results. Below 5 Hz some of the SASSI peaks were higher than the BWSPAN results; however, the first fundamental frequency for the RCS is above 5 Hz. Between approximately 5 Hz and up to about 20 Hz, there was a good match of frequencies, and peak accelerations were comparable. The BWSPAN result at frequencies greater than about 20 Hz had peaks that were higher than the SASSI result, but since the BWSPAN ISRS are being used for the design of piping or equipment attached to the RCS, this input is more conservative than the ISRS obtained from SASSI. Based on these results, the staff concludes that the two analyses are providing comparable results and that, therefore, the modeling of the RBIS stick model in BWSPAN and modeling of the simplified RCS model in SASSI are acceptable. **RAI 201, Question 03.07.02-35 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed change.

3.7.2.4.4 Soil-Structure Interaction Analysis

In FSAR Tier 2, Section 3.7.2.4, "Soil-Structure Interaction," the staff reviewed the modeling methods and methodology used in the seismic analysis to account for SSI effects. In the original analysis, the NI Common Basemat Structures were modeled as stick models attached to a rigid foundation basemat. The embedment of the NI was not modeled in the analysis. Instead, it was treated as a surface-founded structure.

In the June 22, 2011, response to RAI 320, Question 03.07.02-63, the applicant described a revised seismic analysis in which the NI is modeled as a dynamic 3-D FEM. In the revised analysis, embedment effects are included as the NI foundation is located 11.85 m (38 ft 10.5 in.) below the surface of the soil. The seismic SSI analysis is performed using the SASSI computer code. The SHAKE 91 computer code is used to develop a "surface motions" from the input ground motion defined by the CSDRS which are hypothetical "outcrop" motions. In developing the "surface motions," iteration of the soil properties is not performed, since the properties used in the analysis are assumed to be final strain dependent properties. Since the soil properties are hypothetical properties which the COL applicant must compare to site-specific strain dependent properties, the staff finds the assumption used by the applicant acceptable. As discussed in FSAR Tier 2, Section 2.5.2.6, a COL applicant will determine the final site–specific soil properties.

The SASSI computer code solves the equations of motion in a frequency domain analysis (complex frequency response analysis method). The applicant used the subtraction method option in SASSI to determine SSI response. The Defense Nuclear Facilities Safety Board (DNFSB) issued a letter on April 8, 2011, requesting the Department of Energy (DOE) to address technical and software quality assurance issues related to the use of the SASSI subtraction method for seismic analyses. Specifically, in certain circumstances the subtraction method may not provide conservative results. In RAI 489, Question 03.07.02-75, the staff requested that the applicant address the DNFSB issues in the SASSI analyses performed in support of the U.S. EPR design certification application. **RAI 489, Question 03.07.02-75 is being tracked as an open item**.

The SSI analysis for the EPGB and ESWB has been revised from the original analysis. These changes are described in the applicant's July 22, 2011, interim response to RAI 376, Question 03.08.05-31. The changes include revised finite element models to account for design

changes to both structures, the use of a HF input ground motion to reflect the site response for a hard rock site, and changes to the modified CSDRS, which includes SSSI effects from the NI and also accounts for the difference in elevation between the NI basemat and the EPGB and ESWB basemats. The EPGB is a surface founded structure with shear keys, while the ESWB is embedded 6.10 m (20 ft) into the subgrade. Embedment effects are included in both analyses. The modified CSDRS are reviewed in Section 3.7.1 of this report, and the staff considers their development acceptable. The staff finds the description of the SSI analysis consistent with that of the NI, and therefore acceptable, with respect to the approach used for SSI analyses for EPGB and ESWB, pending revision of the FSAR to incorporate the proposed change. **RAI 376, Question 03.08.05-31 is being tracked as an open item** regarding other portions of the question and is confirmatory with respect to proposed changes to the FSAR describing the SSI of the EPGB and ESWB.

In the original seismic analysis of Seismic Category I structures described in FSAR Tier 2, Section 3.7.2.4, different versions of the SASSI code were used for the different Seismic Category I structures. For the NI, the AREVA computer code SASSI, Version 4.1B was used, and for the EPGB and ESWB the Bechtel computer code SASSI 2000, Version 3.1 was used. In RAI 248, Question 03.07.02-43 and in RAI 371, Question 03.07.02-70, the staff requested that the applicant provide comparisons between the codes being used to confirm that they provide consistent results for similar problems.

As discussed further below, the applicant is now using MTR/SASSI for these calculations.

In the August 31, 2010, response to RAI 371, Question 03.07.02-70, the applicant indicated that it had applied the quality assurance measures of ANP-10266, “AREVA NP Inc. Quality Assurance Plan (QAP) for Design Certification of the U.S. EPR,” which the staff approved in an SE dated April 26, 2007, to MTR/SASSI, and that the code satisfied the QAP measures. Since the applicant applied an approved QAP to MTR/SASSI, the staff finds its use in the seismic analysis acceptable.

In a June 22, 2011, response to RAI 320, Question 03.07.02-63, the applicant identified that the only code that would be used for seismic analysis of all Seismic Category I structures was the MTR/SASSI code, Version 8.3. MTR/SASSI, Version 8.3 changed the number of frequencies and elements groups that can be included in the models and the number of load cases that can be analyzed in a single computer run. Some changes to the formatting of input and results files and user guidance related to the element formulation for some plates were also changed. On June 6-10, 2011, the staff performed an audit of MTR/SASSI calculations and verified that the most recent version of the code (MTR/SASSI, Version 8.3) did not yield significantly different results than the previous version of the code. Therefore, the staff finds the use of MTR/SASSI, Version 8.3 acceptable.

Since this is the only code now used for the seismic analysis of Seismic Category I structures, the staff considers RAI 248, Question 03.07.02-43 and RAI 371, Question 03.07.02-70 resolved. **RAI 320, Question 03.07.02-63 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed change.

3.7.2.4.5 Development of In-Structure Response Spectra

The staff reviewed the information provided in FSAR Tier 2, Section 3.7.2.5, “Development of Floor Response Spectra,” for the procedures and methods used for developing the ISRS for Seismic Category I structures. For the NI Common Basemat Structures and the NAB, the floor acceleration time histories in a given direction, due to the three components of input motion, are

combined algebraically to produce a combined floor acceleration time history in the same direction from which the ISRS in that specific direction is computed. This conforms to RG 1.92, Revision 2 for statistically independent earthquake motions specified in each of three directions which are applied simultaneously in the analysis. ISRS are developed following the guidance of RG 1.122, Revision 1 and SRP Acceptance Criteria 3.7.2.II.5.C.

For the EPGB and ESWB, the ISRS at any given location were first computed separately for each of the three components of input ground motion. The three resulting response spectra in a given direction were then combined using the SRSS method. In a July 22, 2011, interim response to RAI 376, Question 03.08.05-31, the applicant revised the development of the ISRS for the EPGB and ESWB. The staff notes that the methodology is now the same as that used for the NI and NAB described above and meets the guidance of RG 1.122 and RG 1.92 and SRP Acceptance Criteria 3.7.2.II.5.C.

Since the methods described for the development of ISRS meet SRP Acceptance Criteria 3.7.2.II.5.C and the guidance of RG 1.92 and RG 1.122, the staff finds the methods used by the applicant acceptable with respect to addressing the development of the ISRS for the EPGB and ESWB, pending revision of the FSAR to incorporate the proposed change. **RAI 376, Question 03.08.05-31 is being tracked as an open item** regarding other portions of the question, and is confirmatory regarding the proposed changes to the FSAR describing the development of ISRS for the EPGB and ESWB.

3.7.2.4.6 Three Components of Earthquake Motion

In FSAR Tier 2, Section 3.7.2.6(1), "NI Common Basemat Structures and NAB," the method for combining the response to three components of earthquake motion is described. For the NI and NAB, the floor acceleration time history in a given direction and the peak structural acceleration are obtained by algebraically combining the three corresponding time history responses in that direction due to simultaneous application of the three earthquake components of ground motion. For the EPGB and ESWB, the zero period acceleration (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 rule is based on the assumption that when the maximum response from one earthquake component is reached, the response due to the other components are no more than 40 percent of their maximum.

In a July 22, 2011, interim response to RAI 376, Question 03.08.05-31, the applicant revised the methodology for the EPGB and ESWB to be consistent with that of the NI in that the floor acceleration time history in a given direction is obtained by algebraically combining the three corresponding time history responses in that direction due to the three earthquake components of ground motion. Since the time histories have been shown to be statistically independent and combining the time history responses in a given direction algebraically meets the guidance of RG 1.92 when the time history inputs from three directions are applied simultaneously, the staff finds the method described for combining the three components of earthquake motion acceptable.

Based on the above discussion, the staff finds the response acceptable with respect to addressing the seismic analyses considering the three directional earthquake components for the EPGB and ESWB**,** pending revision of the FSAR to incorporate the proposed change. **RAI 376, Question 03.08.05-31 is being tracked as an open item** regarding other portions of the question and is confirmatory with respect to the proposed changes to the FSAR describing the combination of three components of earthquake for the EPGB and ESWB.

3.7.2.4.7 Combination of Modal Responses

FSAR Tier 2, Section 3.7.2.7, "Combination of Modal Responses," describes methods used for modal combinations when using the response spectrum method of analysis. When this method is used, the maximum modal responses are combined using the methods specified in RG 1.92, Section C, Revision 2. The effect of missing mass for modes not included in the analysis is accounted for by calculating the residual seismic load in accordance with RG 1.92, Appendix A, Revision 2.

In the modal analysis, the total response of a structure is determined from the response of each of the modes of structural vibration. Associated with each mode is a frequency and modal mass. If an analysis only considers frequencies up to some value (the cutoff frequency), the mass associated with the remaining modal frequencies will be excluded from the result. The missing mass method is used to account for the effect of the remaining modes without actually running the calculations through all of the frequencies above the cutoff frequency.

Since the approaches for combining modal responses and for calculating the effect of missing mass are in conformance with the RG, the staff finds the methods described in FSAR Tier 2, Section 3.7.2.7 acceptable.

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

In FSAR Tier 2, Section 3.7.2.8, "Interaction of Non-Seismic Category I Structures with Seismic Category I Structures," the staff reviewed the methods of analysis and design that prevent an interaction between a non-Seismic Category I structure and a Seismic Category I structure such that the safety function of the Seismic Category I structure is not impaired. FSAR Tier 2, Section 3.2, "Classification of Structures, Systems, and Components," identifies as Seismic Category II those non-safety-related SSCs whose failure under seismic loading could prevent or reduce the functional capability of a Seismic Category I SSC. SSCs in this seismic category are designed to withstand SSE loads without incurring a structural failure that permits deleterious interaction with any Seismic Category I SSC or causes incapacitating injury to main control room occupants. The structures designated as Seismic Category II are the NAB, ACB, and TB.

The separation distance between the NAB and the NI Common Basemat Structures is 45.7 cm (18 in.). The staff noted that the details of the methods used to design a Seismic Category II structure such that it could withstand SSE loads without incurring a structural failure that would permit deleterious interaction with a Seismic Category I structure were not described in the FSAR. The staff also noted that, due to their close proximity, an interaction between the NI structures and the NAB was possible. Therefore, in RAI 370, Question 03.07.02-64, the staff requested that the applicant identify the method used for the analysis and design of the NAB. In addition, the staff also requested that the applicant provide the methods and results for the stability analysis of the NAB.

In an August 10, 2010, interim response to RAI 370, Question 03.07.02-64, the applicant stated that due to the potential for the NAB to interact with a Seismic Category I structure during an SSE, this potential interaction will be analyzed to SSE load conditions and designed to the codes and standards associated with Seismic Category I structures so that the margin of safety is equivalent to that of a Seismic Category I structure. The applicant also indicated that because the NAB does not have a safety function, it may slide or uplift provided that the gap between the NAB and any Seismic Category I structure is adequate to prevent interaction between the two structures. The staff concluded that the applicant's approach for providing a

margin of safety for the NAB that is equivalent to that of a Seismic Category I structure complies with SRP Acceptance Criteria 3.7.2.II.8, and therefore is acceptable.

For the stability evaluation of the NAB, in its June 17, 2011, interim response to RAI 370, Question 03.07.02-64, the applicant proposed the use of a nonlinear dynamic analysis. The analysis will be performed with the ANSYS computer code using horizontal soil springs that will allow the structure to slide if the static coefficient of friction (0.50) used in the sliding analysis is exceeded and will allow the structure to uplift since the vertical soil springs will act in compression only. The static coefficient of friction will be reduced by half if sliding is initiated. For the overturning stability analysis, a higher coefficient of friction will be used (0.70) which will provide a more conservative result for the overturning stability evaluation. In addition to the SSE input motions represented by the CSDRS, the applicant will include input motion based on RG 1.60, “Design Response Spectra for Seismic Design of Nuclear Power Plants,” spectrum. This spectrum was added to include an input motion with low frequency energy as this might provide higher displacements and result in a more conservative evaluation of the sliding stability case. Equivalent springs are developed to model each of the soil cases in FSAR Tier 2, Revision 3-interim, Table 3.7.1-6, “Generic Soil Profiles for the U.S. EPR Standard Plant,” which accompanied the June 22, 2011, response to RAI 320, Question 03.07.02-63. The output from this analysis will include the bearing stresses under the NAB, as well as the reduction in the separation distance between the NI and the NAB. The staff concludes that the assumptions and methodology described in the interim response are based on reasonable, yet conservative, assumptions which should provide an acceptable stability evaluation for this structure. However, **RAI 370, Question 03.07.02-64 is being tracked as an open item** since the applicant has not yet compled the NAB stability analysis and the review of the final RAI response which will include NAB stability analysis results.

In FSAR Tier 2, Section 3.7.2.8, the applicant describes the RWPB as having no significant potential to seismically interact with either the NI Common Basemat Structures or with the nearest non-NI Seismic Category I structures. Therefore, the applicant did not evaluate the RWPB for the SSE. The RWPB is a reinforced concrete shear wall structure designed according to RW-IIa criteria in RG 1.143. As such, the RWPB is designed using the codes and standards associated with Seismic Category I structures and analyzed for one-half of SSE. This provides significant reserve lateral force resistance capacity, and as such the staff concludes that a collapse of the RWPB during an SSE event is unlikely. The closest Seismic Category I structure to the RWPB is one of the EPGB structures. The applicant states that the potential interaction between the RWPB and EPGB is precluded by the separation distance between these two structures. The RWPB is embedded a significant distance below grade and has a clear height above grade of 24.80 m (52.5 ft), while the clearance between the RWPB and EPGB is at least 23.4 m (49.5 ft). Therefore, the separation between the two is only slightly less than the height above grade of the RWPB. The staff believes that failure of the RWPB in a manner that would allow it to strike the EPGB is not considered credible due to the separation distance between the two structures, embedment depth of the RWPB, and the RWPB’s seismic design for a one-half of an SSE event. Accordingly, the staff concludes that the design basis for the RWPB is acceptable.

The ACB and TB are non-Seismic Category I structures for which continued operation during an SSE event is not required by regulations. The ACB and TB are classified as Seismic Category II based on their proximity to the NI Common Basemat Structures. In RAI 370, Question 03.07.02-65, the staff requested that the applicant provide information regarding the interaction analysis for these two structures and describe the methodology used to ensure that

the safety functions of Seismic Category I structures were not impaired by the response of these structures to a seismic event.

In an August 10, 2010, response to RAI 370, Question 03.07.02-65, the applicant stated that the ACB and TB buildings are site-specific structures and as such are the design responsibility of the COL applicant. In FSAR Tier 2, Revision 2, Section 3.7.2.8, the applicant provided conceptual design information for these structures which states that the structures will be analyzed to site-specific SSE load conditions and designed to the codes and standards associated with Seismic Category I structures, so that the margin of safety is equivalent to that of a Seismic Category I structure. The applicant states that because the ACB and TB do not have a safety function, it is permissible for these structures to slide or uplift due to the site SSE provided that the gap with any Seismic Category I structure is adequate to prevent structure-to-structure interaction. The effects from sliding, overturning, and any other calculated building displacements must be considered when demonstrating the adequacy of the gap between these structures and a Seismic Category I structure. The applicant included COL Information Items 3.7-7 and 3.7-8 in FSAR Tier 2, Table 1.8-2, Revision 2, to ensure that the response of the ACB and TB to an SSE event will not impair the ability of Seismic Category I systems, structures, or components to perform their design basis safety functions. In addition, COL Information Item 3.7-2 in FSAR Tier 2, Table 1.8-2, Revision 2 requires that the COL applicant address the site-specific separation distances for the ACB and TB. A COL applicant will either confirm the conceptual design information regarding these structures or propose a different approach, but must ensure that the response of these structures does not impair the ability of Seismic Category I systems, structures, or components to perform their design basis safety functions. Since the applicant has identified COL information items that assure the safety functions of Seismic Category I structures are not impaired by an interaction with a Seismic Category II structure, and the conceptual design information for the ACB and TB is outside the scope of the design certification, the staff finds the approach by the applicant acceptable, and, therefore, considers RAI 371, Question 03.07.02-65 resolved.

The Fire Protection Storage Tanks and Buildings are site-specific structures. In the site layout of the U.S. EPR standard plant, these structures are sufficiently distant from a Seismic Category I structure such that there is no potential for an interaction during a seismic event. A COL applicant that references the U.S. EPR design certification will address the seismic design basis for the sources of fire protection water supply for safe plant shutdown in the event of an SSE. The staff notes this requirement is listed in FSAR Tier 2, Revision 2, Table 1.8-2, COL Information Item 3.7-6, and is therefore acceptable.

3.7.2.4.9 Effects of Parameter Variations on Floor Response Spectra

FSAR Tier 2, Section 3.7.2.9, “Effects of Parameter Variations on Floor Response Spectra,” describes the effects of parameter variations on floor response spectra. For all Seismic Category I structures, ISRS are developed for each of the soil cases with out-of-plane cracked and uncracked concrete properties used for each soil case. The resulting ISRS for any location in a given direction is the envelope of all of the cases. Once the envelope has been developed, the ISRS are peak spread +/- 15 percent in accordance with the guidance of RG 1.122. Since this section meets the guidance of RG 1.122, the staff finds this design approach acceptable.

3.7.2.4.10 Use of Constant Vertical Static Factors

SRP Acceptance Criterion 3.7.2.II.10 states that the use of equivalent static load factors to calculate vertical response loads for the dynamic analysis is acceptable only if it can be shown that the structure is rigid in the vertical direction, or SRP Acceptance Criteria 3.7.2.II.1.b are

satisfied. FSAR Tier 2, Section 3.7.2.10, “Use of Constant Vertical Static Factors,” indicates that constant vertical factors are not used in the design of Seismic Category I or Seismic Category II structures, because the vertical seismic loads are generated directly from a structure’s SSI seismic analysis. Since the vertical seismic response loads are determined from an analysis which is based on a modeling of the mass and stiffness in the vertical direction, as opposed to assuming the structure is rigid in the vertical direction, the staff agrees that constant vertical static factors have not been used to calculate vertical response loads and the acceptance criteria are not applicable.

3.7.2.4.11 Method Used to Account for Torsional Effects

In FSAR Tier 2, Section 3.7.2.11, “Method Used to Account for Torsional Effects,” the staff reviewed the methods to account for the effects of torsion in the seismic response of building structures. The applicant indicates that the torsional effects due to the eccentricity are built into the modeling process and are, therefore, automatically accounted for in the seismic analysis. SRP Acceptance Criteria 3.7.2.II.11 specifies that an additional eccentricity of +/- 5 percent of the maximum building dimention should be assumed in both horizontal directions. Since the model accounts for torsion due to eccentricity, and accidental torsion is included in the design, the staff finds the approach described in the FSAR acceptable.

3.7.2.4.12 Comparison of Responses

FSAR Tier 2, Section 3.7.2.12, “Comparison of Responses,” states that the response spectrum method is used only in the local seismic analyses of selected slabs in the NAB and in the seismic analysis of the vent stack. The time history analysis method is not used. Therefore, the staff notes that a comparison of responses between the response spectrum method and a time history analysis method would not be needed, and the approach used in the FSAR as discussed above is acceptable to the staff because it complies with regulatory guidance.

3.7.2.4.13 Methods for Seismic Analysis of Category I Dams

FSAR Tier 2, Section 3.7.2.13, “Methods for Seismic Analysis of Category I Dams,” references FSAR Tier 2, Section 3.7.3.13, “Methods for Seismic Analysis of Category I Concrete Dams.” FSAR Tier 2, Section 3.7.3.13, states there are no Seismic Category I concrete dams in the U.S. EPR certified design. It concludes that the COL applicant that references the U.S. EPR design certification will need to provide a description of methods used for seismic analysis of site-specific Seismic Category I concrete dams, if such exist. This is identified as a COL information item in FSAR Tier 2, Section 3.7.3.

3.7.2.4.14 Determination of Dynamic Stability of Seismic Category I Structures

The staff reviewed the methods for determinining dynamic stability presented in FSAR Tier 2, Section 3.7.2.14, “Determination of Dynamic Stability of Seismic Category I Structures.” SRP Acceptance Criteria 3.7.2.II.14 states that the determination of the design overturning moment and sliding force should incorporate the three components of input motion and conservative consideration of the simultaneous action of vertical and horizontal seismic forces. However, the description in the FSAR did not provide the methods for determining the overturning moment and sliding forces and said only that overturning of the NI Common Basemat Structures due to a seismic event does not occur due its inherent stability. Since the determination of overturning moment and sliding forces was not provided, in RAI 130, Question 03.07.02-22, the staff requested that the applicant provide the analytical model and methods used for assessing building stability during a seismic event. The staff also requested this information for both the

EPGBs and the ESWBs. The applicant provided the response to RAI 130, Question 03.07.02-22 in the June 24, 2011, response to RAI 371, Question 03.07.02-69, for the NI; and in the July 22, 2011, interim response to RAI 376, Question 03.08.05-31 for the EPGB and ESWB. These responses are discussed below.

In RAI 371, Question 03.07.02-69, the staff requested that the applicant provide information relative to the treatment of lateral passive sidewall soil pressures and their contribution to structural stability of the NI. In a July 8, 2011, response to RAI 371, Question 03.07.02-69, the applicant provided the following methodology for determining the seismic stability of the NI. AREVA indicates the methodology for stability analysis of the ESWB and EPGB is performed in the same manner as that for the NI, as follows:

- The driving forces affecting stability are taken from the SASSI SSI analysis. These forces include the dynamic forces on the embedded portions of the NI as well as the dynamic forces acting on the basemat and shear keys,
- The at-rest static soil pressure is used instead of active soil pressure that is developed by the movement of the NI away from the embedded soil. This is conservative as the at-rest pressure is greater than the active soil pressure.
- The analysis considers the simultaneous application of a horizontal driving force (x or y) with a vertical driving force.
- The resisting forces consist of sidewall passive soil pressures acting on the embedded portion of the building as it moves into the soil as well as the bearing forces and friction forces that are mobilized at the bottom of the basemat and shear key.
- The coefficient of friction between the basemat and soil is assumed to be 0.5.
- For the EPGB, a sidewall friction value of 0.36 in conjunction with the at-rest soil pressure is included in the overturning calculation.
- Passive soil pressure utilized in the analysis is compatible with the displacements determined from the SASSI SSI results.
- Hydrostatic pressures, including buoyancy effects from groundwater, are included in the stability analysis.

The staff finds the applicant's July 8, 2011, response to RAI 371, Question 03.07.02-69 acceptable as it considers the three directions of earthquake motion and the simultaneous application of vertical and horizontal earthquake motions; the determination of the passive pressure is compatible with the displacements from the SASSI SSI analysis; the effect of hydrostatic pressure has been included in the analysis; and an acceptable value for the coefficient of friction between the basemat and supporting soil has been used to determine the available horizontal resisting forces at the bottom of the basemat and shear key. Therefore, the staff considers RAI 130, Question 03.07.02-22 resolved. **RAI 371, Question 03.07.02-69 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed change. The actual factors of safety for overturning and sliding are reported in FSAR Tier 2, Section 3.8.5, "Foundations."

3.7.2.4.15 Analysis Procedure for Damping

The damping ratios used for seismic analysis of the SSCs for the U.S. EPR are described in FSAR Tier 2, Section 3.7.1.3, "Supporting Media for Seismic Category I Structures." The staff evaluation is described in Section 3.7.1.4.3 of this report.

3.7.2.4.16 FSAR Tier 1 Information

FSAR Tier 1: The staff reviewed the FSAR Tier 1 information related to this FSAR section and finds it to be acceptable since the interface requirements identified in FSAR Tier 1, Section 4.0 are consistent with those specified in FSAR Tier 2, Section 3.7.2. The site parameters identified in FSAR Tier 1, Table 5.0-1, have been revised by the June 22, 2011, response to RAI 320, Question 03.07.02-63. In FSAR Tier 1, Revision 3-interim, Table 5.0-1, which accompanied the RAI response, the site parameters are consistent with the values provided in FSAR Tier 2, Revision 3-interim, Section 3.7.2. **RAI 320, Question 03.07.02-63 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed change.

3.7.2.5 *Combined License Information Items*

Table 3.7.2-1 provides a list of seismic system analysis related COL information item numbers and descriptions from FSAR Tier 2, Table 1.8-2:

Table 3.7.2-1 U.S. EPR Combined License Information Items

Item No.	Description	FSAR Tier 2 Section
3.7-1	A COL applicant that references the U.S. EPR design certification will confirm that the site-specific seismic response is within the parameters of Section 3.7 of the U.S. EPR standard design.	3.7.2
3.7-2	A COL applicant that references the U.S. EPR design certification will provide the site-specific separation distances for the access building and turbine building.	3.7.2.8
3.7-6	A COL applicant that references the U.S. EPR design certification will provide the seismic design basis for the sources of fire protection water supply for safe plant shutdown in the event of an SSE.	3.7.2.8
3.7-7	A COL applicant that references the U.S. EPR design certification will demonstrate that the response of the Access Building to an SSE event will not impair the ability of Seismic Category I systems, structures, or components to perform their design basis safety functions.	3.7.2.8
3.7-8	A COL applicant that references the U.S. EPR design certification will demonstrate that the response of the TB (including Switchgear Building on the common basemat) to an SSE event will not impair the ability of Seismic Category I systems, structures, or components to perform their design basis safety functions.	3.7.2.8

The staff finds the above listing to be complete. Also, the list adequately describes actions necessary for the COL applicant. No additional COL information items need to be included in FSAR Tier 2, Table 1.8-2 for seismic system analysis consideration.

3.7.2.6 *Conclusions*

Except for the open items specified above, the staff concludes that the seismic system analysis procedures used in the design of plant SSCs are adequate and meet applicable requirements of 10 CFR Part 50, Appendix A, GDC 2; and 10 CFR Part 50, Appendix S with respect to the capability of structures to withstand the effects of earthquakes. This conclusion is based on the applicant satisfying the following three regulatory requirements:

1. Appropriate consideration for a standard plant SSE earthquake suitable for most sites in Central and Eastern U.S., as provided in FSAR Tier 2, Section 3.7.1.

2. Appropriate combination of the effects of normal and accident conditions with the effects of natural phenomena.

3. Appropriate consideration for the importance of the safety functions to be performed for the different categories of structures as required by GDC 2 and the use of a suitable dynamic analysis to demonstrate that structures, systems, and components can withstand the seismic and other concurrent loads.

As described in detail above, the staff finds that the applicant meets the requirements of Item 1 above by using seismic design parameters that meet the guidelines of SRP Section 3.7.1. With respect to Item 2 above, the combination of earthquake-induced loads with those resulting from normal and accident conditions in the design of Seismic Category I structures meets the guidelines of SRP Sections 3.8.1 through 3.8.5 and is discussed in corresponding sections of this report.

The review of seismic analysis methods included procedures for modeling, SSI analysis, the development of floor response spectra, inclusion of torsional effects, and the effects of parameter variations on floor response spectra. Time history methods using ground motion inputs in three orthogonal directions were used in the analysis of Seismic Category I structures. The review also included design criteria and procedures for evaluation of the interaction of non-Seismic Category I structures with Seismic Category I structures. In view of the foregoing, the use of the seismic analysis procedures and criteria implemented by the applicant provides an acceptable basis for the seismic design which will meet the requirements of GDC 2 and the applicable RGs.

3.7.3 Seismic Subsystem Analysis

3.7.3.1 *Introduction*

For the seismic design of nuclear power plants, 10 CFR Part 50, Appendix A, GDC 2, “Design Bases for Protection against Natural Phenomena,” requires that the design basis shall reflect appropriate consideration of the most severe earthquakes that have been historically reported for a site and the surrounding area. Two levels of design earthquake ground motions are considered: (1) Operating basis earthquake (OBE); and (2) safe shutdown earthquake. For the U.S. EPR design, the OBE has been selected as one-third of the SSE and is not included in the design of safety-related SSCs. This design approach is acceptable since it meets the

requirements of 10 CFR Part 50, Appendix S, “Earthquake Engineering Criteria for Nuclear Power Plants.” For the SSE, Appendix S requires that SSCs remain functional and be within applicable stress, strain, and deformation limits. Section 3.7.3 of this report discusses the seismic analysis of subsystems which include platforms, support frame structures, yard structures, buried piping, tunnels, and atmospheric tanks.

3.7.3.2 *Summary of Application*

FSAR Tier 1: The FSAR Tier 1 information associated with this section is found in FSAR Tier 1, Chapter 4, “Interface Requirements.” FSAR Tier 1, Section 4.6, “Buried Conduit and Duct Banks and Pipe and Pipe Ducts,” addresses the interface requirements for buried conduit and duct banks, and buried pipe and pipe ducts.

FSAR Tier 2: The applicant has provided an FSAR Tier 2 description of the seismic subsystem analysis in Section 3.7.3, “Seismic Subsystem Analysis,” which is summarized as follows:

The seismic analysis of subsystems covers the following: equipment and equipment supports, platforms, support frame structures, yard structures, buried duct banks and conduit, buried piping, tunnels, and atmospheric tanks. The seismic analysis for piping subsystems and the seismic qualification of electrical and mechanical equipment are addressed in FSAR Tier 2, Section 3.9.2, “Dynamic Testing and Analysis of Systems, Components, and Equipment”; Section 3.12, “ASME Code Class 1, 2, and 3 Piping Systems, Piping Components, and their Associated Supports”; Section 3.10, “Seismic and Dynamic Qualification of Mechanical and Electrical Equipment”; and Section 3.11, “Environmental Qualification of Mechanical and Electrical Equipment.” Design criteria for subsystem supports are contained in FSAR Tier 2, Appendix 3A.

The seismic analysis of subsystems is performed using the response spectra, time history, or equivalent static load methods of analysis. Response spectra modal analysis is performed using ISRS which are applied in each of three orthogonal directions (two horizontal and one vertical). Two methods are described for taking analysis uncertainties into account in the development of the ISRS. These are the peak broadening and peak shifting methods.

The time history method of seismic analysis uses modal superposition which is based on the assumption that the dynamic response of a system is linear elastic and the equations of motion can be uncoupled. The response of the system to a given earthquake direction is determined by integrating the decoupled equations for each mode and combining the results of the modes at each time step by algebraic addition. The total response in a given direction from each direction of applied motion is combined by either the square root of the sum of the squares (SRSS) method, if the analysis has been performed for each direction of input separately, or by combining the results algebraically if the input motion for each direction is applied simultaneously.

The equivalent static load method is used for relatively simple subsystems. In this method of analysis, the seismic acceleration coefficient that is applied to the subsystem or component is typically determined from the peak acceleration of the appropriate ISRS which is then multiplied by 1.5 to account for multi-mode effects.

In addition to the methods of seismic analysis, the application describes analytical modeling methods, analysis procedures for calculating damping values, methods for combining modal responses, the interaction of non-Seismic Category I SSCs with Seismic Category I SSCs and the seismic analysis of tanks.

3.7.3.3 *Regulatory Basis*

The relevant requirements of NRC regulations for this area, and the associated acceptance criteria, are given in NUREG-0800, Section 3.7.3, “Seismic Subsystem Analysis,” and are summarized below. Review interfaces with other SRP sections are also identified in this SRP section.

1. 10 CFR Part 50, GDC 2, “Design Bases for Protection against Natural Phenomena.” The design basis shall reflect appropriate consideration of the most severe earthquakes that have been historically reported for the site and surrounding area with sufficient margin for the limited accuracy, quantity, and period of time in which historical data have been accumulated.

2. 10 CFR Part 50, Appendix S, is applicable to applications for a design certification or combined license pursuant to 10 CFR Part 52, “Licenses, Certifications, and Approvals for Nuclear Power Plants,” or a construction permit or operating license pursuant to 10 CFR Part 50, “Domestic Licensing of Production and Utilization Facilities,” on or after January 10, 1997. For SSE ground motions, certain SSCs must remain functional and within applicable stress, strain, and deformation limits. The required safety functions of SSCs must be assured during and after the vibratory ground motion through design, testing, or qualification methods. The evaluation must take into account soil-structure interaction (SSI) effects and the expected duration of the vibratory motion. If the OBE is set at one-third or less of the SSE, an explicit response or design analysis is not required. If the OBE is set at a value greater than one-third of the SSE, an analysis and design must be performed to demonstrate that the applicable stress, strain, and deformation limits are satisfied.

Acceptance criteria adequate to meet the above requirements include:

1. RG 1.92, “Combining Modal Responses and Spatial Components in Seismic Response Analysis,” July 2006, Revision 2, as it relates to combining modal responses and spatial components in seismic response analysis.

2. RG 1.122, “Development of Floor Design Response Spectra for Seismic Design of Floor-Supported Equipment or Components,” February 1978, Revision 1, as it relates to developing floor design response spectra for seismic design of floor-supported components.

3. American Society of Civil Engineers (ASCE) 4-98, “Seismic Analysis of Safety-Related Nuclear Structures and Commentary.”

4. NUREG-1061, “Report of the U.S. NRC Piping Review Committee, Volume 4: Evaluation of Other Loads and Load Combinations,” as it relates to combining individual modal results using the independent support motion method.

5. NUREG/CR-1161, “Recommended Revisions to Nuclear Regulatory Commission Design Criteria, D.W. Coats, May 1980,” as it relates to recommended revisions to the NRC seismic design criteria.

3.7.3.4 *Technical Evaluation*

In this section of the report, the staff reviewed the procedures for the seismic analysis of subsystems. This review included seismic analysis methods and acceptance criteria for support frame structures and platforms; yard structures; buried piping, tunnels, ducts and conduits; concrete dams; and atmospheric tanks. Seismic analysis of piping systems is covered in FSAR Tier 2, Sections 3.9.2 and 3.12 and is not included in the scope of FSAR Tier 2, Section 3.7.3. The design criteria for distributed subsystem supports are provided in FSAR Tier 2, Appendix 3A.

The review of this section includes methods of seismic analysis, procedures for modeling, methods for combining three components of earthquake motion, methods for combining modal responses, criteria for fatigue evaluation, determination of composite damping, and methods for the seismic analysis of tanks. The review also covers design criteria and procedures for evaluating the interaction of non-Seismic Category I SSCs with Seismic Category I SSCs. The methods and procedures are reviewed against the acceptance criteria of SRP Section 3.7.3 and the RGs referenced above. Many of the acceptance criteria in SRP Section 3.7.3 reference the corresponding acceptance criteria of SRP Section 3.7.2 as a basis for acceptance of the seismic analysis of subsystems. As a result, some references to SRP acceptance criteria may directly reference the Acceptance Criteria of 3.7.2.

Many of the staff questions generated during the review either requested clarifications or the technical basis of a proposed methodology. Since RAI questions dealing with clarifications were successfully resolved with the applicant and are considered to not contribute to technical insights, these are not included in discussions of this section of the report.

3.7.3.4.1 Seismic Analysis Methods

In FSAR Tier 2, Section 3.7.3.1, “Seismic Analysis Methods,” the staff reviewed methods for seismic subsystem analysis. The seismic analysis methods reviewed are the response spectrum, time history, and equivalent static load methods.

Response Spectrum Method

The response spectrum method of analysis is used to determine seismic loads on suspended systems, and equipment and components supported within and on building structures. The seismic input for this analysis uses ISRS developed from building response time histories. These, in turn, are obtained from the SSI analysis of building structures described in FSAR Tier 2, Section 3.7.2. According to the description provided in FSAR Tier 2, Section 3.7.3.1.1, “Response Spectrum Method,” ISRS are applied to a subsystem in each of three orthogonal directions, which meets SRP Acceptance Criteria 3.7.2.II.1.A.ii. The ISRS peaks are broadened by a minimum of +/- 15 percent of the frequency at which the peak accelerations occur to account for uncertainties in the structural response. This conforms to the recommendations in RG 1.122, to broaden peak accelerations by this amount, and the staff finds this acceptable.

In addition to the use of peak broadened ISRS, FSAR Tier 2, Section 3.7.3.1.1, described the use of a peak shifting analysis method as an alternative to the peak broadening method. In the peak shifting method of analysis, the natural frequencies of the subsystem that fall within the peak broadened ISRS curve are identified. Then N+3 response spectra analyses are performed using the un-broadened ISRS, where N is the number of subsystem modes within the broadened ISRS frequency range. For each of the N+3 analyses, the peaks of the

unbroadened spectra are shifted to account for the uncertainties in the development of the ISRS as opposed to using the peak broadened spectra described in RG 1.122. The modal results of each of the N+3 analyses are combined using the SRSS approach described in FSAR Tier 2, Section 3.7.3.6, "Three Components of Earthquake Motion."

The peak shifting method is documented in AREVA Topical Report ANP-10264NP-A, Section 4.2.2.1.2, "U.S. EPR Piping Analysis and Pipe Support Design," which has been reviewed and accepted by the staff. Due to this position, in RAI 130, Question 03.07.03-22, the staff requested that the applicant specify in the FSAR that the peak shifting method would only be applied to piping systems.

In a February 20, 2009, response to RAI 130, Question 03.07.03-22, the applicant stated that the FSAR will be revised to state that the use of the peak shifting method is applicable only to piping systems. Based on the fact that the use of the peak shifting method is only used for piping systems and its use for these systems as documented in the AREVA topical report on piping has been approved, the staff finds the method described in the FSAR acceptable. Therefore, the staff considers RAI 130, Question 03.07.03-22 resolved.

<u>Time History Method</u>

FSAR Tier 2, Section 3.7.3.1.2, "Time History Method," describes a time history method of analysis using modal superposition. This method is used in the seismic analysis of more complex systems such as the reactor coolant system where there is a need to generate ISRS at points along the system itself and where it may be important to understand the response of the system as a function of time. The effects of modes that are higher than the cut-off frequency are accounted for by including the missing mass to conform to RG 1.92, Appendix A, Revision 2. In using the time history method, the total response of the system is determined by integrating the decoupled equations for each of the modes. For the Newmark and Wilson θ integration methods, which are two of the most commonly used integration methods, the maximum Δt, as described in ASCE 4-98, Table 3.2-1 is one-tenth of the shortest period of importance (i.e., the reciprocal of the cutoff frequency) for the subsystem in question. Since the effect of missing mass is included in the time history analysis as provided in RG 1.92, and an acceptable time step is used in the time history integration, the staff concludes that the time history method described is acceptable.

FSAR Tier 2, Section 3.7.3.1.2, describes two methods that may be used to account for uncertainties in structural analysis using the time history method. The first method described is similar to peak spreading described in the response spectrum analysis method. This method consists of using three separate time histories with modified time steps to account for analysis uncertainties. The technical justification for this method had not been provided, and the method for accounting for uncertainties embodied in this alternative analysis method was not clear. Therefore, in RAI 108, Question 03.07.03-3, the staff requested that the applicant provide the basis for the time step modification so as to achieve an equivalent +/- 15 percent peak shifting used in the response spectrum method, and to describe how this method satisfies the underlying purpose of RG 1.122 to account for uncertainties in the structural seismic analysis.

In a February 27, 2009, response to RAI 108, Question 03.07.03-3, the applicant indicated that the underlying purpose of RG 1.122 is to broaden computed in-structure response spectra peaks to account for uncertainties in structural modeling due to uncertainties in structure and soil approximations used in seismic analyses. RG 1.122 provides an approach for determining the value (± Δfj) for which each reference structural modal frequency (fj) is broadened. Using this approach, the minimum calculated value of Δfj is 0.1fj, or 10 percent of the reference peak

frequency. RG 1.122 also states that if Δfj is not determined using the formulated approach described above, Δfj should be taken as the minimum of ±0.15fj or 15 percent. RG 1.122 notes that "time history motions that will give results comparable to the floor design response spectra are also acceptable."

To accomplish the variation in the input time histories, the applicant states that the approach outlined in ASCE 4-98, Section 3.4.3.1 is used. In this method, time steps of the structural response are modified so that frequency content of the time history input data is varied by a minimum of ±15 percent. A similar approach is presented in ASME B&PV Code, Section III, Division I, Appendix N, SubSection N-1222.3. Both publications state that this approach is similar to broadening spectral peaks of a smoothed response spectrum. The staff finds the method described using the modified time steps to reflect analysis uncertainties in the structural dynamic analysis satisfies RG 1.122 and, therefore, considers the applicant's response acceptable. Therefore, the staff considers RAI 108, Question 03.07.03-3 resolved.

The second method used to account for uncertainties in time history analysis is to utilize a synthetic time history as a subsystem forcing function. The synthetic time history generated should envelope the peak broadened ISRS determined from the building seismic analysis. When this method is used, additional variation of the frequency content is not needed, as the ISRS have already been broadened in accordance with RG 1.122. The staff requested that the applicant provide verification that the use of this method would be consistent with development of artificial time histories in accordance with SRP Acceptance Criteria 3.7.1.II.1.B. In addition, for systems supported at points having different ISRS, and therefore having different synthetic time histories, the phase relationship between the time histories would be lost, thereby precluding their use for these types of applications. Therefore, in RAI 215, Question 03.07.03-23, the staff requested that the applicant address the phasing issue concerning systems supported at points having different ISRS in the application of this method and to describe how the time histories used in this approach are developed.

In an October 20, 2009, response to RAI 215, Question 03.07.03-23, the applicant stated that the second method is not used for piping or other distribution systems. When the second method is used, time histories are developed to match the enveloped response spectra in accordance with SRP Acceptance Criteria 3.7.1.II.1.B. The applicant indicated that the second method is used only in spent fuel rack design. This method is not employed for subsystems supported at multiple points with different ISRS (i.e., where independent support motion or uniform support motion methods are applied). Since the synthetic time histories are developed in accordance with SRP Acceptance Criteria 3.7.1.II.1.B and the use of synthetic time histories is limited to a single ISRS and not to subsystems that are supported at multiple support points, the staff concluded that their use as described by the applicant is acceptable and complies with regulatory guidance. Based on the above discussion, the staff considers RAI 215, Question 03.07.03-23 resolved.

Inelastic Analysis Methods

FSAR Tier 2, Section 3.7.3.1.3, "Inelastic Analysis Methods," states that inelastic analysis is not used to qualify subsystems for the U.S. EPR standard plant.

Equivalent Static Load Method

FSAR Tier 2, Section 3.7.3.1.4, "Equivalent Static Load Method," describes the equivalent static load method of analysis. This method is used for the analysis of simple systems where a full response spectrum method of analysis may not be warranted and the additional conservatism

built into this approach can be accepted. In this method, the fundamental frequency of the subsystem is assumed to coincide with the peak of the response spectra curve. The peak value of acceleration is multiplied by a factor of 1.5 to account for multi-mode effects and applied to the subsystem or component being analyzed. The staff finds that the equivalent static load method described complies with SRP Acceptance Criteria 3.7.2.II.1.B, and is therefore acceptable.

This FSAR section also describes an alternative to the equivalent static load method which is called the frequency determination method. In this approach, the subsystem fundamental frequency is calculated and if this frequency is higher than the frequency corresponding to the peak of the ISRS, the acceleration corresponding to the fundamental frequency may be used and applied to the system. In the case of the ISRS having multiple peaks, the peak acceleration corresponding to the next higher frequency must be used. Similar to the equivalent static load approach, the seismic acceleration determined by this method is multiplied by a factor of 1.5 to account for multimode response effects. This approach is similar to the equivalent static load approach except that the fundamental frequency for the system is calculated to determine the appropriate seismic acceleration to be applied. Since this approach also applies a factor of 1.5 to the seismic acceleration to account for multi-mode effects and provides a conservative estimate of the response, the staff finds the approach complies with the regulatory guidance of SRP Acceptance Criteria 3.72.II.1.B, and is therefore acceptable.

For single degree of freedom systems having a frequency above the cut-off frequency or for multi-degree-of-freedom systems where the fundamental frequency is above the cut-off frequency, a multiplication factor of 1.0 is used on the acceleration corresponding to the cut-off frequency. Since single degree-of-freedom systems do not have multi-mode effects and as multiple degrees-of-freedom systems having their fundamental frequency above the cutoff frequency will respond essentially as a single degree of freedom system, the staff finds the use of 1.0 as a multiplication factor applied to the cut-off acceleration acceptable and meets the regulatory guidance of SRP Acceptance Criteria 03.7.2.II.1.B where the fundamental frequency is above the cutoff frequency and a multiplication factor of 1.0 is acceptable for use.

3.7.3.4.2 Determination of Number of Earthquake Cycles

SRP Acceptance Criteria 3.7.3.II.2, states that when the OBE is defined as less than one-third of the SSE, the number of earthquake cycles for use in fatigue evaluations should follow the guidance provided in the SRM for SECY-93-087, "Policy, Technical, and Licensing Issues Pertaining to Evolutionary and Advanced Light-Water Reactor (ALWR) Designs," SRM, July 21, 1993, for piping systems. Consistent with this guidance, FSAR Tier 2, Section 3.7.3.2, "Determination of Number of Earthquake Cycles," states that the effects of seismic-induced fatigue are evaluated in accordance with SECY-93-087 and SRP Section 3.7.3, which the staff finds acceptable.

Seismic induced fatigue of piping systems is described in AREVA Topical Report ANP-10264NP-A. As identified earlier in this section, this report has been reviewed and accepted by the staff. Therefore, the staff finds its application to seismic induced fatigue for piping systems acceptable. To qualify electrical and mechanical equipment, SRP Acceptance Criteria 3.7.3.II.2, states that consideration of low level seismic effects is addressed by consideration of the equivalent of five OBE events followed by one SSE event (with 10 maximum stress cycles per event). However, with the elimination of the OBE as a design event (the OBE being one-third of the SSE), the FSAR describes the earthquake cycles included in the fatigue analysis as being composed of five one-half SSE events followed by one

full SSE event. This consideration of five one-half SSE events followed by one full SSE event conforms to the approach outlined in SRP Acceptance Criteria 3.10.III.3.C. Accordingly, the staff concludes that the number of earthquake cycles used in fatigue analysis for electrical and mechanical equipment is acceptable.

The effects of seismically induced fatigue on distributed subsystems other than piping and electrical and mechanical equipment are evaluated using the guidance from SECY-93-087 and SRP Acceptance Criteria 3.7.3.II.2. The SRP acceptance criteria state that the number of earthquake cycles to consider is two SSE events with 10 maximum stress cycles per event. Alternatively, the number of fractional vibratory cycles equivalent to that of 20 full SSE vibratory cycles may be used in accordance with Institute of Electrical and Electronic Engineers Standard (IEEE Std) No. 344-1987, Appendix D, "Test Duration and Number of Cycles." This is consistent with the approach used in FSAR Tier 2, Section 3.7.3.2, except that the FSAR references IEEE Std No. 344-2004, Appendix D, instead of IEEE Std No. 344-1987, Appendix D, "Test Duration and Number of Cycles." Since IEEE Std No. 344-2004 is accepted in RG 1.100, "Seismic Qualification of Electric and Mechanical Equipment for Nuclear Power Plants," September 2009, Revision 3 and the exceptions to the RG acceptance of IEEE Std No. 344-2004, "IEEE Recommended Practice for Seismic Qualification of Class 1E Equipment for Nuclear Power Generating Stations," do not apply to fatigue evaluation, the staff finds the description in the FSAR acceptable.

3.7.3.4.3 Procedures Used for Analytical Modeling

In FSAR Tier 2, Section 3.7.3.3, "Procedures Used for Analytical Modeling," the staff reviewed the procedures used by the applicant for analytical modeling. For dynamic analysis, the model used to represent the subsystem is a sequence of nodes connected by beam elements with appropriate stiffness to represent the subsystem components. Subsystem supports are modeled as springs with appropriate stiffness values. When lumped mass models are used, a mass is placed at each node point. The minimum number of degrees-of-freedom is equal to twice the number of modes with frequencies below the zero period acceleration frequency. Since this approach complies with SRP Acceptance Criteria 3.9.2.II.2.A.i, the staff finds it acceptable.

When developing dynamic models of structural elements that support the system under consideration, masses equal to 25 percent of the floor live load or 75 percent of the roof snow load are included in the model. Also included in the substructure model is an additional 243 kg/m^2 (50 psf) is to account for miscellaneous dead loads. The staff finds this complies with SRP Acceptance Criteria 3.7.3.II.3.D, and is therefore acceptable.

FSAR Tier 2, Section 3.7.3.3, provides decoupling criteria that define when supported systems and the supporting system can be analyzed separately or when they must be included in the same dynamic model. The criteria are a function of the ratio of the total mass of the supported system to the mass of the supporting system and the ratio of the fundamental frequency of the subsystem to the frequency of the support motion. The following criteria are used for decoupling subsystems:

- when $Rm < 0.01$, the mass of a subsystem is included in the primary system model regardless of Rf

- when $0.01 \leq Rm \leq 0.1$, decoupling may be done under two conditions:

1. when Rf ≥ 1.25, the subsystem mass is included in the primary system model at the support point

2. when Rf ≤ 0.8, the subsystem mass is not included in the primary system model

- when 0.01 ≤ Rm ≤ 0.1 and 0.8 < Rf < 1.25 and when Rm > 0.1, a subsystem model is included in the primary system model, and the effects of subsystem stiffness and mass are both considered

In the above, Rm is the ratio of the total supported subsystem to total mass of the supporting system, and Rf is the ratio of fundamental frequency of the supported subsystem to dominant frequency of the input support motion. The staff finds that the criteria provided comply with SRP Acceptance Criteria 3.7.2.II.3.B, and are therefore acceptable.

3.7.3.4.4 Basis for Selection of Frequencies

FSAR Tier 2, Section 3.7.3.4, “Basis for Selection of Frequencies,” describes the basis for selection of frequencies to determine the response of an analyzed system or component. For modes having frequencies above the ZPA, the residual response due to the missing mass effect is calculated using RG 1.92 which the staff finds acceptable. However, the applicant also proposes an alternative criterion which provides that it is sufficient to include enough modes to confirm that inclusion of the remaining modes does not result in more than a 10 percent increase in total response. Since this alternative criterion does not meet the guidance provided in RG 1.92, it was to be deleted in FSAR Tier 2, Section 3.7.3 as discussed in the applicant’s March 9, 2009, response to RAI 187, Question 03.07.03-20, which responded to a request by the staff to provide the technical basis for the dynamic analysis cutoff frequency. The fact that this was not deleted was viewed as an editorial oversight which the applicant corrected as part of the July 29, 2011, response to **RAI 371, Question 03.07.02-68, which is being tracked as a confirmatory item** pending incorporation of proposed changes to the FSAR. The staff notes that RAI 187, Question 03.07.03-20, however, has been resolved by deletion of this alternative criterion.

FSAR Tier 2, Section 3.7.3.4, indicates that certain components are designed to be rigid by establishing that their first fundamental frequency exceeds 40 Hz. However the HF input motion identified in the June 22, 2011, response to RAI 320, Question 03.07.02-63 has a cutoff frequency of 50 Hz. As such, the 40 Hz criterion in FSAR Tier 2, Section 3.7.3 needed to be revised to reflect a higher cutoff frequency of 50 Hz to cover HF input motion. Since this was done for the 10 percent criterion, the change to a 50 Hz cutoff frequency was included in the July 29. 2011, response to RAI 371, Question 03.07.02-68 and is acceptable to the staff. **RAI 371, Question 03.07.03-68 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed changes.

3.7.3.4.5 Analysis Procedure for Damping

FSAR Tier 2, Section 3.7.3.5, “Analysis Procedure for Damping,” describes an analysis procedure for determining composite modal damping values. This is based on either a stiffness-weighted damping or a mass-weighted damping, as the SRP recommends. The staff finds that the method presented in this FSAR section complies with SRP Acceptance Criteria 3.7.2.II.13, and is therefore acceptable.

3.7.3.4.6 Three Components of Earthquake Motion

FSAR Tier 2, Section 3.7.3.6, describes how the response of the subsystem due to three orthogonal earthquake inputs is determined. When the response spectrum method of analysis is used, the seismic loads from all three components of the earthquake are combined by the SRSS method. Since this conforms to RG 1.92, the staff finds the application of this method acceptable. When the time history analysis method is used, the FSAR describes that the analyses of subsystems may be performed separately for each of the three components of earthquake motion, or one analysis may be performed by applying all three components simultaneously. When the components of earthquake motion are applied separately, the maximum response from all three components in a given direction is combined using the SRSS method. If the earthquake motion in all three directions is applied simultaneously, the response is determined by algebraic summation at each time step. Since this conforms to RG 1.92, the staff finds the methods described acceptable.

3.7.3.4.7 Combination of Modal Responses

FSAR Tier 2, Section 3.7.3.7, “Combination of Modal Responses,” describes the combination of modal responses for low frequency modes, as well as for high frequency modes. The low frequency modes consist of every mode with seismic excitation up to the cut-off frequency. Above the cutoff frequency, a missing mass correction is applied to account for its contribution to the subsystem response and to component support loads.

Low Frequency (Non-Rigid) Modes

For subsystems having no closely spaced modes, the modal response at any node is calculated as the SRSS of the modes with natural frequencies below the cut-off frequency as recommended in RG 1.92. The staff finds this satisfies RG 1.92, Revision 2 and is acceptable. When using uniform support motion (USM) and closely spaced modes exist, then one of the two methods in RG 1.92, Regulatory Position C.1.1.2 or C.1.1.3, Revision 2 is used. The staff finds this meets the guidance of RG 1.92, and is therefore acceptable. When using independent support motion (ISM), modes are combined using the methods of NUREG-1061, Volume 4, which the staff finds acceptable.

High Frequency (Rigid) Modes

The high frequency mode contribution to overall response is accounted for by applying a missing mass correction. This is calculated using RG 1.92 for USM and NUREG-1061 for ISM. On this basis, the staff finds the approach for high frequency (rigid) modes acceptable.

3.7.3.4.8 Interaction of Other Systems with Seismic Category I Systems

To meet the requirements of GDC 2, Seismic Category I systems must be protected from adverse interactions with non-Seismic Category I SSCs. FSAR Tier 2, Revision 2, Section 3.7.3.8, “Interaction of Non-Seismic Category I SSC,” describes the methods used to provide assurance that if any part of a Seismic Category I SSC is within the impact zone of a non-Seismic Category I SSC, the safety function of the Seismic Category I SSC is not impaired. These methods are:

1. The two components are isolated from one another so that interaction does not occur.

2. The Seismic Category I subsystem is analyzed to confirm that its safety function is not lost as a result of impact from a non-Seismic Category I component during the SSE event. Impact loads are locally added to the analyzed stress of the Seismic Category I subsystem for load combinations that include seismic loads. Code allowable for the Seismic Category I subsystem with the additional impact load shall not be exceeded. This method shall not be used for vibratory sensitive Seismic Category I subsystems. Isolation or application of a restraint system shall be used for vibratory sensitive Seismic Category I subsystems.

3. A restraint system designed to Seismic Category I standards is used to prevent any interaction between the Seismic Category I subsystem and the non-Seismic Category I subsystem. The restraint system is designed to Seismic Category I standards.

The staff finds that Methods 1 and 3 meet SRP Acceptance Criteria 3.7.3.II.8, and are therefore acceptable. Method 2 provides that for any interaction with a non-Seismic Category I SSC, the code allowable for the Seismic Category I shall not be exceeded and the Seismic Category I safety function shall not be lost. No interaction with a vibratory sensitive subsystem is allowed. Since the safety function must be preserved and code allowable not exceeded using Method 2, the staff finds this method acceptable.

FSAR Tier 2, Section 3.7.3.8 states that in performing an interaction evaluation, impact of a Seismic Category I SSC must be assumed by any non-Seismic Category I SSC lying within a zone defined by the volume contained within a 15 degree angle extending upward from either side of the Seismic Category I SSC. Both falling and overturning of non-Seismic Category I SSCs are considered. In RAI 370, Question 03.07.03-38 and RAI 463, Question 03.07.03-40, the staff requested that the applicant clarify conflicting language in this FSAR section regarding the interaction of Seismic Category I and non-Seismic Category I SSCs and provide the basis for the acceptability of a 15 degree angle used to define the impact zone.

In a September 10, 2010, response to RAI 370, Question 03.07.03-38 and a March 15, 2011, response to RAI 463, Question 03.07.03-40, the applicant made a number of changes to the interaction evaluation criteria. These changes included the following:

- Non-Seismic Category I subsystems located in the main control room would be supported to Seismic Category I criteria

- ITAAC would be added to FSAR Tier 1, Section 3.9 requiring the evaluation of non-Seismic SSC interactions with Seismic Category I SSCs

- The definition of vibratory sensitive Seismic Category I equipment would be expanded to include vibratory equipment qualified by test

The staff concluded that these additions provided additional protection to Seismic Category I SSCs from impact by non-Seismic Category I SSCs and finds them acceptable and in compliance with regulatory guidance.

Regarding the impact zone, in a March 15, 2011, response to RAI 463, Question 03.07.03-40, the applicant modified the definition of an impact zone as follows:

- The impact zone for falling objects is defined by a conical volume with a slope of 45 degrees extending away from, and beginning six feet beyond, the perimeter of a Seismic Category I SSC.

- The impact zone for overturning includes the volume encompassed by the height of the non-Seismic Category I SSC and a radius extending from the perimeter of the non-Seismic Category I SSC equal to 1.5 times its height.

The staff notes that this significantly increases the impact zone and provides a conservative volume within which potential impacts must be considered. **RAI 370, Question 03.07.03-38 and RAI 463, Question 03.07.03-40 are being tracked as confirmatory items** pending revision of the FSAR to incorporate the proposed changes.

For non-Seismic Category I subsystems attached to seismic systems, FSAR Tier 2, Section 3.7.3.8 describes how the dynamic effects of the non-Seismic Category I subsystem are accounted for in the modeling of the seismic subsystem. In addition, the non-Seismic Category I subsystem is designed to preclude it from causing failure of the Seismic Category I subsystem. The attached non-Seismic Category I subsystems up to and including the first anchor or series of restraints acting in three directions beyond the interface are analyzed and designed using the same methods and design criteria for Seismic Category I subsystems. The staff finds this approach complies with SRP Acceptance Criteria 3.7.3.II.8, and is therefore acceptable.

3.7.3.4.9 Multiple-Supported Equipment and Components with Distinct Inputs

FSAR Tier 2, Revision 2, Section 3.7.3.9, "Multiply-Supported Equipment and Components with Distinct Inputs," provides analysis methods for the evaluation of subsystems that span between multiple locations within a structure or between locations in different structures and experience non-uniform support motion. These methods are the uniform support motion (USM) method and the independent support motion (ISM) method. In each method, relative displacements are considered in the analysis of the system. This is done by determining the support displacement from a static analysis or by approximating relative displacements from the floor response spectra according to the equation.

$S_d = S_{ag}/\omega^2$

Where S_d the maximum displacement at each support, S_{ag} is the spectral acceleration in g's at the cutoff frequency and ω is fundamental frequency of the building. This equation meets the guidance in SRP Acceptance Criteria 3.9.2.II.2.G and is acceptable to the staff. The support displacements thus determined are applied in the most unfavorable combination and are combined with the inertial responses of the system, and are therefore acceptable.

Uniform Support Motion

FSAR Tier 2, Section 3.7.3.9.1, "Uniform Support Motion Method," addresses USM in which a single set of spectra is applied at multiple support locations. This set of spectra envelopes the individual response spectra found at each of the individual locations contained in the analysis. The FSAR section states that an enveloped spectrum is applied for each of the orthogonal directions of earthquake motion. The staff finds this acceptable, since it complies with SRP Acceptance Criteria 3.7.3.II.9. The responses due to support motion are combined with the inertial responses by the absolute sum method. Since this is a conservative method based on a bounding approach as discussed above, the staff finds this approach acceptable.

Independent Support Motion

FSAR Tier 2, Section 3.7.3.9.2, “Independent Support Motion Method,” addresses the ISM method in which supports are divided into groups, and separate ISRS appropriate to each group’s location are applied in the analysis. For systems analyzed using this method of analysis, the criteria presented in NUREG-1061 are followed. On this basis, the staff finds the method acceptable.

In addition to the USM and ISM approaches, the FSAR description includes the use of support motion time histories that may be used as system input. If this approach is used, the relative displacements at support points are combined by the SRSS method. In RAI 320, Question 03.07.03-37, the staff requested that the applicant provide additional details regarding an analysis using time history support motion inputs as the FSAR was unclear to the staff under what conditions this would be applied or what assumptions would be used in the analysis.

In a July 24, 2011, response to RAI 320, Question 03.07.03-37, the applicant deleted this provision in the description of ISM. Instead, in FSAR Tier 2, Section 3.7.3.9, Revision 3-interim, provided with this response, the applicant added a statement that SRP Acceptance Criteria 3.7.3.II.9 for multiple-supported equipment will be met as described in Topical Report ANP-10264NP-A, “U.S. EPR Piping Analysis and Pipe Support Design.” Since this FSAR section has been revised to include the acceptance criteria of the SRP and the time history motion methodology identified in FSAR Tier 2, Section 3.7.3.9.2, Revision 2, has been deleted, the staff finds this acceptable. **RAI 320, Question 03.07.03-37 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed changes.

3.7.3.4.10 Use of Equivalent Vertical Static Factors

FSAR Tier 2, Section 3.7.3.10, “Use of Equivalent Vertical Static Factors,” states that equivalent vertical static factors are not used in the design of subsystems.

3.7.3.4.11 Torsional Effects of Eccentric Masses

FSAR Tier 2, Section 3.7.3.11, “Torsional Effects of Eccentric Masses,” describes how the effects due to the eccentricity of a mass connected to a system are included in the system’s dynamic model. For flexible components having a frequency less than the cut-off frequency, the stiffness of the link connecting the center of gravity of the mass to the subsystem is included. When the frequency of the component is greater than the cutoff frequency, a rigid link is used to connect the center of gravity of the mass to the subsystem. Since torsional effects are included in the system analysis and the effect of the eccentric mass for connected components is adequately modeled to reflect the component frequencies, in accordance with the guidance of SRP Acceptance Criteria 3.7.3.II.11, the staff finds the techniques for modeling torsion effects of eccentric masses acceptable.

3.7.3.4.12 Buried Seismic Category I Piping, Conduits, and Tunnels

FSAR Tier 2, Revision 2, Section 3.7.3.12, “Buried Seismic Category I Piping and Conduits,” provides a general description of the analysis and design of Seismic Category I buried pipe, electrical conduit banks, and tunnels used in the U.S. EPR design. The responsibility for design of buried piping, conduit, and tunnels rests with the COL applicant.

Methods for seismic analysis and design of safety-related pipe buried in soil are provided in AREVA Topical Report ANP-10264 NP-A, “U.S. EPR Piping Analysis and Pipe Support Design,”

Section 3.10, which, as stated earlier in this section of the report, the staff has reviewed and finds acceptable.

The seismic design of buried utilities other than piping buried in soil, is done in accordance with ASCE Report, “Seismic Response of Buried Pipe and Structural Components,” ASCE Committee on Seismic Analysis of Nuclear Structures and Material, American Society of Civil Engineers, 1983. This report is provided for use as a guidance document in SRP Acceptance Criteria 3.7.3.II.12 and therefore is acceptable to the staff. Concrete components of buried utilities are designed to satisfy the standards of ACI 349 which is recommended for use in the design of concrete structures for nuclear power plants in RG 1.142 and is, therefore, acceptable to the staff.

FSAR Tier 2, Sections 3.8.4.1.8, “Buried Conduit and Duct Banks,” 3.8.4.1.9, “Buried Pipe and Pipe Ducts,” and 3.8.4.4, “Design and Analysis Procedures,” provide for the design of buried conduit, duct banks, pipe and pipe ducts. FSAR Tier 2, Section 3.8.4.4, states that a COL applicant that references the U.S. EPR design certification will describe the design and analysis procedures used for buried conduit and duct banks, and for buried pipe and pipe ducts. FSAR Tier 2, Table 1.8-2, COL Information Items 3.8-4 and 3.8-5 state that action is required by the COL applicant to provide a description of the Seismic Category I buried conduit and duct banks, and buried pipe and pipe ducts, respectively. COL Information Item 3.8-7 of the same table states that the COL applicant will confirm the specific conditions for Seismic Category I buried conduit, electrical duct banks, pipe, and pipe ducts satisfy the provisions of AREVA Topical Report ANP-10264-NP-A on piping, FSAR Tier 2, Section 3.8.4.4.5, “Buried Conduit and Duct Banks, and Buried Pipe and Pipe Ducts.” Since these sections assign the design responsibility specifically to the COL applicant, and the staff will determine their adequacy in the review of the COL application, the staff finds these acceptable.

Included in FSAR Tier 2, Section 3.7.3.12, are criteria related to the limitation of tensile strains for buried carbon steel and stainless steel pipe. Also discussed in this section are limits on compressive strains, although no compressive limits are provided. No reference is given for this information. As discussed above, it is up to the COL applicant to provide the design for buried pipe. Therefore, it is unclear to the staff why this information was included in the FSAR. Therefore, in RAI 508, Question 03.07.03-41, the staff requested that the applicant provide additional information with respect to this criteria and why it was included in the FSAR.
RAI 508, Question 03.07.03-41 is being tracked as an open item.

3.7.3.4.13 Methods of Seismic Analysis of Category I Concrete Dams

FSAR Tier 2, Revision 2, Section 3.7.3.13, “Methods for Seismic Analysis of Category I Concrete Dams,” states that no Seismic Category I concrete dams are used in the U.S. EPR design. A COL applicant will provide a description of methods used for the analysis of site-specific Category I dams that are used in conjunction with the U.S. EPR design. This has been identified as COL Information Item 3.7-3 in FSAR Tier 2, Table 1.8-2. On this basis, the staff finds this information acceptable.

3.7.3.4.14 Methods for Seismic Analysis of Aboveground Tanks

FSAR Tier 2, Revision 2, Section 3.7.3.14, “Methods for Seismic Analysis of Aboveground Tanks,” describes the methods of analysis for above ground tanks. Seismic analyses of above-ground tanks consider impulsive and convective forces of the contained water, as well as the flexibility of the tank walls, floor, and ceiling. Impulsive forces are calculated by conventional methods for tanks determined to be rigid. For non-rigid tanks, the effect of tank flexibility on

spectral acceleration is included when determining the hydrodynamic pressure on the tank wall for the impulsive mode.

Convective forces resulting from the sloshing of water are calculated based on the natural frequency of the sloshing water. The natural frequency is used with the 0.5 percent damping curve to determine the spectral acceleration. Guidance from USAEC TID-7024, “Nuclear Reactors and Earthquakes, 1963,” is used to calculate the forces, which are applied as pressures and used in the design of the tank structure. Since the description of the analysis meets SRP Acceptance Criteria 3.7.3.II.14, the staff finds it acceptable.

3.7.3.4.15 FSAR Tier 1 Information

FSAR Tier 1: The staff has reviewed the FSAR Tier 1 information related to this FSAR section and finds it is acceptable as the interface requirements identified in Section 4.0 are consistent with those specified in FSAR Tier 2, Section 3.7.3.

3.7.3.5 *Combined License Information Items*

Table 3.7.3-1 provides a list of seismic subsystem analysis related COL information item numbers and descriptions from FSAR Tier 2, Table 1.8-2:

Table 3.7.3-1 U.S. EPR Combined License Information Items

Item No.	Description	FSAR Tier 2 Section
3.7-3	A COL applicant that references the U.S. EPR design certification will provide a description of methods used for seismic analysis of site-specific Category I concrete dams, if applicable.	3.7.3.13

The staff finds the above listing to be complete. Also, the list adequately describes actions necessary for the COL applicant. No additional COL information items need to be included in FSAR Tier 2, Table 1.8-2 for seismic subsystem analysis consideration.

3.7.3.6 *Conclusions*

Except for the open item identified above, the staff concludes that the seismic subsystem analysis method used in the design of plant SSCs is adequate and meets applicable requirements of 10 CFR Part 50, Appendix A, GDC 2;; 10 CFR 52.47(a)(1); and 10 CFR Part 50, Appendix S with respect to the capability of structures to withstand the effects of earthquakes. This conclusion is based on the following considerations:

Appropriate consideration was given for the importance of the safety functions to be performed for the different categories of subsystems as required by GDC 2. The evaluation of subsystem analysis included a review of the procedures for performing seismic analysis, procedures used for analytical modeling, the basis for selection of frequencies, analysis procedures for damping, procedures for uniform support motion and independent support motion, interaction of Non-Seismic Category I SSCs with Seismic Category I SSCs, and the methods for the seismic analysis of above ground tanks. The SSE ground motion occurring in three orthogonal directions is the basis for the input to subsystem seismic analysis and design. The seismic

methods used and the SSE ground motion, which is either a direct input or is an input to the structures which provide support to subsystems, provide assurance that appropriate earthquake loads have been developed for the design of safety-related SSCs. The use of the seismic analysis procedures and criteria implemented by the applicant provides an acceptable basis for the seismic design and complies with the requirements of GDC 2.

3.8 Design of Seismic Category I Structures and Foundations

FSAR Tier 2, Section 3.8, "Design of Category I Structures," describes the Seismic Category I structures and their foundations. The Seismic Category I structures include the structures that are located on the NI common basemat foundation and those that are located separately from the NI. The Seismic Category I structures located on the NI consist of the Reactor Building (RB); Reactor Building Internal Structures; Safeguard Buildings 1, 2, 3, and 4; and Fuel Building (FB). The RB consists of two concrete shell structures, the inner RCB and the outer Reactor Shield Building. The Seismic Category I structures adjacent to the NI are two EPGBs and four ESWBs, which have their own independent basemat foundations. Also, safety-related buried conduit, duct banks, pipes, and pipe ducts, which are Seismic Category I and outside the NI common basemat structure, are described. FSAR Tier 2, Section 3.8 and FSAR Tier 2, Appendix 3E, "Design Details and Critical Sections for Safety-Related Category I Structures," describe the design, construction, testing, and inservice inspection (ISI) requirements of structures and their foundations. FSAR Tier 1 information of the above Seismic Category I structures and foundations is provided in FSAR Tier 1, Sections 2.1.1, "Nuclear Island"; 2.1.2, "Emergency Power Generating Buildings"; 2.1.5, "Essential Service Water Building"; 3.5, "Containment Isolation"; and 4.6, "Buried Conduit and Duct Banks, and Pipe and Pipe Ducts."

3.8.1 Concrete Containment

3.8.1.1 *Introduction*

As described in FSAR Tier 2, Section 3.8.1, "Concrete Containment," the Reactor Containment Building is a post-tensioned concrete pressure vessel located inside the reinforced concrete Reactor Shield Building. The RCB houses the Reactor Building Internal Structures, and also controls the release of airborne radioactivity following the design basis accidents (DBAs) and provides radiation shielding for the reactor core and the reactor coolant system. FSAR Tier 2, Section 3.8.1 addresses the concrete portions of the RCB and the liner plate system, while FSAR Tier 2, Section 3.8.2, "Steel Containment," addresses the steel elements of the RCB that are not backed by concrete (e.g., equipment hatch, personnel airlocks, and other penetrations).

3.8.1.2 *Summary of Application*

FSAR Tier 1: The FSAR Tier 1 information associated with this section is found in FSAR Tier 1, Section 2.1.1 and Section 3.5.

The RCB is a Seismic Category I cylindrical concrete structure, post-tensioned, with an outside diameter of approximately 49.4 m (162 ft) and a height of approximately 66.4 m (218 ft); it has a 6.35 mm (0.25 in.) thick steel liner. The primary functions of the RCB are to protect the safety-related SSCs located within it, to prevent the release of radiation during plant operations, and to prevent the release of radiation and contamination in the event of accident conditions. The RCB is designed to retain its pressure boundary integrity during and following design-basis events.

The function of containment isolation is to isolate fluid system piping that penetrates the RB to prevent the discharge of radioactivity outside containment following design-basis accidents. FSAR Tier 1, Section 3.5 includes containment isolation barriers not included in FSAR Tier 1, Chapter 2, "System Based Design Description of ITAAC." Detailed FSAR Tier 1 mechanical, electrical, and instrumentation information associated with containment isolation equipment is specified in FSAR Tier 1, Tables 3.5-1, "Containment Isolation Equipment Mechanical Design," and 3.5-2, "Containment Isolation Equipment I&C and Electrical Design."

FSAR Tier 2: The applicant has provided an FSAR Tier 2 description of the concrete containment in Section 3.8.1, summarized as follows:

Description of Containment

The RCB is a reinforced concrete post-tensioned shell type structure which consists of an upright cylinder and a spherical dome top. FSAR Tier 2, Section 3.8.1.1, "Description of the Containment," indicates that the RCB has an outside diameter of approximately 49.4 m (162 ft) and a height of approximately 66.4 m (218 ft). The cylindrical wall thickness is 1.3 m (4 ft 3 in.), and the dome thickness is 1.0 m (3 ft 3 3/8 in.). The NI foundation basemat, which supports the containment and is part of the containment pressure boundary, is 3.05 m (10 ft) thick beneath the containment liner plate. Above the liner plate, additional concrete is placed in order to provide support to the RBIS.

There are three vertical concrete buttresses that extend outward from the containment wall to provide a means to anchor the hoop tendons. To anchor the vertical tendons, a tendon gallery is provided at the cylindrical containment wall beneath the basemat, and a ring girder is provided at the intersection point of the top of the containment wall and the containment dome.

The post-tensioning system utilizes hoop, vertical, and gamma tendons which are grouted in place after tensioning. There are a total of 119 horizontal hoop tendons that are installed circumferentially around the containment wall. There are a total of 47 vertical tendons that are installed around the containment wall. These tendons span between the tendon gallery and the ring girder. In the vertical direction, there are also 104 gamma tendons which initiate at the tendon gallery, pass vertically through the RCB wall, and then continue to wrap over the dome, terminating at the ring girder on the opposite side of the wall.

The entire inner surface of the RCB, including the basemat, has a leak-tight carbon steel liner plate. This steel plate is 6.35 mm (0.25 in.) thick and is thickened in localized regions such as containment penetrations, large brackets, and major attachments. The liner plate is stiffened circumferentially and vertically with structural steel shapes which are fully embedded in the concrete.

Applicable Codes, Standards, and Specifications

FSAR Tier 2, Section 3.8.1.2.1, "Codes," identifies a number of different codes as being relevant to the design, fabrication, construction, testing, and ISI of the RCB. Some of the key codes given are the 2004 Edition of the ASME B&PV Code (ASME Code), Section III, Division 2, Subsection CC, "Code for Concrete Reactor Vessels and Containments"; the 2001 Edition of ACI 349 (ACI 349-01), "Code Requirements for Nuclear Safety-Related Concrete Structures"; and the 2005 Edition of ANSI/American Welding Society (ANSI/AWS) D1.4, "Structural Welding of American National Standards Institute/ American Welding Society Code-Reinforcing Steel."

FSAR Tier 2, Section 3.8.1.2.2, “Standards and Specifications,” indicates that industry standards, such as those published by the American Society for Testing and Materials (ASTM), are used for specifying material properties, testing procedures, fabrication, and construction methods. The applicable standards used are given in FSAR Tier 2, Section 3.8.1.6, “Materials, Quality Control, and Special Construction Techniques.” Structural specifications, which address areas related to the design and construction of the RCB, include those for concrete material properties; mixing, placing, and curing of concrete; reinforcing steel and splices; post-tensioning systems; and liner plate systems.

FSAR Tier 2, Section 3.8.1.2.4, “Regulations,” identifies the applicable NRC regulations. These consist of 10 CFR Part 50, “Domestic Licensing of Production and Utilization Facilities”; 10 CFR Part 50, Appendix A, “General Design Criteria for Nuclear Power Plants”; GDC 1, “Quality Standards and Records”; GDC 2, “Design Bases for Protection Against Natural Phenomena”; GDC 4, “Environmental and Dynamic Effects Design Bases”; GDC 16, “Containment Design,” and GDC 50, “Containment Design Basis”; 10 CFR Part 50, Appendix J, “Primary Reactor Containment Leakage Testing for Water-Cooled Power Reactors”; and 10 CFR Part 100, “Reactor Site Criteria.”

FSAR Tier 2, Section 3.8.1.2.5, “NRC Regulatory Guides,” identifies the Regulatory Guides applicable to the design and construction of the RCB. These consist of RG 1.7, “Control of Combustible Gas Concentrations in Containment,” March 2007, Revision 3; RG 1.35.1, “Determining Prestressing Forces for Inspection of Prestressed Concrete Containments,” July 1990; RG 1.84, “Design, Fabrication, and Materials Code Case Acceptability, ASME Code, Section III,” August 2005, Revision 33; RG 1.90, “Inservice Inspection of Prestressed Concrete Containment Structures with Grouted Tendons,” August 1977, Revision 1; RG 1.94, “Quality Assurance Requirements for Installation, Inspection, and Testing of Structural Concrete and Structural Steel During the Construction Phase of Nuclear Power Plants,” April 1976, Revision 1; RG 1.107, “Qualifications for Cement Grouting for Prestressing Tendons in Containment Structures,” February 1977, Revision 1; RG 1.136, “Design Limits, Loading Combinations, Materials, Construction, and Testing of Concrete Containments,” March 2007, Revision 3 (with a noted exception); RG 1.199, “Anchoring Components and Structural Supports in Concrete,” November 2003 (with a noted exception); and RG 1.216, “Containment Structural Integrity Evaluation for Internal Pressure Loadings Above Design-Based Pressure,” August 2010.

Loads and Load Combinations

FSAR Tier 2, Section 3.8.1.3, “Loads and Load Combinations,” describes the loads and load combinations for the RCB. The FSAR indicates that the loads and load combinations are in accordance with the requirements of ASME Code, Section III, Division 2, Article CC-3000 and the recommendations of RG 1.136. The RCB design is based on the 2004 Edition of the ASME Code, including consideration of the exceptions taken in RG 1.136.

FSAR Tier 2, Section 3.8.1.3.1, “Design Loads,” describes the service loads and factored loads used in the design of the RCB. The service loads include dead loads, live loads, hydrostatic loads, normal thermal loads, tendon loads, and construction loads. The factored loads include wind loads, safe-shutdown earthquake loads, DBA pressure and temperature loads, and pipe break loads. Other loads, such as those generated by the combustible gas resulting from a fuel-clad coolant reaction, are described as well.

FSAR Tier 2, Section 3.8.1.3.2, “Design Load Combinations,” identifies the specific load combinations for the RCB. Load combinations are presented for the concrete sections and the

steel liner plate. The FSAR indicates that the load combinations used for design are in accordance with NUREG-0800, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants: LWR Edition," (hereafter referred to as NUREG-0800 or SRP), Section 3.8.1, "Concrete Containment," Revision 2.

Design and Analysis Procedures

FSAR Tier 2, Section 3.8.1.4, "Design and Analysis Procedures," describes the design and analysis procedures of the RCB. The FSAR indicates that the analysis and design of the post-tensioned RCB comply with the requirements of ASME Code, Section III, Division 2, Subarticle CC-3300, and conform to the guidance of RG 1.136.

As described in FSAR Tier 2, Section 3.8.1.4.1, "Computer Programs," the containment structure is included in an overall model developed for analysis of the entire NI common basemat structure. The NI common basemat structure includes the RCB, RBIS, RSB, SBs, FB, and the NI foundation basemat. The RCB structure is modeled and analyzed using the ANSYS computer program. Four and five layers of brick finite elements are primarily used to model the RCB wall and dome. Forces from the tendons, obtained from a separate ANSYS finite element model, are applied to the RCB model in the overall NI common basemat structure. Additional information of the basemat foundation and modeling of the soil as springs is presented separately in FSAR Tier 2, Section 3.8.5, "Foundations."

The ANSYS finite element model is used to analyze the RCB for the loads defined in FSAR Tier 2, Section 3.8.1.3.1. The results from these separate load analyses are then combined and factored using the load combinations defined in FSAR Tier 2, Section 3.8.1.3.2. The FSAR states that the design of the RCB is generally controlled by load combinations which contain the +427/-20.7 kPaG (+62/-3 psig) design internal pressure load and SSE seismic loads.

Localized analyses are performed separately to evaluate local loadings and discontinuities in the structures such as openings and localized changes in member cross-sections. Then, the analysis results from the local analyses are combined with those from the overall global analyses to obtain the final design.

FSAR Tier 2, Appendix 3E provides further details of the analysis and design for the selected critical sections of the RCB.

Structural Acceptance Criteria

FSAR Tier 2, Section 3.8.1.5, "Structural Acceptance Criteria," indicates that the acceptance limits for the RCB in terms of allowable stresses, strains, deformations, and other design criteria are in accordance with ASME Code, Section III, Division 2, Subarticle CC-3400, and RG 1.136. These limits are applicable to the RCB except for those steel elements that are not backed by concrete, which are addressed in FSAR Tier 2, Section 3.8.2. The allowable concrete stresses for factored loads and service loads are in accordance with ASME Code, Section III, Division 2, Subarticles CC-3420 and CC-3430, respectively.

For the liner plate and its anchorage components, the stress and strain limits are in accordance with ASME Code, Section III, Division 2, Tables CC-3720-1 and CC-3730-1. Acceptance criteria for concrete embedments and anchors are in accordance with the 2006 Edition ACI 349 (ACI 349-06), Appendix D and guidance in RG 1.199, with exceptions noted in the FSAR regarding the use of ACI 349-06.

Materials, Quality Control, and Special Construction Techniques

FSAR Tier 2, Section 3.8.1.6.1, “Concrete Materials,” describes the materials, quality control, and special construction techniques. The materials and quality control are in accordance with the ASME Code, Section III, Division 2; RG 1.107, Revision 1; RG 1.136, Revision 3, and RG 1.216.. The concrete minimum compressive strength (f'_c) is equal to 48.3 MPa (7,000 psi). FSAR Tier 2, Section 3.8.1.6.1 identifies the concrete mix design and standards followed for the cement, aggregates, admixtures, water, and placement of the concrete.

FSAR Tier 2, Section 3.8.1.6.2, “Reinforcing Steel and Splice Materials,” identifies the materials for the reinforcing steel and splice materials. The reinforcing steel used for the RCB conforms to ASTM A615 or ASTM A706, and the requirements in the ASME Code, Section III, Division 2, Subarticle CC-2330.

As described in FSAR Tier 2, Section 3.8.1.6.3, “Tendon System Materials,” the tendons of the post-tensioning system meet the requirements of the ASME Code, Section III, Division 2, Subarticle CC-2400. The post-tensioning system used is the Freyssinet C-range system in which the tendons have a minimum ultimate strength of 1,862 MPa (270 ksi). Each tendon consists of 55 strands, with a total cross sectional area equal to 82.3 cm^2 (12.76 $in.^2$).

The liner plate, which is 6.35 mm (0.25 in.) thick, is constructed from SA-516, Grade 55, 60, 65, or 70 materials, in accordance with the ASME Code, Section III Division 2, Subarticle CC-2500.

FSAR Tier 2, Section 3.8.1.6.8, “Special Construction Techniques,” indicates that special techniques are not used for construction of the RCB. Modular construction techniques are used to some extent for prefabricating portions of the containment liner, equipment hatch, airlocks, penetrations, reinforcing steel, tendon conduits, and concrete formwork. As stated in the FSAR, such construction methods have been previously used in the nuclear industry.

Testing and Inservice Surveillance Requirements

FSAR Tier 2, Section 3.8.1.7, “Testing and Inservice Inspection Requirements,” describes the structural integrity test (SIT) and long-term surveillance of the RCB. After completion of construction, a proof test of the RCB is performed at 115 percent of the design pressure. Deflection measurements and concrete crack inspections are performed during this test to confirm that the RCB response is within the limits predicted during the analysis and design of the RCB. The SIT is performed in accordance with the ASME Code, Section III, Division 2, Article CC-6000.

Throughout its service life, the RCB is monitored periodically in accordance with 10 CFR 50.55a, “Codes and Standards”; 10 CFR Part 50, Appendix J; and ASME Code, Section XI, Subsections IWE and IWL. These ISI requirements include pressurization of the containment for identification of leakage, measurements of RCB deformation, and visual examination of the condition of the containment surfaces.

For the ISI inspection of grouted tendons, the applicant provided a procedure for testing and inspections, based on the guidelines of RG 1.90, Revision 1.

ITAAC, and Technical Specifications,

ITAAC: The ITAAC associated with FSAR Tier 2, Section 3.8.1 are given in FSAR Tier 1, Table 2.1.1-8, “Reactor Building Plan Elevation +64 ft.” The ITAAC associated with

containment isolation equipment are contained in FSAR Tier 1, Chapter 2 and in FSAR Tier 1, Table 3.5-3, “Containment Isolation ITAAC (6 Sheets).”

Technical Specifications: The Technical Specifications associated with FSAR Tier 2, Section 3.8.1 are given in FSAR Tier 2, Chapter 16, “Technical Specifications,” Section 3.6, “Containment Systems,” including specifications on the containment, airlocks, containment isolation valves, containment pressure, containment temperature, Shield Building, annulus ventilation system, and pH adjustment.

3.8.1.3 *Regulatory Basis*

The relevant requirements of NRC regulations for this area of review, and the associated acceptance criteria, are given in NUREG-0800, Section 3.8.1, “Concrete Containment,” and are summarized below. Review interfaces with other SRP sections also can be found in NUREG-0800, Section 3.8.1.

1. 10 CFR Part 50, specifically 10 CFR 50.55a, as it relates to codes and standards and the inservice inspection of concrete containments.

2. GDC 1, as it relates to the concrete containment being designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety function to be performed.

3. GDC 2, as it relates to the design of the concrete containment being able to withstand the most severe natural phenomena such as winds, tornadoes, floods, and earthquakes and the appropriate combination of all loads.

4. GDC 4, as it relates to the concrete containment being appropriately protected against dynamic effects, including the effects of missiles, pipe whipping, and discharging fluids due to blowdown loads associated with the loss-of-coolant accident.

5. GDC 16, as it relates to the capability of the concrete containment to act as an essentially leak-tight barrier to prevent the uncontrolled release of radioactive effluents to the environment.

6. GDC 50, as it relates to the concrete containment being designed with sufficient margin of safety to accommodate appropriate design loads.

7. GDC 53, “Provisions for Containment Testing and Inspection,” as it relates to the containment being designed to permit appropriate ISI.

8. 10 CFR 50.44, “Combustible Gas Control for Nuclear Power Reactors,” as it relates to demonstrating the structural integrity of the containment for loads associated with combustible gas generation.

Acceptance criteria adequate to meet the above requirements include:

1. RG 1.7, “Control of Combustible Gas Concentrations in Containment,” as it relates to the capability of the containment to withstand internal pressurization due to 100 percent fuel-clad coolant reaction followed by hydrogen burning.

2. RG 1.90, “Inservice Inspection of Prestressed Concrete Containment Structures with Grouted Tendons,” as it relates to the structural inspection of a concrete containment with grouted tendons.

3. RG 1.91, “Evaluations of Explosions Postulated to Occur on Transportation Routes near Nuclear Power Plants,” as it relates to evaluations of nuclear power plant (NPP) structures affected by nearby explosions postulated for transportation routes.

4. RG 1.107 “Qualifications for Cement Grouting for Prestressing Tendons in Containment Structures,” as it relates to evaluating the quality of cement grouting for prestressing tendons in containment structures.

5. RG 1.115, “Protection Against Low Trajectory Turbine Missiles,” as it relates to evaluation of the effect of low trajectory turbine missiles on structures.

6. RG 1.136, “Design Limits, Loading Combinations, Materials, Construction, and Testing of Concrete Containments,” as it relates to the design of concrete containments.

7. RG 1.216, “Containment Structural Integrity Evaluation for Internal Pressure Loadings Above Design-Basis Pressure,”as it relates to structural loads and acceptance criteria associated with internal pressurization due to 100 percent fuel-clad coolant reaction followed by hydrogen burning, and to the evaluation of internal pressure capacity of the containment.

3.8.1.4 *Staff Evaluation*

The staff's evaluation is based on a complete review of FSAR Tier 2, Section 3.8.1.1, Section 3.8.1.2, Section 3.8.1.3, Section 3.8.1.4, Section 3.8.1.5, Section 3.8.1.6, and Section 3.8.1.7 including all their subsections.

This section describes the staff’s review and evaluation of FSAR Tier 2, Section 3.8.1 and subsequent revisions against the guidance of SRP Section 3.8.1 to ensure that the applicant has adequately addressed identified technical issues.

<u>Description of Containment</u>

The staff reviewed the description of the RCB to ensure that it contains sufficient information to define the primary structural aspects and elements that are relied upon to perform the containment function. With the exception described below, the staff concluded that the description met the applicable regulatory requirements, regulatory guidance, and the ASME Code.

The review of FSAR Tier 2, Section 3.8.1.1.1, “Concrete Wall and Dome Shells and Connection to Foundation,” Revision 0, through Section 3.8.1.1.3, “Liner Plate System,” and Appendix 3E, identified a number of items that were not described in sufficient detail. These included design details of the RCB dome, representative details for attachment of equipment to the RCB wall

and dome, and design details of the RCB liner plate, including anchorage and stiffeners. This information was needed to establish the primary structural elements of the RCB that are relied upon to transfer loads and to perform their intended safety functions. Therefore, in RAI 155, Question 03.08.01-3, the staff requested that the applicant address these concerns and provide additional design details for the items given above.

In an April 21, 2010, response to RAI 155, Question 03.08.01-3, the applicant provided descriptive information for the polar crane support brackets; attachments of miscellaneous components related to electrical and HVAC equipment; and the RCB liner plate, including liner plate thickness, sizes and spacing of stiffeners, and clarification that the stiffeners also function as anchorage. In addition, the applicant indicated that design details for the RCB dome would be provided in their future response to RAI 155, Questions 03.08.01-24 and 03.08.04-6. The applicant addressed the staff's concerns related to the identification of the load path for the structural elements. Issues related to the methodology used to design the RCB liner plate, including its stiffeners and anchorage, are evaluated in RAI 155, Question 03.08.01-8 and RAI 448 Question 03.08.01-54, discussed below in "Design and Analysis Procedures." Issues related to design details for the RCB dome are evaluated in RAI 155, Questions 03.08.01-24 and 03.08.04-6, also discussed below in "Design and Analysis Procedures." Accordingly, based on the applicant's April 21, 2010, response to RAI 155, Question 03.08.01-3, the staff finds that the applicant has provided an adequate description of the containment structure to comply with SRP Section 3.8.1.II.1 Acceptance Criteria. Therefore, the staff considers RAI 155, Question 03.08.01-3 resolved. The technical adequacy of the information provided will be discussed below in various portions of this report.

Applicable Codes, Standards, and Specifications

To ensure that the containment can perform its intended safety function, the staff verified that the applicable codes, standards, and specifications used in the design, construction, testing, and ISI meet the relevant requirements in 10 CFR 50.55a; GDC 1, GDC 2, GDC 4, GDC 16, GDC 50, and GDC 53; and 10 CFR 50.44; and are in accordance with SRP Section 3.8.1.II.2 Acceptance Criteria.

FSAR Tier 2, Section 3.8.1.2, identifies the codes, standards, and specifications used in the design, construction, testing, and ISI surveillance of the RCB. The codes identified include the ASME Code, Sections II, III (Division 2), V, VIII, IX, and XI. The standards and specifications include those for concrete material properties; mixing, placing, and curing of concrete; reinforcing steel and splices; post-tensioning system; and liner plate. These include various ASTM and ACI standards which are specifically identified in various locations in FSAR Tier 2, Section 3.8 where appropriate (e.g., FSAR Tier 2, Section 3.8.1.6).

The staff finds the codes, standards, and specifications in FSAR Tier 2, Section 3.8.1.2, applicable to the RCB, comply with those identified in SRP Section 3.8.1.II.2 Acceptance Criteria and 10 CFR 50.55a, and are therefore acceptable.

Loads and Load Combinations

To ensure that the containment can perform its intended safety function, the staff verified that the loads and load combinations used in the structural design meet the relevant requirements in 10 CFR 50.55a; GDC 1, GDC 2, GDC 4, GDC 16, GDC 50, and GDC 53; and 10 CFR 50.44; and are in accordance with ASME Code, Article CC-3000, and SRP Acceptance Criteria 3.8.1.II.3. With the exceptions described below, the staff finds that the loads and load

combinations meet the regulatory requirements, regulatory guidance, and the ASME Code cited above.

The staff determined that certain loads described in FSAR Tier 2, Section 3.8.1.3.1, were not described in sufficient detail to determine whether they complied with SRP Section 3.8.1.II.3 Acceptance Criteria. These included dead loads associated with the weight of miscellaneous components (e.g., cable tray systems, conduit systems, heating, ventilation, and air condition (HVAC) systems), uniformly distributed dead loads (floor, platform, and wall), uniformly distributed live loads (floor, platform, and roof), and hydrostatic loads. In addition, it was unclear to the staff whether the masses corresponding to all applicable dead loads and a percentage of live loads were included in the masses used in the seismic analysis. The staff was concerned about whether the applicant was considering all applicable gravity and seismic inertial loads. Therefore, in RAI 155, Question 03.08.01-5, the staff requested that the applicant provide the magnitude of the gravity and seismic inertial loads, and confirm that these were included in the structural design as applicable.

In a March 31, 2009, response to RAI 155, Question 03.08.01-5, the applicant provided the requested design information, including the magnitude of the gravity and seismic inertial loads and proposed revisions to FSAR Tier 2, Section 3.8.1.3.1. The staff finds this information technically acceptable, because the treatment of all applicable gravity and seismic inertial loads complies with the acceptance criteria in SRP Sections 3.7.2.II.3.D, "Representation of Floor Loads, Live Loads, and Major Equipment in Dynamic Model"; 3.8.1.II.3, "Loads and Load Combinations (for concrete containment"; and 3.8.3.II.3, "Loads and Load Combinations" (for containment internal structures). For the case of gravity loads, the information meets industry standards such as the 2005 Edition of ASCE 7 (ASCE 7-05), "Minimum Design Loads for Buildings and Other Structures." The staff finds these other standards acceptable because they are the industry consensus standards that give acceptable results for similar loading applications in the staff's experience. The staff also verified that the changes proposed by the applicant were incorporated into FSAR Tier 2, Revision 2. Therefore, the staff considers RAI 155, Question 03.08.05-5 resolved.

FSAR Tier 2, Sections 3.8.1.3.1 and 3.8.2.3.1, "Design Loads," discuss the hydrogen pressurization loads that result from a fuel-clad coolant reaction and uncontrolled hydrogen burn. Referring to RG 1.136, Revision 3, and RG 1.7, Revision 3, the applicant applied a pressure of 310 kPaG (45 psig) combined with dead loads as a minimum design condition for hydrogen pressurization loads. Since the RCB design pressure is higher than 310 kPaG (45 psig), the applicant indicated that the load combinations that include the design pressure subsume the hydrogen pressurization loads.

The staff determined that the applicant's approach did not conform to RG 1.136, Revision 3; RG 1.7, Revision 3; and SRP Acceptance Criteria 3.8.1.II.3 and 3.8.2.II.3, which specify that the pressure used in the evaluation of containment integrity should be the higher of (1) pressure from the hydrogen generation/burn event and (2) 310 kPaG (45 psig). In addition, the applicant referred to stress and strain evaluations of the containment, but, the regulatory guides refer to acceptance criteria based on liner strain limits, not stress, limits for establishing a barrier to fission product releases.

The staff was concerned that the applicant did not adequately evaluate the containment integrity for hydrogen pressurization loads based on the regulatory guides. Therefore, in RAI 155, Question 03.08.01-6, the staff requested that the applicant identify the maximum load from the hydrogen generation/burn event; evaluate the containment integrity for the higher of the

pressure from this event or 310 kPaG (45 psig); and include the proper loads, load combinations, acceptance criteria, and analysis description in the FSAR.

In an April 21, 2010, response to RAI 155, Question 03.08.01-6, the applicant indicated that the maximum pressure load from the hydrogen generation/burn event due to 100 percent fuel-clad coolant reaction is 517 kPaG (75 psig), and the corresponding temperature is 287 °C (549 °F), as indicated in an October 30, 2009, response to RAI 234, Questions 19-305 and 19-306. The applicant performed nonlinear structural analyses for the containment integrity evaluation considering this maximum internal pressure, dead loads, post-tensioning loads, and two sets of reduction factors for tensile strength and modulus of elasticity to account for temperature effects. From these analyses, the applicant calculated maximum liner strains of 0.00044 in tension and 0.00096 in compression, which fall within the strain limits of 0.003 in tension and 0.005 in compression in ASME Code, Subarticle CC-3720. Proposed revisions to FSAR Tier 2, Sections 3.8.1 and 3.8.2 were also included with the response to RAI 155, Question 03.08.01-6.

The staff finds the applicant's containment integrity evaluation for the hydrogen generation/burn event acceptable because the definition of the hydrogen pressure load, load combinations associated with the hydrogen pressure load structural analysis methodology, and strain limit acceptance criteria conform to the guidance in RG 1.136, Revision 3, and RG 1.7, Revision 3; and are in accordance with SRP Sections 3.8.1.II.3 and 3.8.2.II.3 Acceptance Criteria; and also complies with ASME Code, Subarticle CC-3720. Therefore, the response to RAI 155, Question 03.08.01-6 addressed the staff's concerns related to the maximum pressure and temperature load from the hydrogen generation/burn event due to the assumed 100 percent fuel-clad coolant reaction.

During an April 26-30, 2010, onsite audit, the applicant proposed to revise FSAR Tier 2, Sections 3.8.1 and 3.8.2 to correct the definition of hydrogen pressurization loads to comply with 10 CFR 50.44; revise the calculations to address dynamic effects, if applicable; and include a summary of loads, load combinations, models, analyses, results, and acceptance criteria in the relevant sections of the FSAR.

In a May 20, 2011, revised response to RAI 155, Question 03.08.01-6, the applicant provided calculations which demonstrated that dynamic effects were not applicable, because the time-scale of the hydrogen pressurization loads is much larger than the period (inverse of the natural frequency) associated with the breathing mode of the containment. The breathing mode is the dynamic vibrational mode of the containment that expands and contracts in a uniform radial pattern. In addition, the applicant clarified that information regarding loads, load combinations, models, analyses, results, and acceptance criteria were added to FSAR Tier 2, Sections 3.8.1 and 3.8.2, as part of the resolution of RAI 354, Question 03.08.02-11. The staff finds this information technically acceptable, because it complies with SRP Sections 3.8.1.II.3 and 3.8.2.II.3 Acceptance Criteria, as indicated in the staff's evaluation of RAI 354, Question 03.08.02-11, included in Section 3.8.2.4, "Design and Analysis Procedures," of this report. Based on the above discussion, the staff considers RAI 155, Question 03.08.01-6 resolved.

Design and Analysis Procedures

The staff reviewed the structural design and analysis procedures used for the RCB to ensure compliance with the relevant requirements in 10 CFR 50.55a; GDC 1, GDC 2, GDC 4, GDC 16, GDC 50, and GDC 53; and 10 CFR 50.44; and are in accordance with ASME Code Article CC-3000 and SRP Section 3.8.1.II.4 Acceptance Criteria. The staff also reviewed the

FSAR to establish that the structural design of the RCB is essentially complete, as required by 10 CFR 52.47(c). With the exceptions described below, the staff finds that the design and analysis procedures meet the regulatory requirements, regulatory guidance, and the ASME Code cited above.

The staff's initial review of the applicant's design and analysis procedures for the NI structures, described in FSAR Tier 2, Sections 3.8.1 through 3.8.5, identified a number of technical issues including: (1) Identification, analysis, and design of critical sections; (2) equivalent-static seismic analysis methodology; (3) combination of the three directional components of seismic loads using the 100-40-40 percent rule described in ASCE 4-98 and its applicability to nonlinear analysis; and (4) analysis and design of Seismic Category I foundations, including seismic stability analyses.

Items (1) and (2) are discussed in detail in the subsequent paragraphs. Item (3) is discussed in Section 3.8.3.4 of this report and Item (4) is discussed in Section 3.8.5.4 of this report.

The staff also finds that the applicant's design procedures for the NI structures incorporated nonlinear analysis methods. In particular, the static finite element (FE) model of the NI structures (used also for the design of the RCB) explicitly included contact elements at the foundation-soil interface to simulate uplift of the structures under seismic loading conditions, which results in nonlinear structural response. The staff determined this approach did not comply with SRP Section 3.8.1.II.4, Acceptance Criteria, which assumes the design process is based on structural response that is essentially linear. In general, nonlinear analysis procedures should be used to verify whether the completed design satisfies the acceptance criteria but not to perform the design itself.

The above technical issues were conveyed to the applicant in several onsite audits conducted on January 26-30, 2009; June 19, 2009; April 26-30, 2010; and February 14-17, 2011. To address the staff's concerns, the applicant undertook a number of significant modifications to its analysis and design procedures, including: (1) Establishing a systematic critical section selection methodology (see below); (2) modification of the equivalent-static seismic analysis methodology to eliminate the use of seismic modification/reduction factors (see below); (3) use of the square root of the sum of the squares (SRSS) method instead of the 100-40-40 percent rule to combine the three directional components of seismic loads (see Section 3.8.3.4 of this report); and (4) modification of the static FE model of the NI used in design, from a single model with springs and contact elements at the soil-foundation interface to two models: (a) a fixed-base static FE model of the NI for design of the NI superstructure, and (b) an FE model of the NI with springs and contact elements at the soil-foundation interface for design of the NI foundation using dynamic time-history methods (see Section 3.8.5.4 of this report and the AREVA Path to Closure document, provided during the June 9, 2010, public meeting, and attached to the meeting summary,). These modifications were discussed with the staff during the April 26-30, 2010, audit. The staff concluded that implementing these modifications would address the technical issues identified during the initial review.

During the January 26-30, 2009, onsite audit, the staff noted that there was no documented criteria for the selection of critical sections of Seismic Category I structures within the scope of the design certification. The identification, analysis, and design of critical sections is important, because it provides reasonable assurance to the staff that the structural design of Seismic Category I structures is essentially complete, as required by 10 CFR 52.47(c). Therefore, in RAI 155, Questions 03.08.01-20, 03.08.01-24, and 03.08.04-6, the staff requested that the applicant address these concerns.

In response to the staff's questions during the January 26-30, 2009, audit, the applicant proposed a critical section selection methodology to ensure that an appropriate set of critical sections were selected for design of Seismic Category I structures. The staff reviewed this proposed methodology during the onsite audit conducted on June 19, 2009, and provided detailed comments. The applicant presented the final version of this methodology to the staff during a public meeting on September 18, 2009.

The applicant documented the critical section selection methodology in a January 28, 2010, response to RAI 155, Question 03.08.01-20. The methodology consists of three criteria: (1) qualitative; (2) quantitative; and (3) supplemental. The qualitative criterion was used to select critical sections of the NI structures that perform a safety-critical role. The quantitative criterion used the FE model of the NI structures and a numerical algorithm to select critical sections that are highly stressed, but were not chosen under the qualitative criterion. The supplemental criterion used engineering judgment to select critical sections of Seismic Category I structures that were not selected by application of the other two criteria, but are necessary to obtain an adequate representation of all types of structural elements and other buildings beyond the NI. The applicant also implemented the selection methodology and provided a list of critical sections. It should be noted that the supplemental criterion was added to the methodology in response to the staff's comments during the June 19, 2009, audit, which pointed out a potentially insufficient representation of all types of structural elements or other buildings beyond the NI.

The staff concluded that the applicant's critical section selection methodology was technically acceptable, because it provided a systematic methodology to ensure that an appropriately representative set of critical sections was selected for design (i.e., an appropriate combination of safety-critical, highly stressed, and different types of Seismic Category I structural elements, within the NI and beyond the NI). Therefore, the staff considers the issues raised in RAI 155, Question 03.08.01-20 resolved. However, the staff also noted that the list of critical sections provided by the applicant may be subject to change depending on the resolution of the various technical issues discussed in Section 3.8 of this report.

In light of the resolution of the critical section selection methodology documented in the January 28, 2010, response to RAI 155, Question 03.08.01-20, the applicant will address the remaining issues identified in RAI 155, Questions 03.08.01-24, and 03.08.04-6, related to critical sections and will complete the design of the critical sections. The January 28, 2010, response to RAI 155, Question 03.08.01-20 also indicated that the final design will be documented in FSAR Tier 2, Appendix 3E and in a future response to RAI 155, Question 03.08.04-6.

During the February 14-17, 2011, onsite audit, the staff reviewed calculations associated with the critical sections of the EPGB and ESWB, and provided comments. The staff expects to conduct an additional onsite audit to review calculations and design details associated with all critical sections when these are completed.

Based on the above discussion, the issues raised by **RAI 155, Questions 03.08.01-24 (critical section details), and 03.08.04-6 (descriptive information for critical sections) are being tracked as open items** pending the completion of the design for the critical sections.

During the January 26-30, 2009, onsite audit, the staff identified a technical issue regarding the equivalent-static seismic analysis methodology used by the applicant for the design of Seismic Category I structures. The equivalent static analysis method is also described in several location of FSAR Tier 2, Sections 3.7 and 3.8. In the application of this methodology seismic inertial loads are statically applied to an FE model in such a way that the resulting structural

response is equivalent or relatively conservative compared to the dynamic response that would be obtained from a seismic dynamic analysis of the FE model (therefore the term, “equivalent-static”).

In the applicant’s approach, the seismic inertial loads were obtained by statically applying zero-period accelerations (ZPAs, which represent the maximum accelerations at each point of the structure) to the masses defined in the FE models of Seismic Category I structures used for design. The ZPAs were taken from the seismic soil-structure interaction (SSI) analyses described in FSAR Tier 2, Sections 3.7.2.3, “Procedures Used for Analytical Modeling,” 3.7.2.4, “Soil-Structure Interaction,” and 3.7.2.5, “Development of Floor Response Spectra.” Different sets of ZPAs were obtained for each of the generic soil cases analyzed. Since the ZPAs were applied statically, the resulting structural responses were also static. In the applicant’s initial approach, the magnitudes of the ZPAs were reduced by seismic modification/reduction factors. The rationale for using these modification/reduction factors was to account for presumed over conservatisms in the responses of the structures; however, no technical justification was provided in the FSAR regarding the use of these modification/reduction factors. Therefore, in RAI 190, Question 03.08.01-28, the staff requested that the applicant clarify why these seismic modification/reduction factors were used and how they were developed.

In a separate effort, to verify the applicant’s methodology, the staff performed a confirmatory response spectrum seismic analysis of the RBIS and compared the structural responses to those using the ZPAs with the reduced seismic modification factors. The findings of the confirmatory analysis are discussed in Section 3.8.3.4, “Design and Analysis Procedures,” of this report in RAI 222, Question 03.08.03-18.

In a June 12, 2009, response to RAI 190, Question 03.08.01-28, the applicant provided a discussion of how the seismic modification/reduction factors were developed. This discussion included a comparison of results between SSI and equivalent-static analyses of the NI structures, but the comparison was limited to story shears and overturning moments (summation of shear forces and moments) computed at the base of the structures, for seismic loads corresponding to one generic soil case.

The staff determined the applicant’s June 12, 2009, response to RAI 190, Question 03.08.01-28 inadequate, because it did not provide a detailed quantitative description of the methodology used to develop the seismic modification/reduction factors. In particular, the staff could not identify all the steps involved in the methodology from the discussion provided by the applicant. In addition, the comparison of results between SSI and equivalent-static analyses was limited to story shears and overturning moments at the base of the NI structures, for seismic loads corresponding to one generic soil case. The staff notes that a meaningful comparison of results should include story shears and overturning moments at multiple elevations, for all Seismic Category I structures, and for seismic loads corresponding to all generic soil cases. Therefore, in follow-up RAI 376, Question 03.08.01-48, the staff requested that the applicant provide additional information to address these issues.

During the April 26-30, 2010, onsite audit, the applicant proposed to eliminate the use of seismic modification/reduction factors from their equivalent-static seismic analysis methodology and, instead, to use the full magnitude of ZPAs obtained from the 3-D FE SSI models being implemented in response to staff concerns regarding the applicant’s initial seismic SSI analysis based on lumped-mass dynamic models (see Section 3.7.2 of this report). The elimination of the use of seismic modification/reduction factors was confirmed in the applicant’s July 29, 2010, response to RAI 376, Question 03.08.01-48. The staff finds this response acceptable on the

basis of the findings of its confirmatory analysis of the RBIS, which demonstrated that equivalent-static seismic analysis, using full-ZPA values obtained from the more realistic 3-D FE SSI models, yielded conservative or comparable results relative to response spectrum analysis.

The staff also confirmed that the applicant's proposed changes were incorporated into FSAR Tier 2, Revision 2. Based on the above discussion, the staff considers RAI 190, Question 03.08.01-28 and RAI 376, Question 03.08.01-48 resolved.

In FSAR Tier 2, Section 3.8.1.4.11, "Containment Ultimate Capacity," the applicant indicated that analyses were performed to determine the ultimate pressure capacity of the RCB and RCB penetrations, in support of probabilistic risk assessments and severe accident analyses (see Chapter 19, "Probabalistic Risk Assement and Severe Accident Evaluation," of this report), based on the guidance in SRP Section 3.8.1.II.4.K Acceptance Criteria.

The staff determined that the applicant's approach does not comply with SRP Section 3.8.1.II.4.K Acceptance Criteria. SRP Section 3.8.1.II.4.K Acceptance Criteria recommends that the applicant should establish the ultimate pressure capacity of the containment for use as a measure of the safety margin above the DBA pressure. The calculations performed should be deterministic with minimum code-specified material properties corresponding to the DBA temperature. The staff notes that the ultimate pressure capacity calculation for use in probabilistic risk assessments (PRAs) is based on fragility analysis and should be performed separately, using different criteria than those in SRP Section 3.8.1.II.4.K Acceptance Criteria.

The staff was concerned that the applicant's evaluation of the ultimate pressure capacity of the containment did not comply with SRP Section 3.8.1.II.4.K Acceptance Criteria. Therefore, in RAI 155, Question 03.08.01-10, the staff requested that the applicant: (1) Revise the FSAR to reflect the purpose of the ultimate pressure capacity calculation; (2) provide a revised ultimate pressure capacity evaluation for various structural elements of the containment using deterministic criteria, in accordance with SRP Section 3.8.1.II.4.K Acceptance Criteria, or provide technical justification for alternative criteria; (3) provide additional details of the nonlinear structural analyses performed as part of the ultimate pressure capacity calculation; and (4) provide an explanation of the criteria used to evaluate leakage from the various containment elements (e.g., penetrations, bolted connections, seals, hatches, bellows).

In a May 29, 2009, response to RAI 155, Question 03.08.01-10, the applicant indicated that the FSAR will be revised to clarify that the ultimate pressure capacity evaluation was performed in accordance with SRP Section 3.8.1.II.4.K Acceptance Criteria, using deterministic criteria. The applicant also provided a revised ultimate pressure capacity for the RCB structure and the equipment hatch penetration, as well as summary descriptions of the finite element models, material properties, loads, nonlinear structural analyses, strain limits, leakage criteria, and limit states considered in the calculations. The ultimate pressure capacity of the containment based on the governing limit state (leakage of the equipment hatch) was determined to be 861 kPaG (125 psig), approximately 2.0 times the DBA pressure of 427 kPaG (62 psig).

The staff finds the applicant's proposed revision to the FSAR and the deterministic calculation methodology for determining the containment ultimate pressure capacity to be in accordance with SRP Section 3.8.1.II.4.K Acceptance Criteria. The applicant's May 29, 2009, response to RAI 155, Question 03.08.01-10 addressed some of the staff's concerns related to the ultimate pressure capacity of the containment. However, the staff also identified certain issues that warranted additional clarification. In particular, it was unclear to the staff why only the equipment hatch was considered in the calculations and not other major containment

penetrations. In addition, regarding the criteria used to evaluate leakage, the applicant indicated that an FE analysis of the equipment hatch was performed using contact elements. The applicant assumed that loss of contact in these elements was appropriate to simulate loss of leak-tightness; however, the staff concluded that the basis for this assumption was needed. Therefore, RAI 155, Question 03.08.01-10 was resolved, and in follow-up RAI 448, Question 03.08.01-49, the staff requested that the applicant clarify these issues.

During the February 14-17, 2011, onsite audit ,the applicant presented preliminary results of the calculations for determining the ultimate pressure capacity of all major containment penetrations and provided additional details of the criteria used to evaluate leakage, as well as details of the FE analyses used in the evaluations. The applicant indicated that this additional information would be provided in response to RAI 448, Question 03.08.01-49.

In a June 8, 2011, response to RAI 448, Question 03.08.01-49, the applicant documented the following information provided to the staff during the February 14-17, 2011, onsite audit:

1. The FE model used to determine the ultimate pressure capacity of the RCB structure represents a 2-degree slice of the RCB, with the boundary conditions chosen to simulate axisymmetric conditions. The results obtained from the nonlinear analysis of this FE model were compared to closed-form solutions based on the tensile capacity of the prestressing tendons and steel reinforcement (the steel liner has minimal contribution), and found to be in close agreement.

2. Calculations to determine the ultimate pressure capacity of all major containment penetrations (equipment hatch, construction opening, airlocks, fuel transfer tube, and main steam and feedwater lines) were performed using nonlinear FE analyses with material properties corresponding to the DBA temperature, in accordance with the guidance in SRP Acceptance Criteria 3.8.1.II.K.iii. Three different criteria were used to determine the ultimate pressure capacity of the penetrations: (a) Strain limits in ASME Code, Table CC-3720-1; (b) ASME Service Level D allowable buckling pressure; and (c) leak tightness of potential leak paths. For the equipment hatch, construction opening and airlocks, it was found that Item (b) was the controlling mode; for the fuel transfer tube and main steam and feedwater penetrations, it was found that Item (a) was the controlling mode.

3. The revised ultimate pressure capacity of the containment was determined to be 821 kPaG (119 psig), approximately 1.91 times the DBA pressure of 427 kPaG (62 psig). This was based on the controlling mode for the construction opening. The airlocks and equipment hatch also have very similar ultimate pressure capacities (821 kPaG [119 psig] and 886 kPaG [128 psig], respectively) based on the same controlling mode. All other containment structural elements and penetrations have substantially higher ultimate pressure capacities (at least higher than 1,070 kPaG [155 psig]).

4. Leak-tightness is maintained in all steel components of the containment penetrations by limiting the strains to the allowable values given in ASME Code, Table CC-3720-1. In addition, it was determined that the flanged connections in the equipment hatch and fuel transfer tube, as well as the airlock hatch seals, remain under compression at the calculated ultimate pressure, thereby ensuring the leak-tight integrity of the penetrations.

5. FSAR Tier 2, Section 3.8.1.4.11 and FSAR Tier 2, Table 3.8-6 will be revised to incorporate the revised ultimate pressure capacities of the various containment structural elements and penetrations.

The staff finds the above information acceptable, because it complies with ASME Code, Subsection CC and Subsection NE acceptance limits, and complies with SRP Acceptance Criteria Section 3.8.1.II.4.K Acceptance Criteria. In particular, the staff concluded that the ASME Service Level D allowable buckling pressures, which were obtained from the buckling analyses performed in response to RAI 354, Question 03.08.02-13, were an acceptable conservative estimate of the ultimate pressure capacities of the equipment hatch, construction opening, and airlocks because:

1. The staff evaluation of RAI 354, Question 03.08.02-13 finds the buckling analyses acceptable (see Section 3.8.2.4 of this report under “Design and Analysis Procedures”).

2. The computed strains corresponding to these pressures, also reported in the response to RAI 354, Question 03.08.02-13, are below the strain limits in ASME Code, Table CC-3720-1, which correspond to the same strain limits in RGs 1.7 and 1.136 which are used to ensure the structural integrity and leak-tightness of the containment under hydrogen generation/burn loading events.

Based on the above discussion, **RAI 354, Question 03.08.01-49 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed changes.

The staff’s review of FSAR Tier 2, Sections 3.8.1.4.4, “Transient and Localized Loads,” Revision 0, 3.8.1.4.5, “Creep, Shrinkage, and Cracking of Concrete,” and 3.8.1.4.8, “Variation in Physical Material Properties,” identified a number of technical issues associated with the structural analysis and design of the RCB to account for temperature effects, concrete creep and shrinkage, and variation in material properties. In particular, the staff was concerned with the design of the RCB post-tensioning system to account for these effects. The staff identified the following issues:

1. FSAR Tier 2, Section 3.8.1.4.4, Revision 0, discusses the effects of temperature through the RCB wall for the DBA event. It was unclear to the staff how these temperature conditions affect tendon prestress losses. In addition, the FSAR states that the annulus (i.e., the space between the RSB and RCB) temperature of 26 °C (79 °F) was assumed in the thermal transient calculations; however, FSAR Tier 2, Section 3.8.1.3.1 Revision 0, states that the annulus temperature can vary from 45 °C (113 °F) maximum to 7 °C (45 °F) minimum. It was unclear to the staff whether the design of the RCB wall can accommodate these temperature variations. Therefore, in RAI 155, Question 03.08.01-14, the staff requested that the applicant clarify these issues.

   In a February 13, 2009, response to RAI 155, Question 03.08.01-14, the applicant indicated that the calculation of tendon prestress losses due to relaxation were based on an average temperature of 48 °C (117.5 °F) and parameters based on 1,000-hour steel relaxation tests. The applicant also indicated that the temperature of 48 °C (117.5 °F) was selected because it bounds the temperatures due to the DBA event at the tendon locations, for conditions at 1 yr after a DBA event. It was noted that shorter time periods after a DBA event do not contribute significantly to tendon relaxation because total relaxation is calculated over the 60-year life of the plant. For time periods longer than 1 year after a DBA event, the temperature of the tendons would be less than 48 °C (117.5 °F) which would result in lower relaxation. In addition, the applicant indicated that

the parametric study performed in response to RAI 155, Question 03.08.01-16 (see Item (3) below) would address the impact of the annulus temperature variations.

Since the temperature of 48 °C (117.5 °F) assumed in the computation of prestress losses bounds the temperature due to a DBA event at the tendon locations, for conditions at 1 yr after a DBA event, this assumed temperature yields conservative estimates of tendon prestress losses, and is therefore acceptable. However, the staff noted that the 1,000-hour steel relaxation parameter mentioned in the applicant's response appeared to be significantly lower than values reported in the technical literature (e.g., Figure 2-15 in J. R. Libby, "Modern Prestressed Concrete," 4th Edition, Van Nostrand Reinhold, 1990). Therefore, in RAI 248, Question 03.08.01-33, the staff requested that the applicant provide the technical basis for the 1,000-hour steel relaxation parameter used in the calculations.

In a September 29, 2009, response to RAI 248, Question 03.08.01-33, the applicant indicated that the 1,000-hour steel relaxation parameter was increased by 67 percent to be in agreement with values reported in the technical literature. However, it was unclear to the staff that this revised parameter was used in the analysis and design of the RCB. This is a significant issue, because the resulting change in post-tension loads affects many aspects of the analysis and design of the RCB.

During the February 14-17, 2011, onsite audit, the applicant confirmed that the revised 1,000-hour steel relaxation parameter was used in all aspects of the analysis and design of the RCB, and that FSAR Tier 2, Table 3.8-5, "Tendon Losses and Effective Forces with Time," would be revised to incorporate the prestress loss parameters effectively used in the analysis and design of the RCB. The applicant indicated that this additional information would be provided in a supplemental response to RAI 248, Question 03.08.01-33.

In a March 24, 2011, response to RAI 248, Question 03.08.01-33, the applicant formally submitted the information provided to the staff during the February 14-17, 2011, audit. Therefore, the staff considers RAI 155, Question 03.08.01-14 resolved, while **RAI 248, Question 03.08.01-33 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed changes.

2. In FSAR Tier 2, Section 3.8.1.4.5, the applicant indicated that creep and shrinkage were assessed based on past experience. In this regard, the staff noted that SRP Section 3.8.1 recommends that creep and shrinkage values used for concrete should be established by test or from data obtained on completed containments constructed of the same concrete. It was unclear to the staff whether the applicant's past experience includes the use of 48 MPa (7,000 psi) concrete in a prestressed concrete containment and how this experience meets the requirements of ASME Code, Subarticle CC-2231.5 and SRP Section 3.8.1.II.4.D Acceptance Criteria.

   Furthermore, FSAR Tier 2, Table 3.8-5, provides tendon prestress losses due to elastic shortening, and concrete creep and shrinkage. It was unclear to the staff what material properties were used in the calculation of these losses, how the material properties were determined, and whether variations were considered in the selection of the material properties. The staff noted that ASME Code, Article CC-3000 and SRP Section 3.8.1.II.4.D Acceptance Criteria, respectively, provide specific requirements and guidelines for the determination of creep and shrinkage values to be used in the design of the RCB.

Therefore, in RAI 155, Question 03.08.01-15, the staff requested that the applicant address the concerns described above.

In a February 13, 2009, response to RAI 155, Question 03.08.01-15, the applicant indicated that the FSAR will be revised to remove reference to past experience. The applicant explained that the material properties selected for the calculation of prestress losses were based on typical concrete properties under environmental conditions that are typical of the Central and Eastern U.S. (CEUS). A list of the material property values was included in the response. No variations were considered by the applicant. The applicant also indicated that it compared the values of prestress losses given in FSAR Tier 2, Table 3.8-5, with values from several operating NPPs in the U.S. and found the values comparable.

The information provided by the applicant in the February 13, 2009, response to RAI 155, Question 03.08.01-15 addressed the staff's concerns regarding reference to past experience in the estimation of creep and shrinkage losses, and the use of typical concrete properties that are expected under the environmental conditions of the CEUS. The basis for this determination was the applicant's comparison with prestress losses from several operating NPPs of similar construction materials to the RCB. To complete its assessment; however, the staff needed the source of the material property values and equations given in the response, and additional details of the comparison of prestress losses with several operating NPPs. Therefore, in RAI 248, Question 03.08.01-34, the staff requested that the applicant provide this additional information.

In a September 29, 2009, response to RAI 248, Question 03.08.01-34, the applicant provided the additional information requested by the staff. In particular, the applicant provided a discussion of the comparisons performed with three operating NPPs for prestress losses due to creep, shrinkage, and elastic shortening. Based on these comparisons, the staff concluded that the prestress losses due to creep, shrinkage, and elastic shortening for the RCB are comparable to corresponding values from three NPPs of similar construction materials to the RCB.

Based on the above discussion, the staff finds that the applicant has adequately addressed the staff's concerns related to the estimation of prestress losses due to elastic shortening, concrete creep, and shrinkage. Therefore, the staff considers RAI 155, Question 03.08.01-15 and RAI 248, Question 03.08.01-34 resolved.

3. FSAR Tier 2, Section 3.8.1.4.8, states that possible variations of material properties are taken into account in the analysis and design of the RCB. The FSAR further states that material properties were established based on past engineering experience with similar construction and materials. It was unclear to the staff how the variability in material properties was considered in the analysis and design of the RCB and other Seismic Category I structures; in particular, whether the potential effects of high irradiation on structural members close to the reactor pressure vessel (e.g., the reactor vessel concrete support structure) were considered. Therefore, In RAI 155, Question 03.08.01-16, the staff requested that the applicant clarify these issues.

In a May 29, 2009, response to RAI 155, Question 03.08.01-16, the applicant indicated that a parametric analysis of the RCB was performed to identify changes in design forces and moments due to variations in specific heat, modulus of elasticity, and convection parameters. The results of this analysis indicated that, for the range of

variations considered, the design forces and moments do not change significantly. The applicant also clarified that variations in material properties were not considered for structures other than the RCB. In addition, the applicant indicated that potential detrimental effects of irradiation on material properties were considered insignificant in the RBIS, because the reactor pressure vessel is shielded by primary and secondary shield walls.

The information in the May 29, 2009, response RAI 155, Question 03.08.01-16 clarified how the applicant performed the parametric analysis of the RCB, and explained that changes in design forces and moments due to variations in specific heat, modulus of elasticity, and convection parameters are not considered significant. However, the applicant did not address the effect of the annulus temperature variations, as had been asserted in the applicant's February 13, 2009, response to RAI 155, Question 03.08.01-14. In addition, the applicant did not clarify the range of variations used in the parametric study or the technical basis for selecting this range. Finally, the applicant did not clarify how the detrimental effects of radiation were considered for the concrete and steel structures in and within the primary and secondary shield walls. Therefore, in follow-up RAI 306, Question 03.08.01-39, the staff requested that the applicant provide this information.

In a March 12, 2010, response to RAI 306, Question 03.08.01-39, the applicant provided additional information regarding the parametric analysis of the RCB, including the range of values used and the basis for their selection. In addition, the applicant indicated that the results of the parametric study for the annulus temperature variations also indicated insignificant effects on design forces and moments.

Regarding the detrimental effects of radiation on the structures located in or within the primary and secondary shield walls, in the March 12, 2010, response to RAI 306, Question 03.08.01-39, the applicant indicated that primary and secondary shield wall thicknesses were selected based on radiation shielding criteria described in ANSI/ANS-6.4-2006, "Nuclear Analysis and Design of Concrete Radiation Shielding for Nuclear Power Plants," or measures denoted as structural requirements in ACI 349-01 and ACI 349.1R-07. The applicant also indicated that no industry operating experience or applicable structural design codes or standards indicate variations in material properties for structural materials exposed to radiation.

During the February 14-17, 2011, onsite audit, the staff reviewed the results of the aforementioned parametric analysis of the RCB to confirm the applicant's conclusions. The staff determined that the changes in design forces and moments due to variations in specific heat, modulus of elasticity, and convection parameters were indeed not significant; however, changes in design moments due to an assumed minimum annulus temperature of 7 °C (45 °F) were on the order of 20 percent and thus significant. In addition, regarding the detrimental effects of radiation on the structures located in or within the primary and secondary shield walls, the staff noted that several industry codes and technical reports (e.g., NUREG/CR-6927, "Primer on Durability of Nuclear Power Plant Reinforced Concrete Structures – A Review of Pertinent Factors," 2007) provide guidelines and recommend maximum radiation exposure levels for structural materials exposed to radiation. The staff requested that the applicant clarify whether the maximum accumulated lifetime radiation considered in the design of the shield walls was below the recommended maximum exposure levels given in these references or, if not, whether it was necessary to consider degraded material properties due to radiation exposure. Finally, the staff determined that the technical

basis for the magnitude of wobble and curvature coefficients assumed in the estimation of tendon prestress losses due to friction was test data that was not available for audit.

In response to the staff's concerns, the applicant indicated that although the changes in design moments due to an assumed minimum annulus temperature of 7 °C (45 °F) were not insignificant, preliminary calculations indicated that the design of the RCB was sufficiently conservative to accommodate these changes; furthermore, it was asserted that an annulus temperature of 7 °C (45 °F) was an unlikely scenario. Regarding the maximum accumulated lifetime radiation exposure of the structures located in or within the primary and secondary shield walls, the applicant provided preliminary estimates of lifetime radiation exposure that were below the recommended maximum exposure levels. Finally, regarding the test data used to validate the wobble and curvature coefficients assumed in the estimation of tendon prestress losses due to friction, the applicant indicated it would provide a summary of this data and the technical justification for why it is applicable to the design of the RCB.

The applicant indicated that the preliminary information provided to the staff during the February 14-17, 2011, audit (summarized above) would be confirmed and submitted in a July 15, 2011, supplemental response to RAI 306, Question 03.08.01-39, which provided the following additional information.

- Changes in design forces and moments due to the assumed minimum annulus temperature of 7 °C (45 °F) were quantified and added to the various load combinations used for the design of the RCB wall and dome. The increased force and moment demands were compared to the capacity of the RCB wall and dome, at several locations and at the four critical time points used in the design for DBA conditions. The interaction diagrams provided with the RAI response enveloped the design points corresponding to the increased force and moment demands, with significant margin, thus confirming that the design of the RCB is sufficiently conservative to accommodate the changes. Since the design of the RCB wall and dome may be subject to minor changes when the critical section designs are completed (see discussion above, in RAI 155 Questions 03.08.01-24, and 03.08.04-6), the applicant also indicated that the comparison provided in the RAI response will be updated with the final critical section design. The staff conclusion regarding this item of the response, depends on the final critical section design and, therefore, the staff considers this part of the open item established in connection with RAI 155, Questions 03.08.01-24, and 03.08.04-6. . The staff notes, nonetheless that the applicant has demonstrated, in the absence of the completion of the critical section design, that the preliminary design of the RCB wall and dome shows sufficient conservativism to accommodate changes in the annulus temperature between 45 °C (113 °F) maximum and 7 °C (45 °F) minimum. The staff will verify whether any changes in the final critical section design affect this analysis.

- Lifetime neutron fluence and gamma dose rates were evaluated at the peak location on the surface of the reactor cavity biological shield wall and compared to recommended maximum exposure levels in ANSI/ANS-6.4-2006. The gamma dose was calculated to be 2.06E+09 rad, which is below the recommended maximum exposure level of 1.0E+10 rad; the neutron fluence was calculated to be 1.09E+19 n/cm^2, which is nine percent higher than the limit of 1.0E+19 n/cm^2. The staff finds this item of the response acceptable, because it shows that the maximum accumulated lifetime radiation exposure of the structures located within the primary and secondary shield walls will be within or only slightly above the maximum exposure level in ANSI/ANS-6.4-2006. Based on the experimental data summarized in NUREG/CR-6927, Figure 4.7, neutron fluence

that is nine percent above the recommended maximum exposure level is not expected to reduce the concrete strength or stiffness properties in any significant manner.

- Wobble and curvature coefficients assumed in the estimation of tendon prestress losses due to friction were partially based on test data from the European EPR design and partially based on manufacturer's recommendations. A comprehensive summary of test data to validate the wobble and curvature coefficients was not provided. Since ASME Code, Subarticle CC-3542(b), requires the wobble and curvature coefficients assumed in design to be based on experimental data to be verified during stressing operations, the applicant proposed a revision to FSAR Tier 1, Table 2.1.1-8, Item 2.6, Reactor Building ITAAC. The proposed ITAAC indicates that inspections will be performed to verify the existence of analyses which reconcile as-built deviations and also references ASME Code, Section III, Subarticle NCA-3454. The staff determined the proposed ITAAC does not specifically identify the testing required by ASME Code, Subarticle CC-3542(b). Furthermore, it is unclear to the staff whether the reference to ASME Code, Section III, Subarticle NCA-3454, ensures that this testing will be performed since the latter only refers to general "containment acceptance testing." Moreover, the performance of the analysis is the essence of the ITAAC, not an "inspection" of the analysis.

On the basis of the above discussion, the staff considers RAI 155, Question 03.08.01-16 resolved**.** The technical issues associated with RAI 306, Question 03.08.01-39 have been addressed, with the exception of the issue associated with the proposed ITAAC as discussed above to ensure the testing requirements in ASME Code, Subarticle CC-3542(b) are met.

To summarize Items (1), (2), and (3) above, the staff notes that **RAI 306, Question 03.08.01-39 is being tracked as an open item;** the staff considers RAI 155, Questions 03.08.01-14, 03.08.01-15 and 03.08.01-16, and RAI 248, Question 03.08.01-34 resolved; and **RAI 248, Question 03.08.01-33 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed changes.

The staff reviewed the analysis and design procedures used for RCB FSAR Tier 2, Section 3.8.1.4 and Appendix 3E. The staff was unable to determine whether the applicant's FE model was adequate to predict the structural responses of the NI, or whether the various loads applied to this model were analyzed correctly. One particular concern was the approach used by the applicant to account for the effects of concrete cracking in the structural analysis and design for thermal, seismic, and other loads. Therefore, in RAI 155, Questions 03.08.01-8, 03.08.01-9, 03.08.01-22, and 03.08.01-27, the staff requested that the applicant provide more detailed information regarding: (1) The FE model (e.g., type of finite elements, discretization of the finite elements, representation of reinforcement, modeling of the liner and liner anchorage), and how each type of load is applied to the model; (2) how concrete cracking was considered in the various load analyses; (3) methodology used to develop thermal modification factors; and (4) procedure to model the thermal and pressure transients for a LOCA. The staff's evaluation for each of these items is discussed below.

1. In a May 29, 2009, response to RAI 155, Question 03.08.01-8, the applicant provided more detailed information regarding the FE model of the NI and the method for applying loads to this model. The response described the so-called static FE model of the NI, which includes all Seismic Category I structures on the NI common basemat foundation. To model the RCB, four and five layers of the ANSYS finite element "SOLID45," are used through the thickness of the dome and cylindrical shell wall, respectively. Other finite elements consisting of "SOLID95" and "SOLID92" are also used where meshing

with regular SOLID45 elements is not possible. The reinforcement is not explicitly modeled. The liner is modeled with 4-node "SHELL181" finite elements on the inside surface of the RCB for use as pressure load transfer elements. The liner and its anchorages are not structural elements in the design of the RCB, and thus are not explicitly modeled in the RCB. Finally, a mesh-sensitivity study of the RCB was performed to verify the sensitivity of the analysis to FE mesh refinement for thermal and other loads. As a result of this study, the applicant determined that the FE model of the RCB gives conservative results for thermal loads and is sufficiently accurate for other loads.

The May 29, 2009, response to RAI 155, Question 03.08.01-8, also described how the various loads were applied to the NI model. For seismic loads, as an example, ZPAs in the three perpendicular directions for different elevations are applied to the masses in the model, as well as to the pressure distributions used to represent additional items such as equipment.

The staff concluded that most of the FE modeling information in the May 29, 2009, response to RAI 155, Question 03.08.01-8 was acceptable, because it complied with SRP Section 3.8.1.II.4 and 3.7.2.II.3.C.ii, Acceptance Criteria. In particular, the results of a mesh-sensitivity study indicated that the FE model of the RCB is sufficiently accurate for non-thermal loads. The evaluation of the FE model of the RCB for thermal loads is further discussed under Items (3) and (4) below.

The staff identified several inconsistencies in the May 29, 2009, response to RAI 155, Question 03.08.01-8 related to the treatment of live loads, soil loads, hydrostatic loads, pipe reaction loads, tendon loads, and seismic loads applied to the static FE model of the NI. In addition, since the liner and its anchors are not explicitly included in the FE model, it was unclear to the staff how the applicant designed these elements; in particular, for "local" loads that are not applied to the FE model of the RCB (e.g., jet impingement loads). Therefore, in RAI 448, Question 03.08.01-54, the staff requested that the applicant clarify these issues.

During the onsite audit conducted on February 14-17, 2011, the applicant provided additional descriptive information regarding live loads, soil loads, hydrostatic loads, pipe reaction loads, tendon loads, and seismic loads applied to the static FE model of the NI. In addition, the applicant explained the methodology to design the liner and its anchors, which is based on Bechtel Topical Report BC-TOP-01, Revision 1, "Containment Building Liner Plate Design Report," 1971, and was also used in previous NPP certified designs (e.g., Economic Simplified Boiling Water Reactor (ESBWR)). Further, the applicant explained that a pipe break hazard analysis determined that local dynamic loads such as pipe whip, missile impact, and jet impingement are not applicable to the liner. The applicant indicated that this additional information would be provided in response to RAI 448, Question 03.08.01-54.

In an April 27, 2011, response to RAI 448, Question 03.08.01-54, the applicant provided additional descriptive information regarding live loads, soil loads, hydrostatic loads, pipe reaction loads, tendon loads, and seismic loads applied to the static FE model of the NI. In addition, the applicant stated that the methodology to design the liner and its anchors, which is based on Bechtel Topical Report BC-TOP-01, Revision 1, "Containment Building Liner Plate Design Report," 1971, and also was used in previous nuclear power plants (NPP) certified designs. Additionally, the applicant stated that a pipe break

hazard analysis determined that local dynamic loads such as pipe whip, missile impact, and jet impingement are not applicable to the liner. SRP Acceptance Criteria 3.8.1.II.3 and 3.8.1.II.4 indicate that the loads, load combinations, and design and analysis procedures are acceptable if found to be in accordance with ASME Code, Article CC-3000 and RG 1.136. The staff concludes that the local dynamic loads do not apply to the liner because there is sufficient separation between the liner and the sources of such dynamic loads or the sources meet leak-before-break criteria, as discussed in Section 3.6.3 of this report. Furthermore, the applicant considered all other loads identified in RG 1.136. Accordingly, the staff finds this information acceptable, because it complies with SRP Acceptance Criteria 3.8.1.II.3 and 3.8.1.II.4.

In addition, the applicant provided a detailed discussion of how the methodology to design the liner, and in particular its anchors, satisfies ASME Code, Section III, Division 2, Subarticle CC-3810, Items (a) through (h), which complies with SRP Acceptance Criteria 3.8.1.II.4.J. SRP Acceptance Criteria 3.8.1.II.J identifies some of the key provisions in the ASME Code, Section III, Division 2, Article 3000 and describes important aspects of the design and analysis procedures for steel plate liner and anchors that should be considered. The applicant also confirmed the pipe break hazard analysis which determined that local dynamic loads such as pipe whip, missile impact, and jet impingement are not applicable to the liner. These dynamic loads are not applicable to the liner, because there is either sufficient distance between the liner and potential pipe break sources or leak before break considerations, or additional protection (i.e., whip restraints, jet shields) has been provided as described in FSAR Tier 2, Section 3.6 (see FSAR Tier 2, Revision 2, Table 3.6.1-3, “Building, Room, Break, Target, and Protection Required”). In addition, the COL applicant will complete the pipe break hazards analysis per FSAR Tier 2, Table 1.8-2, COL Information Items 3.6-1 and 3.6-2.and ITAAC 3.8-1 will require an as-built pipe break hazards analysis, which will ensure the liner is protected from dynamic loads. This item of the response is acceptable, because it provides the technical basis for not considering local dynamic loads, and because ITAAC 3.8-1 requires an as-built pipe break hazards analysis to ensure that the liner is protected from local dynamic loads even if there is a change in the piping configuration subsequent to the design certification.

Based on the above discussion, the staff considers RAI 155, Question 03.08.01-8 resolved, while **RAI 448, Question 03.08.01-54 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed changes.

2. In a May 29, 2009, response to RAI 155, Question 03.08.01-9, the applicant provided the results of a study intended to justify the use of uncracked concrete properties for non-thermal loads. A nonlinear FE analysis was performed for a circumferential 6-degree vertical slice of the RCB away from discontinuities. ANSYS SOLID65 elements were used to represent reinforced concrete with the ability to crack. The FE model was subjected to a series of incremental loads: first, the structural integrity test (SIT) pressure loads; second, the accident temperature and pressure loads; and finally, additional SIT pressure loads. Dead and prestressing loads were maintained throughout the analysis. Forces and moments were determined at the final stage of SIT pressure loading. The applicant indicated that design forces and moments from this nonlinear analysis, considering cracked section properties, were not significantly modified when compared to the linear FE model without concrete cracking. Therefore, based on this analysis, the applicant justified the use of uncracked concrete properties for non-thermal loads.

The staff' concluded that additional information was needed to determine the adequacy of the study, including: The manner in which the loads were applied in the nonlinear analysis, representative results for the worst case moments and shears, explanation of how this study addresses the effects of cracking for seismic loading, and how this study on the RCB addresses the concerns of cracking for other Seismic Category I structures. Therefore, in RAI 335 Question 03.08.01-44, the staff requested that the applicant provide the additional information.

In a July 13, 2011, response to RAI 335, Question 03.08.01-44, the applicant provided a discussion and results of three different analytical studies performed to investigate the effects of concrete cracking on the structural analysis and design of the RCB, RBIS, and other NI and non-NI Seismic Category I structures, for seismic and non-seismic loads. Each of these three studies is discussed below:

a. Regarding the linear and nonlinear FE analysis of the 6-degree vertical slice of the RCB away from discontinuities, described in the May 29, 2009, response to RAI 155, Question 03.08.01-9, the RAI response provided additional information which demonstrated that the effect of cracking on forces and moments (for non-seismic loads) is relatively minor and does not require changes to the linear analysis procedures used in the design of the RCB, which utilize uncracked section properties.

   - The applicant provided a detailed explanation of the loading history that clarified all the load steps/stages used in the analysis (i.e., load combinations, applied temperatures and pressures). In particular, it was explained that the accidental pressure and temperature loads were applied in 27 different steps corresponding to 1 year of DBA conditions.

   - Graphical comparisons of forces and moments from the linear and nonlinear FE models subjected to the same loading history were provided, for a typical RCB wall section and for a section near the RCB wall-gusset junction. The latter section was chosen, because it corresponds to the region where maximum cracking was predicted in the evaluation.

   - The plots indicated that the differences in axial forces at typical RCB wall sections would be negligible for the entire loading history. For a section near the RCB wall-gusset junction, the plots indicated that maximum differences in moments would be in the order of 15 percent (unconservative) at the beginning of the DBA event (between 1 min and 2 hrs); differences in moments were negligible or conservative for all other steps of the loading history.

   - Although moments would increase due to cracking in some regions of the RCB and for some steps in the loading history, these increased moments would be smaller than the peak moments in the same regions calculated from linear analysis with uncracked section properties (peak moments predicted to occur approximately 24 hrs after the start of DBA conditions).

   SRP Acceptance Criteria 3.8.1.II.4.A through 3.8.1.II.4.D indicate that the design and analysis procedures are acceptable if they are performed in accordance with

ASME Code, Section III, Division 2, Article 3000, RG 1.136 and other criteria detailed in SRP Sections 3.8.1.II.4.A through 3.8.1.II.4.D. The other criteria address consideration of boundary conditions, axisymmetric and nonaxisymmetric loads, transient and localized loads, and cracking of concrete. The staff finds this additional information acceptable, because detailed analyses have been performed to comply with the guidance in SRP Acceptance Criteria 3.8.1.II.4.A through 3.8.1.II.4.D. These analyses demonstrated that the effect of cracking on forces and moments (for non-seismic loads) is relatively minor in regions of the RCB away from discontinuities. Even in regions of discontinuity, such as the RCB wall-gusset junction, predicted increases in moments would not impact the section design using linear analysis procedures. Therefore, based on these analyses, the staff finds the applicant's justification for using uncracked concrete properties and linear analysis procedures in the design of the RCB for non-seismic loads acceptable.

b. The applicant performed seismic time-history analyses of the NI structures and the EPGB to determine the effects of concrete cracking on the magnitude of the seismic inertial loads used in structural design.

The staff noted that ZPAs obtained from the seismic SSI analyses described in FSAR Tier 2, Sections 3.7.2.3, 3.7.2.4, and 3.7.2.5 are used as seismic inertial loads applied to the FE models of the NI structures, EPGB, and ESWB, following the applicant's equivalent-static seismic analysis approach. Since concrete cracking modifies the fundamental frequencies and modes, as well as the damping ratios of the structures, it can also modify the SSI analysis results and the ZPAs; however, it is not always clear whether this modification is conservative or unconservative, because it depends on the frequency content of the input ground motion and on the extent of the cracking. The staff also noted that the SSI analyses performed by the applicant already include the case of a 50 percent reduction in out-of-plane bending stiffness for walls, slabs, columns, and beams to simulate cracking, but no reduction in in-plane bending and shear stiffnesses. In fact, two sets of ZPAs are generated from these SSI analyses: cracked and uncracked, and both sets are used in the structural design of Seismic Category I structures.

To elucidate whether additional cracking is of concern, the applicant performed seismic time-history analyses of the NI structures and the EPGB using the ANSYS computer code. A description of these analyses and corresponding results and conclusions were provided in the RAI response and are summarized below.

- Fixed-base FE models of the NI structures and EPGB were subjected to input ground motions that correspond to EURH (for the NI) and "modified EUR" (for the EPGB) time-histories. These time-histories are consistent with the fixed-based assumption and, in the case of the "modified EUR," consider structure-soil-structure interaction effects between the EPGB and the NI, as described in FSAR Tier 2, Section 3.7.1.1, "Design Ground Motion."
- Four different levels of cracking were considered in the FE analyses: uncracked (lowest level); cracked in out-of-plane bending for walls, slabs,

columns and beams; cracked in out-of-plane bending for walls, slabs, columns and beams, and in-plane bending for walls and slabs; and cracked in out-of-plane bending for walls, slabs, columns and beams and in-plane bending and in-plane shear for walls and slabs (highest level). In all cases, cracking was represented as a 50 percent reduction in the corresponding stiffness properties of the finite elements. No axial stiffness reductions were considered.

- Structural damping for concrete was set to five percent for the RCB and seven percent for the other NI structures, for all levels of cracking. For the EPGB, structural damping was set to four percent for the two lower levels of cracking and seven percent for the two higher levels of cracking.

- Maximum reactions at the base of the different structures were determined for each level of cracking. In most cases, the trend observed was a decrease in the base reactions due to an increase in cracking, which ranged from zero to 39 percent reduction.

- The cracked out-of-plane bending case, which is already modeled in the seismic SSI analyses performed by the applicant, was the only case in which increased base reactions were observed relative to the uncracked case. This increase was in the order of three percent (horizontal) for the RBIS and three percent (vertical) for the EPGB.

- The applicant explained the results obtained as follows: For the NI analysis in the two horizontal directions, a decrease in the dominant frequencies of the structures due to cracking results in a shift towards the region of the EURH seismic input spectra below 4.6 Hz, which has decreasing spectral accelerations as the frequency decreases. The seismic demands are thus lower. For the NI analysis in the vertical direction, there is relatively little change, because no reduction is taken in the axial stiffness and also because the shifted dominant frequencies remain in the plateau region of the EURH seismic input spectra between 4.6 Hz and 14 Hz. In the case of the EPGB, there are additional complexities due to the two different damping ratios used and predicted changes in the participating mass ratios of the dominant modes; however, the results obtained were quantitatively similar as those for the NI structures.

SRP Acceptance Criteria 3.7.2.II.1.A identify acceptable methods for performing dynamic seismic analysis using the time history analysis approach. SRP Acceptance Criteria 3.8.1.II.4.D identify acceptable methods for evaluating the effects of concrete cracking on the structural response of the containment. The staff finds this additional information acceptable, because detailed analyses have been performed to comply with the guidance in SRP Acceptance Criteria 3.7.2.II.1.A and 3.8.1.II.4.D. These analyses demonstrated that, for fixed-base conditions, increased levels of concrete cracking result in approximately equal or reduced seismic demands for the NI structures and EPGB, relative to the uncracked and cracked out-of-plane bending cases which are already modeled in the seismic SSI analyses performed by the applicant.

Since the dynamic structural response of the ESWB is expected to be very similar to that of the EPGB, this conclusion is also applicable to the ESWB.

However, the analyses described above only considered fixed-base conditions applicable to very stiff soils or rock. To determine whether the above conclusions are also applicable to other generic soil cases, the applicant performed an additional investigation.

First, the dominant frequencies of the NI structures and EPGB, for the various soil profiles, were determined from the peaks observed in the in-structure response spectra (ISRS) and the transfer functions computed at key locations as part of the seismic SSI analyses. Soil cases 5ae, 2sn4ue, 4ue, 1n5ae, and 1n2ue, as described in FSAR Tier 2, Revision 2, Section 3.7 were considered. Next, for each different soil case, the dominant frequencies of the structures were superimposed on the seismic input spectra that are applicable to the given soil case and damping ratio, to determine whether they fall in the increasing, decreasing, or plateau regions of the spectra. Finally, for each case, it is qualitatively determined whether a decrease in the dominant frequencies due to increased cracking, similar to the predicted shifts in the fixed-base analyses, would result in an increase, decrease, or no variation in the spectral accelerations and in the seismic demands on the corresponding structures.

Based on the above investigation, the applicant concluded that increased levels of concrete cracking result in approximately equal or reduced seismic demands, for all generic soil cases, comparable to the results obtained from the analyses for fixed-base conditions.

The staff finds the applicant’s investigation acceptable, because a rational and systematic procedure was followed to obtain the dominant frequencies of the NI structures and EPGB for the various soil profiles, and also to qualitatively determine the effect of a decrease in the dominant frequencies due to increased cracking, in conformance with SRP Acceptance Criteria 3.7.2.II.1 through 3.7.2.II.4.

Therefore, based on the analyses discussed above, the staff concludes that the seismic inertial loads used in structural design are conservative relative to the effects of concrete cracking.

c. Whereas the above evaluation described the effects of concrete cracking on the magnitude of the seismic inertial loads used in the structural design, the applicant also performed another study on the effect of concrete cracking on the potential redistribution of member forces for seismic and non-seismic loading. The applicant performed linear and nonlinear FE analyses of a full 360-degree model of the RCB. A description of these analyses and corresponding results and conclusions were provided in the RAI response and are summarized below.

- ANSYS SOLID65 elements were used to represent reinforced concrete with the ability to crack, and these elements included the concrete rebar as smeared reinforcement. Large openings in the containment were considered by reducing the material modulus of elasticity in that region.

- Loads were applied in three steps: First, dead and prestressing loads; second, accident temperature gradient and pressure loads corresponding to one DBA critical time point only (see Item (3) below, in RAI 448, Question 03.08.01-51, for a discussion of the so-called critical time points); and finally, seismic inertial loads (ZPAs) applied in the two horizontal and vertical directions simultaneously (100-100-100 combination). ZPAs corresponding to both cracked and uncracked concrete assumptions were used.

- Interaction diagrams showing the capacities of two typical RCB wall and dome sections (based on current design of RCB critical sections) were included with the response to RAI 448, Question 03.08.01-51. Force and moment results for the three load steps, from the linear and nonlinear FE models, were plotted in these interaction diagrams to visually assess the redistribution due to cracking. The DBA critical time point used in the second load step was 24 hours, which is considered the worst-case scenario relative to the other critical time points of 0, 1.39, and 100 hours. The ZPAs used in the third load step correspond to soil case 4ue.

- A comparison of forces and moments from the linear and nonlinear FE models against the capacities shown in the interaction diagrams demonstrates that redistribution of forces and moments due to cracking are within the capacities of the sections with significant margins. Indeed, it is observed that in most cases the redistribution tends to slightly increase the margins. Therefore, designs based on linear analysis of the RCB with uncracked section properties are adequate.

- The applicant repeated the analyses considering critical time points zero, 1.39, and 100 hours; ZPAs corresponding to soil case 5ae; 75 percent reduction in concrete tensile strength; and 50 percent reduction in RCB wall reinforcement. The applicant determined that the above conclusion was still valid.

- Since the design of the RCB wall and dome may be subject to minor changes when the design of the critical sections is completed (see discussion above, in RAI 155, Questions 03.08.01-24, and 03.08.04-6), the applicant also indicated that a 10 percent margin will be maintained between analysis demands and section capacities in the final critical section design.

The staff finds this additional information acceptable, because detailed analyses have been performed to comply with the guidance in SRP Acceptance Criteria 3.8.1.II.4.A through 3.8.1.II.4.D. These analyses demonstrated that the effect of concrete cracking on the redistribution of forces and moments (for seismic and non-seismic loading) does not impact the section design using linear analysis procedures. Therefore, based on these analyses, the applicant's justification for using uncracked concrete properties and linear analysis procedures in the design of the RCB for seismic and non-seismic loads is acceptable. The staff will need to verify the applicant's commitment that a 10 percent margin will be maintained between analysis demands and section capacities in the final critical section design. The verification will be performed as

part of the resolution of the technical issues associated with critical section design.

Based on the conclusions of the three different analytical studies discussed above (Items (a), (b), and (c)), the staff considers RAI 155, Question 03.08.01-9 resolved, while **RAI 335, Question 03.08.01-44 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate proposed changes.

3. In a May 29, 2009, response to RAI 155, Question 03.08.01-22, the applicant explained that another FE analysis of a circumferential 6-degree vertical slice of the RCB structure (away from discontinuities) was performed to evaluate the change in magnitude of the thermal moments in the RCB resulting from mesh refinement (linear analysis) and cracking of concrete (nonlinear analysis). Details of the FE model were provided, including the computer code, the loading sequence, and the types of finite elements used in the analyses. By comparison of thermal moments from the refined mesh linear slice model to the equivalent mesh linear slice model (mesh density equal to the FE model of the NI), a thermal modification factor due to mesh refinement was determined. Similarly, by comparison of thermal moments of the nonlinear slice model to the thermal moments from the linear slice model, a thermal modification factor due to concrete cracking was determined. The RAI response also indicated that the RCB is the only structure expected to develop a significant thermal gradient across its thickness. Therefore, the applicant did not consider thermal loading for RSB, FB, SB, RBIS, EPGB, or ESWB.

   The staff's review of the RAI response concluded that additional information was needed to determine the adequacy of the thermal analyses performed to obtain the thermal modification factors due to mesh refinement and concrete cracking. This additional information included: Details showing the various FE models, an explanation of how the thermal loading was applied to the FE models including thermal gradient through the thickness of the RCB wall and dome, an explanation of whether there are multiple thermal factors for each element or region of the RCB, and the magnitude of the thermal factors. Therefore, in RAI 448, Question 03.08.01-51, the staff requested that the applicant provide this additional information.

   In a July 13, 2011, response to RAI 448, Question 03.08.01-51, the applicant provided additional information that clarified the methodology used to compute the thermal modification factors. This information is summarized below.

   - Thermal modification factors were developed to account for the effects of mesh refinement and concrete cracking that cannot be reflected in the FE model of the NI used in the design of RCB critical sections. These modification factors are applied as multipliers to the thermal moments obtained from the linear FE analysis of the NI model for DBA conditions.

   - Since the development of the thermal modification factors is based on the structural analysis of a 6-degree vertical slice of the RCB (axisymmetric conditions), these factors are only applicable to regions of the RCB away from discontinuities. For regions of the RCB within or influenced by discontinuities, a modification factor of 1.0 (i.e., no reduction in moments due to mesh refinement and concrete cracking) will be used or additional submodel analyses will be performed, using the same methodology, to develop modification factors (which

may be less than 1.0) for these regions. Documentation of the latter, if used, will be included in the critical section design calculations.

- Details of the refined slice model and the equivalent slice model were provided by the applicant, including representative figures. The refined slice model has 15 and 12 layers of brick elements through the thickness of the RCB wall and dome, respectively. The equivalent slice model has similar element thickness and mesh density as the FE model of the NI: Five and four layers of brick elements through the thickness of the RCB wall and dome, respectively. The equivalent slice model is used as the reference linear model, while the refined slice model is used in the linear and nonlinear analyses to determine the effects of mesh refinement and concrete cracking.

- Transient thermal analyses of the refined and equivalent slice models were performed to calculate corresponding temperature gradients through the thickness of the containment at different time instants during the DBA. Representative plots of the temperature gradients obtained from these thermal analyses were provided in a July 13, 2011, response to RAI 448, Question 03.08.01-52 (see Item (4) below). Based on the relative magnitude of accident pressure loads and temperature gradients obtained from the thermal analyses, four critical time points were chosen for the structural analysis of the RCB: 0, 1.39, 24, and 100 hours. These critical time points are representative of the accident pressure and temperature loads for the duration of DBA conditions.

- In the linear analyses, the calculated temperature gradients at the critical time points were applied to the equivalent and refined slice models to obtain corresponding thermal moments. At the beginning of DBA conditions (t=0 and t=1.39 hours), the temperature distribution through the thickness of the containment has a sharp gradient close to the inner surface and is relatively flat close to the outer surface. As the accident progresses, the sharp gradient dissipates and the distribution becomes closer to linear (t=100 hours). This thermal response is clearly shown in the plots of temperature gradients provided in a July 13, 2011, response to RAI 448, Question 03.08.01-52 (see Item (4) below). The refined slice model can predict the sharp temperature gradient for t=0 and t=1.39 hours, but the equivalent slice model cannot; therefore, the equivalent slice model significantly overestimates the thermal moments for t=0 and t=1.39 hours.

- From the linear analyses described in the bullet above, the ratio of thermal moments between the refined and equivalent slice models were computed for t=0, 1.39, and 24 hours at different locations in the RCB wall and dome. For t=0, the ratio varies between 0.16 in the RCB wall and 0.49 near the RCB wall-gusset junction; for t=1.39 hours, the ratio varies between 0.59 in the RCB wall and 0.72 near the RCB wall-gusset junction; for t=24 hours, the ratio varies between 0.95 near the RCB wall-gusset junction and 1.02 in the RCB dome. No ratios were computed for t=100 hrs, because the temperature gradients for the refined and equivalent slice models are essentially identical. This information confirms that the effect of mesh refinement is only significant for t=0 and t=1.39 hours.

- A nonlinear incremental analysis of the refined slice model was performed with explicit modeling of concrete cracking and reinforcement. Applied loads included

gravity, prestressing, and test pressure (applied and then removed, to simulate cracking due to the SIT); accident pressure loads and temperature gradients were applied quasi-statically and incrementally for 27 different time instants corresponding to 1 yr of DBA conditions (determined from the transient thermal analysis). This analysis is similar to the one described in a July 13, 2011, response to RAI 335, Question 03.08.01-44 (see Item (2) above).

- The ratio of thermal moments between the nonlinear and linear refined slice models were computed for t=0, 1.39, and 24 hours at different locations in the RCB wall and dome. The ratios vary between 0.96 and 1.00 at most locations of the RCB, except near the RCB wall-gusset junction where they vary between 1.01 and 1.28 for t=0; 0.87 and 0.99 for t=1.39 hours; and 0.80 and 0.93 for t=24 hours. This information indicates the effect of concrete cracking is relatively minor in most locations of the RCB away from discontinuities except near the RCB wall-gusset junction, which is consistent with the information provided in a July 13, 2011, response to RAI 335, Question 03.08.01-44 (see Item (2) above).

- Thermal modification factors were computed as the product of the two ratios described in the previous three bullets. The modification factors thus include the effects of mesh refinement and concrete cracking for t=0, 1.39, and 24 hours at different locations in the RCB wall and dome. For t=0, the factor varies between 0.18 in the RCB wall and 0.63 near the RCB wall-gusset junction; for t=1.39 hours, the factor varies between 0.60 in the RCB wall and 0.71 near the RCB wall-gusset junction; for t=24 hours, the factor varies between 0.88 near the RCB wall-gusset junction and 1.02 in the RCB dome. This information indicates that the thermal modification factors are only significant for t=0 and t=1.39 hours, except near the RCB wall-gusset junction where they are significant for t=0, 1.39, and 24 hours. Modification factors are not used for t=100 hours.

- Interaction diagrams showing the capacities of two typical RCB wall and dome sections (based on current design of RCB critical sections) were included with the response to RAI 448, Question 03.08.01-51. Modified and unmodified DBA force and moment demands (at t=0, 1.39, and 24 hours) were plotted in these interaction diagrams to visually assess the impact of the thermal modification factors. It is confirmed that the modification factors are only significant for t=0 and t=1.39 hours. The safety margin for t=24 hours is essentially unchanged; however, the safety margin for t=0 and t=1.39 hours is increased because of the reduced demands. It is also observed that the design of the RCB is sufficiently conservative such that the interaction diagrams envelope the design points even for the unmodified demands.

The staff finds the above information technically acceptable, because detailed thermal and structural analyses have been performed to comply with the guidance in SRP Acceptance Criteria 3.8.1.II.4.A through 3.8.1.II.4.D. These analyses demonstrate that the thermal moments obtained by applying thermal gradients calculated from the equivalent slice model (at the critical time points) to the FE model of the NI, used in the design of RCB critical sections, can be accurately reduced by the computed thermal modification factors. The staff will need to verify the implementation of this methodology to the design of RCB critical sections, and in particular to the design of regions within or influenced by discontinuities. The verification will be performed as part of the resolution of the technical issues associated with critical section design.

Based on the above discussion, the staff considers RAI 155, Question 03.08.01-22 resolved, while **RAI 448, Question 03.08.01-51 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate proposed changes.

4. In a May 29, 2009, response to RAI 155, Question 03.08.01-27, the applicant described the FE analysis procedure to model the thermal and pressure transients from a LOCA. The response described the circumferential 6-degree vertical slice of the containment which was used to study the effect of mesh refinement on the thermal moments. Based on this study, the applicant concluded that the existing 4/5 layers of mesh through the thickness of the RCB overestimates the thermal gradient across the thickness at the beginning of the LOCA period and is a good estimate of the thermal gradient in the later period of the LOCA. To address the staff's concern that only the liner material properties were adjusted for the LOCA temperatures and not the concrete, another study was performed using an axisymmetric model of the RCB to compare changes in member forces corresponding to changes in the mechanical properties of the RCB concrete. The applicant concluded that the reduction in modulus of elasticity decreases the thermal moments; whereas, the design forces and moments due to pressure loads do not change significantly.

   The staff determined that the information provided was insufficient to conclude that the RCB has been properly designed for pressure and thermal loads associated with LOCA. Therefore, in RAI 448, Question 03.08.01-52, the staff requested that the applicant provide the numerical results for the thermal moments at various elements and regions of the RCB in order to evaluate the conservatism of the approach used for the analysis.

   In a July 13, 2011, response to RAI 448, Question 03.08.01-52, the applicant provided the following additional information:

   - Plots were provided to illustrate the temperature gradients through the thickness of the containment, obtained from the transient thermal analyses of the refined slice and equivalent slice FE models described in the July 13, 2011, response to RAI 448, Question 03.08.01-51. The plots correspond to typical sections of the RCB wall and dome for steady state conditions, and time instants 0.33, 1.39, 24, 110, and 8,760 hours during the DBA. These temperature gradients clearly show that, at the beginning of the DBA (t=0.33 and 1.39 hours), the temperature distribution through the thickness of the containment has a sharp gradient close to the inner surface and is relatively flat close to the outer surface. As the accident progresses, the sharp gradient dissipates, and the distribution becomes closer to linear (t=110 and 8,760 hours).

   - Numerical values were provided for the thermal moments obtained from the linear structural analyses of the refined slice and equivalent slice FE models, each subjected to the corresponding temperature gradients described in the bullet above, for the same locations and similar time instants. Since the equivalent slice model cannot model the sharp temperature gradients at the beginning of the DBA (t=0, 0.33 and 1.39 hours), the calculated moments are significantly greater than the corresponding moments from the refined slice model, which can model sharp temperature gradients. For t=24, 100, and 8,760 hours, however, the moments from the equivalent slice and refined slice models are essentially unchanged.

The staff finds the above information technically acceptable, because detailed thermal and structural analyses have been performed by the applicant, in conformance with the guidance in SRP Acceptance Criteria 3.8.1.II.4.A through 3.8.1.II.4.D. This information is also consistent with and complements the July 13, 2011, response to RAI 448, Question 03.08.01-51, which the staff finds technically acceptable as discussed above. Therefore, the staff finds technical issues associated with RAI 155, Question 03.08.01-27 and RAI 448, Question 03.08.01-52 acceptable and considers these RAIs resolved.

To summarize items (1), (2), (3), and (4) above, the staff considers RAI 155, Questions 03.08.01-8, 03.08.01-9, 03.08.01-22, and 03.08.01-27 as well as RAI 448, Question 03.08.01-52 resolved. **RAI 335, Question 03.08.01-44, and RAI 448, Questions 03.08.01-51 and 03.08.01-54 are being tracked as confirmatory items** pending revision of the FSAR to incorporate the proposed changes.

In FSAR Tier 2, Revision 0, Sections 3.8.1.1.3 and 3.8.1.4.10, "Steel Liner Plate and Anchors," the applicant indicated that the RBIS is not anchored to the surrounding portions of the RCB or to the underlying NI basemat. Furthermore, the liner plate at the horizontal interface between the RBIS and the underlying NI basemat is not anchored to either of these two structures, so there is minimal transfer of lateral loads from the RBIS through this interface. Therefore, the main load path for lateral seismic loads and overturning moments from the RBIS to the surrounding structures is through horizontal bearing pressure on the RCB haunch wall and vertical bearing pressure on the underlying NI basemat.

The staff identified the following issues associated with the above design approach:

1. The FSAR did not explain how seismic stability for the RBIS is ensured and what the associated analysis method is, and in particular how the factors of safety given in FSAR Tier 2, Section 3.8.5.5.1, "Nuclear Island Common Basemat Structure Foundation Basemat," had been computed.

2. The FSAR did not describe any analyses performed to determine the magnitude of potential uplift or sliding of the RBIS due to seismic loads, which could damage the RCB liner plate.

3. It was not clear whether the applicant had determined the magnitude of the contact pressures imposed by the RBIS on the surrounding portions of the RCB and the NI basemat, and whether these contact pressures had been taken into consideration in the design.

Therefore, in RAI 155, Question 03.08.01-2; RAI 190, Question 03.08.01-29; and RAI 283, Question 03.08.01-37, the staff requested that the applicant address these issues.

In a March 31, 2009, response to RAI 155, Question 03.08.01-2, the applicant explained that a seismic stability analysis of the RBIS was performed and provided the results of this analysis. The analysis was based on a linear FE model of the RBIS with simplified boundary conditions and statically applied loads. The analysis ignored the lateral restraint provided by the RCB haunch wall. The factor of safety against sliding was found to be 0.16, significantly less than the minimum value of 1.10 specified in SRP Acceptance Criteria 3.8.5.II.5. The factor of safety against overturning was found to be 1.22, which is higher than the minimum value of 1.10 specified in SRP Acceptance Criteria 3.8.5.II.5. From this analysis, the applicant concluded that (1) friction at the horizontal interface between the RBIS and the underlying NI basemat, by itself,

cannot prevent sliding of the RBIS; (2) sliding is prevented by the lateral restraint provided by the RCB haunch wall; (3) the horizontal contact pressures imposed by the RBIS on the RCB haunch wall are significant and need to be considered in the design; and (4) vertical uplift of the RBIS is not credible because of the acceptable factor of safety against overturning.

The staff determined that the applicant's seismic stability analysis did not address all the issues associated with potential uplift and sliding of the RBIS under seismic loads, because it only computed factors of safety and not the actual magnitude of uplift and sliding. The staff noted that damage to the liner plate due to impact or tearing could result if the magnitude of uplift and sliding are significant. In particular, although the applicant's analysis determined an acceptable factor of safety against overturning, the staff did not agree with applicant's conclusion that vertical uplift of the RBIS was not credible, because a structure that is stable with respect to overturning can still rock and sustain vertical uplift. The staff noted that only a realistic nonlinear analysis of the RBIS could determine the magnitude of uplift and sliding.

In a June 30, 2010, response to RAI 190, Question 03.08.01-29, the applicant stated that it performed a nonlinear dynamic analysis of the RBIS and its immediate surrounding structures to evaluate potential RBIS sliding and uplift under seismic loads and provided the analysis results. Frictional contact elements were used to simulate the interface between the RBIS and the surrounding structures. The analysis results enveloped cases corresponding to lower and upper bound friction coefficients. Based on the results of this analysis, the applicant reached the following conclusions:

1. The applicant's analysis concluded that there would be only minor uplift and localized sliding of the RBIS with respect to the NI basemat: maximum uplift was .127cm (0.05 in.) and maximum sliding was .610cm (0.24 in.) The minimum contact area at the interface between the RBIS and the NI basemat was calculated to be approximately 79 percent of the total area.

2. The maximum strains in the liner plate were calculated to be 0.0000464 cm/cm (0.0000183 in./ in.) in tension and 0.000065 cm/cm (0.0000256 in./in.) in compression, which fall within the strain limits of 0.0076 cm/cm (0.003 in/inch) in tension and 0.0127 cm/cm (0.005 in./ in.) in compression in ASME Code, Subarticle CC-3720. No peripheral tearing of the liner plate would occur due to uplift. No damage to the liner plate would occur due to repetitive compressive impact forces.

3. The maximum contact pressures imposed by the RBIS on the surrounding structures were calculated to be 2.94 MPa (427 psi) in the vertical direction and 3.18 MPa (462 psi) in the horizontal direction; both values fall within the stress limit of 16.5 MPa (2,400 psi) derived from ASME Code, Subarticle CC-3421.1. In this regard, the applicant indicated that the RCB haunch wall, which provides most of the horizontal sliding resistance, was designed for a horizontal contact pressure of 5.20 MPa (755 psi); therefore, the contact pressures used in the design of the RCB haunch wall bound the horizontal contact pressures calculated in the analysis.

In an October 5, 2010, response to RAI 283, Question 03.08.01-37, the applicant provided the results of a revised seismic stability analysis of the RBIS, which was based on the nonlinear dynamic analysis performed in a June 30, 2010, response to RAI 190, Question 03.08.01-29. The revised factors of safety against sliding and overturning were found to be 2.8 and 1.9, respectively, higher than the corresponding minimum values of 1.10 specified in SRP Acceptance Criteria 3.8.5.II.5. The response also included proposed revisions to FSAR Tier 2, Revision 2, Section 3.8.5.5.1.

The staff finds the information provided by the applicant in the RAI responses summarized above acceptable because: (1) The methodology for the analysis performed in the June 30, 2010, response to RAI 190, Question 03.08.01-29, was in accordance with SRP Acceptance Criteria 3.7.2.II.1.A, seismic dynamic analysis methods; 3.8.1.II.4, dynamic and analysis procedures for containment; and 3.8.3.II.4, dynamic and analysis procedures for containment internal structures; which describe provisions of the design and analysis procedures for the RBIS that should be considered, (2) the applicant demonstrated that no damage to the liner plate occurs due to uplift or localized sliding of the RBIS under seismic loads; (3) the applicant also demonstrated that no damage to the surrounding structures occurs due to horizontal and vertical bearing pressures imposed by the RBIS under seismic loads; (4) in all cases, significant margins were demonstrated by the applicant with respect to the limits prescribed in ASME Code, Article CC-3000; (5) factors of safety against sliding and overturning were found to be higher than the minimum values specified in SRP Acceptance Criteria 3.8.5.II.5; and (6) horizontal contact pressures used in the design of the RCB haunch wall bounded the horizontal contact pressures calculated from the analysis.

Based on the above discussion, the staff considers RAI 155, Question 03.08.01-2 and RAI 190, Question 03.08.01-29 resolved, while **RAI 283, Question 03.08.01-37 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed changes.

<u>Structural Acceptance Criteria</u>

To ensure that the containment, as designed, can perform its intended safety function, the staff verified that the structural design acceptance criteria meet the relevant requirements in 10 CFR 50.55a; GDC 1, 2, 4, 16, 50, and 53; and 10 CFR 50.44; and are in accordance with SRP Acceptance Criteria 3.8.1.II.5.

The staff reviewed the acceptance limits for the RCB in terms of stresses, strains, deformations, factors of safety, and other criteria in FSAR Tier 2, Revision 2, Section 3.8.1.5, and determined that they are in accordance with the requirements of the ASME Code, Section III, Division 2, Subsection CC and the recommendations of RG 1.136. These acceptance criteria apply to the containment vessel, subassemblies, and appurtenances that are part of the pressure retaining function of the containment, except for those elements that are covered by FSAR Tier 2, Section 3.8.2. For allowable loads on concrete embedments and anchors, the staff concluded that the acceptance criteria are in accordance with ACI 349-06 (with exceptions stated in FSAR Tier 2, Section 3.8.4.4, "Design and Analysis Procedures") and the guidance given in RG 1.199 (with exceptions noted in FSAR Tier 2, Section 3.8.1.4.10). The exceptions noted for ACI 349-06 and RG 1.199, which also appear in FSAR Tier 2, Sections 3.8.3, "Concrete and Steel Internal Structures of Concrete Containment," and 3.8.4, "Other Seismic Category I Structures," are reviewed by the staff under RAI 155, Question 03.08.03-2 and RAI 283, Question 03.08.03-20, in Section 3.8.3.4 of this report for the entire NI.

Therefore, the staff finds that the structural acceptance criteria presented in FSAR Tier 2, Section 3.8.1.5, applicable to the RCB, comply with those identified in SRP Acceptance Criteria 3.8.1.II.5 and RG 1.136, and are therefore acceptable.

<u>Materials, Quality Control, and Special Construction Techniques</u>

The staff reviewed the materials, quality control, and special construction techniques used for the RCB to ensure that they meet the relevant requirements in 10 CFR 50.55a; GDC 1, GDC 2, GDC 4, GDC 16, GDC 50, GDC 53; and 10 CFR 50.44; and are in accordance with the ASME Code and SRP Acceptance Criteria 3.8.1.II.6. With the exceptions described below, the staff

finds that the materials, quality control, and special construction techniques meet the regulatory requirements, regulatory guidance, and the ASME Code cited above.

The staff reviewed information relating to the materials, quality control program, and special construction techniques used to construct the RCB in FSAR Tier 2, Section 3.8.1.6. The review of the RCB post-tensioning system identified that (1) the tendon duct specifications that are applicable to non-ferrous duct material appeared to deviate from the requirements in the ASME Code, Section III, Division 2, Subarticle CC-2441, and (2) the FSAR did not describe the grouting process, which should ensure that no voids are left in the tendon ducts after grouting. Therefore, in RAI 155, Question 03.08.01-18, the staff requested that the applicant address these concerns.

In a February 13, 2009, response to RAI 155, Question 03.08.01-18, the applicant revised the material for the tendon ducts to rely on corrugated steel ducts and rigid metal conduit. The applicant also indicated that the RCB will be post-tensioned using the Freyssinet C-range post-tensioning system. There are two variations of this system: System No. 1 and No. 2, both of which use corrugated steel tendon ductwork, as well as ferrous trumpet and transition cone parts. In some areas, rigid metal pipe will be used instead of corrugated steel ductwork. All of these non-load-carrying materials meet the requirements of ASME Code, Section III, Division 2, Subarticle CC-2441. System No. 2 includes an additional sheathing around each seven-wire strand described in the response as polyurethane sheathing.

Regarding the grouting process, the applicant stated that the corrugated steel ducts have been used extensively in post-tensioned concrete construction. The requirements in ASME Code, Section III, Division 2, Subarticle CC-4282 will be satisfied by developing a grouting procedure designed for the specific job conditions. The grouting procedure will incorporate grout manufacturer and tendon manufacturer recommendations.

From a review of the February 13, 2009, response to RAI 155, Question 03.08.01-18, the staff concluded that the proposed revision to eliminate the use of non-ferrous ducts, and rely on steel ducts and rigid metal conduit is acceptable because this change complies with the provisions in the ASME Code, Section III, Division 2, Subarticle CC-2441 which indicates that the material for tendon ducts shall be ferrous. However, the response did not adequately explain how the grouting process ensures that no void spaces will occur in the ducts, and another concern was identified regarding the use of the polyurethane sheathing around each of the seven-wire strands. Therefore, in follow-up RAI 248, Question 03.08.01-35, the staff requested that the applicant explain (1) how the polyurethane sheathing meets the requirements in the ASME Code, Section III, Division 2, Subarticle CC-2442.3, and also how the polyurethane sheathing affects the assumptions for bonding between the tendon and surrounding grout; and (2) how the grouting process ensures that there will be no void spaces in the corrugated steel tendon ductwork.

In a September 29, 2009, response to RAI 248, Question 03.08.01-35, 2009, the applicant indicated that the sheathing around each seven-wire strand is composed of high-density polyethylene (HDPE), not polyurethane as previously stated. In addition, the tendon manufacturer’s experience has shown that this HDPE sheathing is inert within the post-tensioning system. If System No. 2 is used for the RCB, any grease used will meet the requirements in the ASME Code, Section III, Division 2, Subarticle CC-2442.3. The applicant also indicated that the design of the RCB post-tensioning system does not assume any bonding between the tendons and the surrounding grout.

Regarding the grouting process, the applicant indicated that the corrugated steel ducts have been used extensively in post-tensioned concrete construction. In addition, when using the proper materials, details, equipment, and procedures (items to be provided by the manufacturer), the applicant indicated that grouting can typically be completed with 100 percent fill using conventional techniques. Also, inspection ports can be provided at locations where the potential exists for voids to occur. In some cases, the manufacturer may recommend an alternative grouting procedure to fill the tendon ducts completely.

The staff finds the information for the HDPE sheathing and grease acceptable because the coatings will meet the requirements in ASME Code, Section III, Division 2, Subarticle CC-2442.3 with regard to the type of coatings permitted, the properties of the coatings, and the physical and chemical analysis of the coatings. However, the RAI response did not provide the grouting procedures that would be followed to ensure that no significant voids will be present or the operating experience with the use of the Freyssinet C-Range post-tensioning system in terms of its overall performance and potential aging degradation effects. This additional information is important because the use of grouted tendons in the U.S. for NPPs is very limited, and in particular, the use of the Freyssinet C-Range post-tensioning system is new to U.S. nuclear power plants.

During the February 14-17, 2011, onsite audit, the applicant indicated that it would address the staff's concerns regarding grouting procedures, performance of the Freyssinet C-Range post-tensioning system, and potential aging degradation effects by committing to RG 1.90, Revision 1, and RG 1.107, Revision 1, without exceptions. The applicant indicated that this additional information would be provided in a supplemental response to RAI 248, Question 03.08.01-35.

In a March 30, 2011, response to RAI 248, Question 03.08.01-35, the applicant confirmed the information provided to the staff during the February 14-17, 2011, audit. The staff finds this information acceptable because the ISI program and the grouting procedures for the post-tensioning system of the RCB will conform to the guidance in RG 1.90, Revision 1, and RG 1.107, Revision 1, respectively, in addition to meeting the requirements in the ASME Code. In this regard, conforming to RG 1.90, Revision 1, establishes that the ISI program will include force monitoring of ungrouted test tendons, monitoring the performance of grouted tendons by either reading instrumentaion to determine the prestress level or measuring deformation of the containment under the test pressure, and visual examination of the critical areas of the containment. Furthermore, conforming to RG 1.107, Revision 1, establishes that the grouting procedures will meet the regulatory positions in the guide for materials (e.g., cement, aggregate, water, admixtures) properties of the grout equipment for grouting, grouting process, and tendon condition. Therefore, the staff considers RAI 155, Question 03.08.01-18 resolved, while **RAI 248, Question 03.08.01-35 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed changes.

Testing and Inservice Surveillance Requirements

To ensure that the containment can perform its intended safety function, the staff verified that the testing and ISI surveillance requirements meet the relevant requirements in 10 CFR 50.55a; GDC 1, GDC 16, GDC 50, and GDC 53; and are in accordance with ASME Code, Article CC-6000, Subsections IWL and IWE, and SRP Acceptance Criteria 3.8.1.II.7. With the exceptions described below, the staff finds that the testing and ISI surveillance requirements meet the regulatory requirements, regulatory guidance, and the ASME Code cited above.

From the information provided in FSAR Tier 2, Section 3.8.1.7.2, it was unclear to the staff whether the ISI program for the RCB, a prestressed concrete structure with grouted tendons, meets the guidance in RG 1.90, Revision 1. The following issues were of particular concern to the staff: (1) The dimensions of the RCB imply that it is not a “reference containment” type, as defined in RG 1.90, Section B, so it was unclear to the staff whether RG 1.90, Appendix A, was being followed; (2) without sufficient technical justification, the FSAR stated that force monitoring of ungrouted test tendons recommended by RG 1.90 would not be used as part of the ISI program; (3) the FSAR did not clearly state that measuring deformation of the RCB under test pressure (Alternative B in RG 1.90) would be used to monitor the performance of the grouted tendons; (4) without sufficient technical justification, the FSAR stated that the ISI test pressure at Years 3 and 7 would be the accident pressure (379 kPaG (55 psig)), which is significantly lower than 1.15 times the design pressure (491 kPaG (71 psig)) recommended by RG 1.90; (5) it was unclear to the staff whether the RCB design considered the additional design criteria in RG 1.90, such as maintaining membrane compression under peak test pressure and limiting the maximum tensile stress in the reinforcement; and (6) the FSAR did not describe the locations for measuring deformation of the RCB under test pressure. Therefore, in RAI 155, Question 03.08.01-12 and RAI 211, Question 03.08.01-31, the staff requested that the applicant clarify these issues.

In a May 29, 2009, response to RAI 155, Question 03.08.01-12, the applicant provided technical justification for Items (2) and (4) stated above.

Regarding Item (2), the applicant indicated that ungrouted test tendons may not accurately reflect the prestress losses in the entire containment. The applicant also indicated that, rather than using ungrouted test tendons for force monitoring of prestress losses, the ISI program would implement deformation monitoring of the RCB under test pressure and the results would be compared with expected deformations. The staff determined the applicant’s justification of Item (2) inadequate because, according to RG 1.90, the ISI program should consist of three distinct activities: (1) Force monitoring of ungrouted test tendons; (2) monitoring the performance of grouted tendons by either periodically reading instrumentation to determine prestress level (Alternative A) or measuring deformation under test pressure (Alternative B) at pre-established sections; and (3) visual examination. These three activities provide complementary information; therefore, measuring deformation of the RCB under test pressure does not eliminate the need to provide force monitoring of ungrouted test tendons.

Regarding Item (4), the applicant indicated that continued pressurization of the containment to 1.15 times the design pressure would induce unnecessary cyclic loading of the structure. The applicant also indicated that using the lower accident pressure would establish a continuous basis for comparison of results, minimize gradual propagation of cracking during subsequent pressure tests, and be in compliance with the ISI requirements of ASME Code, Subarticle IWL-5220. The staff determined the applicant’s justification of Item (4) inadequate, because ASME Code, Article IWL-5000 of the ASME Code is applicable to pressure testing of the containment following repair/replacement activities and not the periodic ISI pressure tests. Furthermore, the applicant implied that using 1.15 times the design pressure as ISI test pressure for Years 3 and 7 would be unnecessarily conservative and possibly detrimental to the structure. However, the staff noted that one of the considerations for using Alternative B in RG 1.90 is that the design of the containment should be conservative so that cracking under repeated ISI pressure tests is minimized; therefore, the containment should be designed to minimize cracking under repeated ISI pressure tests at the test pressures prescribed in RG 1.90. To address some of the outstanding issues related to the ISI program, additional RAIs were developed which are discussed below.

In a November 4, 2009, response to RAI 211, Question 03.08.01-31, the applicant provided additional information regarding items (1), (3), (5), and (6) given above.

Regarding Item (1), the applicant confirmed that the RCB has larger dimensions than the "reference containment" type specified in RG 1.90; however, the applicant also provided numerical calculations to show that the number of deformation monitoring locations considered in the FSAR is greater than the number recommended by RG 1.90, Appendix A. Regarding Item (3), the applicant confirmed that Alternative B in RG 1.90 would be used to monitor the performance of the grouted tendons. Regarding Item (6), the applicant confirmed that the locations for measuring deformation of the RCB under test pressure would be based on RG 1.90. The staff finds this information for these three items technically acceptable, because it conforms to the guidance in RG 1.90.

Regarding Item (5), the applicant indicated that the RCB design was in accordance with ASME Code, Article CC-3000; in particular, with ASME Code, Subarticle CC-3432.1(c), which allows an increase in the allowable tensile stress of the reinforcement when the temporary pressure loads are combined with other loads. The staff determined that this does not conform to the use of Alternative B in RG 1.90, which requires a more conservative containment design and requires limits on the maximum tensile stress of the reinforcement.

To address the outstanding issues (Items (2), (4), and (5) given above), in RAI 448, Questions 03.08.01-50 and 03.08.01-53, the staff requested that the applicant either conform to RG 1.90 or provide additional technical justification for the proposed alternative approach. Additionally, the staff requested that the applicant provide information regarding the visual inspection component of the ISI program, which is not mentioned in the FSAR or in the May 29, 2009, response to RAI 155, Question 03.08.01-12, or the November 4, 2009, response to RAI 211, Question 03.08.01-31.

During the February 14-17, 2011, onsite audit, the applicant explained that to address the staff's concerns regarding the ISI program, it would revise the FSAR to clearly indicate conformance to RG 1.90, Revision 1, without exceptions, and include additional information regarding the visual inspection component of the ISI program. An additional concern of the staff regarding sufficient physical access to perform the visual inspection would also be addressed by the applicant. The applicant indicated that this additional information would be provided in its response to RAI 448, Questions 03.08.01-50 and 03.08.01-53.

In May 20, 2011, and April 27, 2011, responses to RAI 448, Questions 03.08.01-50 and 03.08.01-53, respectively, the applicant confirmed the information provided to the staff during the February 14-17, 2011, audit. The staff finds this information acceptable, because the ISI program of the RCB will conform to the guidance in RG 1.90, Revision 1, without exceptions.

In addition, the applicant also confirmed there is sufficient physical access to perform the visual inspection from the tendon gallery, annular space between the RCB exterior wall and the RSB wall, and the annular space between the RCB dome and RSB dome. The minimum space is approximately 0.46 m (18 in.) around the RCB ring girder. Although the space is restricted, it does not physically impede the visual inspection. ASME Code, Subarticle IWL-2310(c) gives the option of performing the visual inspection by either direct or indirect means (e.g., cameras). Therefore, the staff finds this information acceptable.

Based on the above discussion, the staff considers RAI 155, Question 03.08.01-12 resolved, while **RAI 211, Question 03.08.01-31, and RAI 448, Questions 03.08.01-50 and 03.08.01-53**

are being tracked as confirmatory items pending revision of the FSAR to incorporate the proposed changes.

<u>ITAAC and Technical Specifications</u>

ITAAC: The staff has reviewed the ITAAC associated with FSAR Tier 2, Section 3.8.1 in FSAR Tier 1, Table 2.1.1-8 and FSAR Tier 1, Table 3.5-3. With the exception of Item 2.6 in FSAR Tier 1, Table 2.1.1-8, which needs to be revised as part of the resolution of RAI 306, Question 03.08.01-39 discussed above, the staff finds the ITAAC acceptable for this area of review. The next revision of FSAR Tier 1, Table 2.1.1-8 needs to be reviewed by the staff to confirm that the applicant has incorporated the proposed changes.

Technical Specifications: The staff has reviewed the Technical Specifications associated with FSAR Tier 2, Section 3.8.1 as provided in FSAR Tier 2, Chapter 16, Section 3.6 and finds them consistent with the information presented in FSAR Tier 2, Section 3.8.1, and are therefore acceptable for this area of review.

3.8.1.5 *Combined License Information Items*

Table 3.8.1-1 provides a list of concrete containment related COL information item numbers and descriptions from FSAR Tier 2, Table 1.8-2:

Table 3.8.1-1 U.S. EPR Combined License Information Items

Item No.	Description	FSAR Tier 2 Section
3.8-1	A COL applicant that references the U.S. EPR design certification will confirm that site-specific loads lie within the standard plant design envelope for the Reactor Containment Building, or perform additional analyses to verify structural adequacy.	3.8.1.3
3E-1	A COL applicant that references the U.S. EPR design certification will address critical sections relevant to site-specific Seismic Category I structures.	3E

The staff finds the above listing to be complete. Also, the list adequately describes actions necessary for the COL applicant. No additional COL information items need to be included in FSAR Tier 2, Table 1.8-2 for concrete containment design consideration.

3.8.1.6 *Conclusions*

As discussed above, the staff reviewed FSAR Tier 2, Revision 2, Section 3.8.1, and associated subsections of FSAR Appendix 3E, to determine whether the design, construction, testing, and ISI of the concrete containment comply with 10 CFR 50.55a; GDC 1, GDC 2, GDC 4, GDC 16, GDC 50, GDC 53; and 10 CFR 50.44. Except for the open and confirmatory items described above, the staff concludes that the concrete containment meets the relevant requirements of 10 CFR 50.55a; GDC 1, GDC 2, GDC 4, GDC 16, GDC 50, GDC 53; and 10 CFR 50.44. The basis for this conclusion follows.

The applicant meets the requirements of 10 CFR 50.55a and GDC 1 to ensure that the concrete containment is designed to quality standards commensurate with the importance of the safety function to be performed, by meeting the guidelines in SRP Section 3.8.1; RG 1.7, RG 1.90, RG 1.107, RG 1.136; and the requirements of ASME Code, Section III, Division 2, Subsection CC, and Section XI, Subsections IWE and IWL. In addition, the applicant has met these requirements to ensure the containment is fabricated, erected, and tested to quality standards commensurate with the importance of the safety function to be performed, insofar as the applicant has proposed commitments in this regard.

As discussed above, the applicant meets the requirements of GDC 2 to ensure that the concrete containment is able to withstand the most severe natural phenomena by analyzing and designing the containment to withstand the most severe earthquake that has been established for the U.S. EPR certified design, in load combinations that include the earthquake loading. Protection from other natural phenomena such as winds, tornadoes, and floods is provided by other Seismic Category I structures that shield the RCB.

As discussed above, the applicant meets the requirements of GDC 4 to ensure that the concrete containment is appropriately protected against other dynamic effects, by analyzing and designing the containment, where applicable, to withstand the dynamic effects associated with pipe whipping, missiles, and discharging fluids associated with the LOCA.

As discussed above, the applicant meets the requirements of GDC 16 to ensure that the concrete containment acts as a leak-tight barrier to prevent the uncontrolled release of radioactive effluents to the environment, by meeting the design, testing, and ISI guidelines in SRP Section 3.8.1; RGs 1.7, 1.90, 1.107, and 1.136; and the requirements of ASME Code, Section III, Division 2, Subsection CC, and Section XI, Subsections IWE and IWL. Furthermore, the FSAR also indicates that throughout its service life, the containment is monitored periodically in accordance with 10 CFR 50.55a, which provides supplemental requirements to the ASME Code, Section XI, Subsections IWE and IWL, and in accordance with 10 CFR Part 50, Appendix J, for containment leakage testing. In addition, the applicant has met these requirements to ensure the containment is constructed to act as a leak-tight barrier to prevent the uncontrolled release of radioactive effluents to the environment, insofar as the applicant has proposed commitments in this regard.

As discussed above, the applicant meets the requirements of GDC 50 to ensure that the concrete containment is designed with a sufficient margin of safety to accommodate appropriate design loads, by meeting the design guidelines in SRP Section 3.8.1; RG 1.7, RG 1.90, RG 1.107, RG 1.136; and the requirements of ASME Code, Section III, Division 2, Subsection CC.

As discussed above, the applicant meets the requirements of GDC 53 to ensure that the concrete containment is designed to permit appropriate ISI, by meeting the guidelines in SRP Section 3.8.1; ASME Code, Section III, Division 2, Subsection CC; and the requirements of ASME Code, Section XI, Subsections IWE and IWL.

As discussed above, the applicant meets the requirements of 10 CFR 50.44 to ensure that the structural integrity of the containment has been demonstrated for loads associated with combustible gas generation, by meeting the guidelines in SRP Section 3.8.1 and RG 1.7, RG 1.107, RG 1.136, and RG 1.216.

3.8.2 Steel Containment

3.8.2.1 *Introduction*

FSAR Tier 2, Section 3.8.2, “Steel Containment,” describes the steel RCB penetrations and portions of penetrations that are not backed by structural concrete and are intended to resist pressure. FSAR Tier 2, Section 3.8.1, “Concrete Containment,” addresses the concrete portions of the RCB and the steel liner backed by concrete.

3.8.2.2 *Summary of Application*

FSAR Tier 1: The FSAR Tier 1 information associated with this section is found in FSAR Tier 1, Section 2.1.1, “Nuclear Island.” This section requires that the RCB, including the steel penetration assemblies, maintains its pressure boundary integrity at the design pressure. This section also requires that guard pipes are placed around high energy pipelines that pass through RB annulus penetrations so that consequential failures to other safeguard systems cannot occur.

FSAR Tier 2: The applicant has provided an FSAR Tier 2 description of the steel RCB penetrations and portions of penetrations that are not backed by structural concrete and are intended to resist pressure in FSAR Tier 2, Section 3.8.2, which is summarized as follows:

Description of Containment

FSAR Tier 2, Section 3.8.2.1, “Description of the Containment,” describes the following steel items that form the RCB pressure boundary not backed by concrete: Equipment hatch; airlocks; construction opening; piping penetration sleeves; electrical penetration sleeves; and fuel transfer tube penetration sleeve.

FSAR Tier 2, Section 3.8.2.1.1, “Equipment Hatch, Dedicated Spare Penetration, Airlocks, and Construction Opening,” describes the equipment hatch, spare penetration, airlocks, and construction opening.

The equipment hatch is a welded steel assembly with a double-sealed, flanged, and bolted cover. The cover for the equipment hatch seats against the sealing surface of the penetration sleeve mating flange when subjected to containment internal pressure. The penetration sleeves in the RCB and the RSB are connected by an expansion joint to allow for differential movement between the RCB and RSB walls. The equipment hatch is protected from external environmental hazards. The equipment hatch sleeve is approximately 8.31 m (27 ft 3 in.) in diameter.

The continment penetrations also include a 0.91 M (36 in.) diameter spare containment penetration dedicated for post-accident conditions as described in FSAR Tier 2, Section 19.2.3.3.8 “Containment Venting.”

One personnel airlock and one emergency airlock provide for personnel access to the RCB. Each airlock is a welded steel assembly that includes two doors, with double seals in each door. The airlock doors seat against the sealing surfaces when subjected to containment internal pressure. The personnel airlock and emergency airlock are attached to the RSB wall through the use of expansion joints which allow for differential movement. Both airlocks are protected from external environmental hazards such as high wind, tornado, and other site hazards.

The personnel airlock and the emergency airlock are approximately 3.10 m (10 ft 2 in.) in diameter.

The construction opening is at the heavy load operating floor level, and provides access for personnel and material into the RCB during construction. The construction opening is approximately 2.90 m (9 ft 6 in.) in diameter. After completion of construction, the construction opening is sealed permanently using a metal closure cap that is welded to an embedded sleeve.

The equipment hatch, dedicated spare penetration, two airlocks, and construction opening closure cap and sleeve are designated as Class MC components, which comply with ASME Code, Section III, Division 1, Article NE-3000.

FSAR Tier 2, Section 3.8.2.1.2, “Piping Penetration Sleeves,” describes piping penetration sleeves. All pipe sleeves comply with ASME Code, Section III, Division 1, Subsection NE.

High-energy penetrations are provided for the safety injection and chemical and volume control lines, as well as other high energy piping. Pipe sleeves, which are fabricated from carbon or stainless steel, consist of the portion of the penetration that projects into the RCB and supports the connecting part.

Main steam and feedwater penetrations are a special adaptation of the high-energy penetrations. The design is the same as the high-energy penetration except for the use of a guard pipe over the process pipe in the inner containment sleeve that is designed to dissipate heat and prevent the concrete from overheating. The protection pipes are connected to the RSB penetration sleeve by expansion bellows, which permit differential movement and minimizes load transfer between the RCB and RSB.

Standard piping penetrations are used for moderate or low energy piping lines where the basic configuration consists of an inline flued head component attached to the inner containment embedded pipe sleeve. A guard pipe is not utilized, but an expansion joint attached to the pipe and sleeve allows differential movement and minimizes load transfer between the RCB and RSB. Pipe sleeves are made from carbon or stainless steel and comprise the portion of the penetration that extends into the RCB and supports the flued head.

Spare penetrations are provided for future use. Spare penetrations contain the following key items: (1) Closure plates and pipe caps fabricated from carbon or stainless steel which complies with the requirements of Subsection NC of the ASME Code, Section III, Division 1, Subsection NC; and (2) pipe sleeves fabricated from carbon or stainless steel, consisting of the portion of the penetration that projects into the RCB.

FSAR Tier 2, Section 3.8.2.1.3, “Electrical Penetration Sleeves,” describes electrical penetration sleeves. Sleeves for electrical penetrations consist of the portion of penetrations that projects into the RCB and supports the electrical assembly. Electrical penetration sleeves comply with ASME Code, Section III, Division 1, Subsection NE.

FSAR Tier 2, Section 3.8.2.1.4, “Fuel Transfer Tube Penetration Sleeve,” describes the fuel transfer tube penetration sleeve. This penetration contains an approximately 0.51 m (20 in.) diameter stainless steel pipe installed inside a larger 0.91 m (36 in.) diameter penetration sleeve that is supported off the concrete RCB. The penetration sleeve complies with Subsection NE of the ASME Code, Section III, Division 1. The inner pipe acts as the transfer tube. Expansion joints are used around the fuel transfer tube where it passes through the RBIS refueling canal concrete wall and the RSB and FB concrete walls; thereby permitting differential movement

between these structures, and also maintaining leak-tight boundaries for the refueling pools and the annulus ventilation system.

Applicable Codes, Standards, and Specifications

FSAR Tier 2, Section 3.8.2.2, “Applicable Codes, Standards, and Specifications,” identifies the codes, standards, specifications, design criteria, regulations, and regulatory guides used for design, fabrication, construction, testing, and ISI of steel portions of the RCB that are intended to resist pressure, but are not backed by structural concrete. The boundaries between the reinforced concrete and the steel pressure boundary components are defined in ASME Code, Section III, Division 1, Subarticle NE-1132.

FSAR Tier 2, Section 3.8.2.2.1, “Codes,” identifies the following codes as applicable to the steel pressure retaining components of the RCB: The 2004 Edition of ASME Code, Sections II, III (Division 1), V, VIII, and IX; acceptable ASME Code Cases per RG 1.84, “Design, Fabrication, and Materials Code Case Acceptability, ASME Code, Section III”; the 2000 Edition of ANSI/AWS D1.1, “Structural Welding Code-Steel”; and the 1999 Edition of ANSI/AWS D1.6, “Structural Welding Code-Stainless Steel.”

FSAR Tier 2, Section 3.8.2.2.2, “Standards and Specifications,” identifies that ASTM standards are used to define material properties, testing procedures, fabrication, and construction methods. The applicable ASTM standard specifications for materials correspond to those identified in Article NE-2000 of ASME Code, Section III, Division 1. Applicable ASTM standard specifications for nondestructive methods of examination correspond to those identified in ASME Code, Section III, Division 1, Appendix X and Article X-3000.

FSAR Tier 2, Section 3.8.2.2.3, “Design Criteria,” identifies that the design of steel pressure retaining components of the RCB complies with ASME Code, Section III, Division 1, Article NE-3000.

FSAR Tier 2, Section 3.8.2.2.4, “Regulations,” identifies the applicable NRC regulations. These consist of 10 CFR Part 50; 10 CFR Part 50, Appendix A, GDC 1, GDC 2, GDC 4, GDC 16, and GDC 50; and 10 CFR 50, Appendix J.

FSAR Tier 2, Section 3.8.2.2.5, “NRC Regulatory Guides,” identifies the following regulatory guides applicable to the design and construction of the steel pressure retaining components of the RCB: RG 1.7, “Control of Combustible Gas Concentrations in Containment,” Revision 3; RG 1.57, “Design Limits and Loading Combinations for Metal Primary Reactor Containment System Components,” Revision 1; RG 1.84, “Design, Fabrication, and Materials Code Case Acceptability, ASME Code, Section III, Revision 33; RG 1.136 ”Design Limits, Loading Combinations, Materials, Construction , and Testing of Concrete Containments,” March 2007, Revision 3 (with noted exceptions); RG 1.193, “ASME Code Cases Not Approved for Use,” Revision1; and RG 1.216, “Containment Structural Integrity Evaluation for Internal Pressure Loadings Above Design-Basis Pressure,” August 2010.

Loads and Load Combinations

FSAR Tier 2, Section 3.8.2.3, “Loads and Load Combinations,” describes the loads and load combinations for the steel pressure retaining components of the RCB. The FSAR indicates that the loads and load combinations are in accordance with the requirements of ASME Code, Section III, Division 1, Subsection NE, as augmented by the applicable provisions of RG 1.57.

FSAR Tier 2, Section 3.8.2.3.1, “Design Loads,” describes the individual loads considered in the design of the steel pressure retaining components of the RCB. These include dead loads, live loads, normal operating pressure and thermal loads, construction loads, SSE loads, DBA pressure and temperature loads, pipe break loads, and loads generated by combustible gas resulting from a metal-water reaction of the fuel cladding.

Since the RCB is completely shielded by the RSB and by other Seismic Category I structures, loads due to external events and natural phenomena (except earthquakes) are not applicable.

FSAR Tier 2, Section 3.8.2.3.2, “Design Load Combinations,” identifies the specific load combinations used in design of the steel pressure retaining components of the RCB.

Design and Analysis Procedures

FSAR Tier 2, Section 3.8.2.4, “Design and Analysis Procedures,” describes the design and analysis procedures for the steel pressure retaining components of the RCB. The FSAR indicates that the analysis and design comply with ASME Code, Article NE-3000, Section III, Division 1, Subsection NE, as augmented by the applicable provisions of RG 1.57.

Containment penetrations, or portions thereof, within the jurisdictional boundaries of ASME Code, Section III, Division 1, Subsection NE satisfy the stress intensity limits defined by Subarticles NE-3221.1, NE-3221.2, NE-3221.3, and NE- 3221.4. Evaluations of the containment penetrations use three dimensional finite element modeling techniques (using the ANSYS computer program). The steel components are screened for cyclic service analysis in accordance with the criteria in ASME Code, Section III, Division 1, Subarticle NE-3221.5.

Buckling analyses under earthquake, thermal, and pressure loads are performed. For simple geometries, such as piping penetrations, evaluations are performed in accordance with ASME Code, Section III, Division 1, Subarticle NE-3133 or ASME Code Case N-284-1. The resulting stresses are compared to the allowable buckling limits for the design conditions: Design; testing; and Service Levels A, B, C, and D. More complex geometries such as airlocks are analyzed using rigorous finite element buckling analyses.

Containment ultimate capacity analysis results, which include evaluation of major containment steel penetrations, are described in FSAR Tier 2, Section 3.8.1.4.11, “Containment Ultimate Capacity.”

Structural Acceptance Criteria

FSAR Tier 2, Section 3.8.2.5, “Structural Acceptance Criteria,” indicates that the structural acceptance criteria for the steel pressure retaining components of the RCB are in accordance with Subsections NC and NE of the ASME Code, Section III, Division 1, including allowable stress limits, strain limits, deformation limits, and factors of safety. These are augmented by the requirements of RG 1.57 and RG 1.216.

Steel items that are an integral part of the RCB pressure boundary are designed to meet minimum leakage rate requirements, in accordance with the applicable technical specification.

A SIT of the containment is performed, as described in FSAR Tier 2, Section 3.8.2.7, “Testing and Inservice Inspection Requirements.”

Steel components that form part of the containment pressure boundary are stamped in accordance with the ASME Code, Section III, Division 1, Article NE-8000.

Materials, Quality Control, and Special Construction Techniques

FSAR Tier 2, Section 3.8.2.6, “Materials, Quality Control, and Special Construction Techniques,” describes the materials, quality control, and special construction techniques for the steel pressure retaining components of the RCB.

The steel pressure retaining components of the RCB are fabricated to meet the requirements specified in ASME Code, Section III, Division 1, Article NE-2000, except as modified by applicable and acceptable ASME Code Cases. The major steel components of the major penetration assemblies are constructed from SA-516 Grade 70 material. The steel materials are defined in FSAR Tier 2, Table 6.1-1, “Pressure-Retaining Material Specifications for Engineered Safety Features.”

FSAR Tier 2, Section 3.8.1.6, “Materials, Quality Control, and Special Construction Techniques,” is referenced for additional information related to welding requirements for steel items for the RCB; quality control for steel items for the RCB; and materials used for penetration sleeves, steel embedments, and corrosion retarding compounds.

For the airlocks and the equipment hatch seals, an elastomer seal material (Dupont Viton, or equal) is used, which is compressed by the action of the mechanical closure devices. This material is recessed into two concentric grooves forming a double seal around the perimeter of the airlock doors and around the equipment hatch flange. This material is selected, because it maintains elasticity at elevated temperatures and complies with the materials tested for severe accident conditions, as specified in NUREG/CR-5096, “Evaluation of Seals for Mechanical Penetrations of Containment Buildings,” August 1988.

Quality control complies with ASME Code, Section III, Division 1, Articles NE-2000, NE-4000, and NE-5000. The equipment hatch, airlocks, fuel transfer tube, and penetrations are prefabricated and installed as subassemblies during construction. No special construction techniques are used.

Testing and Inservice Surveillance Requirements

FSAR Tier 2, Section 3.8.2.7 identifies that a SIT of the steel containment components not backed by concrete is performed in accordance with ASME Code, Section III, Division 1, Subsection NE, Article NE-6000.

The ISI for the steel pressure retaining subassemblies is conducted in accordance with the requirements of the ASME Code, Section XI, Subsection IWE, supplemented by the additional requirements specified in 10 CFR 50.55a, “Codes and Standards.” Containment leakage testing and the associated acceptance criteria are described in FSAR Tier 2, Section 6.2.6, “Containment Leakage Testing.”

The seals for mechanical closures are subject to both vendor testing and in-situ testing to ensure seal performance for normal operating conditions and for temperature and pressure conditions associated with a loss of coolant accident. Equipment installed in containment maintains the air space between the double seals under a negative pressure by connection to the leak-off system. When performing in-situ leak rate testing, this system is also used to pressurize the air space between the seals.

ITAAC: The ITAAC associated with FSAR Tier 2, Section 3.8.2 are given in FSAR Tier 1, Table 2.1.1-8, "Reactor Building ITAAC."

Technical Specifications: The Technical Specifications associated with FSAR Tier 2, Section 3.8.2 are given in FSAR Tier 2, Chapter 16, Section 3.6, "Containment Systems," specifically, Section 3.6.2, "Containment Airlocks."

Site Parameters: This section of the FSAR contains information related to the following site parameter that will be addressed in COL designs: Item No. 3-4 in FSAR Tier 2, Table 1.8-1, "Summary of U.S. EPR Plant Interfaces with Remainder of Plant," Site-specific loads that lie within the standard plant design envelope for Seismic Category I structures.

3.8.2.3 *Regulatory Basis*

The relevant requirements of NRC regulations for this area of review, and the associated acceptance criteria, are given in NUREG-0800, Section 3.8.2, "Steel Containment," Revision 2, and are summarized below. Review interfaces with other SRP sections also can be found in NUREG-0800, Section 3.8.2.

1 GDC 1, "Quality Standards and Records," as it relates to steel components of containment being designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety function to be performed.

2 GDC 2, "Design Bases for Protection against Natural Phenomena," as it relates to the design of the steel components of containment being able to withstand the most severe natural phenomena such as winds, tornadoes, floods, and earthquakes and the appropriate combination of all loads.

3 GDC 4, "Environmental and Dynamic Effects Design Bases," as it relates to the steel components of containment being appropriately protected against dynamic effects, including the effects of missiles, pipe whipping, and discharging fluids due to blowdown loads associated with the loss-of-coolant accident.

4 GDC 16, "Containment Design," as it relates to the capability of the steel components of containment to act as a leak-tight barrier to prevent the uncontrolled release of radioactivity to the environment.

5 GDC 50, "Containment Design Basis," as it relates to the steel components of containment being designed with sufficient margin of safety to accommodate appropriate design loads.

6 GDC 53, "Provisions for Containment Testing and Inspection," as it relates to the containment being designed to permit appropriate ISI and pressure testing of penetrations.

7 10 CFR Part 50, "Domestic Licensing of Production and Utilization Facilities," specifically, 10 CFR 50.55a, "Codes and Standards," as they relate to codes and standards.

8 10 CFR 50.44, "Combustible Gas Control for Nuclear Power Reactors," as it relates to demonstrating the structural integrity of the containment for loads associated with combustible gas generation.

Acceptance criteria adequate to meet the above requirements include:

1. RG 1.7, “Control of Combustible Gas Concentrations in Containment,” as it relates to the capability of the containment to withstand internal pressurization due to a 100 percent fuel-clad coolant reaction followed by hydrogen burning.

2. RG 1.57, “Design Limits and Loading Combinations for Metal Primary Reactor Containment System Components,” as it relates to the design requirements for steel portions of concrete containments that are intended to resist pressure, but are not backed by structural concrete.

3. RG 1.84, “Design, Fabrication, and Materials Code Case Acceptability, ASME Code, Section III,” as it relates to ASME Code Cases that have been determined by NRC to be acceptable alternatives to applicable parts of the ASME Code, Section III.

4. RG 1.193, “ASME Code Cases Not Approved for Use,” as it relates to ASME Code Cases that the NRC has determined not to be acceptable on a generic basis.

5. RG 1.216, “Containment Structural Integrity Evaluation for Internal Pressure Loadings Above Design-Basis Pressure,” as it relates to structural loads and acceptance criteria associated with internal pressurization due to 100 percent fuel-clad coolant reaction followed by hydrogen burning, and to the evaluation of internal pressure capacity of the containment.

3.8.2.4 *Technical Evaluation*

The staff's evaluation is based on the review of FSAR Tier 2, Sections 3.8.2.1, 3.8.2.2, 3.8.2.3, 3.8.2.4, 3.8.2.5, 3.8.2.6, and 3.8.2.7, including all subsections.

This section describes the staff's review and evaluation of FSAR Tier 2, Section 3.8.2, against the guidance of SRP Section 3.8.2 to ensure that the applicant has adequately addressed identified technical issues.

Description of Steel Containment

The staff reviewed the description of the steel portions of the RCB pressure boundary, not backed by concrete, to ensure that it contains sufficient information to define the primary structural aspects and elements that are relied upon to perform the containment function. With the exception described below, the staff found that the description meets the applicable regulatory requirements, regulatory guidance, and the ASME Code.

The staff’s review of FSAR Tier 2, Section 3.8.2.1.1 through 3.8.2.1.4, including Figures 3.8-25, “Equipment Hatch General Assembly,” through 3.8-31, “Fuel Transfer Tube Penetration (Conceptual View),” revealed that the major steel RCB penetrations (equipment hatch, airlocks, construction opening, fuel transfer tube penetration sleeve, and high energy penetration sleeves) were not described with sufficient detail to establish the primary structural elements that are relied upon to transfer loads and to perform the intended safety functions, in accordance with SRP Acceptance Criteria 3.8.2.II.1. The missing information included materials of construction and geometry of the penetration assemblies, including element thicknesses and dimensions as well as connection details. Therefore, in RAI 155, Question 03.08.02-1 (information for equipment hatch, airlocks, and high energy penetrations), RAI 155, Question 03.08.02-2 (information for the construction opening), RAI 155, Question 03.08.02-3

(information for fuel transfer tube); RAI 354, Question 03.08.02-11 (follow-up to RAI 155, Questions 03.08.02-1 and 03.08.02-2) and RAI 354, Question 03.08.02-12 (follow-up to RAI 155, Question 03.08.02-3), the staff requested that the applicant address the staff's concern regarding the missing information stated above.

In a June 30, 2009, response to RAI 155, Question 03.08.02-1, a July 31, 2009, response to, RAI 155, Question 03.08.02-2, an April 30, 2009, response to RAI 155, Question 03.08.02-3 an April 18, 2011, response to RAI 354, Question 03.08.02-11, and a March 4, 2011, response to RAI 354, Question 03.08.02-12, the applicant provided the requested detailed design description information which the staff finds acceptable. The staff considers RAI 155, Questions 03.08.02-1, 03.08.02-2, and 03.08.02-3, and RAI 354 Question 03.08.02-12 resolved. The resolution of these RAIs is discussed below under "Design and Analysis Procedures." **RAI 354, Question 03.08.02-11 is being tracked as a confirmatory item.**

<u>Applicable Codes, Standards, and Specifications</u>

To ensure that the steel portions of the RCB within the jurisdictional boundary of the ASME Code, Subsection NE, can perform their intended safety function, the staff verified that the applicable codes, standards, and specifications used in the design, fabrication, construction, testing, and ISI meet the relevant requirements in 10 CFR 50.55a; GDC 1, GDC 2, GDC 4, GDC 16, GDC 50, GDC 53; and 10 CFR 50.44, and comply with the guidance in SRP Acceptance Criteria 3.8.2.II.2.

FSAR Tier 2, Revision 2, Section 3.8.2.2, identifies the codes, standards, and specifications used in the design, materials, fabrication, erection, testing, and ISI surveillance of steel portions of the RCB within the jurisdictional boundary of the ASME Code, Subsection NE. The codes identified include the ASME Code, Sections II, III (Division 1), V, VIII, and IX. The standards and specifications include ASTM standards used to define material properties, testing procedures, fabrication, and construction methods.

The staff finds that the codes, standards, and specifications in FSAR Tier 2, Revision 2, Section 3.8.2.2, applicable to the steel portions of the RCB, comply with those identified in SRP Acceptance Criteria 3.8.2.II.2 and comply with those in 10 CFR 50.55a, and are therefore acceptable.

<u>Loads and Load Combinations</u>

To ensure that the steel portions of the RCB within the jurisdictional boundary of the ASME Code, Subsection NE, can perform their intended safety function, the staff verified that the loads and load combinations used in the design meet the relevant requirements in 10 CFR 50. 55a; GDC 1, GDC 2, GDC 4, GDC 16, GDC 50, GDC 53; and 10 CFR 50.44; and are in accordance with ASME Code, Subsection NE, and SRP Acceptance Criteria 3.8.2.II.3. With the exception described below, the staff finds that the loads and load combinations meet the regulatory requirements, regulatory guidance, and the ASME Code cited above.

In FSAR Tier 2, Section 3.8.2.3.2, Revision 0, under "Level B Service Limits," the applicant states that if a component is screened based on an analysis for cyclic operation, Service Level B load combinations may be eliminated. However, the technical basis for the screening could not be identified by the staff. Therefore, in RAI 155, Question 03.08.02-9, the staff requested that the applicant provide this technical basis.

In a March 31, 2009, response to RAI 155, Question 03.08.02-9, the applicant indicated that steel components are screened for cyclic service analysis according to the criteria in Subarticle NE-3221.5 of the ASME Code. If the specified service loads for the component meet all of the requirements of Subarticle NE-3221.5(d), then analysis for cyclic service is not required. The applicant also indicated that FSAR Tier 2, Section 3.8.2.3.2 would be revised to specifically reference ASME Code, Subarticle NE-3221.5. The staff finds this response acceptable, because it is in accordance with ASME Code criteria. The staff also verified that the proposed changes were incorporated into FSAR Tier 2, Revision 2. On this basis, the staff considers RAI 155, Question 03.08.02-9 resolved.

Design and Analysis Procedures

The staff reviewed the design and analysis procedures used for the steel portions of the RCB within the jurisdictional boundary of the ASME Code, Subsection NE, to ensure that they meet the relevant requirements in 10 CFR 50.55a; GDC 1, GDC 2, GDC 4, GDC 16, GDC 50, GDC 53; and 10 CFR 50.44; and are in accordance with ASME Code, Article NE-3000 and SRP Acceptance Criteria 3.8.2.II.4. The staff also reviewed the FSAR to establish that the design is essentially complete, as required by 10 CFR 52.47(c). Except for the exceptions described below, the staff finds that the design and analysis procedures meet the regulatory requirements, regulatory guidance, and the ASME Code cited above.

The staff's review of FSAR Tier 2, Sections 3.8.2.4, 3.8.2.4.1, and 3.8.2.4.2, including Figures 3.8-25 through 3.8-31, revealed that insufficient design information was provided for the major steel RCB penetrations (equipment hatch, airlocks, construction opening, fuel transfer tube penetration sleeve, and high energy penetration sleeves). The missing information included jurisdictional boundaries; materials of construction; detailed geometry, thicknesses, and dimensions; design-basis analysis conducted, including buckling; computer codes used in the analysis; summary of analysis results; comparison of results to ASME Code, Subsection NE, acceptance criteria; and an explanation of how the differential movement between the RCB and RSB was considered in the analysis for design-basis accident pressure and temperature conditions. Therefore, in RAI 155, Questions 03.08.02-1 (information for equipment hatch, airlocks, and high energy penetrations), 03.08.02-2 (information for the construction opening), 03.08.02-3 (information for fuel transfer tube), 03.08.02-4 (information regarding buckling analyses), 03.08.02-5 (information regarding analysis methods and computer codes), and 03.08.02-8 (information regarding design for differential movement between the RCB and RSB), the staff requested that the applicant provide the design information given above for the major steel RCB penetrations.

In a June 30, 2009, response to RAI 155, Question 03.08.02-1, the applicant provided information regarding jurisdictional boundaries, which the staff finds acceptable, because it is in accordance with ASME Code, Subarticle NE-1132. However, the applicant indicated that the major steel RCB penetrations were considered vendor supplied items to be designed in the detailed design phase; thus, the requested design information (i.e., materials, geometric information, design basis analyses, etc.) was not yet available.

In a July 31, 2009, response to RAI 155, Question 03.08.02-2, the applicant provided descriptive information for the construction opening. However, the applicant did not provide the requested information regarding materials, geometric information, and design-basis analyses performed.

In an April 30, 2009, response to RAI 155, Question 03.08.02-3, the applicant provided descriptive information for the fuel transfer tube. However, the applicant indicated that the fuel

transfer tube, like the other major steel RCB penetrations, was considered a vendor supplied item; thus, the requested design information (i.e., materials, geometric information, design basis analyses, etc.) was not yet available.

In a June 30, 2009, response to RAI 155, Question 03.08.02-4, the applicant explained that, since the major steel RCB penetrations were considered vendor supplied items, the required buckling analyses would be performed by the vendor during the detailed design phase.

In a May 29, 2009, response to RAI 155, Question 03.08.02-5, the applicant explained that, since the major steel RCB penetrations were considered vendor supplied items, information regarding analysis methods and computer codes was not available.

In a July 31, 2009, response to RAI 155, Question 03.08.02-8, the applicant provided descriptive information regarding how the equipment hatch, the airlocks, and the construction opening of the European EPR were designed to allow for differential movement between the RCB and RSB without compromising the leak-tightness of the containment. However, the applicant indicated that since the major steel RCB penetrations were considered vendor supplied items, the requested design information (i.e., geometric information, design basis analyses, etc.) for the U.S. EPR was not yet available.

The staff determined that the applicant's responses to RAI 155, Questions 03.08.02-1, 03.08.02-2, 03.08.02-3, 03.08.02-4, 03.08.02-5, and 03.08.02-8 stated above did not provide the information requested by the staff to define the primary structural aspects and elements that are relied upon to perform the containment function, to comply with the guidance in SRP Acceptance Criteria 3.8.2.II.1, 3.8.2.II.4, and 3.8.2.II.6. In particular, the applicant did not provide information regarding materials of construction; detailed geometry, thicknesses, and dimensions; design-basis analysis conducted, including buckling; computer codes used in the analysis; summary of analysis results; comparison of results to ASME Code, Subsection NE, acceptance criteria; and an explanation of how the differential movement between the RCB and RSB was considered in the analysis for design-basis accident pressure and temperature conditions. The staff was concerned that the applicant had not finalized the design of the major steel RCB penetrations. An incomplete design of these safety-critical elements prevents the staff from determining whether the requirements in 10 CFR 50.55a; GDC 1, GDC 2, GDC 4, GDC 16, GDC 50, GDC 53; and 10 CFR 50.44 could be met. Therefore, in follow-up RAI 354, Questions 03.08.02-11, 03.08.02-12, 03.08.02-13, and 03.08.02-15, the staff requested that the applicant address the staff's concerns.

In follow-up RAI 354, Question 03.08.02-11, the staff again requested that the applicant provide design information for the major steel RCB penetrations (equipment hatch, airlocks, construction opening, fuel transfer tube penetration sleeve, and high energy penetration sleeves) within the jurisdictional boundary of the ASME Code, Subsection NE. In addition, the staff also requested that this information be included in the relevant sections of the FSAR, in particular: (1) Materials of construction and detailed geometry; (2) description of the design-basis analyses conducted; (3) summary of analysis results; and (4) comparison of results to ASME Code, Subsection NE, acceptance criteria.

During the April 26-30, 2010, onsite audit, the applicant indicated that it would no longer consider the major steel RCB penetrations as vendor supplied items; the applicant would provide their design and update the FSAR to include the requested design information.

In an August 31, 2010, response to RAI 354, Question 03.08.02-11, the applicant indicated that the penetrations for the equipment hatch, airlocks, construction opening, MS and FW lines, and

fuel transfer tube had been evaluated to verify the integrity of the containment pressure boundary, in accordance with ASME Code, Subsection NE requirements. The following information was provided with the RAI response:

- The material used for the major components and penetration assemblies is carbon steel SA-516, Grade 70, which is an acceptable material per ASME Code, Table NE-2121(a)-1. FSAR Tier 2, Revision 2, Table 6.1-1was revised to include all materials used in the various penetration assemblies. FSAR Tier 2, Revision 2, Figures 3.8-25, 3.8-26, 3.8-27, 3.8-31, 3.8-119, 3.8-120, and 3.8-123 were revised or added to provide dimensional information and element thicknesses for the various penetration assemblies. The staff finds this information acceptable because it fully describes the penetration assemblies and allows the identification of a complete load path in accordance with SRP Acceptance Criteria 3.8.2.II.1, 3.8.2.II.4, and 3.8.2.II.6.

- The applicant's design-basis analysis methods relied on FE analysis using the ANSYS computer program. FE models represent the penetration assemblies and interfaces at the containment boundary. Loads and load combinations for design, testing, and service levels A, B, C, and D were evaluated in accordance with ASME Code, Article NE-3000. Additional load cases were included to address different operating conditions; for example, loads associated with ancillary equipment such as hatch doors, built-up floors, and inner hatch closed and open. The applicant compared the stresses resulting from the design-basis analyses to the stress intensity limits determined in accordance with ASME Code, Article NE-3000 for each loading condition, and comply with these limits. The RAI response, however, did not provide a summary of analysis results or a comparison to stress intensity limits and other acceptance criteria specified in Article NE-3000 of the ASME Code, as requested in the RAI. This outstanding issue was subsequently resolved by the staff's review of the applicant's calculations as discussed below.

The staff reviewed the applicant's calculations associated with RAI 354, Question 03.08.02-11, during the October 28-29, 2010, onsite audit. As a result of this review, the staff concluded that the applicant's design-basis analysis methods and evaluations meets the ASME Code, Subsection NE (steel portions not backed by concrete), Subsection CC (steel portions backed by concrete), and SRP Acceptance Criteria 3.8.2.II.4 and 3.8.2.II.5. However, the staff identified the following technical issues that needed further clarification by the applicant:

- The applicant's evaluations did not consider loads associated with combustible gas generation from a metal-water reaction of the fuel cladding, as described in SRP Acceptance Criteria 3.8.2.II.3 and RG 1.57.

- The applicant's evaluations did not consider loads associated with local dynamic loads such as jet impingement or associated missile impact, as described in SRP Acceptance Criteria 3.8.2.II.3 and RG 1.57.

- In a May 26, 2010, response to RAI 372, Question 06.02.04-9, the applicant added a 0.91 m (36 in.) diameter penetration with both ends covered with welded caps to use as vent for containment pressure release under beyond-design-basis accident conditions, as described in FSAR Tier 2, Revision 2, Section 19.2.3.3.8, "Containment Venting," (also discussed in Section 19.2.4.4.1 of this report). However, the staff could not identify design information related to this additional penetration.

- The staff could not determine the criteria used for selecting the MS and FW penetrations evaluated in the design as representative of all MS and FW penetrations and as the bounding case of all high-energy penetrations.

- The seismic loads used in the design-basis analyses were taken from a lumped-mass dynamic model using the BWSPAN computer program, not from the in-structure response spectra. The staff could not determine whether the BWSPAN model had been benchmarked and reconciled with the seismic analyses for the NI described in FSAR Tier 2, Section 3.7, "Seismic Design."

The applicant indicated that additional information to address the technical issues given above would be included in a revised response to RAI 354, Question 03.08.02-11, which was subsequently submitted on April 18, 2011. The following additional information was provided with the revised RAI response:

- Loads and load combinations associated with combustible gas generation were included in the updated design-basis evaluations of all containment penetrations. The staff confirmed this information during the February 14-17, 2011, onsite audit, and also confirmed that stresses (Subsection NE) and strains (Subsection CC) were within ASME Code limits. Additionally, the applicant indicated that FSAR Tier 2, Section 3.8.2 would be modified to clarify that load combinations corresponding to combustible gas generation were included in all design-basis evaluations. Therefore, the staff finds this item of the response acceptable.

- A pipe break hazard analysis determined that local dynamic loads such as pipe whip, missile impact, and jet impingement, are not applicable to the containment penetrations, because there is either sufficient distance between the penetrations and potential pipe break sources, leak before break considerations, or additional protection (i.e., whip restraints, jet shields) has been provided as described in FSAR Tier 2, Section 3.6, "Protection Against Dynamic Effects Associated with Postulated Rupture of Piping" (see FSAR Tier 2, Revision 2, Table 3.6.1-3,"Building, Room, Break, Target, and Protection Required"). In addition, the COL applicant is required to address an as-built pipe break hazards analysis which will ensure the containment penetrations are protected from dynamic loads per FSAR Tier 2, Table 1.8-2, Revision 2, COL Information Item 3.6-1 and 3.6-2 and ITAAC 3.8-1 will require an as-built pipe break hazards analysis, which will ensure the containment penetrations are protected from dynamic loads. The staff finds this part of the response acceptable because it provides the technical basis for not considering local dynamic loads in the design-basis evaluations, and because ITAAC 3.8-1 requires as-built pipe break hazards analysis to ensure that the penetrations are protected from local dynamic loads even if there is a change in the piping configuration subsequent to the design certification.

- Additional descriptive information related to the penetration for containment pressure release would be added to the relevant sections of the FSAR. The design of the portion of the penetration not backed by concrete (ASME Code, Subsection NE) is bounded by the design of the construction opening, which has the same cap thickness but is approximately 2.90 m (9.51 ft) in diameter (i.e., significantly larger than 0.91 m (36 in.) and thus, more highly stressed). The design of the portion of the penetration backed by concrete (ASME Code, Subsection CC) is bounded by the design of the MS penetration, which has a similar elevation, same thickness, but is approximately 1.42 m (4.66 ft) in diameter (i.e., larger than 0.91 m [36 in.] and, thus, sustains higher strains). This item of

the response is acceptable, because it provides the technical basis for a conservative design of the penetration for containment pressure release.

- Regarding the criteria for selecting the MS and FW penetrations as representative of all MS and FW penetrations and as the bounding case for all high-energy penetrations, the revised RAI response indicated that these lines are the controlling process piping penetrations based on energy content, buckling, seismic loading, and concrete loading on the penetration. There are four MS and FW lines; one MS penetration and one FW penetration are analyzed, and the loading selected for each of the analyses is the maximum of the four lines. This item of the response is acceptable, because the applicant's approach results in a conservative design of the MS and FW penetrations, which is bounding for all high-energy penetrations.

With respect to using the BWSPAN program to develop the seismic loads for the containment penetrations, during the February 14-17, 2011, onsite audit, the applicant explained that benchmarking of the BWSPAN model was complete, and reconciliation of the BWSPAN analysis with the NI seismic analysis is addressed in the response to RAI 201, Question 03.07.02-35. The resolution of this RAI is discussed in Section 3.7.2 of this report.

On the basis of the above discussions, the technical issues associated with RAI 155, Questions 03.08.02-1, 03.08.02-2, and 03.08.02-5 are resolved; and **RAI 354, Question 03.08.02-11 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed changes.

In follow-up RAI 354, Question 03.08.02-12, the staff again requested that the applicant provide design information for the fuel transfer tube penetration within the jurisdictional boundary of the ASME Code, Subsection NE. In addition, the staff also requested that this information be included in the relevant sections of the FSAR; in particular: (1) Sufficient information to determine the adequacy of the load path, including detailed geometry; and (2) sufficient information to determine the adequacy of the expansion bellows identified in FSAR Tier 2, Figure 3.8-31, "Fuel Transfer Tube Penetration (Conceptual View)," and how the differential movement between the RBIS, RCB, RSB, and FB was considered in the design, while ensuring leak-tightness of the containment pressure boundary.

During the April 26-30, 2010, onsite audit, the applicant indicated that it would perform the design of the fuel transfer tube penetration and update the FSAR to include the requested design information.

In an August 31, 2010, response to RAI 354, Question 03.08.02-12, the applicant provided the following information:

- The response included an explanation of the jurisdictional boundary between the fuel transfer tube components backed by concrete (ASME Code, Subsection CC), and the components not backed by concrete (ASME Code, Subsection NE); a list of fuel transfer tube components that are part of the containment pressure boundary; dimensional information of the fuel transfer tube components and fuel transfer tube penetration sleeve, anchors and connection to the containment liner plate; and reference to revised FSAR Tier 2, Figure 3.8-31, which contains additional dimensional information. The staff finds this information acceptable, because it fully describes the fuel transfer tube penetration assembly and allows the identification of a complete load path, in accordance with SRP Acceptance Criteria 3.8.2.II.1, 3.8.2.II.4, and 3.8.2.II.6.

- The applicant clarified that the expansion bellows at the fuel transfer tube compartment in the RBIS, RSB wall, and FB transfer pit wall are not part of the containment pressure boundary; moreover, the expansion bellows at the fuel transfer tube compartment in the RBIS and FB transfer pit wall are only designed to be leak-tight for the water column pressure that exists when the compartments are filled. The staff finds this information acceptable, because it addressed the staff's concern regarding lack of information on the load path and leak-tightness, especially the possibility that the expansion bellows could be subjected to LOCA pressure differential.

During the October 28-29, 2010, onsite audit, the staff reviewed the applicant's calculations associated with RAI 354, Question 03.08.02-12. As a result of the review, the staff identified the following technical issues that needed further clarification and, possibly, additional design and analysis work:

- The seismic analysis reported in the applicant's calculations did not consider the relative seismic anchor motions (SAMs) between the RCB, RSB, RBIS, and FB structures. The staff requested that the applicant provide the technical justification for not including SAMs in the design.

- The applicant's calculations did not consider the hydrostatic and hydrodynamic loads associated with water that could be stored in the fuel transfer pit during operations for water inventory control.

The applicant indicated that additional information to address the technical issues given above would be included in a revised response to RAI 354, Question 03.08.02-12, which was subsequently submitted on March 4, 2011. The revised RAI response indicated that the stress analyses for the fuel transfer tube were updated to take into account the SAMs for the RBIS, RCB, RSB, and FB, and the hydrostatic and hydrodynamic loads associated with water stored in the fuel building transfer pit. This information was confirmed by the staff during the February 14-17, 2011, onsite audit.

The staff finds the revised RAI response acceptable, because it complies with the guidance in SRP Acceptance Criteria 3.8.2.II.4 and 3.8.2.II.5, and is also consistent with the response to RAI 354, Question 03.08.02-11, which provides additional information regarding the design-basis evaluations of the fuel transfer tube in accordance with ASME Code, Subsection NE, including a discussion of the stress analyses performed.

On the basis of the above discussion, the staff considers the technical issues associated with RAI 155, Question 03.08.02-3 and RAI 354, Question 03.08.02-12 resolved.

In follow-up RAI 354, Question 03.08.02-13, the staff again requested that the applicant (1) describe the buckling analyses performed for the major steel penetrations of the RCB within the jurisdictional boundary of the ASME Code, Subsection NE; (2) provide a summary of analysis results; (3) provide a comparison of results to ASME Code, Subsection NE, acceptance criteria; and (4) include this information in the relevant sections of the FSAR.

During the April 26-30, 2010, onsite audit, the applicant indicated that it would perform the buckling analyses for the major steel penetrations of the RCB and update the FSAR to include the requested design information.

During the October 28-29, 2010, onsite audit, the staff reviewed the calculations associated with RAI 354, Question 03.08.02-13. Due to its simple geometry, the construction opening was

designed for buckling using the simplified approach in ASME Code, Subarticles NE-3133.4(e) and (d). In the case of the equipment hatch and airlocks, with more complicated geometries, nonlinear FE analyses were performed taking into account large deformations and bilinear material properties. In accordance with ASME Code, Subarticle NE-3222.1(a)(1), the applicant determined the allowable buckling load as the ultimate load from the FE analyses divided by a factor that depends on the service level. Also, for the equipment hatch and airlocks, additional calculations were performed in accordance with ASME Code Case N-284-1 and RG 1.193 to determine the effect of initial geometric imperfections on the buckling loads.

As a result of the review, the staff concluded that the applicant's buckling analysis methods and evaluations meet the ASME Code, Subsection NE, and SRP Acceptance Criteria 3.8.2.II.4 and 3.8.2.II.5. However, the staff identified the following technical issues that need further clarification by the applicant and, possibly, additional design and analysis work:

- ASME Code, Subarticle NE-3222.1(a)(1) clearly states that nonlinear FE analyses must consider the effects of initial geometric imperfections. However, these effects were not included in the calculations. The staff noted that, in the ASME Code Case N-284-1 calculations, the knockdown factors corresponding to initial geometric imperfections (α) appeared to be quite low in several instances. This may indicate sensitivity to initial geometric imperfections.

- The staff noted a discrepancy between the design margins computed from the nonlinear FE analyses and the ASME Code Case N-284-1 calculations for the airlocks and airlock hatches. The code case calculations should typically provide lower (conservative) estimates of the allowable buckling loads; however, in this instance, the opposite was the case.

During the February 14-17, 2011, onsite audit, the applicant provided preliminary results of its calculations to determine the sensitivity of the computed buckling loads to initial geometric imperfections. These calculations were performed to address the staff's concerns with the buckling analysis for the airlocks. For the airlock hatch, the magnitude of the assumed imperfections was based on standard dimensional tolerances; however, for the airlock cylinder, an arbitrary value was assumed. In both cases, the shape of the assumed imperfections was based on the buckling mode shape obtained from an eigenvalue analysis. The staff indicated that the approach was acceptable, but the magnitude of the assumed imperfections would need to be comparable to standard dimensional tolerances.

The applicant indicated that the justification for the magnitude of the geometric imperfection assumed in buckling analysis of the airlocks would be provided in its response to RAI 354, Question 03.08.02-13.

In a July 14, 2011, response to RAI 354, Question 03.08.02-13, the applicant formally documented the following information provided to the staff during the October 28-29, 2010, and February 14-17, 2011, onsite audits:

- Buckling is not a failure mechanism for the MS and FW penetrations, because their slenderness ratios (equivalent short columns under compression) are less than 89; therefore, yielding will occur before buckling.

- Due to its simple geometry, the buckling analysis for the construction opening was performed in accordance with ASME Code, Subarticle NE-3133. The allowable external

pressure is 545 kPaG(79 psig), which is 27 percent higher than the containment design pressure of 427 kPaG (62 psig). In accordance with the ASME Code, a reduction due to geometric imperfections is not required for this case.

- Nonlinear FE analyses for the equipment hatch were performed taking into account large deformations and bilinear material properties corresponding to accident temperature conditions, but without geometric imperfections. In accordance with ASME Code, Subarticle NE-3222.1(a)(1), the allowable external pressure was determined as the ultimate load from the FE analyses (1,773 kPaG [257 psig]) divided by a factor that depends on the service level. The calculated allowable external pressure for design and service Levels A and B (the most conservative) is 593 kPaG (86 psig), which is 38 percent higher than the containment design pressure of 427 kPaG (62 psig). Using the guidance in ASME Code Case N-284-1 and RG 1.193, the allowable external pressure for design and service Levels A and B is reduced to 552 kPaG (80 psig) due to the effect of geometric imperfections. This value is still 30 percent higher than the containment design pressure of 427 kPaG (62 psig).

- Nonlinear FE analyses for the airlock assemblies (cylinder, torispherical head, penetration sleeve, and hatch cover) were performed taking into account large deformations, bilinear material properties corresponding to accident temperature conditions, and initial geometric imperfections. In accordance with ASME Code, Subarticle NE-3222.1(a)(1), the allowable external pressure was determined as the ultimate load from the FE analyses (1,732 kPaG [251 psig] for the cylinder, torispherical head, and penetration sleeve assembly, and 1,649 kPaG [239 psig] for the hatch cover) divided by a factor that depends on the service level. The calculated allowable external pressure for design and service Levels A and B is 577 kPaG (84 psig) for the cylinder, torispherical head, and penetration sleeve assembly, and 550 kPaG (80 psig) for the hatch cover, which are 35 percent and 28 percent higher than the containment design pressure of 427 kPaG (62 psig). The shape of the geometric imperfections assumed in the analysis was based on the linear buckling mode shapes obtained from the reference configuration, while the magnitude of the imperfections was based on standard dimensional tolerances. For the case of the airlock cylinder, the magnitude of the imperfections was in the order of 0.5 mm (0.020 in.) deviation from the reference configuration (approximately 0.016 percent of the cylinder diameter). An additional analysis was performed in which the geometric imperfection was based on an ovalized cylindrical geometry with the difference between major and minor diameters set to 31 mm (1.220 in.) (approximately one percent of the cylinder diameter). This ovalized cylindrical geometry models the tolerances specified in ASME Code, Subarticle NE-4221.1. For this additional analysis, the allowable external pressure for design and service levels A and B is only reduced by three percent to 557 kPaG (81 psig). This value is still 31 percent higher than the containment design pressure of 427 kPaG (62 psig) and demonstrates that the design is relatively insensitive to initial geometric imperfections.

The staff finds the above information acceptable, because it meets the ASME Code, Subsection NE, and SRP Acceptance Criteria 3.8.2.II.4 and 3.8.2.II.5; in particular, SRP Acceptance Criteria 3.8.2.II.4.B.

Based on the above discussion, the staff considers RAI 155, Question 03.08.02-4 resolved, and **RAI 354, Question 03.08.02-13 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed changes.

In follow-up RAI 354, Question 03.08.02-15, the staff again requested that the applicant describe the loads that are imposed on the major steel RCB penetrations due to differential movement between the RCB and the RSB, and to explain how these loads are considered in the design of the penetrations.

In an August 31, 2010, response to RAI 354, Question 03.08.02-15, the applicant provided the following information: (1) The construction opening is not attached to the RSB so there are no loads imposed by differential movement; (2) the airlocks and equipment hatch assemblies contain flexible expansion joints that allow differential movement and minimize the load transfer between the RCB and RSB; (3) MS and FW penetration sleeves contain expansion bellows that allow differential movement and minimize the load transfer between the RCB and RSB; (4) the fuel transfer tube contains expansion bellows with finite stiffness that do provide a load path for forces associated with the relative movement of the RBIS, RCB, RSB, and FB; (5) the FE model of the fuel transfer tube used in design explicitly modeled the stiffness of these bellows so the induced forces due to relative movement are considered; and (6) revisions are proposed to the FSAR to incorporate this additional information. The staff finds this information acceptable, because it addresses the staff's concern regarding how the differential movement between the RCB and RSB was considered in the design-basis evaluations of the penetrations.

The staff also confirmed that the proposed changes were incorporated into FSAR Tier 2, Revision 2. Therefore, on the basis of the above discussion, the staff considers RAI 155, Question 03.08.02-8 and RAI 354, Question 03.08.02-15 resolved.

Structural Acceptance Criteria

To ensure that the steel portions of the RCB within the jurisdictional boundary of the ASME Code, Subsection NE, as designed, can perform their intended safety function, the staff verified that the structural design acceptance criteria meet the relevant requirements in 10 CFR 50.55a; GDC 1, GDC 2, GDC 4, GDC 16, GDC 50, GDC 53; and 10 CFR 50.44; and comply with SRP Acceptance Criteria 3.8.2.II.5.

The staff reviewed the acceptance limits for the steel portions of the RCB in terms of stresses, strains, deformations, factors of safety, and other criteria in FSAR Tier 2, Section 3.8.2.5, and concluded that they are in accordance with the requirements in ASME Code, Subsection NE and RG 1.57. These acceptance criteria apply to the steel portions of the RCB that are part of the pressure retaining function of the containment and are not backed by concrete. Steel portions of the RCB that are backed by concrete (including the containment steel liner and portions of the penetration sleeves or assemblies) are covered by FSAR Tier 2, Section 3.8.1.

The staff finds that the structural acceptance criteria presented in FSAR Tier 2, Section 3.8.2.5, applicable to the steel portions of the RCB within the jurisdictional boundary of the ASME Code, Subsection NE, comply with those identified in SRP Acceptance Criteria 3.8.2.II.5 and RG 1.57, and are therefore acceptable.

Materials, Quality Control, and Special Construction Techniques

The staff reviewed the materials, quality control, and special construction techniques used for the steel portions of the RCB within the jurisdictional boundary of the ASME Code, Subsection NE, to ensure that they meet the relevant requirements in 10 CFR 50.55a; GDC 1, GDC 2, GDC 4, GDC 16, GDC 50, GDC 53; and 10 CFR 50.44; and are in accordance with ASME Code, Articles NE-2000, NE-4000, NE-5000, and SRP Acceptance Criteria 3.8.2.II.6. With the exceptions described below, the staff finds that the materials, quality control, and

special construction techniques meet the regulatory requirements, regulatory guidance, and the ASME Code cited above.

FSAR Tier 2, Section 3.8.2.1.1, describes the equipment hatch and airlocks as having doors with sealed double gaskets. However, the technical basis for the qualification of these seals under the design-basis LOCA pressure and temperature conditions could not be identified. This information was needed to ensure that the equipment hatch and airlocks maintain a leak-tight boundary during the design-basis LOCA event. Therefore, in RAI 155, Question 03.08.02-10, the staff requested that the applicant address this issue.

In a May 29, 2009, response to RAI 155, Question 03.08.02-10, the applicant indicated that the equipment hatch and airlocks were considered vendor supplied items to be designed in the detailed design phase; therefore, detailed information on vendor qualification and testing of the seals could not be provided. The staff determined that the applicant's response did not comply with SRP Acceptance Criteria 3.8.2.II.1 and 3.8.2.II.6, which indicate that the FSAR should include sufficient information to define the primary structural aspects and elements that are relied upon to perform the containment function; in this case, leak-tightness. Therefore, in follow-up RAI 354, Question 03.08.02-16, the staff requested that the applicant provide design details and methods of qualification for the seals, including materials and detailed geometry, description of vendor qualification tests, and description of in-situ tests that would be conducted prior to operation.

In a July 29, 2010, response to RAI 354, Question 03.08.02-16, the applicant provided the following additional information: (1) A description of the elastomeric materials used in the seals, to comply with the guidance in NUREG/CR-5096, as well as a description of the seal configuration; (2) a description of vendor qualification tests to provide assurance of seal performance for normal operating conditions and for temperature and pressure conditions associated with a LOCA; (3) a description of in-situ tests that would be conducted, after installation in the containment, to provide assurance of seal performance for normal operating conditions and for temperature and pressure conditions associated with a LOCA; and (4) proposed revisions to the FSAR to incorporate this additional information.

The staff finds the above information acceptable, because it complies with the recommendations in NUREG/CR-5096, which reports testing of gasket seals for severe accident conditions, significantly more severe than the design-basis LOCA pressure and temperature conditions considered for the U.S. EPR. The staff also verified that the proposed changes were incorporated into FSAR Tier 2, Revision 2. Accordingly, the staff considers RAI 155, Question 03.08.02-10 and RAI 354, Question 03.08.02-16 resolved.

In FSAR Tier 2, Section 3.8.2.6, Revision 0, the applicant stated that steel portions of the RCB within the jurisdictional boundary of ASME Code, Subsection NE, are fabricated from materials that meet the requirements specified in ASME Code Article NE-2000. However, the applicant did not identify the specific materials used in the fabrication of the major steel RCB penetrations.

As indicated above in "Design and Analysis Procedures," the staff was concerned that the applicant had not finalized the design of the major steel RCB penetrations. The staff could not draw a safety conclusion on these safety-critical elements based on incomplete designs. Therefore, in RAI 155, Questions 03.08.02-1 and 03.08.02-2, and RAI 354, Question 03.08.02-11 (follow-up to RAI 155, Questions 03.08.02-1 and 03.08.02-2), the staff requested that the applicant provide a list of the specific materials used for the major steel RCB penetrations, as well as the extent of compliance with ASME Code Article NE-2000.

In a June 30, 2009, response to RAI 155, Question 03.08.02-1, a July 31, 2009, response to RAI 155, Question 03.08.02-2, and an April 18, 2011, response to RAI 354, Question 03.08.02-11, the applicant provided the requested information which the staff finds acceptable. The staff considers RAI 155, Questions 03.08.02-1 and 03.08.02-2 resolved. **RAI 354, Question 03.08.02-11 is being tracked as a confirmatory item.** The resolution of these RAIs is discussed above in "Design and Analysis Procedures."

Testing and Inservice Surveillance Requirements

To ensure that the steel portions of the RCB within the jurisdictional boundary of the ASME Code, Subsection NE, can perform their intended safety function, the staff verified that the testing and ISI surveillance requirements meet the relevant requirements in 10 CFR 50.55a; GDC 1, GDC 2, GDC 4, GDC 16, GDC 50, GDC 53; and are in accordance with ASME Code, Article NE-6000 and Subsection IWE, and SRP Acceptance Criteria 3.8.2.II.7. With the exception described below, the staff finds that the testing and ISI surveillance requirements meet the regulatory requirements, regulatory guidance, and the ASME Code cited above.

10 CFR 50.55a requires the ISI surveillance be conducted in accordance with ASME Code Subsection IWE. However, FSAR Tier 2, Section 3.8.2.7, did not reference this requirement. Therefore, in RAI 155, Question 03.08.02-7, the staff requested that the applicant identify how it would meet the requirements in ASME Code, Subsection IWE and the supplemental requirements of 10 CFR 50.55a.

In a July 31, 2009, response to RAI 155, Question 03.08.02-7, the applicant indicated that FSAR Tier 2, Section 3.8.2.7 would be revised to specifically reference ASME Code, Subsection IWE requirements, including the supplemental requirements of 10 CFR 50.55a. The staff finds this information acceptable, because it complies with SRP Acceptance Criteria 3.8.2.II.7. The staff also verified that the proposed changes were incorporated into FSAR Tier 2, Revision 2. Accordingly, the staff considers RAI 155, Question 03.08.02-7 resolved.

ITAAC, Technical Specifications, and Site Parameters

ITAAC: The staff has reviewed the ITAAC associated with FSAR Tier 2, Section 3.8.2 in FSAR Tier 1, Table 2.1.1-8 and finds them acceptable for this area of review.

Technical Specifications: The staff has reviewed the Technical Specifications associated with FSAR Tier 2, Section 3.8.2 in FSAR Tier 2, Chapter 16, Section 3.6, specifically, Section 3.6.2, and finds them acceptable for this area of review.

Site Parameter: The staff has reviewed the following site parameter that will be addressed in COL designs: FSAR Tier 2, Table 1.8-1, Item No. 3-4, Site-specific loads that lie within the standard plant design envelope for Seismic Category I structures, and finds it acceptable for this area of review.

3.8.2.5 *Combined License Information Items*

There are no combined license information items related to this area of review. The staff determined that no COL information items need to be included in FSAR Tier 2, Table 1.8-2, "U.S. EPR Combined License Information Items," for steel containment consideration.

3.8.2.6 *Conclusions*

The staff reviewed FSAR Tier 2, Section 3.8.2, to determine whether the design, fabrication, construction, testing, and ISI of the steel portions of the RCB that are not backed by concrete, and are intended to resist pressure, comply with 10 CFR 50.55a; GDC 1, GDC 2, GDC 4, GDC16, GDC 50, GDC 53, and 10 CFR 50.44. The staff concludes that, pending revision of the FSAR to incorporate the proposed changes in the confirmatory items described above, the applicant meets the relevant requirements of 10 CFR 50.55a; GDC 1, GDC 2, GDC 4, GDC 16, GDC 50, GDC 53, and 10 CFR 50.44. The basis for this conclusion is the following:

The applicant meets the requirements of 10 CFR 50.55a and GDC 1 to ensure that the steel portions of the RCB that are not backed by concrete are designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety function to be performed, by meeting the guidelines in SRP Section 3.8.2; RG 1.7 and 1.57; and ASME Code, Section III, Division 1, Subsection NE, and Section XI, Subsection IWE.

The applicant meets the requirements of GDC 2 to ensure that the steel portions of the RCB that are not backed by concrete are able to withstand the most severe natural phenomena, by analyzing and designing these items to withstand the most severe earthquake that has been established for the U.S. EPR certified design, in load combinations that include earthquake loading. Protection from other natural phenomena such as winds, tornadoes, and floods is provided by other Seismic Category I structures that shield the RCB.

The applicant meets the requirements of GDC 4 to ensure that the steel portions of the RCB that are not backed by concrete are appropriately protected against other dynamic effects, by analyzing and designing these items, where applicable, to withstand the dynamic effects due to pipe whipping, missiles, and discharging fluids associated with the LOCA.

The applicant meets the requirements of GDC 16 to ensure that the steel portions of the RCB that are not backed by concrete act as a leak-tight barrrier to prevent the uncontrolled release of radioactivity to the environment, by meeting the design, construction, testing, and ISI guidelines in SRP Section 3.8.2; RGs 1.7 and 1.57; and ASME Code, Section III, Division 1, Subsection NE, and Section XI, Subsection IWE. Furthermore, the FSAR also indicates that throughout its service life, the steel portions of the RCB are monitored periodically in accordance with 10 CFR 50.55a, which provides supplemental requirements to the ASME Code, Section XI, Subsection IWE, and in accordance with 10 CFR Part 50, Appendix J, for containment leakage testing.

The applicant meets the requirements of GDC 50 to ensure that the steel portions of the RCB that are not backed by concrete are designed with sufficient margin of safety to accommodate appropriate design loads, by meeting the design guidelines in SRP Section 3.8.2; RGs 1.7 and 1.57; and ASME Code, Section III, Division 1, Subsection NE.

The applicant meets the requirements of GDC 53 to ensure that the steel portions of the RCB that are not backed by concrete are designed to permit appropriate ISI and pressure testing of penetrations, by meeting the guidelines in SRP Section 3.8.2; ASME Code, Section III, Division 1, Subsection NE; and ASME Code, Section XI, Subsection IWE.

The applicant meets the requirements of 10 CFR 50.44 to ensure the structural integrity of the steel portions of the RCB that are not backed by concrete, for loads associated with combustible gas generation, by meeting the guidelines in SRP Section 3.8.2 and RG 1.7, RG 1.57, and RG 1.216.

As mentioned above, the acceptability of the design, fabrication, construction, testing and ISI of the steel portions of the RCB that are not backed by concrete is predicated on the adequate revision of the FSAR to incorporate the proposed changes for the confirmatory items discussed in this section.

3.8.3 Concrete and Steel Internal Structures of Concrete Containment

3.8.3.1 *Introduction*

FSAR Tier 2, Section 3.8.3, “Concrete and Steel Internal Structures of Concrete Containment,” describes the RBIS, which are Seismic Category I structures that provide support for the reactor vessel (RV), reactor coolant system, pressurizer (PZR), steam generators (SGs), and other components housed within the RCB. The RBIS also provide radiation shielding for the RCS, as well as shielding during refueling operations.

3.8.3.2 *Summary of Application*

FSAR Tier 1: The FSAR Tier 1 information associated with this section is found in FSAR Tier 1, Section 2.1.1, “Nuclear Island.” This section requires, in part, that the NI structural supports, including critical sections, be Seismic Category I and constructed to withstand design basis loads without loss of structural integrity and safety-related functions.

FSAR Tier 2: The applicant has provided an FSAR Tier 2 description of the RBIS in Section 3.8.3, summarized as follows:

<u>Description of Concrete and Steel Internal Structures of Concrete Containment</u>

FSAR Tier 2, Section 3.8.3.1, “Description of the Internal Structures,” describes the concrete and steel internal structures located within the RCB. The internal structures include reinforced concrete walls and floors, steel framing members, and other concrete and structural steel elements that provide support for components and radiation shielding. The foundation basemat, inside the RCB, provides support for the internal structures. Clearance is maintained between the containment wall and the internal structures to preclude interaction under design basis loading conditions (e.g., safe-shutdown earthquake).

The internal structures support the nuclear steam supply system (NSSS) components, provide access for plant operation and maintenance, and support safety-related functions of the plant. The major internal structures are:

- RV support structure and reactor cavity
- SG support structures
- Reactor coolant pump (RCP) support structures
- PZR support structure
- Operating floor and intermediate floors
- Secondary shield walls

- Refueling canal walls
- Polar crane support structure
- Internal structures basemat
- In-containment refueling water storage tank (IRWST)
- Core melt retention area
- Convection and Rupture Foils
- Reactor Containment Building Doors

FSAR Tier 2, Sections 3.8.3.1.1, "Reactor Vessel Support Structure and Reactor Cavity"; 3.8.3.1.2, "Steam Generator Support Structures"; 3.8.3.1.3, "Reactor Coolant Pump Support Structures"; 3.8.3.1.4, "Pressurizer Support Structure"; 3.8.3.1.5, "Operating Floor and Intermediate Floors"; 3.8.3.1.6, "Secondary Shield Walls"; 3.8.3.1.7, "Refueling Canal Walls"; 3.8.3.1.8, "Polar Crane Support Structure"; 3.8.3.1.9, "Reactor Building Internal Structures Basemat, In-Containment Refueling Water Storage Tank, and Core Melt Retention Area"; 3.8.3.1.10, "Distribution System Supports"; 3.8.3.1.11, "Platforms and Miscellaneous Structures," 3.8.3.1.12, "Reactor Containment Building Rupture and Convection Foils"; and 3.8.3.1.13, "Reactor Containment Building Doors;" provide details for each of the major internal structures. FSAR Tier 2, Section 5.4, "Component and Subsystem Design," further describes the steel supports for the RV, four SGs, four RCPs, and the PZR. The RBIS also include pipe supports, equipment supports, cable tray and conduit supports, and HVAC duct supports. Platforms, ladders, stairs, guard rails, and other miscellaneous structures are provided inside the RCB for equipment access and maintenance.

FSAR Tier 2, Section 3.2, "Classification of Structures, Systems, and Components," specifies the seismic classification of SSCs. The containment internal structures are Seismic Category I, except for miscellaneous structures such as platforms, stairs, guard rails, and other ancillary items. The miscellaneous structures are designated as Seismic Category II, and are designed to ensure that, in the event of an SSE, there is no impairment of the intended function of safety-related SSCs.

<u>Applicable Codes, Standards, and Specifications</u>

FSAR Tier 2, Section 3.8.3.2, "Applicable Codes, Standards, and Specifications," identifies the codes, standards, specifications, design criteria, regulations, and regulatory guides applicable to the design, fabrication, construction, testing, and ISI of concrete and steel internal structures.

FSAR Tier 2, Sections 3.8.3.2.1, "Codes and Standards," and 3.8.3.2.2, "Specifications," address codes, standards, and specifications. Twenty-two codes and standards are given. The two key standards are ACI 349-01 for reinforced concrete structures and the 1994 Edition of ANSI/AISC N690, "Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures for Nuclear Facilities," including Supplement 2 issued in 2004 (hereafter referred to as ANSI/AISC N690-1994, including S2004), for structural steel. The remaining documents supplement these key standards.

The specifications address details related to the design and construction of the RBIS, including the following: Concrete material properties; mixing, placing, and curing of concrete; reinforcing steel and splices; structural steel; stainless steel liner plate and embedments; miscellaneous and embedded steel; anchor bolts; expansion anchors; polar crane; and miscellaneous cranes and hoists.

FSAR Tier 2, Section 3.8.3.2.3, "Design Criteria," identifies the following standards for the design criteria applicable to RBIS: ACI 349-01, ACI 349-06, and ANSI/AISC N690-1994, including S2004.

FSAR Tier 2, Section 3.8.3.2.4, "Regulations," identifies the following regulations applicable to RBIS: 10 CFR Part 50, Appendix A, "General Design Criteria for Nuclear Power Plants"; GDC 1, "Quality Standards and Records"; GDC 2, "Design Bases for Protection Against Natural Phenomena"; GDC 4, "Environmental and Dynamic Effects Design Bases"; GDC 5, "Sharing of Structures, Systems, and Components"; GDC 50, "Containment Design Basis"; 10 CFR Part 50, Appendix B, "Quality Assurance Criteria for Nuclear Power Plants and Fuel Reprocessing Plants"; and 10 CFR Part 50, Appendix S, "Earthquake Engineering Criteria for Nuclear Power Plants."

FSAR Tier 2, Section 3.8.3.2.5, "NRC Regulatory Guides," identifies the following Regulatory Guides applicable for the design and construction of RBIS: RG 1.61, "Damping Values for Seismic Design of Nuclear Power Plants," March 2007, Revision 1 (with a noted exception); RG 1.69, "Concrete Radiation Shields for Nuclear Power Plants," December 1973; RG 1.136, "Design Limits, Loading Combinations, Materials, Construction, and Testing of Concrete Containments," March 2007, Revision 3, (with a noted exception); RG 1.142, "Safety-Related Concrete Structures for Nuclear Power Plants (Other than Reactor Vessels and Containments)," November 2001, Revision 2 (with a noted exception); RG 1.160, "Monitoring the Effectiveness of Maintenance at Nuclear Power Plants," March 1997, Revision 2; and RG 1.199, "Anchoring Components and Structural Supports in Concrete," November 2003 (with a noted exception).

Loads and Load Combinations

FSAR Tier 2, Section 3.8.3.3, "Loads and Load Combinations," describes the loads and load combinations for the RBIS. The FSAR indicates that the loads are classified as normal loads, severe environmental loads, extreme environmental loads, or abnormal loads. It is necessary for a COL applicant referencing the U.S. EPR design certification to confirm that the site-specific loads on the RBIS are enveloped by the loads assumed for design certification. If this is not the case, then it is necessary for the COL applicant to perform additional analyses to verify structural adequacy of the RBIS.

FSAR Tier 2, Section 3.8.3.3.1, "Design Loads," describes the individual loads that are classified as normal loads, severe environmental loads, extreme environmental loads, or abnormal loads. The FSAR indicates that loads on RBIS are defined in conformance with ACI 349-01 and the guidelines of RG 1.142 for concrete structures, and ANSI/AISC N690-1994, including S2004, for steel structures.

FSAR Tier 2, Section 3.8.3.3.2, "Load Combinations," identifies the specific load combinations for the RBIS. The FSAR indicates that load combinations for design of RBIS are defined to conform to ACI 349-01 and guidelines of RG 1.142 for concrete structures, and ANSI/AISC N690-1994, including S2004, for steel structures.

FSAR Tier 2, Section 3.8.3.3.2, also discusses the NI common basemat structures. Various portions of the NI basemat have different classifications (i.e., RCB, RBIS, or other Seismic Category I structures) and different design requirements based on the corresponding applicable codes. Additional loads due to structure-to-structure interaction (e.g., post tension loads and buoyant loads) are defined for the RBIS basemat.

Design and Analysis Procedures

FSAR Tier 2, Section 3.8.3.4, "Design and Analysis Procedures," describes the design and analysis procedures for the RBIS. The FSAR indicates that Seismic Category I concrete structures are designed according to ACI 349-01 and its appendices, considering the exceptions delineated in RG 1.142. The FSAR further indicates that Seismic Category I steel structures are designed according to the criteria of ANSI/AISC N690-1994, including S2004; and that the design of concrete embedments and anchors conforms to ACI 349-06, Appendix D, and the guidelines of RG 1.199, with the exceptions identified in FSAR Tier 2, Section 3.8.1, "Concrete Containment."

The RBIS are analyzed using both computer modeling and classical solutions for the loads and load combinations described in FSAR Tier 2, Section 3.8.3.3. The overall computer model of the NI structures includes the RBIS. Local analyses of specific structural walls, slabs, and members are also performed to account for local effects of equipment loads, pipe break loads, hydrostatic and hydrodynamic loads, and other conditions. The global response from the overall computer model is combined with the results of the local analyses to obtain the total loads for detailed design.

FSAR Tier 2, Sections 3.8.3.4.1, "Overall Analysis and Design Procedures," 3.8.3.4.2, "Local Analysis and Design," 3.8.3.4.3, "Static Analysis and Design," 3.8.3.4.4, "Seismic and Other Dynamic Analyses and Design," and 3.8.3.4.5, "Design Report," provide details of the design and analysis procedures.

FSAR Tier 2, Appendix 3E, "Design Details and Critical Sections for Safety-Related Category I Structures," provides further details for the analysis and design of the selected critical sections of the RBIS.

Structural Acceptance Criteria

FSAR Tier 2, Section 3.8.3.5, "Structural Acceptance Criteria," indicates that the acceptance limits for reinforced concrete internal structures, in terms of allowable stresses, strains, deformations, and other design criteria, are in accordance with ACI 349-01, and its appendices, including the exceptions specified in RG 1.142. Acceptance limits for concrete embedments and anchors are in accordance with ACI 349-06, Appendix D, and guidance given in RG 1.199, with noted exceptions. Acceptance limits for structural steel internal structures are in accordance with ANSI/AISC N690-1994, including S2004.

The FSAR indicates that load combinations including SSE seismic loads generally control the design of internal structures. FSAR Tier 2, Appendix 3E provides design results for the critical sections of the internal structures.

The FSAR further indicates that deviations from the approved design will be summarized in an as-built report, and it will be confirmed by COL applicants that the as-built internal structures are capable of withstanding the design basis loads described in FSAR Tier 2, Section 3.8.3.3.

Materials, Quality Control, and Special Construction Techniques

FSAR Tier 2, Section 3.8.3.6, “Materials, Quality Control, and Special Construction Techniques,” describes the materials, quality control, and special construction techniques applicable to the U.S. EPR.

FSAR Tier 2, Section 3.8.3.6.1, “Concrete Materials,” indicates that concrete materials for the RBIS conform to ACI 349-01, Chapter 5, supplemented by RG 1.142, and the 2005 Edition of ACI 301, “Specifications for Structural Concrete for Buildings.” In addition, where used for radiation shielding, the concrete conforms to RG 1.69. The concrete minimum compressive strength (f'c) is equal to 41.4 MPa (6,000 psi). The FSAR describes the concrete mix design and standards followed for the cement, aggregates, admixtures, water, and placement of the concrete.

FSAR Tier 2, Section 3.8.3.6.2, “Reinforcing Steel and Splice Materials,” describes the reinforcing steel and splice materials. Reinforcing steel materials for the RBIS conform to ACI 349-01. Fabrication and placement of reinforcing bars for RBIS is in accordance with ACI 349-01, Chapter 7. Specific standards covering welded and mechanical splices are also identified.

FSAR Tier 2, Section 3.8.3.6.3, “Structural Steel,” identifies that the structural steel materials for the RBIS conform to ANSI/AISC N690-1994, including S2004, and the 2000 Edition of AISC 303, “Code of Standard Practice for Steel Buildings and Bridges.” The FSAR identifies the specific standards that cover materials for steel members, bolting, and welding; and the specific standards that cover fabrication and erection.

FSAR Tier 2, Section 3.8.3.6.4, “Quality Control,” refers to FSAR Tier 2, Chapter 17, “Quality Assurance and Reliability Assurance,” for a description of the quality assurance program for the U.S. EPR.

FSAR Tier 2, Section 3.8.3.6.5, “Special Construction Techniques,” indicates that construction of RBIS uses proven methods, and does not rely on unique construction techniques. Well-established modular construction methods are used for prefabricating portions of the IRWST liner, refueling canal liner, reinforcing, concrete formwork, and other portions of the RBIS.

Testing and Inservice Surveillance Requirements

FSAR Tier 2, Section 3.8.3.7, “Testing and Inservice Inspection Requirements,” summarizes the testing and ISI requirements for the RBIS. Monitoring and maintenance of the RBIS is conducted in accordance with 10 CFR 50.65, “Requirements for monitoring the effectiveness of maintenance at nuclear power plants,” as supplemented by the guidance in RG 1.160.

Specific requirements for the RCS component supports and the polar crane are addressed in FSAR Tier 2, Section 5.4.1.5, “Inspection and Testing Requirements,” and FSAR Tier 2, Section 9.1.5, “Overhead Heavy Load Handling System,” respectively.

The FSAR indicates that physical access is provided to perform ISI of the RBIS; and that space is provided between the containment liner and concrete RBIS to allow for inspection of the liner at wall and floor locations.

ITAAC: The ITAAC associated with FSAR Tier 2, Section 3.8.3 are given in FSAR Tier 1, Table 2.1.1-8, “Reactor Building ITAAC.”

Site Parameter: This section of the FSAR contains information related to the following site parameter that will be addressed in COL designs: Item No. 3-4, Site-specific loads that lie within the standard plant design envelope for Seismic Category I structures, in FSAR Tier 2, Table 1.8-1, “Summary of U.S. EPR Plant Interfaces with Remainder of Plant,”

3.8.3.3 *Regulatory Basis*

The relevant requirements of NRC regulations for this area, and the associated acceptance criteria, are given in NUREG-0800, Section 3.8.3, “Concrete and Steel Internal Structures of Steel or Concrete Containments,” and are summarized below. NUREG-0800, Section 3.8.3 also identifies review interfaces with other SRP sections.

1. GDC 1, “Quality Standards and Records,” as it relates to the RBIS being designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety function to be performed.

2. GDC 2, “Design Bases for Protection against Natural Phenomena,” as it relates to the capability of the containment internal structures to perform their safety function, to withstand the effects of natural phenomena, such as earthquakes, tornadoes, floods, and the appropriate combination of all loads.

3. GDC 4, “Environmental and Dynamic Effects Design Bases,” as it relates to the protection of containment internal structures against dynamic effects, including the effects of missiles, pipe whipping, and discharging fluids, that may result from equipment failures and from events and conditions outside the nuclear power unit.

4. GDC 5, “Sharing of Structures, Systems, and Components,” as it relates to safety-related structures not being shared among nuclear power units, unless it can be shown that such sharing will not significantly impair their ability to perform their safety functions.

5. GDC 50, “Containment Design Basis,” as it relates to the design of containment internal structures with sufficient margin of safety to accommodate appropriate design loads.

6. 10 CFR Part 50, “Domestic Licensing of Production and Utilization Facilities,” specifically, 10 CFR 50.55a, “Codes and Standards,” as they relate to codes and standards

7. 10 CFR Part 50, Appendix B, “Quality Assurance Criteria for Nuclear Power Plants and Fuel Processing Plants,” as it relates to the quality assurance criteria for nuclear power plants.

Acceptance criteria adequate to meet the above requirements include:

1. RG 1.57, “Design Limits and Loading Combinations for Metal Primary Reactor Containment,” as it relates to the design requirements for steel portions of concrete containments that are intended to resist pressure, but are not backed by structural concrete.

2. RG 1.69, “Concrete Radiation Shields for Nuclear Power Plants,” as it relates to the design requirements for concrete radiation shields in nuclear power palnts.

3. RG 1.136, “Design Limits, Loading Combinations, Materials, Construction, and Testing of Concrete Containments,” as it relates to the design requirements for concrete containments.

4. RG 1.142, “Safety-Related Concrete Structures for Nuclear Power Plants (Other Than Reactor Vessels and Containments),” as it relates to the design requirements for safety-related concrete structures, excluding concrete containments.

5. RG 1.143, “Design Guidance for Radioactive Waste Management Systems, Structures, and Components Installed in LWR Plants,” as it relates to the design requirements for radioactive waste management structures.

6. RG 1.160, “Monitoring the Effectiveness of Maintenance at Nuclear Power Plants,” as it relates to the monitoring and maintenance of safety-related structures.

7. RG 1.199, “Anchoring Components and Structural Supports in Concrete,” as it relates to the design, evaluation, and quality assurance of anchors (steel embedments) used for component and structural supports on concrete structiures.

3.8.3.4 *Technical Evaluation*

The staff's evaluation is based on a complete review of FSAR Tier 2, Section 3.8.3.1 through Section 3.8.3.7, including all their subsections.

This section describes the staff's review and evaluation of FSAR Tier 2, Section 3.8.3, against the guidance of SRP Section 3.8.3 to ensure that the applicant has adequately addressed identified technical issues.

<u>Description of the Internal Structures</u>

The staff reviewed the descriptions of the structures inside the containment to ensure that they contain sufficient information to define the primary structural aspects and elements that are relied upon to perform the safety-related functions of these structures. To perform safety-related functions, these structures must be capable of resisting loads and load combinations to which they may be subjected and should not become the initiator of a LOCA. If a LOCA were to occur, the structures should be able to mitigate its consequences by protecting the containment and other engineered safety features from effects such as jet forces and whipping pipes.

With the exception described below, the staff finds that the descriptions meet the applicable regulatory requirements and the regulatory guidance in SRP Acceptance Criteria 3.8.3.II.1.

The staff's review identified that insufficient information was provided in FSAR Tier 2, Section 3.8.3.1, to describe the design details of the structural elements, connections, and load path for certain components of the RBIS, including the supports of the RV, SGs, RCPs, and PZRs, as well as the polar crane support brackets. This information was needed to establish the primary structural elements that are relied upon to transfer loads and to perform their intended safety functions. Therefore, in RAI 155, Question 03.08.03-1, the staff requested that the applicant provide additional design details for the items discussed above.

In a March 31, 2009, response to RAI 155, Question 03.08.03-1, the applicant provided descriptive information and figures for the RV supports, including RV ring plate, anchorage details, lateral restraint, cavity seal ring, and neutron shield; SG anchorage and lateral restraints; RCPs anchorage; PZR anchorage and lateral restraints; and polar crane support brackets.

This information addressed the staff's concerns, because it fully described the structural elements and connections, and allowed for the identification of a complete load path, in accordance with SRP Acceptance Criteria 3.8.3.II.1. Therefore, the staff finds the applicant's March 31, 2009, response to RAI 155, Question 03.08.03-1 acceptable and considers RAI 155, Question 03.08.03-1 resolved.

Applicable Codes, Standards, and Specifications

To ensure that the containment internal structures can perform their intended safety function, the staff verified that the applicable codes, standards, and specifications used in the design, fabrication, construction, testing, and ISI meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; GDC 1, GDC 2, GDC 4, GDC 5, and GDC 50; and comply with SRP Acceptance Criteria 3.8.3.II.2. With the exceptions described below, the staff finds that the applicable codes, standards, and specifications meet the regulatory requirements and regulatory guidance cited above.

The staff's review of FSAR Tier 2, Section 3.8.3, could not conclude whether the design and installation of concrete anchors were in accordance with (1) ACI 349-01, Appendix B, subject to the conditions and limitations specified in RG 1.199; and (2) NRC document IE Bulletin 79-02, "Pipe Support Base Plate Designs Using Concrete Expansion Anchor Bolts," Revision 2; or comply with SRP Acceptance Criteria 3.8.3.II.2. The staff was concerned whether the applicant's criteria resulted in the adequate design of structural connections that utilize these anchors, with the appropriate safety margins. Therefore, in RAI 155, Question 03.08.03-2, the staff requested that the applicant explain the criteria used for design and installation of concrete anchors.

The above technical issue was also identified in the review of FSAR Tier 2, Sections 3.8.1, 3.8.4, "Other Seismic Category I Structures," and 3.8.5, "Foundations."

In a March 31, 2009, response to RAI 155, Question 03.08.03-2, the applicant indicated that ACI 349-06, Appendix B was being followed with the limitations described in RG 1.199. The response also indicated that the issues raised in NRC IE Bulletin 79-02, specifically the criteria for anchor bolts factors of safety and base plate flexibility under prying, were adequately addressed in ACI 349-01 and subsequent editions including ACI 349-06. However, the staff noted that ACI 349-01 was still referenced in FSAR Tier 2, Revision 1, Section 3.8.4.5, "Structural Acceptance Criteria," and that ACI 349-06, Appendix D (not Appendix B) provides the design for concrete anchors. Therefore, in RAI 283, Question 03.08.03-20, the staff requested that the applicant clarify which edition of ACI 349 was being followed. If ACI 349-06

was intended, the applicant was further requested to confirm that the provisions of ACI 349-06 that are used for concrete anchor design and installations are in conformance with ACI 349-01 and RG 1.199.

In a November 12, 2009, response to RAI 283, Question 03.08.03-20, the applicant clarified that ACI 349-06, Appendix D, is used for the design and installation of concrete anchors with the exceptions noted in RG 1.199. The use of ACI 349-06 was introduced in the May 29, 2009, response to RAI 155, Question 03.08.03-3 (discussed below), and was reconciled with ACI 349-01 and ACI 349-97 in a proprietary document. The staff confirmed this reconciliation during the April 26-30, 2010, onsite audit and determined the consistency of provisions between ACI 349-06, Appendix D, and ACI 349-01, Appendix B, provided that the strength reduction (phi) factors and load combinations used with ACI 349-06, Appendix D, are consistent with those in ACI 349-01. The July 26, 2010, response to RAI 376, Question 03.08.03-21 (discussed below) clarified that the strength reduction (phi) factors and load combinations used in the design of concrete anchors under ACI 349-06, Appendix D, are consistent with those in ACI 349-01.

The staff finds the use of ACI 349-06, Appendix D, for the design and installation of concrete anchors acceptable because (1) the strength reduction factors and load combinations used in the design are consistent with ACI 349-01 and SRP Acceptance Criteria 3.8.3.II.2, and therefore acceptable; and (2) the conditions and limitations specified in RG 1.199 were accounted for appropriately. Regarding NRC IE Bulletin 79-02, the staff did not agree that conformance to ACI 349 and RG 1.199 automatically implies conformance with the guidance provided by NRC IE Bulletin 79-02. However, the staff acknowledged that ACI 349 and RG 1.199 capture the important provisions in the IE bulletin applicable for design certification; for example, the criteria for anchor bolt safety factors and base plate flexibility under prying. Therefore, the staff finds this item of the response acceptable.

The staff verified that the changes proposed by the applicant were incorporated into FSAR Tier 2, Revision 2. Accordingly, based on the above discussion, the staff considers RAI 155, Question 03.08.03-2 and RAI 283, Question 03.08.03-20 resolved.

In FSAR Tier 2, Section 3.8.3.2, the applicant identified ACI 349-01 as the code used for the design and construction of reinforced concrete structures inside the containment. However, RG 1.142 endorses ACI 349-97, with certain regulatory positions. Since ACI 349-01 is not endorsed by RG 1.142, the staff was concerned with the use of a later edition of ACI 349 that has not been reviewed for applicability to nuclear power plants. To ensure the safety of the reinforced concrete structures, the staff would need to review the applicability of ACI 349-01 on a case-by-case basis. Therefore, in RAI 155, Question 03.08.03-3, the staff requested that the applicant: (1) Identify the differences between ACI 349-01 and ACI 349-97; (2) identify which differences were relaxed provisions; (3) provide the technical basis for using any relaxed provisions; and (4) provide the technical basis for using an isolated provision from ACI 349-06: the shear strength reduction factor of 0.85, which is different from both ACI 349-01 and ACI 349-97.

The review of FSAR Tier 2, Section 3.8.4.4.1, “General Procedures Applicable to Other Seismic Category I Structures,” and Section 3.8.5.4, “Design and Analysis Procedures,” also identified ACI 349-01 as the code used for the design and construction of reinforced concrete structures and foundations outside the containment.

In May 29, 2009, response to RAI 155, Question 03.08.03-3, the applicant addressed the staff issues as follows:

The most significant differences between ACI 349-01 and ACI 349-97 were identified.

1. The applicant explained that design approaches and methodologies in ACI 349-01 remain essentially unchanged with respect to ACI 349-97, with the exception of code-based design of sway frames.

2. The applicant provided a detailed technical explanation of the code-based design of sway frames used in ACI 349-01, which demonstrated that the latter methodology is more realistic relative to the reduction factor method for long columns specified in ACI 349-97.

3. The shear strength reduction allowed in ACI 349-06 is based on the assumption that expected ductility demands for safety-related nuclear structures are low compared to conventional building structures.

The staff finds the applicant's response to Items (1), (2), and (3) above technically acceptable. However, the response to Item (4) was a qualitative assumption without numerical evidence, which did not provide sufficient technical justification. In addition, the staff concluded that using isolated provisions (i.e., reduced shear strength reduction factor) of one code (ACI 349-06) together with the remaining provisions of another code (ACI 349-01) was not recommended, since it could result in undesirable inconsistencies in the design. It was noted that there may be other provisions in ACI 349-06 which could have an impact on the design but would not be utilized following the approach described by the applicant. Therefore, in follow-up RAI 376, Question 03.08.03-21, the staff requested that the applicant identify one version of the ACI 349 standard and to follow the provisions of that code for the design and construction of reinforced concrete structures inside and outside the containment, without making selected exceptions.

In a July 26, 2010, response to RAI 376, Question 03.08.03-21, the applicant indicated that ACI 349-01 is utilized in the design and construction of reinforced concrete structures inside and outside of the containment with the only exception being the design and installation of concrete anchors, which will follow ACI 349-06, Appendix D, as described in the March 31, 2009, response to RAI 155, Question 03.08.03-2 (discussed above), and the November 12, 2009, response to RAI 283, Question 03.08.03-20 (discussed above). The applicant stated that the regulatory positions of RG 1.142 are followed as applicable.

The staff accepted the use of ACI 349-06, Appendix D for the design and installation of concrete anchors in its evaluation of RAI 155, Question 03.08.03-2 and RAI 283, Question 03.08.03-20. The concern regarding the use of isolated provisions of one code (ACI 349-06) together with the remaining provisions of another code (ACI 349-01) is not applicable to ACI 349-06, Appendix D because this appendix, as used by the applicant, is essentially self-contained.

The applicant's July 26, 2010, response to RAI 376, Question 03.08.03-21 also explained that NI structures have limited number of interior columns that are laterally braced by the NI shear walls; therefore, for NI structures, column design does not rely on the revised sway-frame design methodology in ACI 349-01. Seismic Category I structures outside of the NI, however, may be designed using the revised sway frame design methodology in ACI 349-01.

The applicant's July 26, 2010, response to RAI 376, Question 03.08.03-21 also addressed the staff's concerns regarding the use of ACI 349-01 for the design of concrete structures inside and outside of the containment because, as indicated above, it is consistent with the staff review of other NPP certified designs that have utilized ACI 349-01 after case-by-case evaluations by the staff, and also because the FSAR explicitly references the regulatory positions of RG 1.142.

In addition, during the April 26-30, 2010, onsite audit, the staff confirmed the line-by-line comparison between ACI 349-01 and ACI 349-97 performed by the applicant in a proprietary document to ensure consistency of applicable provisions between the two revisions.

The staff verified that the changes proposed by the applicant were incorporated into FSAR Tier 2, Revision 2. Therefore, based on the above discussion, the staff considers RAI 155, Question 03.08.03-3 and RAI 376, Question 03.08.03-21 resolved.

Loads and Load Combinations

To ensure that the containment internal structures can perform their intended safety function, the staff verified that the loads and load combinations used in the design meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; GDC 1, GDC 2, GDC 4, GDC 5, and GDC 50; and are in accordance with SRP Acceptance Criteria 3.8.3.II.3. With the exceptions described below, the staff found that the loads and load combinations meet the regulatory requirements and regulatory guidance cited above.

In FSAR Tier 2, Section 3.8.3.3.2, the applicant described the load combinations used for concrete and steel structures inside the containment. The staff reviewed this section against SRP Acceptance Criteria 3.8.3.II.3 and identified the following inconsistencies: (1) For reinforced concrete elements, pipe reaction loads (R_o) were not included as required by ACI 349-01; (2) deviations from ANSI/AISC N690-1994, including S2004, related to the plastic section modulus of steel shapes were noted; (3) it was unclear to the staff whether all footnotes to ANSI/AISC N690-1994, including S2004, Table Q1.5.7.1, were applicable to the FSAR; and (4) it was unclear to the staff whether a load factor of zero was applicable to loads that reduce the effects of other loads in certain load combinations, unless the former are always known to be present together with the latter. The staff was concerned that all required loads and load combinations were not being adequately considered, which could lead to an unconservative structural design. Therefore, in RAI 155, Question 03.08.03-5, the staff requested that the applicant clarify these issues.

In an April 30, 2009, response to RAI 155, Question 03.08.03-5, the applicant addressed the staff issues as follows:

1. The load combinations given in FSAR Tier 2, Sections 3.8.3.3.2, 3.8.4.3.2, “Loading Combinations,” and 3.8.5.3, “Loads and Load Combinations,” would be revised in the next revision of the FSAR to incorporate the guidance in SRP Sections 3.8.3, 3.8.4, and 3.8.5.

2. FSAR Tier 2, Section 3.8.3.3.2, would be revised to delete the inconsistent criteria related to the plastic section modulus of steel shapes.

3. All footnotes of ANSI/AISC N690-1994, including S2004, Table Q1.5.7.1, are applicable to the design of the U.S. EPR.

4. For live loads, the variation from zero to full value was not considered in the static finite element model of the NI but was considered in local analysis and design.

The staff finds the applicant’s response to Items (1), (2), and (3) acceptable because it complies with SRP Acceptance Criteria 3.8.3.II.3, 3.8.4.II.3, and 3.8.5.II.3. However, upon review of the proposed FSAR revisions, the staff identified that the footnotes to ANSI/AISC N690-1994, including S2004, Table Q1.5.7.1, were still not being referenced in a consistent manner in

FSAR Tier 2, Sections 3.8.3.3.2 and 3.8.4.3.2, as committed to in the response. In addition, regarding Item (4), since the static FE model is being used to obtain member forces for design, it was unclear to the staff why full live load and no live load were not considered in the static FE model of the NI. Finally, the response did not address why a load factor of zero is not considered for (non-live) loads where the use of the full value may reduce the effects of the other loads, as required in ACI 349-01. Therefore, in follow-up RAI 376, Question 03.08.03-22, the staff requested that the applicant supplement its response to clarify the issues given above.

In a July 26, 2010, response to RAI 376, Question 03.08.03-22, the applicant clarified that ANSI/AISC N690-1994, including S2004, was being followed without exceptions and that all footnotes in FSAR Tier 2, Sections 3.8.3.3.2 and 3.8.4.3.2 would be removed for clarity. The applicant further clarified that it will revise the load combinations specified in FSAR Tier 2, Sections 3.8.1, 3.8.2, "Steel Containment"; 3.8.3 and 3.8.4, "Other Seismic Category I Structures," to comply with SRP Acceptance Criteria 3.8.1.II.3, 3.8.2.II.3, 3.8.3.II.3, and 3.8.4.II.3. The staff finds this portion of the response acceptable, because it complies with the above SRP acceptance criteria.

The response also explained that variations in live loads between full and zero values were not considered in the static FE model of the NI, because this would require a very large number of load permutations that would not necessarily contribute to the validation of the structural performance of critical elements. However, local analysis and design does consider variations in live load values, as well as patterned loading conditions, which are used to identify worst case loading conditions for individual structural elements. The same limitations of the static FE model of the NI explain why similar variations are not considered for (non-live) loads where the use of full values may reduce the effects of other loads for certain structural elements. Potential load reduction effects are identified in local analysis, as needed, in accordance with ACI 349-01 Section 9.2.3.

The staff finds the above response acceptable, because it complies with SRP Acceptance Criteria 3.8.1.II.3, 3.8.2.II.3, 3.8.3.II.3, and 3.8.4.II.3. The staff will verify how the variations in live and other loads (between zero and full values) are implemented in local analysis and design. This verification will be performed as part of the resolution of the technical issues associated with the design of critical sections, which are discussed in RAI 155, Questions 03.08.01-24, and 03.08.04-6 (see Section 3.8.1.4 of this report, under "Design and Analysis Procedures").

The staff verified that the changes proposed by the applicant were incorporated into FSAR Tier 2, Revision 2. Therefore, based on the above discussion, the staff considers RAI 155, Question 03.08.03-5 and RAI 376, Question 03.08.03-22 resolved.

In FSAR Tier 2, Section 3.8.3.4.1, the applicant described "localized" abnormal loads applicable to the RBIS, which are not included in the global analysis. These localized loads include sub-compartment pressure loads, pipe break thermal loads, accident pipe reactions, pipe break loads, and local flood loads. Local analyses are used to address these localized loads. However, the staff could not determine how these local analyses and designs are performed for the RBIS, as well as for other Seismic Category I structures described in FSAR Tier 2, Sections 3.8.4 and 3.8.5. The staff was concerned that without a clear methodology for applying localized loads to the RBIS, it would not be possible to determine the safety of the RBIS in relation to localized structural failures which could damage the RBIS to the extent that their safety margin could be compromised. Therefore, in RAI 155, Question 03.08.03-8, the staff requested that the applicant: (1) Provide the method and basis for performing local

analyses for each type of abnormal load, including the potential effects of concrete cracking due to accident thermal loads and redistribution of member forces due to cracking of concrete if significant; and (2) describe how the results of the local analyses are combined with the results of the global structural analyses for other loads.

In a March 31, 2009, response to RAI 155, Question 03.08.03-8, the applicant addressed the staff issues as follows:

Abnormal loads generated by a postulated high-energy pipe break accident were identified.

1. The NSSS is the only high-energy line considered in developing abnormal loads (i.e., accident pipe reaction loads and pipe break loads).

2. Pipe break jet impingement loads and pipe break missile impact loads were not considered in the global analysis of the RBIS.

3. The methodology for evaluating thermal stresses, which follows ACI 349-01, Appendix A, was explained.

4. The design conditions and number of load combinations corresponding to normal plus abnormal loads, as well as normal plus extreme environmental plus abnormal loads, were summarized.

The staff determined that the response did not adequately address the questions raised in the RAI. In particular, the applicant did not explain: (1) Why certain abnormal loads are considered localized and not included in the load combinations for global analysis; (2) how the member forces and stresses due to these localized loads were determined (e.g., describe the more refined FE sub-models briefly mentioned in FSAR Tier 2, Section 3.8.3.4.2); (3) how the potential effects of concrete cracking due to accident thermal loads were considered in the FE models or sub-models; and (4) how the results (e.g., member forces and stresses from the differing global and local FE models and sub-models) of the localized analyses are combined with the results of the global structural analyses for other loads, since location of element forces from the two models do not necessarily match. The staff believes that these issues could impact the design of the RBIS to withstand the abnormal loads. Therefore, in follow-up RAI 306, Question 03.08.01-40, the staff requested that the applicant supplement its response to clarify the issues discussed above.

In a March 12, 2010, response to RAI 306, Question 03.08.01-40, the applicant explained that there are two categories of loads which were not included in the global analysis model: Loads which were not available at the time the global analysis model was run (e.g., relief valve loads and pipe rupture loads), and loads which would result in excessive load combination permutations (e.g., compartment flood loads, containment wall pressure variant loads, and sub-compartment pressure loads). The staff finds this item of the response acceptable, because it explains why certain abnormal loads are considered localized, and complies with the guidance in SRP Acceptance Criteria 3.8.3.II.3 and 3.8.3.II.4.

The applicant further indicated that the future response to RAI 155, Question 03.08.04-6 would address (1) the analysis methodology for determination of member forces and moments due to localized loads not included in global analysis model, (2) potential effects of concrete cracking due to accident thermal loads, and (3) combination of local analysis results with global analysis results. In addition, critical section design utilizing the results of global and local analyses,

including the potential effects of concrete cracking due to accident thermal loads, will be presented in the revised FSAR Tier 2, Appendix 3E.

Based on the above discussion, the staff considers RAI 155, Question 03.08.03-8 resolved with respect to clarifying localized abnormal loads. The remaining issues are subsumed in **RAI 306, Question 03.08.01-40 and RAI 155, Question 03.08.04-6, which are being tracked as open items** pending the completion of the critical section design. The staff evaluation of the critical section design is discussed under RAI 155, Questions 03.08.01-24, and 03.08.04-6 (see Section 3.8.1.4 of this report, under "Design and Analysis Procedures").

The staff identified that certain loads were not described in FSAR Tier 2, Section 3.8.3.3.1, in sufficient detail to determine whether they complied with SRP Acceptance Criteria 3.8.3.II.3. These included dead loads associated with the weight of miscellaneous components (e.g., cable tray systems, conduit systems, HVAC systems), uniformly distributed dead loads (floor, platform, and wall), uniformly distributed live loads (floor, platform, and roof), and hydrostatic loads. In addition, it was unclear to the staff whether the masses corresponding to all applicable dead loads and a percentage of live loads were included in the masses used in the seismic analysis. The staff was concerned that the applicant was not taking into consideration all applicable gravity and seismic inertial loads. Therefore, in RAI 155, Question 03.08.01-5, the staff requested that the applicant provide the magnitude of the gravity and seismic inertial loads, and confirm that these were included in the structural design as applicable. The resolution of this RAI is discussed in Section 3.8.1.4 of this report, under "Loads and Load Combinations," where the staff concluded that RAI 155, Question 03.08.01-5 is resolved.

<u>Design and Analysis Procedures</u>

The staff reviewed the design and analysis procedures used for the containment internal structures to ensure that they meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, GDC 5, and GDC 50; and are in accordance with SRP Acceptance Criteria 3.8.3.II.4. The staff also reviewed the FSAR to establish that the design is essentially complete, as required by 10 CFR 52.47(c). With the exceptions described below, the staff finds that the design and analysis procedures meet the regulatory requirements and regulatory guidance cited above.

During the January 26-30, 2009, onsite audit, the staff identified a technical issue regarding the use of the equivalent-static seismic analysis methodology for the design of Seismic Category I structures. The technical issue concerns the use of seismic modification/reduction factors in the analysis methodology, which were used by the applicant to reduce the magnitude of the zero-period accelerations taken from the applicant's initial seismic SSI analysis based on lumped-mass dynamic models. To verify the applicant's methodology, the staff performed a confirmatory response spectrum seismic analysis of the RBIS using the same shell and brick FE model of the RBIS used by the applicant in its analyses, as described in FSAR Tier 2, Section 3.8.3.4.1 and FSAR Tier 2, Figures 3.8-32, "Reactor Building Internal Structures ANSYS Model," through 3.8-37, "Reactor Building Internal Structures ANSYS Model – View of IRWST and Internal Structures Basemat." In addition, for comparison purposes, the staff also performed an equivalent-static seismic analysis of the same RBIS FE model. Load files, model data, and analysis results were provided by the applicant in a July 20, 2009, response to RAI 222, Question 03.08.03-18.

Two sets of seismic loading data were provided by the applicant. For the equivalent-static analysis, the applicant provided ZPAs with corresponding seismic modification/reduction factors,

for the soil case that resulted in the highest loads. For the response spectrum analysis, the applicant provided seismic floor response spectra at the base of the RBIS model, for a seven percent damping value and the same soil case as the equivalent-static analysis. In both instances, the seismic loading data was obtained from the applicant's initial seismic SSI analysis, which was based on the lumped-mass dynamic model shown in FSAR Tier 2, Figure 3.7.2-3, "Schematic Elevation View of Stick Model for Nuclear Island Common Basemat Structures in Global Y-Z Plane," Revision 1. Fixed base boundary conditions were assumed.

The results of the two confirmatory analyses, equivalent-static and response spectrum, were compared to the results provided by the applicant. Significant differences were identified in both cases.

The results of the staff's equivalent-static analysis, in terms of nodal displacements and member forces, were not identical to those provided by the applicant. Differences in some responses approached eight percent for displacement. The staff noted that the equivalent-static analysis results should have been identical to those provided by the applicant given that the FE model and the loads had not been modified by the staff, and the analysis was linear and static.

The staff's modal analysis of the applicant supplied FE model revealed that the first two frequencies (0.451 Hz, 0.476 Hz) to be very small and possibly spurious (excitation at concentrated mass points). Frequencies of the three most significant modes in the x-direction were 3.81, 5.20, and 5.60 Hz, with corresponding modal participation mass ratios of 0.148, 0.172, and 0.0950, respectively. These results were significantly different from those listed in FSAR Tier 2, Table 3.7.2-3, "Frequencies and Modal Mass Ratios for Reactor Building Internal Structures STICK-1T with All Masses Included," which indicated frequencies of 6.38, 6.77, and 12.53 Hz, and modal participating mass ratios of 0.304, 0.132, and 0.157, respectively. This demonstrated substantial differences in dynamic responses between the RBIS shell and brick FE model and the RBIS lumped-mass dynamic model described in FSAR Tier 2, Section 3.7.2, "Seismic System Analysis." This also demonstrated the limitations of the applicant's RBIS lumped-mass dynamic model and its inability to capture the flexibility of individual slabs and walls.

The results of the staff's response spectrum analysis were compared to the staff's equivalent-static analysis with ZPAs and seismic modification/reduction factors. This comparison would identify the degree of conservatism or non-conservatism of the equivalent-static analysis with ZPAs and seismic modification/reduction factors, which is the applicant's approach, relative to the conventional response spectrum analysis. The comparison of displacements and member forces at representative locations showed response spectrum analysis results larger than equivalent static analysis results (i.e., non-conservative), with differences greater than 50 percent in some cases.

The staff discussed the results of its confirmatory analyses with the applicant during the April 26-30, 2010, onsite audit. The applicant could not clearly explain the discrepancies noted in the equivalent-static analysis. Regarding the results of the modal analysis, there was agreement that the first two frequencies indicated spurious modes, though the actual source of this discrepancy could not be identified. Subsequently, the applicant performed a similar modal analysis of the RBIS shell and brick FE model, and obtained frequencies and modal participation factors which are consistent with those obtained by the staff's analysis, for the three most significant modes in the x-direction. It was concluded that the large differences in the frequency comparison were related to limitations of the RBIS lumped-mass dynamic model (i.e., inability to capture flexibility of individual slabs and walls, especially the SG cubicle walls),

and to the fact that the NSSS could not be realistically represented in the RBIS shell and brick FE model.

Regarding the comparison between the response spectrum and equivalent-static analyses, the applicant provided results of an equivalent-static analysis of the RBIS shell and brick FE model using equivalent-static seismic loads obtained from the more realistic 3-D FE SSI model, which appeared to capture flexibility of slabs and walls. Comparisons of displacement and member force results with the staff's response spectrum analysis results, although improved, still showed discrepancies in the order of 30 percent (non-conservative). However, these discrepancies appeared localized in the region of the flexible SG cubicle walls. The staff concluded that, if seismic modification/reduction factors were not utilized, then the concern with the non-conservatism could be eliminated.

The main objective of the confirmatory analysis was to determine whether the approach initially used by the applicant for the design of the NI structures (i.e., equivalent-static seismic analysis using ZPAs taken from a seismic SSI analysis based on a lumped-mass dynamic model, together with seismic modification/reduction factors) was conservative in comparison to analysis results obtained using the conventional response spectrum analysis and a more realistic dynamic model.

As described in Section 3.8.1.4 of this report in, "Design and Analysis Procedures," the resolution of RAI 190, Question 03.08.01-28 and RAI 376, Question 03.08.01-48 was the modification of the applicant's design approach, which no longer uses seismic modification/reduction factors. Furthermore, FSAR Tier 2, Section 3.7.2, Revision 1, has been further revised to indicate that the seismic SSI analysis of the NI structures utilizes a 3-D FE model instead of the previous lumped-mass dynamic model. Since the applicant's equivalent-static analysis would be utilizing ZPAs taken from a more realistic dynamic model, without any seismic modification/reduction factors, the issues raised by the confirmatory analysis were no longer a technical issue. Therefore, the staff considers RAI 222, Question 03.08.03-18 resolved without further analysis.

In FSAR Tier 2, Sections 3.8.3.4.4, the applicant indicated that the three components of seismic loads are combined using the square root of the sum of the squares (SRSS) method or the 100-40-40 percent rule described in ASCE 4-98, "Seismic Analysis of Safety-Related Nuclear Structures." As described in RG 1.92, Revision 2, the SRSS method and the 100-40-40 percent rule are two different acceptable methodologies that can be used to combine the seismic analysis responses of the structure from each of the three orthogonal directions. However, the staff has noted from past experience that depending on how the 100-40-40 rule is implemented, it may not always give results that are consistent with the guidance provided in RG 1.92. Therefore, in RAI 155, Question 03.08.03-10, the staff requested that the applicant provide further explanation of its implementation of the 100-40-40 rule, including quantitative demonstration which shows that its implementation conforms to the guidance described in RG 1.92.

The above technical issues were also identified in the review of FSAR Tier 2, Section 3.8.4.4, "Design and Analysis Procedures," and Section 3.8.5.4.1, "General Procedures Applicable to Seismic Category I Foundations."

In a May 29, 2009, response to RAI 155, Question 03.08.03-10, the applicant indicated that the 100-40-40 rule described in ASCE 4-98 is mathematically equivalent to the 100-40-40 rule described in RG 1.92, and that FSAR Tier 2, Sections 3.8.3.4.4 and 3.8.4.4.1 would be revised to clarify any ambiguity regarding the 100-40-40 rule.

However, it was unclear to the staff that the 100-40-40 rule was being correctly implemented in the U.S. EPR design. For example, the response to RAI 248, Question 03.07.02-26 and FSAR Tier 2, Revision 0, Tables 3E.2-1, “Governing Forces and Moments for the EPGB Basemat Foundation,” through 3E.2-5, “Governing Forces and Moments for EPGB Typical Wall at Column Line 11 for Horizontal Reinforcement (Local Cut),” appeared to indicate that each 100-40-40 rule permutation was being considered as a different load combination in the design. The staff’s interpretation of this rule is that each 100-40-40 rule permutation should not be used as a different load combination in the design but, rather, should be used only in the determination of maximum individual force or moment resultants. Also, the staff noted that there appeared to be conflicting results between the combination of responses due to multiple ground motion directions and the combination of multiple interacting force or moment resultants. This is not acceptable because it could lead to a non-conservative structural design. Finally, FSAR Tier 2, Section 3.8.5, described the equivalent-static seismic analyses of the NI structures as being nonlinear because they accounted for potential uplift of the structure at the NI basemat-soil interface; however, it appeared that the applicant was utilizing the 100-40-40 rule to combine the results of these nonlinear analyses, which is not valid.

Therefore, in follow-up RAI 376, Question 03.08.03-24, the staff requested that the applicant compare their implementation of the 100-40-40 rule with the staff’s interpretation of this rule, provided in an illustrative example. The staff also requested that the applicant confirm that the 100-40-40 rule was only being used in the context of linear analysis, since the principle of superposition is no longer valid when nonlinear behavior is assumed.

During the April 26-30, 2010, onsite audit, the applicant indicated it would only follow the SRSS method in the design of all Seismic Category I structures, and eliminate the use of the 100-40-40 rule.

In a July 29, 2010, response to RAI 376, Question 03.08.03-24, the applicant confirmed that it would only follow the SRSS method in the design of all Seismic Category I structures. The response also provided a detailed discussion of how the SRSS method is implemented in the design of concrete wall and slab-type structures. The staff finds the applicant’s response acceptable, because it meets the design requirements in ACI 349-01 and ASME Code, Section III, Division 2, Subsection CC, and the guidance in RG 1.92 regarding combination of multi-directional seismic effects.

The issue of the validity of the SRSS method or the 100-40-40 percent rule in the context of nonlinear analysis no longer applies to the NI structures because, as described in Sections 3.8.1 and 3.8.5 of this report, the applicant has modified its design and analysis procedures for the NI structures. Nonlinear analysis is only performed for the design of the NI basemat; however, the seismic design of the NI basemat is based on time-history analyses for which the combination of multi-directional seismic effects can be done by simple algebraic summation, in accordance with the guidance in RG 1.92.

The staff verified that the changes proposed by the applicant were incorporated into FSAR Tier 2, Revision 2. Therefore, based on the above discussion, the staff considers RAI 155, Question 03.08.03-10 and RAI 376, Question 03.08.03-24 resolved.

In FSAR Tier 2, Section 3.8.3, the applicant described the FE models used for the structural analysis and design of the containment internal structures, utilizing the ANSYS computer code. However, the staff could not determine the acceptability of these FE models without additional descriptive information. Therefore, in RAI 155, Question 03.08.03-16, the staff requested that

applicant provide the additional information described below for all Seismic Category I structures.

1. From the information provided in the FSAR, it was unclear to the staff whether the FE discretization was sufficiently refined. The staff was concerned that without a sufficiently refined FE discretization of the structures, the member forces, strains, and stresses determined from the computer analysis may not be sufficiently accurate, which could ultimately lead to an non-conservative design. The FSAR does not describe what procedures were used to select the appropriate number of elements for meshing concrete regions such as walls and slabs. The mesh density used for both the global and local FE models, described in FSAR Tier 2, Section 3.8.3, and FSAR Appendix 3E, in many cases appeared coarse for 4-noded and 3-noded shell elements. The staff requested that the applicant clarify how the mesh refinement was determined and validated for each model, and describe any FE options that were selected to improve the accuracy of the results.

2. Since triangular finite elements were used in addition to rectangular elements, and it is recognized that triangular elements are not generally as accurate as rectangular elements, the staff requested that the applicant clarify what steps were taken in the FE model development to ensure that sufficient accuracy was achieved. As indicated above, an insufficiently accurate FE model could lead to a non-conservative design. Also, since the angle between some of the finite elements in the model appear to deviate from optimum angles for triangular and rectangular finite elements (e.g., FSAR Tier 2, Figure 3.8-34, "Reactor Building Internal Structures ANSYS Model – Section through Reactor Cavity and Refueling Canal," lower right hand region of elevated slab), the staff also requested that the applicant clarify how it was assured that the results using such finite elements were still accurate.

3. The ANSYS FE models of the RBIS are shown in FSAR Tier 2, Figure 3.8-32 with the cut models visible in FSAR Tier 2, Figures 3.8-33, "Reactor Building Internal Structures ANSYS Model – Section through Center of Building Looking West," to 3.8-37, and FSAR Appendix 3E. While most of the internal structures use shell elements, the staff could not clearly define which structures use solid brick-type finite elements. The staff requested that the applicant clarify how the shell/solid interfaces are modeled and how that approach ensures acceptable compatibility at the interface, since solid elements do not have rotational degrees of freedom. The staff was concerned that the regions of the FE model around this interface would not provide an accurate representation of member forces, strains, and stresses to be used in design. The staff also requested that the applicant clarify how solution accuracy is ensured for both linear and nonlinear analyses.

4. FSAR Tier 2, Section 3.8.3.4.1 discusses when creep, shrinkage, and differential settlement are considered. The staff requested that the applicant clarify the criterion used to distinguish when these effects need to be considered and how they are included in a particular analysis.

The above technical issues were also identified in the review of FSAR Tier 2, Sections 3.8.4, 3.8.5, and Appendix 3E, for the FE models of other Seismic Category I structures.

In a July 31, 2009, response to RAI 155, Question 03.08.03-16, the applicant provided the following information regarding FE models used for the structural analysis and design of the containment internal structures and other Seismic Category I structures:

1. FE mesh density was refined to optimize computing time while providing accurate results. The applicant further indicated that two mesh validation studies were performed. For the RBIS static model, two critical sections were analyzed and it was found that further mesh refinement produced negligible effect on design results (i.e., required area of steel reinforcement). The staff finds this item of the response acceptable, because it provided an adequate explanation of the approach to FE mesh refinement and validation, which justifies the mesh refinements for the models and also complies with SRP Acceptance Criteria 3.7.2.II.3.C.ii.

2. The applicant indicated that the number of triangular finite elements with suboptimal geometry (i.e., degenerated elements or elements that violate shape limits) represent a minimal fraction of the FE mesh of the NI static model. Although these triangular elements may be less accurate, they adequately transfer forces and moments and provide acceptable accuracy in selected applications. The applicant also indicated that design results from these elements are carefully considered in section design. The staff finds this item of the response acceptable, because it provided an adequate explanation of the approach used to address suboptimal geometries in the FE mesh, which complies with SRP Acceptance Criteria 3.7.2.II.3.C.ii.

3. The RBIS base and primary shield wall up to elevation 3.0 m (9.84 ft) are modeled with brick elements; all other parts of the RBIS are modeled with shell elements. In addition, compatibility at the shell/solid interface is achieved through an ANSYS multi-point constraint approach. The applicant also indicated that nonlinearity in the NI static model (RCB portion for thermal analysis, nonlinear soil springs, if applicable) does not affect the shell/solid interface. The staff finds this item of the response acceptable, because it provided an adequate explanation of the approach used to address shell/solid interface compatibility in the FE model of the RBIS, which complies with SRP Acceptance Criteria 3.7.2.II.3.C.ii.

4. Creep, shrinkage, and other time-dependent considerations are included in an analysis when they reduce the strength of the structure or cause serviceability concerns. In particular, the effects of creep and shrinkage on deflections of concrete slabs are estimated in accordance with ACI 349-01, Section 9.5. No additional considerations are given to creep and shrinkage in the analysis of the RBIS. The staff finds this item of the response acceptable, because it provided sufficient explanation of the design approach used to account for the effects of creep and shrinkage in the RBIS, and complies with SRP Acceptance Criteria 3.8.3.II.4 and ACI 349-01. The staff also noted that the issue of differential settlements and its effects on the design of the NI structures is addressed in Section 3.8.5 of this report.

Based on the above discussion, the staff considers RAI 155, Question 03.08.03-16 resolved.

In FSAR Tier 2, Section 3.8.3.1.9, "Reactor Building Internal Structures Basemat, In-Containment Refueling Water Storage Tank, and Core Melt Retention Area," the applicant indicated that the RBIS is not anchored to the sides of the RCB or to the underlying NI basemat. Furthermore, the liner plate at the horizontal interface between the RBIS and the underlying NI basemat is not anchored to either of these two structures, so there is minimal transfer of lateral loads from the RBIS through this interface. Therefore, the main load path for lateral seismic loads and overturning moments from the RBIS to the surrounding structures is through horizontal bearing pressure on the RCB haunch wall and vertical bearing pressure on the underlying NI basemat.

The staff identified the following issues associated with the above design approach:

1. The FSAR did not explain how the seismic stability for the RBIS is ensured and what the associated analysis method is, and, in particular, how the factors of safety given in FSAR Tier 2, Section 3.8.5.5.1, "Nuclear Island Common Basemat Structure Foundation Basemat," had been computed.

2. The FSAR did not describe any analyses performed to determine the magnitude of potential uplift or sliding of the RBIS due to seismic loads, which could damage the RCB liner plate.

3. It was unclear to the staff whether the applicant had determined the magnitude of the contact pressures imposed by the RBIS on the surrounding portions of the RCB and the NI basemat, and whether these contact pressures had been taken into consideration in the design.

Therefore, in RAI 155, Question 03.08.01-2, RAI 190, Question 03.08.01-29, and RAI 283, Question 03.08.01-37, the staff requested that the applicant address these issues. The resolution of these RAIs is discussed in Section 3.8.1.4 of this report in, "Design and Analysis Procedures," where the staff concluded that RAI 155, Question 03.08.01-2 and RAI 190, Question 03.08.01-29 are resolved, while **RAI 283, Question 03.08.01-37 is being tracked as a confirmatory item**.

The staff's review of FSAR Tier 2, Section 3.8.3.4, could not determine whether the potential effects of high irradiation on structural members close to the RV (e.g., the RV concrete support structure) were considered in the design. Therefore, in RAI 155, Question 03.08.01-16 and RAI 306, Question 03.08.01-39, the staff requested that the applicant clarify this issue. The staff considers RAI 155, Question 03.08.01-16 and RAI 306, Question 03.08.01-39 resolved with respect to the potential effects of high irradiation on structural members close to the RV. The resolution of these RAIs is discussed in Section 3.8.1.4 of this report, under "Design and Analysis Procedures."

During the January 26-30, 2009, onsite audit, the staff could not locate documented criteria for the selection of critical sections corresponding to the RBIS. The identification, analysis, and design of critical sections is important, because it provides reasonable assurance to the staff that the structural design of Seismic Category I structures is essentially complete, as required by 10 CFR 52.47(c). Therefore, in RAI 155, Question 03.08.01-20, Question 03.08.01-24, and Question 03.08.04-6, the staff requested that the applicant address these concerns. RAI 155, Question 03.08.01-20 is resolved. **RAI 155 Question 03.08.01-24 and Question 03.08.04-6 are being tracked as open items** pending the completion of the critical section design. The resolution of these RAIs is discussed in Section 3.8.1.4, "Design and Analysis Procedures," of this report.

Structural Acceptance Criteria

To ensure that the containment internal structures, as designed, can perform their intended safety function, the staff verified that the structural design acceptance criteria meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; GDC 1, GDC 2, GDC 4, GDC 5, and GDC 50; and are in accordance with SRP Acceptance Criteria 3.8.3.II.5. With the exception described below, the staff finds that the structural acceptance criteria meet the regulatory requirements and regulatory guidance cited above.

In FSAR Tier 2, Section 3.8.3.1, the applicant indicated that the RBIS are Seismic Category I structures except for miscellaneous structures such as platforms, stairs, guard rails, and other ancillary items, which are designed as Seismic Category II to prevent adverse impact on the Seismic Category I structures in the event of an SSE. However, the staff could not identify the analysis and design methods and acceptance criteria for Seismic Category II SSCs. Therefore, in RAI 155, Question 03.08.03-17, the staff requested that the applicant describe the analysis methods and acceptance criteria that have been implemented for the seismic design of Seismic Category II miscellaneous structures inside containment and other Seismic Category II structures covered in FSAR Tier 2, Sections 3.8.3 and 3.8.4.

In a July 31, 2009, response to RAI 155, Question 03.08.03-17, the applicant provided general information regarding acceptance criteria used in the analysis and design of Seismic Category II SSCs. The staff determined that the information provided was not sufficiently detailed to assess the adequacy of its implementation. In particular, the response indicated that Seismic Category II SSCs are analyzed and designed to prevent interaction with Seismic Category I SSCs in a manner that could impair design basis safety functions, but are not necessarily designed to Seismic Category I standards; however, the response did not explain how this would be accomplished.

Since the technical issue described above is related to the potential seismic interaction of Seismic Category II miscellaneous structures with Seismic Category I structures, the staff's review of FSAR Tier 2, Section 3.7.3 also identified the same issue. Therefore, in RAI 370, Question 03.07.03-38 and RAI 463, Question 03.07.03-40, the staff requested that the applicant address the potential impact of these Seismic Category II miscellaneous structures on the Category I structures under SSE load. RAI 155, Question 03.08.03-17 was superseded by the above RAI questions requested in the review of FSAR Tier 2, Section 3.7.3. Since these FSAR Tier 2, Section 3.7.3 RAI issues were resolved, the staff considers RAI 155, Question 03.08.03-17 resolved. The detailed resolution of these RAIs is discussed in Section 3.7.3 of this report.

Materials, Quality Control, and Special Construction Techniques

The staff reviewed the materials, quality control, and special construction techniques used for the containment internal structures to ensure that they meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; GDC 1, GDC 2, GDC 4, GDC 5, and GDC 50 and are in accordance with SRP Acceptance Criteria 3.8.3.II.6. With the exception described below, the staff finds that the materials, quality control, and special construction techniques meet the regulatory requirements and regulatory guidance cited above.

In FSAR Tier 2, Section 3.8.3.6.5, the applicant provided a brief description of modular construction methods and composite-type structural members used in the U.S. EPR standard design. The staff was concerned about the structural design, fabrication, configuration, layout, and connections of designated modules because of past experience with other NPP certified designs (e.g., AP 1000), which utilized composite-type modules and structural units that were new to U.S. NPPs and required extensive case-by-case review. Therefore, in RAI 155, Question 03.08.03-13, the staff requested that the applicant provide a more detailed description of each specific type of module or composite member used in the U.S. EPR standard design, and to provide a description of the analysis and design approach used for each type of module and composite member.

The above technical issue was also identified in the review of FSAR Tier 2, Sections 3.8.4.6.3, "Special Construction Techniques," and 3.8.5.6.3, "Special Construction Techniques."

In a March 31, 2009, response to RAI 155, Question 03.08.03-13, the applicant indicated that, even though modular construction methods are used as noted in FSAR Tier 2, Sections 3.8.3.6.5, 3.8.4.6.3, and 3.8.5.6.3, no designated modules or three dimensional structural units are used in the U.S. EPR design. The response also clarified that the construction of the RBIS uses proven methods, and does not rely on unique construction techniques. Well-established modular construction methods are used for prefabricating portions of the IRWST liner, refueling canal liner, reinforcing, concrete formwork, and other portions of the RBIS. The staff finds this response acceptable, because it complies with SRP Acceptance Criteria 3.8.3.II.6 and 3.8.3.II.4.E.

Based on the above discussion, the staff considers RAI 155, Question 03.08.03-13 resolved.

Testing and Inservice Surveillance Requirements

To ensure that the containment internal structures can perform their intended safety function, the staff verified that the testing and ISI surveillance requirements meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; GDC 1, GDC 2, GDC 4, GDC 5, and GDC 50 and are in accordance with SRP Acceptance Criteria 3.8.3.II.7. With the exception described below, the staff finds that the testing and ISI surveillance requirements meet the regulatory requirements and regulatory guidance cited above.

In FSAR Tier 2, Section 3.8.3.7, the applicant indicated that monitoring and maintenance of structures is performed in accordance with RG 1.160, without reference to 10 CFR 50.65. In RAI 155, Question 03.08.03-14, the staff requested that the applicant clarify why monitoring and maintenance of structures is not performed in accordance with the requirements of 10 CFR 50.65 and supplemented with the guidance in RG 1.160.

The above technical issue was also identified in the review of FSAR Tier 2, Sections 3.8.4.7, "Testing and Inservice Inspection Requirements," and 3.8.5.7, "Testing and Inservice Inspection Requirements."

In an April 30, 2009, response to RAI 155, Question 03.08.03-14, the applicant indicated that FSAR Tier 2, Sections 3.8.3.7, 3.8.4.7, and 3.8.5.7 would be modified to clarify that monitoring and maintenance of structures is performed in accordance with 10 CFR 50.65 and supplemented with the guidance in RG 1.160. The staff verified that the proposed changes were incorporated into FSAR Tier 2, Revision 2. On this basis, the staff considers RAI 155, Question 03.08.03-14 resolved.

ITAAC,and Site Parameters

ITAAC: The staff has reviewed the ITAAC associated with FSAR Tier 2, Section 3.8.3 in FSAR Tier 1, Table 2.1.1-8 and found them acceptable for this area of review.

Site Parameter: The staff has reviewed the following site parameter that will be addressed in COL designs: Item No. 3-4, Site-specific loads that lie within the standard plant design envelope for Seismic Category I structures, in FSAR Tier 2, Table 1.8-1, and finds it acceptable for this area of review.

3.8.3.5 *Combined License Information Items*

Table 3.8.3-1 provides a list of concrete and steel internal structures of concrete containment related COL information item numbers and descriptions from FSAR Tier 2, Table 1.8-2:

Table 3.8.3-1 U.S. EPR Combined License Information Items

Item No.	Description	FSAR Tier 2 Section
3.8-6	A COL applicant that references the U.S. EPR design certification will confirm that site-specific loads lie within the standard design envelope for RB internal structures, or perform additional analyses to verify structural adequacy.	3.8.3.3

The staff finds the above listing to be complete. Also, the list adequately describes actions necessary for the COL applicant. No additional COL information items need to be included in FSAR Tier 2, Table 1.8-2 for concrete and steel internal structures of concrete containment consideration.

3.8.3.6 *Conclusions*

The staff reviewed FSAR Tier 2, Revision 2, Section 3.8.3 to determine whether the design, fabrication, construction, testing, and ISI of the containment internal structures comply with 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, GDC 5, and GDC 50. Except for the open and confirmatory items described above, the staff finds that the applicant meets the relevant requirements of 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, GDC 5, and GDC 50. The basis for this conclusion is the following:

The applicant meets the requirements of 10 CFR 50.55a and GDC 1, to ensure that the containment internal structures are designed, fabricated, erected, constructed, tested, and inspected to quality standards commensurate with the importance of the safety function to be performed, by meeting the guidelines in SRP Section 3.8.3; RG 1.69, RG 1.142, RG 1.160, and RG 1.199; ACI 349 with additional guidance provided by RG 1.142 and RG 1.199; and ANSI/AISC N690-1994, including S2004.

The applicant meets the requirements of GDC 2 to ensure that the containment internal structures are able to withstand the most severe natural phenomena, by analyzing and designing these structures to withstand the most severe earthquake that has been established for the U.S. EPR certified design, in load combinations that include earthquake loading. Protection from other natural phenomena such as winds, tornadoes, and floods is provided by other Seismic Category I structures that shield the containment.

The applicant meets the requirements of GDC 4 to ensure that the containment internal structures are appropriately protected against other dynamic effects, by analyzing and designing these structures to withstand the dynamic effects due to pipe whipping, missiles, and discharging fluids associated with the LOCA.

The applicant meets the requirements of GDC 5 to ensure that SSCs are not shared between units or that sharing will not impair their ability to perform their intended safety functions, by analyzing and designing these structures for the design loads and load combinations described in SRP Section 3.8.3; ACI 349 with additional guidance provided by RG 1.142 and RG 1.199; and ANSI/AISC N690-1994, including S2004.

The applicant meets the requirements of GDC 50 to ensure that the containment internal structures are designed with sufficient margin of safety to accommodate appropriate design loads, by meeting the design guidelines in SRP Section 3.8.3; RG 1.69, RG 1.142, RG 1.160, and RG 1.199; ACI 349 with additional guidance provided by RG 1.142 and RG 1.199; and ANSI/AISC N690-1994, including S2004.

The applicant meets the requirements of 10 CFR Part 50, Appendix B, to provide a quality assurance program for the design, construction, and operation of SSCs, by meeting the guidelines in SRP Section 3.8.3; RG 1.69, RG 1.142, RG 1.160, and RG 1.199; ACI 349 with additional guidance provided by RG 1.142 and RG 1.199; and ANSI/AISC N690-1994, including S2004.

As mentioned above, the acceptability of the design, fabrication, construction, testing, and ISI of the containment internal structures is predicated on the adequate resolution of the open and confirmatory items discussed in this section.

3.8.4 Other Seismic Category I Structures

3.8.4.1 *Introduction*

FSAR Tier 2, Section 3.8.4, "Other Seismic Category I Structures," describes other Seismic Category I structures, which include the RSB, FB, Safeguard Buildings 1, 2, 3, and 4, and vent stack, all located on the NI foundation basemat, as well as the Emergency Power Generating Buildings and the Essential Service Water Buildings, that have their own independent basemat foundations.

3.8.4.2 *Summary of Application*

FSAR Tier 1: FSAR Tier 1 information associated with this section is found in FSAR Tier 1, Sections 2.1.1, "Nuclear Island," 2.1.2, "Emergency Power Generating Building," 2.1.5, "Essential Service Water Building," and 4.6, "Buried Piping and Pipe Ducts."

FSAR Tier 1, Section 2.1.1 provides a physical and functional description of the RSB, FB, and SBs which are located on the NI foundation basemat. It describes the arrangement and key design features of each structure. Key mechanical design features, including seismic classifications, are specified.

FSAR Tier 1, Section 2.1.2 provides a physical and functional description of the EPGBs. It describes the arrangement and key design features of the EPGBs. Key mechanical design features, including seismic classifications, are also specified.

FSAR Tier 1, Section 2.1.5 provides a physical and functional description of the ESWBs. It describes the arrangement and key design features of the ESWBs. Key mechanical design features, including seismic classifications, are also specified.

FSAR Tier 2: The applicant has provided an FSAR Tier 2 description of other Seismic Category I structures in FSAR Tier 2, Section 3.8.4, summarized as follows:

<u>Description of Other Seismic Category I Structures</u>

FSAR Tier 2, Section 3.8.4.1, "Description of the Structures," describes the other Seismic Category I structures. These include:

- Reactor Shield Building and annulus
- Fuel Building
- Safeguard Buildings 1, 2, 3, and 4
- Vent Stack
- Emergency Power Generating Buildings 1/2 and 3/4
- Essential Service Water Buildings 1, 2, 3, and 4
- Distribution system supports
- Platforms and miscellaneous structures

FSAR Tier 2, Section 3.8.4.1, also identifies that structures described within this section are not shared with any other power plant units. FSAR Tier 2, Section 3.7.2, “Seismic System Analysis,” describes the design requirements applicable to non-safety-related structures to preclude adverse interaction effects on Seismic Category I structures. A COL applicant that references the U.S. EPR design certification will describe any differences between the standard plant layout and design of Seismic Category I structures required for site-specific conditions; and a COL applicant that references the U.S. EPR design certification will address site-specific Seismic Category I structures that are not described in this section.

FSAR Tier 2, Sections 3.8.4.1.1, “Reactor Shield Building and Annulus”; 3.8.4.1.2, “Fuel Building”; 3.8.4.1.3, “Safeguard Buildings”; 3.8.4.1.4, “Emergency Power Generating Buildings”; 3.8.4.1.5, “Essential Service Water Buildings,” 3.8.4.1.6, “Distribution System Supports,” and 3.8.4.1.7, “Platforms and Miscellaneous Structures,” provide details for the Seismic Category I structures given above. FSAR Tier 2, Section 3.8.4.1.8, “Buried Conduit and Duct Banks,” identifies that the design of buried conduit and duct banks is site-specific. A COL applicant that references the U.S. EPR design certification needs to describe Seismic Category I buried conduit and duct banks. FSAR Tier 2, Section 3.8.4.1.9, “Buried Pipe and Pipe Ducts,” identifies that the design of buried pipes and pipe ducts is site-specific. A COL applicant that references the U.S. EPR design certification needs to describe Seismic Category I buried pipe and pipe ducts. FSAR Tier 2, Section 3.8.4.1.10, “Masonry Walls,” identifies that no masonry walls are used inside Seismic Category I structures.

<u>Applicable Codes, Standards, and Specifications</u>

FSAR Tier 2, Section 3.8.4.2, “Applicable Codes, Standards, and Specifications,” identifies the codes, standards, specifications, design criteria, regulations, and regulatory guides applicable to the design, fabrication, construction, testing, and ISI of other Seismic Category I structures.

FSAR Tier 2, Sections 3.8.4.2.1, “Codes and Standards”; and 3.8.4.2.2, “Specifications,” address codes, standards, and specifications. Twenty-seven documents are provided. The two key standards are ACI-349-01 for reinforced concrete and ANSI/AISC N690-1994, including S2004, for structural steel. The remaining documents supplement these key standards.

The specifications cover details related to the design and construction of other Seismic Category I structures, including the following: concrete material properties; mixing, placing, and curing of concrete; reinforcing steel and splices; structural steel; stainless steel liner plate and embedments; miscellaneous and embedded steel; anchor bolts; expansion anchors; and cranes and hoists.

FSAR Tier 2, Section 3.8.4.2.3, "Design Criteria," identifies the following standards for the design criteria applicable to other Seismic Category I structures: ACI 349-01, ACI 349-06, and ANSI/AISC N690-1994, including S2004.

FSAR Tier 2, Section 3.8.4.2.4, "Regulations," identifies the following regulations applicable to other Seismic Category I structures: 10 CFR Part 50, Appendix A, "General Design Criteria for Nuclear Power Plants"; GDC 1, "Quality Standards and Records"; GDC 2, "Design Bases for Protection Against Natural Phenomena"; GDC 4, "Environmental and Dynamic Effects Design Bases"; GDC 5, "Sharing of Structures, Systems, and Components"; 10 CFR Part 50, Appendix B, "Quality Assurance Criteria for Nuclear Power Plants and Fuel Reprocessing Plants"; and 10 CFR Part 50, Appendix S, "Earthquake Engineering Criteria for Nuclear Power Plants."

FSAR Tier 2, Section 3.8.4.2.5, "NRC Regulatory Guides," identifies the following Regulatory Guides applicable to the design and construction of other Seismic Category I structures: RG 1.61, "Damping Values for Seismic Design of Nuclear Power Plants ," March 2007, Revision 1 (with a noted exception); RG 1.69, "Concrete Radiation Shields for Nuclear Power Plants," December 1973; RG 1.115, "Protection Against Low-Trajectory Turbine Missiles," July 1977, Revision 1; RG 1.142, "Safety-Related Concrete Structures for Nuclear Power Plants (Other than Reactor Vessels and Containments)," November 2001, Revision 2 (with a noted exception); RG 1.160, "Monitoring the Effectiveness of Maintenance at Nuclear Power Plants," March 1997, Revision 2; RG 1.199, "Anchoring Components and Structural Supports in Concrete ," November 2003 (with a noted exception).

<u>Loads and Load Combinations</u>

FSAR Tier 2, Section 3.8.4.3, "Loads and Load Combinations," describes the loads and load combinations for other Seismic Category I structures. The FSAR indicates that loads are classified as normal loads, severe environmental loads, extreme environmental loads, or abnormal loads. It is necessary for a COL applicant referencing the U.S. EPR design certification to confirm that the site-specific loads are enveloped by the loads assumed for design certification. If this is not the case, then it is necessary for the COL applicant to perform additional analyses to verify structural adequacy of the other Seismic Category I structures.

FSAR Tier 2, Section 3.8.4.3.1, "Design Loads," describes the individual design loads that are classified as normal loads, severe environmental loads, extreme environmental loads, or abnormal loads. The FSAR indicates that design loads on the other Seismic Category I structures are defined to conform to ACI 349-01 and the guidelines of RG 1.142 for concrete structures, and ANSI/AISC N690-1994, including S2004 for steel structures. The FSAR also identifies other loads that are not included in the design basis. These are aircraft hazards; explosion pressure wave; and turbine missiles. The first two are considered as part of the plant safeguards and security measures. Turbine missiles and conformance to RG 1.115 are addressed in FSAR Tier 2, Section 3.5, "Missile Protection."

FSAR Tier 2, Section 3.8.4.3.2, "Loading Combinations," identifies the specific load combinations for other Seismic Category I structures. The FSAR indicates that load

combinations for design are defined to conform to ACI 349-01 and the guidelines of RG 1.142 for concrete structures, and ANSI/AISC N690-1994, including S2004 for steel structures.

FSAR Tier 2, Section 3.8.4.3.2, also discusses the NI common basemat structure. Various portions of the basemat have different classifications (i.e., RCB, RBIS, or other Seismic Category I structures) and different design requirements based on the corresponding applicable code. To account for structure-to-structure effects, additional loads are considered for the other Seismic Category I portion of the common basemat.

Design and Analysis Procedures

FSAR Tier 2, Section 3.8.4.4, "Design and Analysis Procedures," describes the design and analysis procedures for the other Seismic Category I structures. The FSAR indicates that other Seismic Category I concrete structures are designed according to ACI 349-01 and its appendices, with the exceptions delineated in RG 1.142. The FSAR further indicates that other Seismic Category I steel structures are designed according to the requirements of ANSI/AISC N690-1994, including S2004; and that design of concrete embedments and anchors conforms to ACI 349-06, Appendix D, and the guidelines of RG 1.199, with the exceptions identified in FSAR Tier 2, Section 3.8.1, "Concrete Containment."

The other Seismic Category I structures are analyzed using both computer modeling and standard solutions, for the loads and loading combinations described in FSAR Tier 2, Section 3.8.4.3. The overall computer model of the NI structures includes the RSB, the four SBs, the FB, as well as the RCB, RBIS, and the NI Common Basemat Structure foundation basemat which are described in the other FSAR Tier 2 sections. Separate computer models are used for the EPGBs and for the ESWBs. Local analyses of specific structural walls, slabs, and members are also performed to account for local effects of equipment loads, pipe break loads, hydrostatic and hydrodynamic loads, and other conditions. The global responses from the overall computer models are combined with the results of the local analyses, to obtain the total loads for detailed design.

FSAR Tier 2, Section 3.8.4.4.1, "General Procedures Applicable to Other Seismic Category I Structures"; 3.8.4.4.2, "Reactor Shield Building and Annulus, Fuel Building, and Safeguard Buildings – NI Common Basemat Structure"; 3.8.4.4.3, "Emergency Power Generating Buildings"; 3.8.4.4.4, "Essential Service Water Buildings"; 3.8.4.4.5, "Buried Conduit and Duct Banks, and Buried Pipe and Pipe Ducts"; and 3.8.4.4.6, "Design Report," provide details of the design and analysis procedures.

FSAR Tier 2, Appendix 3E, "Design Details and Critical Sections for Safety-Related Category I Structures," provides additional details for the analysis and design of the selected critical sections of the other Seismic Category I structures.

FSAR Tier 2, Section 3.8.4.4.5 identifies that the design of buried conduit and duct banks, and buried pipe and pipe ducts is site-specific, but also provides a very detailed description of design and analysis procedures. Following the detailed description, three COL applicant actions are defined.

Structural Acceptance Criteria

FSAR Tier 2, Section 3.8.4.5, "Structural Acceptance Criteria," indicates that the acceptance limits for other reinforced concrete Seismic Category I structures, in terms of allowable stresses, strains, deformations and other design criteria, are in accordance with ACI 349-01, and its

appendices, including the exceptions specified in RG 1.142. Acceptance limits for concrete embedments and anchors are in accordance with ACI 349-06 (Appendix D) and guidance given in RG 1.199, with noted exceptions. Acceptance limits for other steel Seismic Category I structures are in accordance with ANSI/AISC N690-1994, including S2004.

The FSAR indicates that load combinations including SSE seismic loads generally control the design of other Seismic Category I structures. FSAR Tier 2, Appendix 3E provides design results for the critical sections of the other Seismic Category I structures.

The FSAR further indicates that deviations from the approved design will be summarized in an as-built report, and it will be confirmed that the as-built of other Seismic Category I structures are capable of withstanding the design basis loads described in FSAR Tier 2, Section 3.8.4.3.

The FSAR also specifies that a COL applicant referencing the U.S. EPR design certification will confirm that site-specific Seismic Category I buried conduit, electrical duct banks, pipe, and pipe ducts satisfy the criteria in FSAR Tier 2, Section 3.8.4.4.5, and in AREVA Topical Report ANP-10264NP-A, "U.S. EPR Piping Analysis and Pipe Support Design Topical Report."

Materials, Quality Control, and Special Construction Techniques

FSAR Tier 2, Section 3.8.4.6, "Materials, Quality Control, and Special Construction Techniques," describes the materials, quality control, and special construction techniques applicable to other Seismic Category I structures.

FSAR Tier 2, Section 3.8.4.6.1, "Materials," indicates that concrete, reinforcing steel, and structural steel materials for other Seismic Category I structures are the same as described in FSAR Tier 2, Section 3.8.3.6, "Materials, Quality Control, and Special Construction Techniques," except that the concrete minimum compressive strength (f'c) is equal to 41.4 MPa (6,000 psi) for the RSB, FB and SBs; 34.5 MPa (5,000 psi) for the EPGBs and ESWBs; and 27.6 MPa (4,000 psi) for buried duct banks and pipe ducts.

The FSAR also indicates that the use of epoxy coated reinforcing steel and waterproofing membranes for exterior walls and slabs will be evaluated on a site-specific basis, as described in FSAR Tier 2, Section 3.8.5.6, "Materials, Quality Control, and Special Construction Techniques."

FSAR Tier 2, Section 3.8.4.6.2, "Quality Control," refers to FSAR Tier 2, Section 3.8.3.6, for quality control procedures for other Seismic Category I structures.

FSAR Tier 2, Section 3.8.4.6.3, "Special Construction Techniques," indicates that construction of other Seismic Category I structures uses proven methods, and does not rely on unique construction techniques. Well-established modular construction methods are used for prefabricating portions of the spent fuel pool liner, other tank liners, distribution system supports, reinforcement, and concrete formwork, for other Seismic Category I structures. Permanent and temporary stiffeners are used on liner plate sections and other modular items to ensure structural integrity during rigging operations.

Where steel decking and plates and supporting steel beams are used to form concrete floors, the decking thickness is not considered in determining the required floor thickness. Decking, plates, and beams left in place are designed for applicable seismic loads and other loading conditions. Other types of formwork that are left in place meet code requirements and are designed to prevent their failure from affecting Seismic Category I SSCs.

Testing and Inservice Surveillance Requirements

FSAR Tier 2, Section 3.8.4.7, “Testing and Inservice Inspection Requirements,” summarizes the testing and inservice inspection requirements for the other Seismic Category I structures. Monitoring and maintenance of structures is performed in accordance with 10 CFR 50.65, “Requirements for monitoring the effectiveness of maintenance at nuclear power plants,” as supplemented by the guidance in RG 1.160.

FSAR Tier 2, Section 9.1, “Fuel Storage and Handling,” addresses testing and ISI of the spent fuel pool leak chase channels in the FB. FSAR Tier 2, Section 9.1.5, “Overhead Heavy Load Handling System,” addresses testing and ISI requirements applicable to cranes.

The FSAR indicates that physical access is provided to perform ISI of exposed portions of other Seismic Category I structures. Examination of inaccessible portions of below-grade concrete structures for degradation and monitoring of ground water chemistry are addressed in FSAR Tier 2, Section 3.8.5.7, “Testing and Inservice Inspection Requirements.”

ITAAC: The ITAAC associated with FSAR Tier 2, Section 3.8.4 are given in FSAR Tier 1, Table 2.1.1-8, “Reactor Building ITAAC”; Table 2.1.1-10, “Safeguard Buildings ITAAC”; and Table 2.1.1-11, “Fuel Building ITAAC,” for the RSB, Safeguard Buildings, and FB, respectively. The ITAAC associated with the EPGBs are given in FSAR Tier 1, Table 2.1.2-3, “Emergency Power Generating Building ITAAC.” The ITAAC associated with the ESWBs are given in FSAR Tier 1, Table 2.1.5-3, “Essential Service Water Building ITAAC.”

Interface Requirements: This section of the FSAR contains information related to the following interface requirement that will be addressed in COL applications: FSAR Tier 2, Table 1.8-1, Item No. 3-5, Buried conduit duct banks, pipe ducts, and piping.

3.8.4.3 *Regulatory Basis*

The relevant requirements of NRC regulations for this area, and the associated acceptance criteria, are given in NUREG-0800, Section 3.8.4, “Other Seismic Category I Structures,” Revision 2 and are summarized below. NUREG-0800, Section 3.8.4 also identifies review interfaces with other SRP sections.

1. GDC 1, “Quality Standards and Records,” as it relates to Seismic Category I structures being designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety function to be performed.

2. GDC 2, “Design Bases for Protection against Natural Phenomena,” as it relates to the ability of Seismic Category I structures, without loss of capability to perform their safety function, to withstand the effects of natural phenomena, such as earthquakes, tornadoes, floods, and the appropriate combination of all loads.

3. GDC 4, “Environmental and Dynamic Effects Design Bases,” as it relates to the protection of Seismic Category I structures against dynamic effects, including the effects of missiles, pipe whipping, and discharging fluids, that may result from equipment failures and from events and conditions outside the nuclear power unit.

4. GDC 5, “Sharing of Structures, Systems, and Components,” as it relates to safety-related structures not being shared among nuclear power units, unless it can be

shown that such sharing will not significantly impair their ability to perform their safety functions.

5. 10 CFR Part 50, “Domestic Licensing of Production and Utilization Facilities,” specifically, 10 CFR 50.55a, “Codes and Standards,” as they relate to codes and standards.

6. 10 CFR Part 50, Appendix B, “Quality Assurance Criteria for Nuclear Power Plants and Fuel Processing Plants,” as it relates to the quality assurance criteria for nuclear power plants.

Acceptance criteria adequate to meet the above requirements include:

1. RG 1.69, “Concrete Radiation Shields for Nuclear Power Plants,” December 1973, as it relates to the design requirements for concrete radiation shields in nuclear power plants.

2. RG 1.91, “Evaluations of Explosions Postulated to Occur on Transportation Routes Near Nuclear Power Plants,” February 1978, Revision 1, as it relates to evaluations of nuclear power plant structures affected by nearby explosions postulated for transportation routes.

3. RG 1.115, “Protection Against Low-Trajectory Turbine Missiles,” July 1977, Revision 1 as it relates to the evaluation of the effect of low trajectory turbine missiles on structures.

4. RG 1.127, “Inspection of Water-Control Structures Associated with Nuclear Power Plants,” March 1978, Revision 1, as it relates to the inservice inspection and surveillance program for water-controlled structures associated with emergency cooling water systems or flood protection of nuclear power plants.

5. RG 1.142, “Safety-Related Concrete Structures for Nuclear Power Plants (Other Than Reactor Vessels and Containments),” November 2001, Revision 2, as it relates to the design requirements for safety-related concrete structures, excluding concrete containments.

6. RG 1.143, “Design Guidance for Radioactive Waste Management Systems, Structures, and Components Installed in Light-Water-Cooled Nuclear Power Plants,” November 2001, Revision 2, as it relates to the design requirements for radioactive waste management structures.

7. RG 1.160, “Monitoring the Effectiveness of Maintenance at Nuclear Power Plants” March 1997, Revision 2, as it relates to the monitoring and maintenance of safety-related structures.

8. RG 1.199, “Anchoring Components and Structural Supports in Concrete,” November 2003, as it relates to the design, evaluation and quality assurance of anchors (steel embedments) used for component and structural supports on concrete structures.

3.8.4.4 *Technical Evaluation*

The staff’s evaluation is based on a complete review of FSAR Tier 2, Sections 3.8.4.1 through 3.8.4.7, including all subsections.

This section describes the staff's review and evaluation of FSAR Tier 2, Section 3.8, against the guidance of SRP Section 3.8.4 to ensure that the applicant has adequately addressed identified technical issues.

Description of the Structures

The staff reviewed the descriptions of the other Seismic Category I structures to ensure that they contain sufficient information to define the primary structural aspects and elements that are relied upon to perform the safety-related functions of these structures. With the exceptions described below, the staff finds that the descriptions of the structures meet the applicable regulatory requirements and the regulatory guidance in SRP Acceptance Criteria 3.8.4.II.1.

In FSAR Tier 2, Section 3.8.4, the applicant did not describe the design of Radwaste Structures defined in RG 1.143. The staff noted, however, that FSAR Tier 2, Tables 3.2.2-1, "Classification Summary," and 3.7.2-29, "Seismic Structural Interaction Criteria for Building Structures," describe the NAB and the RWPB as Radwaste Structures, designed in accordance with the guidance for RW-IIa structures in RG 1.143. Since the RWPB and NAB are part of the U.S. EPR design certification and are designed in accordance with RG 1.143, in RAI 155, Question 03.08.04-1 and RAI 248, Question 03.08.04-7, the staff requested that the applicant provide design information for these structures and add this information to FSAR Tier 2, Section 3.8.4.

In a March 31, 2009, response to RAI 155, Question 03.08.04-1, the applicant confirmed that the RWPB and NAB are RW IIa structures. Furthermore, the applicant indicated that the RWPB and the NAB were not included in FSAR Tier 2, Section 3.8.4, because they are neither Seismic Category I nor safety-related structures.

In a November 18, 2009, response to RAI 248, Question 03.08.04-7, the applicant reiterated that the NAB and RWPB are not safety-related structures and, therefore, not included in FSAR Tier 2, Section 3.8.4. The applicant also indicated that the NAB and RWPB will be designed and constructed in accordance with RG 1.143, and that design details will be developed later in the design process.

SRP Acceptance Criteria 3.8.4.II.2 and SRP Evaluation Findings 3.8.4.IV.6 identify that RG 1.143 is applicable to the design of Radwaste Structures. In addition, RG 1.143 indicates that Radwaste Structures categorized as RW-IIa, such as the NAB and RWPB described above, have a "safety classification" of "High Hazard" facilities. As such, RG 1.143 refers to 10 CFR Part 50, Appendix B, and GDC 1, GDC 2, and GDC 60, "Control of Releases of Radioactive Materials to the Environment," for the proper design of these structures, to mitigate accidents, and to suitably control the release of radioactive materials. Therefore, the RAI responses did not provide sufficient information for the staff to assess whether the technical issues described in the RG were being addressed in the structural design.

During the April 26-30, 2010, onsite audit, the applicant provided additional clarifications regarding its design approach for the NAB and RWPB. These can be summarized as follows: (1) The NAB is designed as a Seismic Category II structure, using full SSE loads; (2) the distance between the NAB and adjacent Seismic Category I structures is adequate to prevent interaction under full SSE loads; (3) the RWPB is designed in accordance with RG 1.143 Category RW-IIa, using one-half SSE loads; and (4) it will be qualitatively shown that there is no potential for adverse interaction between the RWPB and any other Seismic Category I structures under full SSE loads. The applicant documented the above information in an August 10, 2010, interim response to RAI 370, Question 03.07.02-64.

The staff concluded that the approach being used to design the Radwaste Structures is technically acceptable, because the NAB is designed as a Seismic Category II structure to prevent adverse interaction with adjacent Seismic Category I structures and the RWPB is designed in accordance with RG 1.143, Category RW-IIa.

The technical issues associated with the potential interaction under seismic loads of Non-seismic Category I structures, such as the NAB and RWPB, and adjacent Seismic Category I structures are addressed in Section 3.7.2 of this report in RAI 370, Question 03.07.02-64.

Based on the above discussion, the staff considers RAI 155. Question 03.08.04-1 and RAI 248, Question 03.08.04-7 with respect to the design of the NAB and the RWPB resolved, because the applicant has committed to the design of the NAB as a Seismic Category II structure and because the RWPB is designed in accordance with RG 1.143 Category RW-IIa. The potential for seismic interaction of these structures with adjacent Seismic Category I structures is addressed in RAI 370, Question 03.07.02-64, which is discussed in Section 3.7.2 of this report.

<u>Applicable Codes, Standards, and Specifications</u>

To ensure that the other Seismic Category I structures can perform their intended safety function, the staff verified that the applicable codes, standards, and specifications used in the design, fabrication, construction, testing, and ISI meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, and GDC 5 are in accordance with SRP Acceptance Criteria 3.8.4.II.2. With the exceptions described below, the staff finds that the applicable codes, standards, and specifications meet the regulatory requirements and regulatory guidance cited above.

The staff's review of FSAR Tier 2, Section 3.8.4, could not conclude whether the design and installation of concrete anchors were in accordance with (1) ACI 349-01, Appendix B, subject to the conditions and limitations specified in RG 1.199, and (2) IE Bulletin 79-02, Revision 2; or comply with SRP Acceptance Criteria 3.8.4.II.2. The staff was concerned whether the applicant's criteria resulted in the adequate design of structural connections that utilize these anchors, with the appropriate safety margins. The same concerns were also raised in the review of FSAR Tier 2, Section 3.8.3, "Concrete and Steel Internal Structures of Concrete Containment," Revision 0. Therefore, in RAI 155, Question 03.08.03-2, and RAI 283, Question 03.08.03-20, the staff requested that the applicant explain the criteria used for design and installation of concrete anchors. The resolution of these RAI issues is discussed in Section 3.8.3.4 of this report, under "Applicable Codes, Standards, and Specifications," where the staff concluded that RAI 155, Question 03.08.03-2 and RAI 283, Question 03.08.03-20 are resolved.

In FSAR Tier 2, Section 3.8.4.4.1, the applicant identified ACI 349-01 as the code used for the design and construction of reinforced concrete structures outside the containment. However, RG 1.142 endorses ACI 349-97, with certain regulatory positions. Since ACI 349-01 is not endorsed by RG 1.142, the staff was concerned with the use of a later edition of the ACI 349 code that has not been reviewed for applicability to nuclear sampling systems (NPPs). To ensure the safety of the reinforced concrete structures, the staff would need to review the applicability of ACI 349-01 on a case-by-case basis. The same issue was also raised in the review of FSAR Tier 2, Section 3.8.3.2, "Applicable Codes, Standards, and Specifications," Revision 0. In RAI 155, Question 03.08.03-3 and RAI 376, Question 03.08.03-21, the staff requested that the applicant: (1) Identify the differences between ACI 349-01 and ACI 349-97; (2) identify which differences were relaxed provisions; (3) provide the technical basis for using

any relaxed provisions; and (4) provide the technical basis for using an isolated provision from ACI 349-06: the shear strength reduction factor of 0.85, which is different from either ACI 349-01 or ACI 349-9**7.** The resolution of these RAI issues is discussed in Section 3.8.3.4 of this report, under "Applicable Codes, Standards, and Specifications," where the staff concluded that RAI 155, Question 03.08.03-3 and RAI 376, Question 03.08.03-21 are resolved.

<u>Loads and Load Combinations</u>

To ensure that the other Seismic Category I structures can perform their intended safety function, the staff verified that the loads and load combinations used in the design meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, and GDC 5; and are in accordance with SRP Acceptance Criteria 3.8.4.II.3. With the exceptions described below, the staff finds that the loads and load combinations meet the regulatory requirements and regulatory guidance cited above.

The staff notes that certain loads described in FSAR Tier 2, Section 3.8.4.3.1, were not described in sufficient detail to determine whether they comply with SRP Acceptance Criteria 3.8.4.II.3. The loads that needed additional clarification included floor live loads, live loads on buried piping and conduit, assumed ground temperature, and roof precipitation loads. The staff could not identify whether all required loads were being adequately taken into consideration, which could lead to a non-conservative structural design. Therefore, in RAI 155, Question 03.08.04-3 and RAI 335, Question 03.08.04-8, the staff requested that the applicant provide the magnitude of these loads, confirm that these were included in the structural design as applicable, and add this information to the FSAR.

In a May 29, 2009, response to RAI 155, Question 03.08.04-3, the applicant provided the following information:

1. The magnitude of live loads for concrete floors and steel platforms assumed in the design of RBIS, FB, SBs, ESWB, and EPGB, which is based on past experience at existing NPPs and is conservative relative to industry standards such as ASCE 7-05.

2. A description of the procedure for determining live loads on buried items such as pipe, pipe ducts, conduit, and duct banks, which follows the standard, "Guidelines for the Design of Buried Steel Pipe," (American Lifelines Alliance, 2001) and considers all relevant traffic and construction loads.

3. A clarification that the design of buried items is site-specific, to be performed by the COL applicant.

4. A ground temperature of 10 °C (50 °F) is assumed in the design.

5. The magnitude of 4.8 KPa (100 psf) roof precipitation load is used in the design of all Seismic Category I structures, conservatively assumed for both normal and extreme live load combinations.

6. An explanation that the assumed roof precipitation load is consistent with current NRC DC/COL-ISG-007, "Interim Staff Guidance on Assessment of Normal and Extreme Winter Precipitation Loads on the Roofs of Seismic Category I Structures," July 2009, and ASCE 7-05, Chapter 7 and is conservative for most of the central and eastern portions of the U.S.

The staff finds the above response items (1), (2) (3), (5), and (6) acceptable, because they meet SRP Acceptance Criteria 3.8.4.II.3, NRC DC/COL-ISG-007, and ASCE 7-05. Regarding item (4), the staff could not determine the soil depth at which the ground temperature was being specified nor the regions in the U.S. to which the stated ground temperature applies. In addition, the staff requested that the applicant add to the FSAR the procedure for determining live loads on buried items so it could be available to COL applicants.

In a March 12, 2010, response to RAI 335, Question 03.08.04-8, the applicant explained that the procedure for determining live loads on buried items would be added to the FSAR. The response also clarified that the assumed ground temperature of 10 °C (50 °F) corresponds to a soil depth of 3.05 m (10 ft), which is consistent with a ground surface temperature between 4 °C (40 °F) to 21 °C (70 °F), valid for the central and eastern portions of the U.S.

Subsequent to the above RAI responses, in its response to FSAR Tier 2, Chapter 2 RAI 453, Question 02.03.01-17, the applicant further clarified that the assumed 4.8 KPa (100 psf) roof precipitation load corresponds to a ground load of 6.8 KPa (143 psf), in accordance with ASCE 7-05.

The staff finds the May 29, 2009, response to RAI 155, Question 03.08.04 -3 and the March 12, 2010, response to RAI 335, Question 03.08.04-8 acceptable, because they meet SRP Acceptance Criteria 3.8.4.II.3, NRC DC/COL-ISG-007, and ASCE 7-05.

The staff verified that the changes proposed by the applicant were incorporated into FSAR Tier 2, Revision 2. Therefore, on the basis of the above discussion, the staff considers RAI 155, Question 03.08.04-3 and RAI 335, Question 03.08.04-8 resolved.

In FSAR Tier 2, Section 10.4.7.3, "Safety Evaluation," the applicant indicated that non-safety-related portions of FW piping located outside the structures may be impacted from external missiles. This appeared to also be the case for the MS piping and possibly other high energy lines. The staff was concerned with external missiles, which may cause direct damage to high energy lines that could result in pipe whip or jet impingement loads on safety-related SSCs. Therefore, in RAI 155, Question 03.08.04-2, the staff requested that the applicant clarify which Seismic Category I structures are susceptible to such loading conditions and explain how these structures are designed to withstand such loads.

In an April 30, 2009, response to RAI 155, Question 03.08.04-2, the applicant stated that the RSB and the SBs are the only Seismic Category I structures that are susceptible to pipe whip or jet impingement loads deriving from external missile damage to high energy piping in the open environment. Furthermore, these structures are designed to withstand high energy line break impact loads, as well as external design-basis and beyond-design-basis missiles and blast loads, which are significantly larger than the energy associated with pipe-break loads.

The staff finds the above response acceptable, because it identified which Seismic Category I structures are susceptible to pipe whip or jet impingement loads from external missile damage to high energy piping in the open environment, and it confirmed that these structures are designed to withstand such loads, in conformance with SRP Acceptance Criteria 3.8.4.II.3. On this basis, the staff considers RAI 155, Question 03.08.04-2 resolved.

In FSAR Tier 2, Section 3.8.4.3.2, the applicant described the load combinations used for other Seismic Category I concrete and steel structures. The staff reviewed this section against SRP Acceptance Criteria 3.8.4.II.3 and identified the following inconsistencies: (1) For reinforced concrete elements, pipe reaction loads (R_o) were not included as required by ACI 349-01;

(2) deviations from ANSI/AISC N690-1994, including S2004, related to the plastic section modulus of steel shapes were noted; (3) it was unclear to the staff whether all footnotes to ANSI/AISC N690-1994, including S2004, Table Q1.5.7.1, were applicable to the FSAR; and (4) it was unclear to the staff whether a load factor of zero was applicable to loads that reduce the effects of other loads in certain load combinations, unless the former are always known to be present together with the latter. The staff was concerned that all required loads and load combinations were not being adequately considered, which could lead to a non-conservative structural design. The same concerns were also raised in the review of FSAR Tier 2, Section 3.8.3.3.2. Therefore, in RAI 155, Question 03.08.03-5 and RAI 376, Question 03.08.03-22, the staff requested that the applicant clarify these issues. The resolution of these RAIs is discussed in Section 3.8.3.4 of this report, under "Loads and Load Combinations," where the staff concluded that RAI 155, Question 03.08.03-5, and RAI 376, Question 03.08.03-22 are resolved.

Design and Analysis Procedures

The staff reviewed the design and analysis procedures used for the other Seismic Category I structures to ensure that they meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, and GDC 5, and are in accordance with SRP Acceptance Criteria 3.8.4.II.4. The staff also reviewed the FSAR to establish that the design is essentially complete, as required by 10 CFR 52.47(c). With the exceptions described below, the staff finds that the design and analysis procedures meet the regulatory requirements and regulatory guidance cited above.

In FSAR Tier 2, Section 3.8.4.4.2, the applicant indicated that separation gaps are specified between Seismic Category I structures and adjacent Non-seismic Category I structures to allow for structural movements during seismic events, containment pressurization, missile strikes, explosions, and other loading conditions. Adjacent Non-seismic Category I structures that could have potential adverse interaction with Seismic Category I structures are the NAB, the Access Building, and the Turbine Building, which the applicant has designated as Seismic Category II structures. In addition, the exterior walls and roofs of the hardened Safeguard Buildings 2 and 3, RSB, and the FB are designed to be independent of their internal structures, because there is no physical connection of internal walls and slabs in these structures with the outside walls and roof. However, the staff could not identify whether the magnitude of the specified gaps was sufficient to accommodate all possible structural movements.

Since insufficient separation gaps could result in undesirable impact of adjacent structures from the various loading conditions mentioned above, in RAI 155, Question 03.08.04-4, the staff requested that the applicant: (1) Specify the gaps provided between all Seismic Category I structures and adjacent Non-seismic Category I structures (i.e., NAB, ACB, and TB), and compare these to calculated structural responses; and (2) specify the gaps provided between the hardened Seismic Category I structures noted above (i.e., Safeguard Buildings 2 and 3, RSB, and FB) and their internal structures, and compare these to calculated structural responses.

In a May 29, 2009, response to RAI 155, Question 03.08.04-4, the applicant indicated that the gaps provided between Seismic Category I structures and adjacent Non-seismic Category I structures range between 0.30 m (12 in.) (gap between ACB and Safeguard Building 3 shield structure) and 0.45 m (18 in.) (gap between NAB and FB shield structure, and between NAB and Safeguard Building 4). In addition, the applicant indicated that maximum predicted seismic "interaction distance" between Seismic Category I structures and adjacent Non-seismic

Category I structures was initially estimated at 0.28 m (11.2 in.), including maximum tilt due to differential settlement. The applicant also indicated that the maximum interaction distance corresponding to the various non-seismic loading conditions is enveloped by the seismic load case.

Regarding the gaps between hardened Seismic Category I structures (i.e., Safeguard Buildings 2 and 3, RSB, and FB) and their internal structures, the applicant indicated that these range between 0.6 m (23.62 in.) and 2.0 m (78.74 in.), compared to the maximum predicted seismic interaction distance of 45 mm (1.78 in.), which is a very small value due to the high lateral stiffness of these structures and the fact that they share a common basemat.

The above information addressed the staff's concerns regarding gaps provided between hardened Seismic Category I structures and their internal structures, because the maximum interaction distances are very small compared to the gaps provided. However, the staff noted that the maximum interaction distance between Seismic Category I structures and adjacent Non-seismic Category I structures was initially estimated at 0.28 m (11.2 in.), which is relatively close to the 0.30 m (12 in.) gap provided between ACB and Safeguard Building 3 shield structure. Therefore, in follow-up RAI 335, Question 03.08.04-9, the staff requested that the applicant describe in detail the seismic analyses used to calculate the interaction distances, and clarify whether the FSAR includes sufficient information to assure the gaps assumed in design can be verified by the COL applicant.

During the April 26-30, 2010, onsite audit, the applicant provided additional clarifications regarding its design approach for the NAB, TB, and ACB. These can be summarized as follows: (1) NAB, TB, and ACB are designed as Seismic Category II structures (for seismic analysis: NAB to certified seismic design response spectra, TB and ACB to site-specific SSE); (2) the distance between NAB, TB, and ACB and adjacent Seismic Category I structures is adequate to prevent adverse interaction under seismic loads; and (3) TB and ACB are site-specific structures and their design criteria will be placed in brackets in the FSAR. The applicant documented the above information in an August 10, 2010, response to RAI 370, Questions 03.07.02-64 (interim response) and 03.07.02-65.

In a June 15, 2011, response to RAI 335, Question 03.08.04-9, the applicant indicated that the interaction distances provided in the May 29, 2009, response to RAI 155, Question 03.08.04-4 are no longer valid. The applicant further indicated that the seismic analyses used to compute the interaction distances between the NAB and adjacent Seismic Category I structures, specifically the NI structures, are described in detail in the August 10, 2010, response to RAI 370, Question 03.07.02-64. These analyses consider lateral deformations of the NI structures, rigid-body displacements from the NI stability analyses, as well as due to tilt settlement of the NI structures, and NAB sliding and overturning.

Regarding the interaction distances between the ACB and TB and adjacent Seismic Category I structures, the applicant stated that the ACB and TB are site-specific structures and, as such, are the design responsibility of the COL applicant. FSAR Tier 2, Table 1.8-2, COL Information Item 3.7-2 requires that the COL applicant provide the site-specific separation distances for the ACB and TB. In addition, FSAR Tier 2, Table 1.8-2, COL Information Items 3.7-7 and 3.7-8 require the COL applicant to demonstrate that the response of the ACB and TB to an SSE event will not impair the ability of Seismic Category I SSCs to perform their design-basis safety functions. The applicant also referred to its August 10, 2010, response to RAI 370, Question 03.07.02-65 for additional information regarding the ACB and TB.

The applicant’s response to RAI 370, Question 03.07.02-65 addressed the staff’s concerns regarding separation gaps provided between ACB and TB and adjacent Seismic Category I structures, because these are site-specific structures and a licensing mechanism has been provided to ensure that adequate gaps are provided by the COL applicant to prevent adverse interaction under seismic loads. The remaining technical issues regarding the potential seismic interaction between the NAB and adjacent Seismic Category I structures are discussed in Section 3.7.2 of this report.

Based on the above discussion, RAI 155, Question 03.08.04-4 and RAI 335, Question 03.08.04-9 with respect to separation gaps are resolved. The potential for seismic interaction between the NAB and adjacent Seismic Category I structures is addressed under RAI 370, Question 03.07.02-64. The demonstration that the response of the ACB and TB to an SSE event will not impair the ability of Seismic Category I SSCs to perform their design-basis safety functions is addressed under RAI 370, Question 03.07.02-65. The resolution of these RAIs is discussed in Section 3.7.2 of this report.

In FSAR Tier 2, Sections 3.8.4.1.2 and 3.8.4.4.2, the applicant indicated that FSAR Tier 2, Section 9.1.2, “New and Spent Fuel Storage,” addresses the design of fuel storage racks. However, in FSAR Tier 2, Section 9.1.2, the applicant indicated that the design of the spent fuel racks is the responsibility of the COL applicant, and provides only general statements regarding design loads (FSAR Tier 2, Section 9.1.2.1, “Design Bases,” Item 10) and required dynamic and stress analyses (FSAR Tier 2, Section 9.1.2.3, “Safety Evaluation”).

Since it appeared that the design of the fuel storage racks had not been completed, the staff could not determine how the applicant had estimated the loads imposed by the fuel storage racks on the spent fuel pool (SFP). In addition, it was unclear to the staff how the hydrodynamic pressures due to the seismic excitation of the water were being considered in the applicant’s equivalent-static seismic analysis of the SFP. The staff was concerned that all required loads were not being adequately taken into consideration, which could lead to a non-conservative structural design of the SFP. Therefore, in RAI 155, Question 03.08.04-5 and RAI 335, Question 03.08.04-10, the staff requested that the applicant clarify (1) whether the spent fuel racks are free standing or anchored to the SFP, (2) whether the analysis and design of the spent fuel racks are performed in accordance with the guidance provided in SRP Section 3.8.4, Appendix D, (3) how the loads imposed by the spent fuel racks on the SFP have been determined, and (4) the procedure used to determine the equivalent-static loads imposed on the SFP bottom slab and walls due to the spent fuel racks and the seismic excitation of the water.

In a May 29, 2009, response to RAI 155, Question 03.08.04-5, and the applicant indicated that the spent fuel racks are free standing. The applicant also indicated that the FB, including the SFP, was identified as a critical section and that design details would be provided under a response to RAI 155, Question 03.08.04-6 (critical sections are discussed below).

In a May 25, 2011, response to RAI 335, Question 03.08.04-10, the applicant indicated that the detailed analysis and design of the fuel racks is provided in Technical Report TN-Rack.0101, “U.S. EPR New and Spent Fuel Storage Rack Technical Report.” This report also includes a coupled fluid-structure time-history analysis of the fuel racks using the LS-DYNA code. This analysis determined the hydrodynamic pressures and loads imposed by the fuel racks on the SFP slab and walls during an SSE. The staff notes that the fuel racks do not generate impact loads on the perimeter SFP walls and have a minimum factor of safety against tipping of 1.1. The staff evaluation of Technical Report TN-Rack.0101 is given below in the subsection under the heading “Structural Evaluation of U.S. EPR Fuel Storage Racks.”

Regarding the procedure used to determine the equivalent-static loads imposed on the SFP slab and walls, used in the design of these structural elements, the applicant stated that these were based on conservative estimates of hydrodynamic pressures and loads imposed by the fuel racks. The hydrodynamic pressures were calculated following the standard methodology described in TID-7024, "Nuclear Reactors and Earthquakes." The loads imposed by the fuel racks were calculated based on the estimated pool fuel capacity and previous NPP rack analytical data. Bounding ZPAs obtained from the seismic analysis of the Nuclear Island were used in both cases.

Since the estimated equivalent-static loads described above were calculated prior to the completion of the Technical Report TN-Rack.0101, the applicant performed reconciliation with the LS-DYNA time-history analysis included in the report. The applicant determined that the estimated equivalent-static loads, applied as uniform pressures on the SFP slab and walls, were significantly greater than the peak pressures (hotspots) computed in the LS-DYNA time-history analysis, and are thus acceptable. The applicant also indicated that impact loads identified in Technical Report TN-Rack.0101 are used in the SFP design for local effects such as bending and punching shear.

The staff finds the above RAI responses acceptable because they comply with SRP Acceptance Criteria 3.8.4.II.4.G, applied accepted methodologies such as TID-7024, and provided conservative estimates of equivalent-static loads applicable to the design of SFP slab and walls. On this basis, the staff considers RAI 155, Question 03.08.04-5 resolved, while **RAI 335, Question 03.08.04-10 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed changes.

During the January 26-30, 2009, onsite audit, the staff could not locate documented criteria for the selection of critical sections corresponding to other Seismic Category I structures. The identification, analysis, and design of critical sections provides reasonable assurance that the structural design of Seismic Category I structures is essentially complete, as required by 10 CFR 52.47(c). The same concerns were also identified in the review of FSAR Tier 2, Sections 3.8.1, 3.8.3, and 3.8.5, "Foundations." Therefore, in RAI 155, Questions 03.08.01-20, 03.08.01-24, and 03.08.04-6, the staff requested that the applicant address the issue regarding the identification, analysis, and design of critical sections. RAI 155 Question 03.08.01-20 is resolved. **RAI 155, Questions 03.08.01-24, and 03.08.04-6 are being tracked as open items** pending completion of the critical section design. The resolution of these RAI issues is discussed in Section 3.8.1.4 of this report, under "Design and Analysis Procedures."

In FSAR Tier 2, Section 3.8.4.4, the applicant indicated that the three components of seismic loads are combined using the SRSS method or the 100-40-40 percent rule described in ASCE 4-98. As described in RG 1.92, "Combining Modal Responses and Spatial Components in Seismic Response Analysis," July 2006, Revision 2, the SRSS method and the 100-40-40 percent rule are two different acceptable methodologies that can be used to combine the seismic analysis responses of the structure from each of the three orthogonal directions. However, the staff has also noted that the guidance provided in RG 1.92 only refers to a single design parameter. For a design involving multiple parameters, the implementation of the 100-40-40 rule is rather complex, and the FSAR description was unclear to the staff with respect to how the multiple design parameters were considered in the application of the 100-40-40 rule. The same issue was also raised in the review of FSAR Tier 2, Section 3.8.3.4.4, "Seismic and Other Dynamic Analyses and Design." Therefore, in RAI 155, Question 03.08.03-10 and RAI 376, Question 03.08.03-24, the staff requested that the applicant provide a detailed explanation of its implementation of the 100-40-40 rule, including quantitative demonstration

which shows that its implementation conforms to the guidance described in RG 1.92, as well as its applicability to nonlinear analysis procedures. The resolution of RAI 155, Question 03.08.03-10 and RAI 376, Question 03.08.03-24 is discussed in Section 3.8.3.4 of this report, under "Design and Analysis Procedures," where the staff concluded that these questions are resolved.

In FSAR Tier 2, Section 3.8.4, Revision 0, and Appendix 3E, the applicant described the FE models used for the structural analysis and design of other Seismic Category I structures, utilizing the ANSYS computer code. However, the staff could not determine the acceptability of these FE models without additional descriptive information. In particular, the staff was concerned that without a sufficiently refined FE discretization of the structures, the member forces, strains, and stresses determined from the computer analysis may not be sufficiently accurate, which could ultimately lead to an non-conservative design. The same issue was also raised in the review of FSAR Tier 2, Section 3.8.3. Therefore, in RAI 155, Question 03.08.03-16, the staff requested that the applicant provide additional information on the FE models used in the analysis of other Seismic Category I structures. The staff considers RAI 155, Question 03.08.03-16 resolved. The resolution of this RAI is discussed in Section 3.8.3.4 of this report, under "Design and Analysis Procedures."

Structural Acceptance Criteria

To ensure that the other Seismic Category I structures, as designed, can perform their intended safety function, the staff verified that the structural design acceptance criteria meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, and GDC 5, and are in accordance with SRP Acceptance Criteria 3.8.4.II.5. Except for the items described below, the staff finds that the structural acceptance criteria meet the regulatory requirements and regulatory guidance cited above.

In FSAR Tier 2, Section 3.8.4.1, the applicant indicated that Seismic Category II miscellaneous structures such as platforms, stairs, guard rails, and other ancillary items are designed to prevent adverse impact on the Seismic Category I structures in the event of an SSE. However, the staff could not identify the analysis and design methods and acceptance criteria for Seismic Category II SSCs. The same issue was also raised in the review of FSAR Tier 2, Section 3.8.3.1, "Description of the Internal Structures." Therefore, in RAI 155, Question 03.08.03-17, the staff requested that the applicant describe the analysis methods and acceptance criteria that have been implemented for the seismic design of Seismic Category II miscellaneous structures inside containment, and other Seismic Category II structures covered in FSAR Tier 2, Sections 3.8.3 and 3.8.4.

In a July 31, 2009, response to RAI 155, Question 03.08.03-17, the applicant provided general information regarding acceptance criteria used in the analysis and design of Seismic Category II SSCs. As discussed in Section 3.8.3 of this report, the staff determined that the information provided was not sufficiently detailed to assess the adequacy of its implementation.

Since the technical issue described above is related to the potential seismic interaction of Seismic Category II miscellaneous structures with Seismic Category I structures, the staff's review of FSAR Tier 2, Section 3.7.3, "Seismic Subsystem Analysis," also identified the same issue. In RAI 370, Question 03.07.03-38 and RAI 463, Question 03.07.03-40, the staff requested that the applicant address the potential impact of these Seismic Category II miscellaneous structures on the Seismic Category I structures under SSE load. Therefore, RAI 155, Question 03.08.03-17 was superseded by the above RAI questions requested in the review of FSAR Tier 2, Section 3.7.3. The staff considers the FSAR Tier 2, Section 3.7.3

RAI issues resolved, and, therefore, consider RAI 155, Question 03.08.03-17 resolved. The detailed resolution of these RAIs is discussed in Section 3.7.3 of this report.

Materials, Quality Control, and Special Construction Techniques

The staff reviewed the materials, quality control, and special construction techniques used for the other Seismic Category I structures to ensure that they meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, and GDC 5, and are in accordance with SRP Acceptance Criteria 3.8.4.II.6. Except for the item described below, the staff finds that the materials, quality control, and special construction techniques meet the regulatory requirements and regulatory guidance cited above.

In FSAR Tier 2, Section 3.8.4.6.3, the applicant provided a brief description of modular construction methods and composite-type structural members used in the U.S. EPR standard design. The staff was concerned about the structural design, fabrication, configuration, layout, and connections of designated modules which may be new to U.S. NPPs and may require extensive case-by-case review. The same issue was also identified in the review of FSAR Tier 2, Section 3.8.3.6.5, “Special Construction Techniques.” Therefore, in RAI 155, Question 03.08.03-13, the staff requested that the applicant provide a more detailed description of each specific type of module or composite member used in the U.S. EPR standard design, and to provide a description of the analysis and design approach used for each type of module and composite member. The staff considers RAI 155, Question 03.08.03-13 resolved**.** The resolution of this RAI is discussed in Section 3.8.3.4 of this report, under “Materials, Quality Control, and Special Construction Techniques.”

Testing and Inservice Surveillance Requirements

To ensure that the other Seismic Category I structures can perform their intended safety function, the staff verified that the testing and ISI surveillance requirements meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, and GDC 5; and are in accordance with SRP Acceptance Criteria 3.8.4.II.7. With the exception described below, the staff finds that the testing and ISI surveillance requirements meet the regulatory requirements and regulatory guidance cited above.

In FSAR Tier 2, Section 3.8.4.7, the applicant indicated that monitoring and maintenance of structures is performed in accordance with RG 1.160, without reference to 10 CFR 50.65. The same issue was raised in the review of FSAR Tier 2, Section 3.8.3.7, “Testing and Inservice Inspection Requirements.” In RAI 155, Question 03.08.03-14, the staff requested that the applicant clarify why monitoring and maintenance of structures is not performed in accordance with the requirements of 10 CFR 50.65 and supplemented with the guidance in RG 1.160. The resolution of this question is discussed in Section 3.8.3.4 of this report, under “Testing and Inservice Surveillance Requirements,” where the staff concluded that RAI 155, Question 03.08.03-14 is resolved.

Structural Evaluation of U.S. EPR Fuel Storage Racks

The analysis and design of the EPR new and spent fuel storage racks are contained in the AREVA Transnuclear, Inc. Report TN-Rack.0101, Revision 0, hereinafter referred to as the TR for fuel racks. The TR for fuel racks documents the structural analysis, thermal hydraulics analysis, criticality evaluation, qualification and testing, and material selection of the racks. This report section describes the staff’s evaluation of the structural adequacy of the fuel racks to support and protect the housed fuel assemblies as documented in Chapters 1, 2, and 3 of the

TR for fuel racks. The structural adequacy of the reinforced concrete pool structure is not within the scope of this TR for fuel racks, but is evaluated above as part of other Seismic Category I structures in this section of the report.

Subsequent to the issuance of FSAR Tier 2, Revision 2, the applicant decided to submit the TR for fuel racks to the staff for review and to incorporate the TR into the FSAR by reference in FSAR Tier 2, Chapter 9. Markup changes to the FSAR were provided, along with the TR for fuel racks, for staff review in the applicant's December 8, 2009, letter to the NRC.

The staff's determination of the structural adequacy of the fuel racks is based on reviewing the analysis and design of the racks in accordance with the applicable regulatory requirements and regulatory guidance, as well as applicable codes and standards, described in Section 3.8.4.3 of this report and as discussed below. The staff review included evaluating the analysis and design of the racks in accordance with the criteria presented in SRP Section 3.8.4, Revision 2, Appendix D, entitled "Guidance on Spent Fuel Pool Racks"; the provisions of the ASME Code, Section III, Division 1, Subsection NF; and the guidance in applicable NRC Regulatory Guides such as RG 1.61, RG 1.92, and RG 1.124. This section describes the staff's review of the TR for fuel racks including technical issues as discussed below.

Description of Fuel Racks

As described in the TR for fuel racks, the racks which are located in the spent fuel storage pool consist of an assembly of tubes with either a flux trap (for Region 1 racks) or neutron absorber plates sandwiched between the tubes (for Region 2 racks). The tube assembly is supported by a stainless steel frame structure that includes four vertical corner angles, base plate, several horizontal bands, and top and bottom support grid assemblies that are welded to the steel frame. The two grid assemblies provide restraint laterally to the enclosed tube assembly, while the top grid assembly also provides restraint in the vertical direction. A total of seventeen racks, which vary in size from 7 X 8 to 10 X 9 individual tubes for each rack are placed in the spent fuel pool. The racks are supported by six legs which have bearing plates that provide an interface with the spent fuel pool liner plate. The storage racks are free standing on the spent fuel pool floor. Therefore, they may slide, tip, and/or rotate from their position due to a seismic event. Due to gaps between the racks and between the racks and the pool walls, under a seismic event, the gaps may close and the racks may impact each other and may impact the pool walls. In addition, the fuel assemblies may rattle inside the tubes and impact the tube walls. The racks are submerged in the spent fuel pool by a minimum of 12.06 m (39' ft 7 in.).

The new fuel racks in the new fuel storage vault use the same Region 1 racks that are stored in the spent fuel pool. Only two racks (7 X 8 and 8 X 8) are placed in the vault, which are supported laterally at the top and bottom to the concrete walls and vertically by the concrete floor. Since these racks will contain new fuel, they are not submerged in water.

The design and fabrication of the racks is performed in accordance with the ASME Code, Section III, Division 1, Subsection NF, Class 3 Component supports. The rack is designed for dead load, live load, thermal, seismic, accidental drop load, and postulated stuck fuel assembly upward force. For seismic analysis, a series of nonlinear time history dynamic analyses are performed using the whole pool multi-rack model. This multi-rack model includes all of the individual racks, each of which is a simplified representation of an individual rack. The simplified rack model was benchmarked against a detailed finite element model of a single rack structure. The finite element model included the water in the pool, considered hydrodynamic coupling between the adjacent racks and between the racks and the pool wall, and was analyzed for different coefficient of friction values between the rack legs and the pool floor.

A separate detailed finite element model of two governing racks was used to perform the detailed stress analysis and design using the response of the racks from the whole pool multi-rack model.

The staff reviewed the description of the fuel racks provided in the TR for fuel racks to ensure that it contains sufficient information to define the primary structural aspects and elements that are relied upon for the racks to perform their intended function. With the exceptions described below, the staff finds that the description meets the applicable regulatory requirements, regulatory guidance, and the ASME Code.

The staff review of the TR for fuel racks, Chapters 1, 2, and 3, and the review of the proposed markups for FSAR Tier 2, Chapter 9, "Auxiliary Systems," identified that some figures and descriptive information for the fuel racks were provided. However, the staff could not find detailed descriptive information including plans and sections showing the racks, pool walls, liner, and details of the fuel-handling system (for review of the parameters associated with the postulated drop accident). This information is needed to permit the staff to evaluate the load path for the various structural elements, to ensure that the mathematical models capture these key structural elements, and to assess the design of these members. Therefore, in RAI 445, Question 03.08.04-15, the staff requested that the applicant provide the items listed below to conduct an adequate review of the structural/seismic analysis and design of the fuel racks.

1. Sketches to show all the major structural features should be provided with sufficient information to describe the racks, including the steel crates, rack tube walls, base plate, support legs, bearing pads, pool liner, vertical angles, bottom bands, grid assemblies, neutron absorber sheathing, welds connecting these parts, and any other elements in the load path of the racks. These sketches should indicate related information including the north arrow, cutouts, dimensions, material thicknesses, and weld size/thickness, as well as gaps between the fuel to cell walls, rack to rack, rack to pool walls, rack to equipment area in both horizontal directions, and between the tubes to steel crate in the horizontal and vertical directions.

2. Horizontal support details for the new fuel racks are not described and should be provided.

3. For gaps provide the following: (1) Gaps between racks, rack to wall, and rack to equipment area boundary should be presented in FSAR Tier 2, Section 9.1, Figure 9.1.2-7; (2) clarify whether there is any gap between the two racks in the new fuel storage area, and what the basis is for concluding that the anchorage of these racks eliminates amplification of the seismic response as stated in the TR for fuel racks, Section 2.2.1; (3) identify the gap tolerances for each of the gaps between the fuel to cell wall, rack to rack, rack to equipment area, and rack to wall; (4) clarify whether any studies were done for different initial gap conditions considering the potential tolerances, and if not, explain why; and (5) identify the combined license application (COLA) requirements in the FSAR to ensure that the assumed gaps (considering tolerances) will be maintained throughout the licensing period, especially after an earthquake.

4. A sketch with details of the fuel-handling system, including the structural elements in the fuel handling load path (e.g., the sling, lifting beam, cables, and point of connection to the racks) should be provided to facilitate the review of the postulated drop accident parameters. As described in SRP Section 3.8.4, Appendix D.I.1.B, this information is needed to ensure the integrity of the racks, fuel pool, and fuel pool liner, for a postulated fuel-handling accident.

To address the above items, the applicant provided information which was discussed during the April 27-29, 2011, structural audit. The information discussed, the staff evaluation, and the applicant's proposed resolution are described below.

1. The applicant indicated that details of the rack design are shown in new figures for Region 1 and Region 2 racks with part numbers for the structural elements and corresponding tables identifying the parts. However, from this information the staff could not determine the structural configuration and load paths for the rack foot assembly subcomponents, how the poison plates are restrained, and the gaps between the racks and between the fuel assemblies and the tubes. In order to understand the load path through the rack structural members, the applicant indicated that details will be provided for the rack support legs in the response to this RAI question and in the TR for fuel racks. To demonstrate that the poison plates are adequately restrained in the racks, in both horizontal and vertical directions, the applicant indicated that a sketch/figure and a description will be provided. The gaps between the spent fuel racks and the equipment area and between the fuel assembly and tube walls will also be provided. The TR for fuel racks, Figures 1-2 and 1-3 and FSAR rack layout figures, in FSAR Tier 2, Chapter 9 will be revised to show the required boundary of the equipment area and gaps between the racks and the equipment area.

2. The applicant provided some descriptive information that explained that the new fuel racks are supported at the top and bottom to the adjacent concrete walls by using bolts cast into the concrete at each corner of the racks. However, the staff determined that because of the large gaps between the racks and the concrete walls and the relatively thin structural elements of the rack structure, information should be provided to show how the bolts and any other associated steel elements (e.g., shims, nuts, etc.) are used for the connection to the racks and how they transfer the localized seismic induced loads from the racks to the vault concrete structure. To demonstrate that the racks in the new fuel storage (NFS) area are adequately restrained, the applicant indicated that a sketch will be provided that will show the details of the lateral connections from the racks to the concrete walls at the top and bottom of the racks. Conceptual design criteria to adequately establish the load paths will be provided. In the TR for fuel racks, Figure 1-3 and corresponding figures in FSAR Tier 2, Chapter 9, will be revised to show the edge of the concrete walls in the NFS area. To understand the structural connection between the two racks in the NFS area, a sketch will also be provided showing the stainless steel straps at the top of the racks.

3. To address the question about the gaps, the applicant indicated that FSAR Tier 2, Figure 9.1.2-7, "Spent Fuel Storage Pool Layout," will be revised to identify the gaps between the racks, racks to wall, and racks to equipment area. Since the figure in the FSAR will be revised to incorporate the gaps, the staff finds this acceptable pending staff review of the revised figure.

   The applicant provided information that indicates that the two new fuel racks are butted together with no gap, leaving the tops of the racks separated by the structural set back at the baseplates. The applicant indicated that revised figures for the new fuel racks showing this information will be provided in the TR for fuel racks and FSAR Tier 2; therefore, the staff finds this acceptable, pending staff's review of the revised figures.

   The question in RAI 445, Question 03.08.04-15 is discussed above, regarding the basis for concluding that the anchorage of these racks eliminates amplification of the seismic

response is addressed separately in this report under the heading below titled, “Design and Analysis Procedures for Fuel Racks.”

The applicant indicated that the gap tolerances for installation of the racks are plus or minus 6.35 mm (0.25 in.). Following a seismic event, the minimum separation between fuel cells is over 7.62 cm (3 in.), and, therefore, no COLA requirements are needed in the FSAR to verify that the assumed gaps considering tolerances will be maintained. The staff review of this information determined that the gap tolerances requested for the gaps between the fuel to cell wall, rack to wall, and rack to equipment area should be provided along with an explanation of how these tolerances were considered in the seismic analysis of the racks. In addition, the RAI response should address the concern that following a seismic event, the locations of the racks would no longer be consistent with the assumed locations and gaps of the racks used in the existing seismic qualification design basis of the racks.

The applicant indicated that the FSAR will be revised to include the need to evaluate the spent fuel rack positions following an earthquake at the plant to avoid an unanalyzed condition of the racks. For the racks in the spent fuel pool, new seismic analyses will be performed based on the extreme values for the gaps considering the permissible tolerances.

4. The spent fuel machine and the auxiliary crane are utilized for handling of the fuel assemblies. In the case of the auxiliary crane, the applicant indicated that this crane is designed as a single failure-proof crane; therefore, a drop of a fuel assembly over the spent fuel storage racks is not postulated. Also, the fuel assembly handling tools, used with this crane are designed to hold the load during normal conditions plus the SSE. The staff determined that this does not provide a technical basis for not evaluating the fuel assembly handling tools for a fuel drop loading case as described in SRP Section 3.8.4, Appendix D.I.4.

   The applicant indicated that when using the auxiliary crane to handle fuel assemblies, the maximum accidental drop height will be provided. The technical basis for accepting this drop height, in terms of the adequacy of the fuel racks or commitment that this drop height will be less than or equal to the drop height from the spent fuel machine (which was used in the fuel drop analysis), will be provided. Also, procedure requirements in FSAR Tier 2, Chapter 9, will be identified or revised to (1) indicate that the use of the auxiliary crane, to transfer new fuel to the new fuel elevator in the spent fuel pool area, will preclude travel over the spent fuel pool, and (2) handling of any fuel assembly over the spent fuel racks will be limited to the maximum accidental drop height discussed above. To clarify the analysis performed for lifting of the racks, the TR for fuel racks, Section 3A.3 will be expanded to explain how the lifting assembly will be used to move the racks.

In a July 8, 2011, interim response to RAI 445, Question 03.08.04-15, the applicant provided information to address the items requested in the RAI and the items discussed during the audit. The information provided in the RAI response and the staff evaluation is provided below.

1. The applicant provided details of the rack design in new figures showing a Region 1 and Region 2 rack with part numbers for the structural elements and corresponding tables identifying the individual parts. Separate details were provided for the rack support legs and a description and figure were presented to explain how the poison plates are adequately restrained in the racks in all directions. The RAI response also provided

information on the gaps between the racks, between the racks and the pool walls, and between the racks and the equipment area in the pool.

The staff concluded that the details and figures provided are acceptable, because the figures provide the requested design information needed to understand the load path through the spent fuel rack structural elements. However, the gaps between fuel racks shown in the markup for FSAR Tier 2, Figure 9.1.2-7 and the TR for fuel racks, Figure 1-2 should be revised to identify a 1.27 cm (one-half in.) typical gap rather than a 1.27 cm (one-half in.) minimum typical gap to be consistent with the seismic analysis of the racks, the gaps should be identified from the edge of the rack baseplates, and the gap between the fuel assemblies and the rack tubes was not provided in the RAI response. Therefore, the staff requested that the applicant provide this information.

2. The applicant provided a description and markup figures for the TR for fuel racks and the FSAR that show that the two fuel racks in the NFS area are restrained at the top and bottom to the concrete walls and supported on the concrete floor, are butted to each other at the rack baseplate, and are also connected to each other at the top of the racks using stainless steel straps. The staff finds this information acceptable, because it shows how the new fuel racks are restrained for horizontal and vertical seismic loads.

3. The applicant provided proposed wording for FSAR Tier 2, Section 3.7.4, regarding post-earthquake activity for the racks, which the staff finds acceptable, because it indicates that the racks will be repositioned or an evaluation of the as-found condition will be performed following an earthquake. The RAI response also indicated that a study will be performed to determine the effects of erection tolerances on the gaps used in the seismic analyses. The results of this study will be reported in the TR for fuel racks when completed. The staff determined that the additional analysis to consider the effects of the tolerances would address this question.

4. The applicant’s response to RAI 445, Question 03.08.04-15 also clarified that the maximum vertical drop height for the fuel assembly, while it is being handled by the auxiliary crane over the spent fuel storage racks, will not be more than the drop height used for the inadvertent drop of a fuel assembly while handled by the spent fuel machine. However, the markups for revising FSAR Tier 2, Section 9.1.4.2.1, “General Description,” only discussed control for the movement of new fuel assemblies and not spent fuel assemblies. Since the RAI response indicates that the auxiliary crane can also be used to handle spent fuel assemblies over the spent fuel racks, the staff requested that the applicant revise the markups to FSAR Tier 2, Section 9.1.4.2.1 to also cover this situation.

   The staff notes that an acceptable description was provided in the RAI response for the fuel rack lifting assembly, since it identified the structural components, configuration of these components, and the load path through the lifting assembly which is used to handle the fuel racks.

Based on the above discussion, **RAI 445, Question 03.08.04-15 is being tracked as an open item** pending receipt of the additional information regarding the dimensional gaps, additional analysis for evaluation of the erection tolerances, description for the auxiliary crane handling of spent fuel assemblies, and revisions to the FSAR and TR for fuel racks to incorporate the applicable items.

<u>Applicable Codes, Standards, and Specifications for Fuel Racks</u>

To ensure that the fuel racks can perform their intended safety function, the staff verified that the applicable codes, standards, and specifications used in the design, fabrication, construction, and installation meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, and GDC 5, and are in accordance with the guidance provided in SRP Section 3.8.4, Appendix D, and RG 1.61, RG 1.92, and RG 1.124.

The staff reviewed the applicable codes, standards, and specifications identified in the TR for fuel racks, related to the seismic analysis and structural design of the fuel racks. This information was presented in the TR for fuel racks, Section 1.3. The applicable codes, standards, and specifications identified included 10 CFR Part 50, Appendix A; 10 CFR Part 50, Appendix B; ASME Code Section II, Materials and Section III, Division 1, Subsection NF, Class 3 Component supports; NUREG-0800, Standard Review Plan; NRC RG 1.60, RG 1.61, and RG 1.92.

The staff review of the list of applicable codes, standards, and specifications determined that these meet the requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, and GDC 5; and regulatory guidance described in SRP Section 3.8.4, Appendix D, and applicable regulatory guides with the exception of RG 1.124, "Service Limits and Loading Combinations for Class 1 Linear-Type Supports." The use of RG 1.124 for the fuel rack design is described later in this report under the heading, "Structural Acceptance Criteria for Fuel Racks."

Based on the above discussion, the staff concludes that the applicant's use of codes, standards, and specifications used in the design, fabrication, construction, and installation for fuel racks is acceptable.

Loads and Load Combinations for Fuel Racks

To ensure that the fuel racks can perform their intended safety function, the staff verified that the loads and load combinations used in the design meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, and GDC 5, and are in accordance with the guidance provided in SRP Section 3.8.4, Appendix D. With the exceptions described below, the staff finds that the loads and load combinations meet the regulatory requirements and regulatory guidance in SRP Section 3.8.4, Appendix D, and RG 1.61, RG 1.92, and RG 1.124.

Section 3.2 of the TR for fuel racks lists the loads and load combinations used in the structural design of the fuel racks. Although the load combinations given in the TR for fuel racks comply with those stated in SRP Section 3.8.4, Appendix D, Table 1, the TR for fuel racks did not provide a detailed breakdown of forces and stresses in load combinations due to different types of loads. Therefore, in RAI 445, Question 03.08.04-17, the staff requested that the applicant provide additional information as provided below in order for the staff to ensure that the proper loads and stresses were considered in the structural analysis of the fuel storage racks.

1. Breakdown of forces and stresses are either not given or not clearly indicated in the TR for fuel racks, Appendices 3A through 3C, especially for dead, live, and temperature loads, which should be included in most of the load combinations. The staff requested that the applicant provide a breakdown of forces and stresses for all applicable load combinations.

2. In the TR for fuel racks, Section 3.2 the values of To and Ta are given as 65.6 °C (150 °F) and 100 °C (212 °F), respectively. Explain why, in Appendix 3C of the

Technical Report, Service Level D allowable stresses are checked at 93.3 °C (200 °F) instead of at 100 °C (212 °F).

To address the above items, the applicant provided information which was discussed during the April 27-29, 2011, structural audit. The applicant provided a new Table 3-7 in the TR for fuel racks to summarize the stress results for components of the racks. However, the staff determined that the new table should include the summary of the critical stress results for all of the key structural components, for each Service Level/Loading Condition. For example, regarding Service Level D load combination, the table does not provide the calculated stress results and comparison to allowable for the aluminum tubes, support legs, and welds of the various key components. Also, it appears that some of the allowable stresses presented in the table may not reflect the governing (i.e., lowest) allowable values. For example, in the proposed revision to the TR for fuel racks, Table 3-7, the allowable stress is given as 342 MPa (49.7 ksi) for the top and bottom grids. However, if the allowable stress is determined using the criteria given in the revised Table 3-2 for Service Level D, for SA-240, Type 304 stainless steel, then the allowable stress would be 310 MPa (45 ksi). If consideration of the alternate material of Type 304L is used, then the allowable stress would be reduced further, because the yield stress and ultimate stress for this material are much lower than the Type 304 material. The staff determined that the applicant should identify the lowest governing allowable stresses for all alternate materials and include this information in the RAI response and in proposed markups for the TR for fuel racks.

In response to the staff's concerns, the applicant indicated that the TR for fuel racks will be updated to include the stress results for all of the key structural components, including the rack corner angles, rack legs, and aluminum tubes. Also, the specification for stainless steel material Type 304L will be removed from the TR for fuel racks. This is needed, because the allowable stress limits for this material are less than the stress limits for stainless steel, Type 304, and the intent was to only use materials purchased to Type 304 stainless steel specification.

Regarding the use of 93.3 °C (200 °F) instead of 100 °C (212 °F) for Ta (designated for accident temperature), the applicant indicated that the differences in material properties and allowable stress limits at these two temperatures are negligible and that the calculated stresses are within the applicable 100 °C (212 °F) as well.

In a July 8, 2011, response to RAI 445, Question 03.08.04-17, the applicant provided information to address the items requested in the RAI and the items discussed during the audit. The RAI response indicated that the TR for fuel racks, Table 3-7 will be revised to include the stress results for the corner angles, support legs, and the fuel assembly tubes. Markups for the TR for fuel racks were provided that show the callout for stainless steel material Type 304L was removed. This is needed, because the allowable stress limits for this material are less than the stress limits for stainless steel, Type 304, and the intent was to only use materials purchased to Type 304 stainless steel specification. The staff concluded that the inclusion of stress results in a new Table 3-7 for the key structural components is acceptable, because it provides a basis for concluding that the stresses in the rack structural elements are less than the material stress limits, and thus the rack structural adequacy would be demonstrated.

Regarding the use of 93.3 °C (200 °F) instead of 100 °C (212 °F) for Ta, the RAI response demonstrated that the differences in material properties and allowable stress limits based on 93.3 °C (200 °F) rather than 100 °C (212 °F) are negligible. In addition, the calculated stresses are within the applicable 100 °C (212 °F) as well. Since the differences are negligible and the

calculated stresses are still within the stress limits corresponding to 100 °C (212 °F), the staff concluded that this item is acceptable.

Based on the above discussion, **RAI 445, Question 03.08.04-17 is being tracked as an open item** pending receipt of the updated TR for fuel racks, Table 3-7, for staff review and revision of the TR for fuel racks to incorporate the markups.

Design and Analysis Procedures for Fuel Racks

The staff reviewed the design and analysis procedures used for the fuel racks to ensure that they meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, and GDC 5, and are in accordance with the guidance provided in SRP Section 3.8.4, Appendix D. The staff also reviewed the FSAR to establish that the design is essentially complete, as required by 10 CFR 52.47(c). With the exceptions described below, the staff finds that the design and analysis procedures meet the regulatory requirements and regulatory guidance in SRP Section 3.8.4, Appendix D, and RG 1.61, RG 1.92, and RG 1.124.

The TR for fuel racks, Appendix 3B addresses two fuel drop analysis cases corresponding to the shallow drop and deep drop. However, the staff identified that the applicant's shallow fuel drop case only evaluated a fuel assembly drop at the center of the top of the fuel rack. SRP Section 3.8.4, Appendix D, Section 1.4 indicates that the fuel pool racks and the fuel pool structure should be evaluated for accident load combinations. One of the accident loading combinations is the shallow drop where the fuel assembly is assumed to drop on top of the fuel rack. Since the shallow drop was only considered to impact the center of the top grid on top of the tube plates, in RAI 445, Question 03.08.04-18, the staff requested that the applicant, clarify why a fuel assembly drop at the corner angle was not also considered.

In a July 8, 2011, response to RAI 445, Question 03.08.04-18, the applicant provided a description of a shallow drop analysis assuming that the fuel assembly drops on top of the rack support angle at the corner of the rack. This additional analysis will be added to the TR for fuel racks, Appendix 3B. A buckling analysis of the support angle will also be included because this shallow drop case produces the maximum compressive force in the angles.

The staff review of the shallow drop analysis concluded that the additional evaluation of the fuel assembly on top of the rack support angle coupled with the existing drop analysis at the center of the rack ensures that all potential shallow drop cases have been considered. Therefore, **RAI 445, Question 03.08.04-18 is being tracked as a confirmatory item** pending revision of the TR for fuel racks to incorporate the markups contained in the RAI response.

The TR for fuel racks, Appendix 3C describes the seismic analysis of the fuel racks located in the spent fuel pool. The staff review of the seismic modeling of the fuel racks and water in the pool identified a number of questions regarding the adequacy of the structural models of the racks and representation of the water in the pool. This information was needed to ensure that the model can predict the expected dynamic behavior of the racks during a seismic event. Therefore, in RAI 445, Question 03.08.04-19, the staff requested that the applicant provide the information listed below:

1. Provide a detailed description of the Arbitrary-Lagrangian-Eulerian (ALE) analysis algorithms used in the whole pool analysis; for example, explain how the important parameters were obtained and how ALE analysis algorithms compare with traditional potential theory-based added mass/damping methods. Clarify whether water in the tube cells and water below the racks were also modeled with fluid elements, and whether

hydrodynamic coupling was taken into account in the separate stress analysis using the detailed single rack model. If not, provide the technical basis for not doing so.

2. Explain how the dead weight mass was treated for submerged rack structures and how the effective damping of the fuel assemblies and water damping were considered.

3. The friction coefficient between the bearing pads and the pool liner is an important factor affecting the seismic response of the racks. Provide the technical basis for only considering the two bounding values and not other intermediate values for the fully loaded configuration. For the partially loaded configurations, several runs may be needed in order to adequately consider the randomness of the friction coefficient and the configuration of the loaded cells within each rack. Therefore, provide a description of the various runs analyzed for each of the two cases of partially loaded configurations and the technical basis for selection of these cases.

To address the above items, the applicant provided information which was discussed during the April 27-29, 2011, structural audit. The information discussed, the staff evaluation, and the applicant's proposed resolution are described below.

1. The applicant provided a description of the ALE analysis approach which illustrated how this methodology was used in the whole pool multi-rack analysis. The applicant also described how hydrodynamic coupling was considered in the separate stress analysis of the detailed single rack model. The staff determined that sufficient information was provided to describe the ALE approach; however, no information was provided to address how the ALE analysis approach compares with traditional potential theory-based mass/damping methods which have been used in other spent fuel licensing applications reviewed and accepted by the staff. For example, have any sample problems been analyzed using both methods and how do the results compare? Also, since the fuel assemblies and water in the tubes were not modeled explicitly in the whole pool analysis, a technical basis or results of a study should be provided which demonstrates that excluding the explicit modeling of the fuel assemblies and water in the tubes has been shown to have a negligible or conservative effect on the fuel rack response.

   To address the staff's concerns, the applicant indicated that an explanation of how the ALE analysis approach compares with traditional potential theory-based mass/damping methods will be provided. Some sample problems contained in the validation and verification documentation for LS-DYNA were reviewed by the staff during the audit. However, another sample problem will be performed to compare the LS-DYNA code, for a rectangular tank structure (without racks), to the approach used in the ASCE 4-98 Standard. To address concerns with not including the rattling of fuel assemblies and water within the tube in the whole pool multi-rack analyses, another analysis will be performed which will include a simplified representation of the fuel assembly inside the tube cells with water. The results from this additional analysis will be evaluated and compared to the same case where the fuel assemblies were modeled as added density to the tubes.

2. For consideration of dead weight, a 1g gravity acceleration was applied to the mass of the rack components and the water. Damping is accounted for in the rack analysis through the use of discrete dashpots (damping elements) at each fuel assembly spacer grid elevation which acts in series with the spacer grid compression only spring. The staff determined that the explanation describing how the dead weight mass was

considered in the seismic analysis demonstrates that the appropriate mass is included for the various elements submerged in water. However, the damping value in percent for the fuel assemblies was not available. This information is needed in order for the staff to determine whether the damping of the fuel assemblies have been properly included in the model.

To confirm that the appropriate damping value was used for the fuel assemblies in the seismic analyses, the applicant indicated that the damping value in terms of percent of critical damping will be determined. If the damping value is found to be significantly greater than the damping values in RG 1.61 for comparable materials, then a technical basis will be provided.

3. To address the questions on the coefficient of friction, the applicant described an additional analysis performed for the fully loaded whole pool model using an intermediate value of 0.5 for the coefficient of friction between the rack bearing pads and the pool liner. The results showed that for the maximum controlling values of all the racks, the 0.2 and 0.8 coefficient of friction cases bound the results. To address the question of the intermediate coefficient of friction value for the partially loaded racks, two additional analyses were performed, one for each of the two partially loaded racks analysis cases. The results showed that the original partially loaded cases controlled the response of the partially loaded racks.

   The staff review of the above information finds that the additional analyses demonstrated that the set of coefficient of friction values selected for the seismic analysis bound the seismic response of the racks and are thus acceptable.

In a July 8, 2011, interim response to RAI 445, Question 03.08.04-19, the applicant provided information to address the items requested in the RAI and the items discussed during the audit. The information provided in the RAI response and the staff evaluation is provided below.

1. The RAI response explained that the multi-rack whole pool analysis model utilizes a combination of the Lagrangian, Eulerian, and Arbitrary-Lagrangian-Eulerian (ALE) methodologies to represent the fluid-structure interactions between the racks and between the racks and the pool structure. In the whole pool model, the racks utilize Lagrangian finite element meshes. In the Lagrangian formulation, the mesh is deformable as the rack structure displaces in space and time under the applied external loads. The displaced rack structure is tracked explicitly and is known at any point in time. In the Eulerian approach, the mesh is fixed and does not deform. The mesh allows the fluid to flow through the mesh grid. The Eulerian approach does not track the individual fluid particles, but instead, measures flux or flow through the mesh grid.

   In the LS-DYNA computer code application of the ALE approach, each rack is modeled using Lagrangian finite element meshes. The fluid is modeled using an Eulerian mesh. As the Lagrangian structure displaces away from or into the fluid-structure boundary, due to inertial seismic loads or fluid pressures, the fluid flows into the void mesh that becomes exposed due to displacements/deformations of the Lagrangian structures.

   In the whole pool model, the region outside of each rack, including the water below the rack baseplates, was modeled with fluid finite elements, and the region occupied by each rack was defined as void whereby, water can flow into it once the rack displaces or deforms. Rack-to-rack and rack-to-pool structure hydrodynamic coupling is explicitly considered. The water inside the tube cells is not explicitly modeled with fluid elements.

Instead, its mass and the mass of the fuel assemblies are represented as added mass to the tubes.

In the separate stress analysis using the detailed single rack models, hydrodynamic coupling is considered by applying the displacement field resulting from the whole pool model fluid structure interaction analysis. The mapped displacement field includes the hydrodynamic coupling effects from the multi-rack whole pool analysis. The hydrodynamic coupling between the fuel assembly and the water inside the tube cell was considered by including the water mass inside the tube as added mass along with the fuel assembly. The RAI response considered this to be a conservative approach, because only the inertial effect of the water is considered, with no credit taken for water resistance forces and damping effects that take place inside the tubes.

The water fluid is modeled using the LS-DYNA finite element with a hydrodynamic material law, where the fluid is defined using two properties: Its density and elastic bulk modulus. The RAI indicated that the modeling of the water is consistent with that of an ideal fluid (i.e., fluid is inviscid (frictionless) and essentially incompressible, which is consistent with potential theory principles).

To validate the ALE analysis methodology, the RAI response described a number of sample problems that were executed in LS-DYNA using the ALE approach and presented the comparison of results with those based on potential theory. These sample problems are documented as part of TN's Validation and Verification (V&V) package for LS-DYNA. The V&V package contains comparisons of the LS-DYNA ALE results against those obtained using other computer codes, closed form solutions, and experimental tests. As discussed during the audit, an additional sample problem was performed consisting of a small rectangular tank filled with water fluid and subjected to dynamic motions. The purpose of this problem was to evaluate the response of the tank using the LS-DYNA ALE approach against the industry wide accepted formulation method presented in the ASCE 4-98 Standard. A finite element model of the rectangular tank filled with water was developed using the LS-DYNA code in which the ALE approach was used. This model was subjected to an input consisting of a triangular shape acceleration pulse applied in the lateral direction. The force resulting from the dynamic pressures on the tank walls along with the fluid response were determined and compared to the solution procedure for a rectangular tank described in the ASCE 4-98 Standard. The results, in terms of the impulsive and convective hydrodynamic forces on the tank walls, the fluid oscillation period, and the sloshing height between the two cases compared very well.

The staff's review of the above information determined that as requested by the staff, the RAI response provided a detailed description of the ALE analysis approach which explained how this methodology was used in the whole pool multi-rack analysis to capture the fluid structure interaction. In addition, the validation of the LS-DYNA code with other computer codes, closed form solutions, and experimental tests, as well as the additional sample problem analyzed for a rectangular tank filled with water demonstrate that the ALE analysis approach used in LS-DYNA for the multi-rack pool analysis is acceptable.

To address concerns with not modeling the fuel assemblies and water within the tube in the various rack analyses, the RAI response indicated that the whole pool model will be modified to include a simplified representation of the fuel assembly inside the tube cells

with water. An additional analysis will be performed for the bounding time history run, and the results will be evaluated and compared to the same case where the fuel assemblies were modeled as added density to the tubes. The staff determined that the additional analysis should address the staff's concern, because it would include a discrete representation of the fuel assembly with the water treated as hydrodynamic added mass in the whole pool multi-rack analysis.

2. The RAI response explained that the dead weight mass is applied to the model as body forces with 1g gravity acceleration to the parts of the whole pool model, including the individual rack structures and the fluid below, in between, and above the racks. The RAI response indicated that this modeling approach accurately accounts for gravity, hydrostatic pressure, and buoyancy effects. The fuel assemblies in the detailed rack model have individual dashpots at each spacer grid elevation. The spacer grid dashpots are modeled as discrete damping elements in series with the spacer grid compression-only spring stiffness. The equivalent damping value at each of the spacer grid locations ranges from 6.3 percent to 7.1 percent with an average value of 6.9 percent.

   The staff review of the above information concluded that the use of 1 g applied to all of the parts in the LS-DYNA whole pool model is acceptable, because it would account for the dead load effect of the masses, hydrostatic pressures, and buoyancy effects. The staff finds the damping values identified in the RAI response acceptable, because they are comparable to SSE damping values for bolted structures tabulated in RG 1.61; additional damping due to the effects of the submerged components in water have been conservatively neglected.

3. The RAI response described the additional analyses performed to address the questions related to the coefficient of friction. An additional analysis of the fully loaded whole pool model was performed using an intermediate value of 0.5 for the coefficient of friction between the rack bearing pads and the pool liner. The results show that on an individual rack basis, generally the results (e.g., rack accelerations, sliding, friction forces, and impact forces) corresponding to the 0.2 and 0.8 coefficients of friction bound the results from the intermediate case of 0.5. However, for the maximum controlling values of all the racks, in all instances, the 0.2 and 0.8 cases bound the results. To address the question on the partially loaded racks, two additional analyses have been performed, one for each of the two partially loaded racks. These additional cases also used an intermediate coefficient of friction of 0.5 for comparison with the original two partially loaded cases which used a random distribution of the coefficient of friction under the various rack legs. The results showed that generally there were no significant differences with the original partially loaded cases, and where differences did occur, the random coefficient of friction cases controlled.

The staff review of the above information determined that the additional analyses, for the fully loaded and partially loaded whole pool analyses, demonstrate that the range of the coefficient of friction values between 0.2 and 0.8 were considered; and therefore, this item is acceptable. The applicant should include these additional analyses in the TR for fuel racks.

Based on the above discussion, **RAI 445, Question 03.08.04-19 is being tracked as an open item** pending completion of the additional analysis which models the fuel assemblies discretely and revision of the TR for fuel racks to describe this additional analysis and the additional analyses that were performed for the intermediate coefficient of friction values.

The staff review of the seismic analysis of the whole pool rack model, described in the TR for fuel racks, Appendix 3C raised some questions regarding the seismic input loads and the detailed stress analysis. These questions need to be addressed to ensure that the correct seismic input loads are used and to ensure that a complete stress evaluation has been performed for all rack structural components. Therefore, in RAI 445, Question 03.08.04-20, the staff requested that the applicant provide the information listed below.

1. Clarify why only input time history set 1, which produces the bounding response for the fully loaded cases, was used to analyze the partially loaded cases. The staff notes that input time histories that yield the bounding response for the fully loaded cases may not yield the bounding response for the partially loaded cases. Explain why the seismic input developed for the U.S. EPR Fuel Building analysis/design (stated in FSAR Tier 2, Section 3.7) was not used. Clarify whether an envelope of the spectra at the pool floor and along the height of the pool wall was used.

2. The stress analysis discussed in the TR for fuel racks, Section 3C.3.6 does not include stress evaluations for the welds and bearing pads. Clarify whether these have been performed.

3. Provide the technical basis showing that a 9x10 rack provides the bounding stresses in the stress evaluations.

To address the above items, the applicant provided information which was discussed during the April 27-29, 2011, structural audit. The information discussed, the staff evaluation, and the applicant's proposed resolution are described below.

1. The applicant stated that time history Set 1 has the largest energy content of the five sets of time histories and is the only time history that produced rack-to-rack impacts. Regarding the selection of the response spectra, the applicant indicated that the response spectra used to develop the time history motions bound the response spectra provided in the FSAR corresponding to the floor elevation of the pool structure.

   The staff review of the above information determined the applicant needs to submit the technical basis in the RAI response to demonstrate that time history Set 1 is the most severe earthquake motion of the five sets of time histories. Regarding the use of the proper seismic input motion for the rack analyses, a comparison of the floor response spectra used as input at the base of the pool to the response spectra on the pool wall (at the top elevation of the fuel racks) obtained from the seismic SSI analysis should be made. This comparison is intended to determine if there is any amplification in the spectra between the location of the pool base and the pool wall at the top of the rack elevation, due to the SSI analyses of the nuclear island. Also, since the response spectra used in the fuel rack analyses are based on the previous seismic SSI analyses, a reconciliation of these seismic input loads needs to be made to the new updated seismic SSI analyses discussed in FSAR Tier 2, Section 3.7.

2. The applicant indicated that the evaluation for the rack component welds is covered by the response to RAI 445, Question 03.08.04-21. The evaluation of the support leg welds and bearing pads is presented in the TR for fuel racks, Appendix B, Sections 3B.6 and 3B.7, respectively. The staff review of this item noted that Appendix 3B only addresses fuel drop accident analyses and not the seismic rack analysis. Therefore, a separate stress evaluation for the legs is needed for the rack seismic analysis.

3. The applicant indicated that the two racks selected for the detailed stress analysis had the largest resultant accelerations for fuel racks. The staff review of this information determined that the information provided does not explain how the resultant accelerations are calculated and why is the use of the single resultant acceleration for the racks appropriate for selection of the most highly stressed rack(s). Also, depending on the particular component being evaluated for stress, this approach may not pick up the worst case (e.g., a different rack may have higher impact forces for rack-to-floor, or rack-to-rack interaction).

To address the above concerns, the applicant indicated that a review of the maximum key stresses from all 15 cases analyzed in the whole pool multi-rack analysis cases will be reviewed. If any other case(s) result in higher stresses than the two cases already evaluated in the detailed rack stress analyses, then they would also be evaluated in additional detailed stress analyses.

In a July 8, 2011, interim response to RAI 445, Question 03.08.04-20, dated July 8, 2011, the applicant provided information to address the items requested in the RAI and the items discussed during the audit. The information provided in the RAI response and the staff evaluation is provided below:

1. The RAI response explained that time history Set 1 has the largest energy content of the five sets of time histories and is the only time history that produced rack-to-rack impacts. The largest energy content of this time history set was demonstrated by comparing the Arias intensities of this time history set to the other time history sets.

   Regarding the question of only using the response spectra corresponding to the floor elevation of the spent fuel pool as input to the rack analyses, the RAI response indicated that a separate study was performed.

   In this study, the first case analyzed the racks in a pool structure modeled with the actual pool concrete wall flexibilities. In the second case, the same model was used; however, the pool walls were made rigid to simulate the approach used in the current analysis of the whole pool multi-rack seismic analysis. The applicant indicated that the results, in terms of the resultant forces on the racks, are slightly lower for the flexible wall case compared to the rigid wall case. To further demonstrate the acceptability of the analysis approach, the RAI response indicated that acceleration time history responses at the elevation of the top of the racks, resulting from the flexible pool wall model, will be used to generate response spectra. These response spectra will be compared with the response spectra obtained from the SSI design basis seismic analysis and the seismic reconciliation analysis of the spent fuel building. This will provide the basis for assessing the acceptability of the analysis model and target spectra used for the seismic evaluations of the racks. The seismic reconciliation analysis will be added to the TR for fuel racks.

   The staff review of the above information determined that the use of time history Set 1 to perform the seismic analyses for the partially loaded rack cases is acceptable, because the RAI response demonstrated that time history Set 1 is the most severe earthquake motion of the five sets of time histories. Regarding the use of the spectra at the pool floor elevation, the staff is concerned that there were several assumptions made in the separate study considering flexible and rigid pool walls. These include modeling all of the racks so that they respond in unison (i.e., the 17 individual racks are merged together), performing the analysis for only the 0.2 value for the coefficient of friction, and

only comparing the maximum resultant forces at the faces of the periphery rack faces adjacent to the pool walls to demonstrate that the results for the flexible wall case are slightly lower than the rigid pool wall case. As a result of these concerns, and as previously requested, the applicant should compare the floor response spectra at the elevation of the pool floor (used as input for the design basis seismic analysis of the racks) to the envelope of the response spectra obtained from the seismic SSI analysis at the pool floor and wall at the elevation of the top of the racks. If any exceedances occur, then the enveloped set of seismic response spectra should be used or technical justification provided to demonstrate the adequacy of these exceedances.

2. The RAI response indicated that the evaluation of the welds is covered by the response to RAI 445, Question 03.08.04-21; therefore, this item is evaluated by the staff under the staff's review of this response discussed later in this section of the report. The RAI 445, Question 03.08.04-20 response also indicates that the seismic stress evaluation of the rack legs, including welds and bearing pads, will be added to the TR for fuel racks in Appendix C, Section 3C.3.6.4, upon completion of the analysis. Since the applicant will include the seismic stress evaluation of the rack in the TR for fuel racks, this item is acceptable pending staff review of the revised Appendix C.

3. The RAI response indicated that the two racks selected for the detailed stress analysis are the 9X10 Group 1 and the 9X10 Group 20 racks on the basis that (1) they had the largest resultant accelerations as shown in Table 3C-11 of the TR for fuel racks, (2) the 9X10 Group 20 rack was the rack that experienced rack to rack impacts, and (3) the 9X10 racks are the largest racks, and thereby, have the largest inertial loading. The RAI response also explained that the resultant accelerations summarized in Table 3C-11 of the RAI response are calculated using the SRSS of the individual x, y, and z component accelerations at each time step and the maximum resultant accelerations result from the averaging of the nodal accelerations. However, to confirm the selection criteria for these two racks, the maximum stresses from all 17 racks for each of the 15 whole pool analysis cases will be reviewed. If needed additional detailed stress analysis models will be developed and a stress analysis will be performed for the rack(s) identified during the review.

   The staff review of the above information determined that the approach is acceptable, because the applicant will review the responses of all 17 racks for all 15 analysis cases in the whole pool analysis and if any racks, beyond the two already identified for detailed analysis, have larger stresses, then a detailed stress analysis will also be performed for those racks.

Based on the above discussion, **RAI 445, Question 03.08.04-20 is being tracked as an open item** pending receipt of the demonstration of the adequacy of the seismic response spectra used as input to the fuel rack analyses, the stress evaluation for some additional rack components, the additional evaluation to ensure that the detailed stress analyses have been performed for all of the critical racks, and revision of the TR for fuel racks for the evaluations.

The TR for fuel racks, Appendix 3C describes the seismic analysis and design of the fuel storage racks. However, the staff noted that insufficient information was provided to demonstrate the design adequacy for the tubes, rack vertical angles, rack baseplate, welds, new fuel rack, and fuel assembly. Additional information was needed to ensure that the design of these structural elements is adequate to withstand all applicable loads. For the fuel assembly, SRP Section 3.8.4, Appendix D.I.3, indicates it should be demonstrated that the

consequent loads on the fuel assembly do not lead to damage of the fuel. Therefore, in RAI 445, Question 03.08.04-21, the staff requested that the applicant provide the items listed below.

1. Buckling analysis for the components, such as aluminum tube walls and rack vertical angles, subjected to significant compression forces should be provided.

2. Complete weld checks for all welds except full penetration welds should be provided.

3. Describe how the qualification of the fuel assemblies is maintained due to the dynamic response of the fuel racks.

4. Explain why analysis of the dry new fuel storage racks is not presented in the TR for fuel racks.

To address the above items, the applicant provided information which was discussed during the April 27-29, 2011, structural audit. The information discussed, the staff evaluation, and the applicant's proposed resolution are described below.

1. To demonstrate the design adequacy of the corner posts in the racks due to the seismic load combinations, the applicant indicated that a new calculation will be performed and a summary added to the TR for fuel racks. The buckling evaluation for the aluminum tubes will be added to the TR for fuel racks.

2. The applicant indicated that where full penetration welds are used between the rack components, the weld is as strong as the structural elements it is connecting. For the other structural connections, where a full penetration weld is not used (e.g., fillet welds), the applicant indicated that the stresses in the welds satisfy the applicable stress limits. The staff noted that the acceptance of these non-full penetration welds is dependent on the type of electrodes used. Therefore, the applicant should identify the weld electrode types or the corresponding yield and ultimate strength values in the TR for fuel racks. The applicant indicated that the TR for fuel racks will be updated to clarify that the yield and ultimate strength of the weld material will be equal to or greater than the base metal.

3. To develop the seismic loading on the fuel assemblies, the applicant described the seismic analysis performed for an individual rack with representative fuel assemblies. The maximum impact force between the fuel assembly and the tubes of the rack at the gird locations was obtained and determined to be less than the minimum allowable loading for the fuel assemblies.

   The seismic analysis described above was determined to be acceptable, because it utilized a separate more detailed rack model with individual fuel assemblies in order to calculate the impact forces between the fuel assemblies and the rack, and compared these forces to the allowable loading for the fuel assemblies.

4. No analysis was performed for the racks in the NFS area. The applicant indicated that these racks have the same structure as the racks in the spent fuel pool which are larger and are free to move (i.e., not anchored to the pool structure). The racks in the NFS area are laterally restrained to the concrete walls and supported by the concrete floor.. The applicant indicated that this configuration provides reasonable assurance that the input accelerations are limited to the zero period accelerations. Therefore, the racks in the NFS area are qualified by comparison to those in the spent fuel pool.

The staff determined that response did not demonstrate why connecting the racks in the NFS area to the sides of the pool structure, at the base and top, ensures that there will be no amplification of the seismic motion to the racks (i.e., they will only be subjected to ZPAs). This would be true if the racks are "dynamically rigid"; however, this has not been demonstrated. In addition, making a comparison of the expected accelerations for the racks in the NFS area and the accelerations of the racks in the spent fuel pool may not be appropriate, because in the spent fuel pool the racks are free to slide and tip, and the water would absorb some of the imposed inertia of the racks; whereas, in the NFS area the racks are supported at discrete locations without water being present. The load path of the inertial loads is also different for the two cases. Therefore, an adequate technical basis for concluding that the racks in the NFS area are bounded by the analyses of the racks in the spent fuel pool is needed or a specific seismic evaluation for the racks in the NFS area should be performed.

To address the staff's concerns, the applicant indicated that a new model will be developed for the racks in the NFS area, and a seismic analysis will be performed for these racks. A preliminary seismic analysis approach was discussed during the audit which will be based on either a response spectrum analysis or equivalent static analysis. The details of the analysis will be provided in the RAI response.

In a July 8, 2011, interim response to RAI 445, Question 03.08.04-21, the applicant provided information to address the items requested in the RAI and the items discussed during the audit. The information provided in the RAI response and the staff evaluation is provided below:

1. The RAI response indicated that the fuel assembly drop on top of the rack corner angle is addressed in the response to RAI 445, Question 03.08.04-18 and a buckling evaluation of the corner angle under seismic load combinations will be added to the TR for fuel racks, Appendix 3C. A buckling evaluation of the tube walls subjected to the seismic loads will be added to the TR for fuel racks, Appendix 3C.

   The staff evaluation of the fuel assembly drop on top of the rack corner angle is discussed in the response to RAI 445, Question 03.08.04-18, which the staff finds acceptable. The buckling evaluation of the corner angle will need to be reviewed by the staff when it is completed. The staff reviewed the buckling evaluation of the tube walls was reviewed and find it acceptable, because it was performed in accordance with the ASME Code, Section III, Division 1, Subsection NF and Appendix F. A summary of these analyses will need to be included in the TR for fuel racks.

2. The RAI response identified where full penetration welds are used between the rack components. In these cases, the penetration weld is as strong as the structural elements it is connecting. For the other structural connections where a full penetration weld is not used (e.g., fillet welds), calculations show that the stresses in the welds satisfy the applicable stress limits. To address the staff's concern regarding the type of electrodes to be used, the RAI response indicated that the TR for fuel racks will be updated stating "the yield and ultimate strength of the weld material will be equal to or greater than the base metal." The staff determined the approach used to design the welds, including the markup proposed for the TR for fuel racks acceptable, because it complies with ASME Code, Section III, Subsection NF and Appendix F.

3. A detailed seismic analysis was performed for an individual rack with representative fuel assemblies using the motions of the loaded racks in the spent fuel pool. The maximum impact force between the fuel assembly and the tubes of the rack at the gird locations

was obtained. The staff determined the maximum impact load was less than minimum allowable loading for the fuel assemblies obtained from test results.

The staff finds the seismic analysis described above acceptable, because it utilized a separate more detailed rack model with individual fuel assemblies in order to calculate the impact forces between the fuel assemblies and the rack. However, no information was provided to demonstrate whether the allowable loading is based on new fuel assemblies or spent fuel assemblies having thermal and mechanical properties reflecting many years of storage and further irradiation in the spent fuel pool. The applicant should provide the technical basis for the determining the allowable load of the spent fuel assembly under these conditions.

4. To address the staff's concern regarding the seismic qualification of the racks in the NFS area, applicant indicated that a detailed finite element model for the racks in the NFS area will be developed where all of the components including the fuel assembly will be modeled. In addition to the restraint in the vertical direction at the rack legs, the racks will be restrained horizontally where the racks are connected to the walls. Either an equivalent static or response spectra seismic analysis will be performed to demonstrate the seismic adequacy of the racks. This analysis will be described in the TR for fuel racks, Appendix 3D. The staff review of this information determined that the approach to qualify the racks in the NFS area would be acceptable pending review by the staff when it is completed.

Based on the above discussion, **RAI 445, Question 03.08.04-21 is being tracked as an open item** pending completion of the buckling evaluation of the corner angle, evaluation demonstrating the adequacy of the spent fuel assemblies for thermal and mechanical properties when stored in the spent fuel pool, seismic qualification of the racks in the NFS area, and revision to the TR for fuel racks to incorporate the evaluations.

The TR for fuel racks, Appendix 3A describes the rack evaluation for dead weight, lifting of the racks, thermal, and stuck fuel assembly. Some stress analysis results were presented; however, a description of the specific component analyzed was not provided. This information is needed in order for the staff to determine whether the stress analysis for this component is acceptable. Therefore, in RAI 445, Question 03.08.04-26, the staff requested that the applicant describe the rack structural component that was being evaluated.

During the April 27-29, 2011, structural audit, the applicant indicated that the stress results presented in Appendix 3A of the TR for fuel racks are applicable to the rack bottom plate. The maximum calculated stress is located in the midspan of the baseplate. The model used to calculate this stress assumes that the baseplate is fixed in all degrees of freedom at the locations of the support legs. The staff determined that this assumption does not appear to be reasonable, because in the actual rack structure, this location could rotate due to rotation of the leg itself and due to bending flexibility of the leg. With rotation released at the intersection of the baseplate and support leg, the maximum moment would be larger than with full fixity. To address this issue, the applicant indicated that it will perform an additional analysis to bound the support condition at the rack leg. The staff concluded that if the bounding support condition is considered (i.e., worst case between fixed and rotationally free boundary), then the calculated stress for the baseplate would be acceptable.

In a July 8, 2011, response to RAI 445, Question 03.08.04-26, the applicant indicated that the base plate, support plates, and support angles are modeled for the deadweight analysis. The stress result presented is the bounding stress in the three components. The TR for fuel racks

will be revised to identify which structural components are being evaluated. The maximum bending stress is calculated in the base plate at the support leg location, corresponding to the fixed boundary condition. However, an additional analysis will be performed assuming that the support leg is simply supported in order to envelope all boundary conditions. The results of the additional analysis will be added to the TR for fuel racks, Section 3A.2.

Based on the above discussion, the staff concluded that the use of the simply supported boundary condition in addition to the fixed condition would bound the maximum stress in the baseplate, and thereby, this approach is acceptable. Therefore, **RAI 445, Question 03.08.04-26 is being tracked as open item** pending completion of this additional analysis and revision of the TR for fuel racks to incorporate the evaluation.

SRP Section 3.8.4, Appendix D, Section I.4 indicates that the temperature gradient across the rack structure that results from differential heating effects should be incorporated in the design of the rack structure. The TR for fuel racks, Appendix 3A, Section 3A.4 presents design checks of the support grid and tubes subjected to differential heating effects between a full and an empty cell. Based on the review of the information presented in the TR for fuel rack, Appendix 3A, in RAI 445, Question 03.08.04-27, the staff requested that the applicant provide the technical basis for the statement made in the TR for fuel racks that an isolated fuel assembly stored in a fuel compartment will produce the maximum thermal gradient in the top and bottom support grids. This information is needed to verify whether the specific thermal case analyzed bounds the other possible thermal cases with multiple hot and cold cells.

During the April 27-29, 2011, structural audit, the applicant indicated that an isolated fuel assembly was selected, because one hot cell will cause a localized effect, whereas the use of multiple hot cells will cause the entire rack to expand and relieve the thermal stresses. However, an additional analysis was performed to consider two adjacent heated cells and some of the conservatism present in the thermal analysis approach was removed.

The staff review of this information noted that when the number of hot cells, adjacent to empty cells, is increased from one to two hot cells, then the maximum stress increased from 92.4 to 142 MPa (13.4 to 20.6 ksi). Therefore, it is possible that if more than two hot cells are considered then the stresses could increase even more. The applicant addressed this concern by indicating that additional thermal analyses will be performed which will consider various hot cell configurations. This will determine the governing case(s) for checking the stresses in the racks for thermal loadings of hot cells with fuel assemblies and other cells without fuel assemblies. The staff concluded that if additional thermal analyses are performed which will consider additional hot cell and cold cell configurations located at different locations of a given rack, then the thermal evaluation would be acceptable.

In a July 8, 2011, interim response to RAI 445, Question 03.08.04-27, the applicant indicated that finite element analyses will be performed for three cases containing loaded fuel assemblies. These cases bound the various configurations for heated cells, because with the addition of more fuel assemblies, the thermal stresses would be alleviated due to a more uniform expansion of the entire rack. The model would include the base plate, corner posts, grid straps, and top grid. A temperature of 100 °C (212 °F) will be used for the cells that contain the fuel assemblies, while a temperature of 21.1 °C (70 °F) will be applied to the rest of the rack. A summary of this evaluation will be included in the TR for fuel racks.

The staff review of the above information determined that the additional thermal analysis cases would address the staff's concern, because it would consider a sufficient number of thermal cases of hot and cold cells which would bound the variations that would be expected to occur.

Therefore, **RAI 445, Question 03.08.04-27 is being tracked as an open item** pending completion of this additional analysis and revision of the TR for fuel racks to incorporate the evaluation.

Structural Acceptance Criteria for Fuel Racks

The staff reviewed the structural acceptance criteria used for the fuel racks to ensure that the criteria meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, and GDC 5, and are in accordance with the guidance provided in SRP Section 3.8.4, Appendix D. With the exceptions described below, the staff finds the structural acceptance criteria meet the regulatory requirements and regulatory guidance in SRP Section 3.8.4, Appendix D, and RG 1.61, RG 1.92, and RG 1.124.

The TR for fuel racks, Tables 3-1 and 3-2 summarize allowable stresses used for design of the racks. The staff review of these tables determined that these allowable stresses did not comply with the applicable provisions of the ASME Code, Section III, Division 1, Subsection NF, which is identified in SRP Acceptance Criteria 3.8.4, Appendix D.I.6. Therefore, in RAI 445, Question 03.08.04-16, the staff requested that the applicant address the items listed below, related to the TR for fuel racks, Tables 3-1 and 3-2:

1. In the TR for fuel racks, Table 3-1, the allowable stress for tension from elastic analysis under accident conditions is given as the lesser of $1.2S_y$ or S_u. However, ASME Code, Section III, Division 1, Appendix F, Article F-1334.1 states: “The tensile stress on the net section, except at pin holes and in the through-plate thickness direction, shall not exceed the lesser of $1.2S_y$ and $0.7S_u$.”

2. Explain what is meant by “elastic plastic analysis” in the TR for fuel racks, Table 3-1, Note 2, since ASME Code, Section III, Division 1, Appendix F has separate provisions for elastic and plastic analysis. Also, explain the basis for the limits specified in the tables, since they do not seem to match the limits specified for austenitic steels in Appendix F to Section III of the ASME Code.

3. In the TR for fuel racks, Table 3-2, explain the basis for the allowable stresses for P_L and (P_m or P_L) + P_b + Q under normal conditions, which are given as $1.5S_m$ and $3.0S_m$, respectively.

4. In the TR for fuel racks, Table 3-2, explain why the allowable stresses given for elastic analysis under accident conditions seem to follow ASME Code. Section III, Division 1, Subsection NF, Article F-1331 instead of ASME Code, Section III, Division 1, Subsection NF, Article F-1332.

5. In view of the above questions, the staff requested in this RAI that the applicant review and confirm that all stress limits for the fuel racks are utilized in accordance with the ASME Code Section III, Division 1, Subsection NF and Appendix F, as appropriate.

During the April 27-29, 2011, structural audit, the applicant provided a proposed revision in the TR for fuel racks, Tables 3-1 and 3-2 to address the above items. Based on this information, the staff determined that not all of the items were addressed. Some examples are (1) for Service Levels B and C, in addition to specifying K_m, the other stress limit Factors K_v and K_{bk} were not provided (subject to the buckling limitation) in the ASME Code, (2) for Service Level D, (Plastic Analysis), under compression, the basis for applying the increase values shown in the table to the allowable stress of 2/3 F_a was not provided, (3) for Service Levels A, B, and C, the

partial penetration groove/fillet weld entry corresponding to "shear stress on fillet weld, normal tension on partial penetration groove weld, shear stress on plug/slot weld" only presents the stress limit for the weld material and does not also include the stress limit for the base metal, and (4) some of the criteria in RG 1.124 have not been incorporated.

To address the staff's concerns, the applicant indicated that the rack allowable stress limits, shown in the TR for fuel racks, Tables 3-1 and 3-2 will be revised to be in agreement with the ASME Code, Section III, Division 1, Subsection NF and Appendix F; NRC RG 1.124; and SRP Acceptance Criteria 3.8.4, Appendix D.I.6. The specific items that need to be corrected were discussed and agreed to during the audit.

In a July 8, 2011, response to RAI 445, Question 03.08.04-16, the applicant provided markups to the TR for fuel racks, Table 3-1 and Table 3-2 . These tables were revised so that the stress limits would be in accordance with ASME Code Section III, Division 1, Subsection NF and Appendix F. In addition, the stress limits were modified so they would conform to the criteria in RG 1.124, Revision 2. The staff finds the revised tables acceptable, because the stress limits meet the applicable ASME Code Section III, Division 1, Subsection NF and Appendix F, as well as RG 1.124, Revision 2. **RAI 445, Question 03.08.04-16 is being tracked as a confirmatory item** pending revision of the TR for fuel racks to incorporate the markups contained in the RAI response.

Materials, Quality Control, and Special Construction Techniques for Fuel Racks

The staff reviewed the materials, quality control, and special construction techniques used for fuel racks to ensure that the techniques meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, and GDC 5; and are in accordance with the guidance provided in SRP Section 3.8.4, Appendix D.

Information regarding the specific materials used for various components of the fuel racks was provided by the applicant in the July 8, 2011, response to RAI 445, Question 03.08.04-15. The staff identified some questions from the materials used for the racks which are evaluated above under the headings, "Design and Analysis Procedures for Fuel Racks," and, "Loads and Load Combinations for Fuel Racks."

The staff finds the fabrication and quality control procedures used in the fabrication acceptable on the basis that they are performed to comply with the requirements of the ASME Code, Section III, Division 1, Subsection NF.

In an April 14, 2011, response to RAI 445, Question 03.08.04-25, the applicant stated that there are no special construction techniques required in the fabrication of the rack modules and that the installation of the racks into the pool may be performed in any sequence, because there is no fuel present that would require special control of rack placement for criticality concerns. On this basis, the staff concluded that the construction techniques for the racks do not need to be reviewed further during the design certification stage.

Testing and Inservice Requirements for Fuel Racks

To ensure that the fuel racks can perform their intended safety function, the staff reviewed the testing and ISI surveillance requirements to determine whether the testing and surveillance requirements meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, and GDC 5; and are in accordance with SRP Acceptance Criteria 3.8.4.II.7.

The staff reviewed SRP Section 3.8.4, including Appendix D, and the TR for fuel racks. The staff concluded that there are no structural related testing requirements applicable to the design certification of the racks.

In FSAR Tier 2, Revision 2, Section 3.8.4.7, the applicant revised the ISI requirements to indicate that monitoring and maintenance of other Seismic Category I structures is performed in accordance with 10 CFR 50.65 and supplemented with the guidance in RG 1.160. However, the TR for fuel racks only discusses ISI of the neutron absorbing material, but not for the rack structure. Therefore, in RAI 445, Question 03.08.04-25, the staff requested that the applicant clarify the ISI requirements for the fuel racks.

In an April 14, 2011, response to RAI 445, Question 03.08.04-25, the applicant indicated that the ISI examination of the rack modules is limited to verifying the efficacy of the poison material. This is accomplished by installing a sampling tree in the pool as described in the TR for fuel racks, Chapter 6.

The staff reviewed the applicant's April 14, 2011, response to RAI 445, Question 03.08.04-25 and determined that justification for limiting the ISI to verifying the efficacy of the poison material was not provided. ISIs of structures, systems, and components, within the scope of 10 CFR 50.65, are subject to monitoring of their performance or condition in a manner sufficient to provide reasonable assurance that these SSCs are capable of fulfilling their intended functions. Also, FSAR Tier 2, Sections 3.8.3.7, 3.8.4.7, and 3.8.5.7 indicate that monitoring and maintenance of Seismic Category I structures are performed in accordance with the requirements of 10 CFR 50.65 and supplemented with the guidance in RG 1.160, without any limitation for the fuel racks.

In an August 15, 2011, response to RAI 445, Question 03.08.04-25, the applicant revised the prior RAI response to indicate that the requirements for the ISI of the racks will be developed in accordance with 10 CFR 50.65 supplemented with the guidance in RG 1.160, Revision 2. Therefore, the staff considers RAI 445, Question 03.08.04-25 resolved.

ITAAC, Site Parameters, and Interface Requirements

ITAAC: The staff has reviewed the ITAAC associated with FSAR Tier 2, Section 3.8.4 in FSAR Tier 1, Table 2.1.1-8, Table 2.1.1-10, Table 2.1.1-11, Table 2.1.2-3, and Table 2.1.5-3, and finds them acceptable for this area of review.

Site Parameters and Interface Requirements: The staff has reviewed the following site parameter that will be addressed in COL applications: Item No. 3-4, Site-specific loads that lie within the standard plant design envelope for Seismic Category I structures, in FSAR Tier 2, Chapter 1, Table 1.8-1,; and finds it acceptable for this area of review. The staff reviewed interface requirement: Item No.3-5, Buried conduit and duct banks, and pipe and pipe ducts, and piping, in FSAR Tier 2, Table 1.8-1,; and finds it acceptable for this area of review.

3.8.4.5 *Combined License Information Items*

Table 3.8.4-1 provides a list of other Seismic Category I structures related COL information item numbers and descriptions from FSAR Tier 2, Table 1.8-2:

Table 3.8.4-1 U.S. EPR Combined License Information Items

Item No.	Description	FSAR Tier 2 Section
3.8-2	A COL applicant that references the U.S. EPR design certification will describe any differences between the standard plant layout and design of Seismic Category I structures required for site-specific conditions.	3.8.4.1
3.8-3	A COL applicant that references the U.S. EPR design certification will confirm that site-specific loads lie within the standard design envelope for other Seismic Category I structures, or perform additional analyses to verify structural adequacy.	3.8.4.3
3.8-4	A COL applicant that references the U.S. EPR design certification will provide a description of Seismic Category I buried conduit and duct banks.	3.8.4.1.8
3.8-5	A COL applicant that references the U.S. EPR design certification will provide a description of Seismic Category I buried pipe and pipe ducts.	3.8.4.1.9
3.8-7	A COL applicant that references the U.S. EPR design certification will confirm that site-specific conditions for Seismic Category I buried conduit, electrical duct banks, pipe, and pipe ducts satisfy the requirements specified in Section 3.8.4.4.5 and those specified in AREVA NP Topical Report ANP-10264NP-A.	3.8.4.5
3.8-8	A COL applicant that references the U.S. EPR design certification will address site-specific Seismic Category I structures that are not described in this section.	3.8.4.1
3.8-14	A COL applicant that references the U.S. EPR design certification will describe the design and analysis procedures used for buried conduit and duct banks, and buried pipe and pipe ducts.	3.8.4.4.5

Item No.	Description	FSAR Tier 2 Section
3.8-15	A COL applicant that references the U.S. EPR design certification will use results from site-specific investigations to determine the routing of buried pipe and pipe ducts.	3.8.4.4.5
3.8-16	A COL applicant that references the U.S. EPR design certification will perform geotechnical engineering analyses to determine if the surface load will cause lateral and/or vertical displacement of bearing soil for the buried pipe and pipe ducts and consider the effect of wide or extra heavy loads.	3.8.4.4.5
3E-1	A COL applicant that references the U.S. EPR design certification will address critical sections relevant to site-specific Seismic Category I structures.	3E

The staff finds the above listing to be complete. Also, the list adequately describes actions necessary for the COL applicant. No additional COL information items need to be included in FSAR Tier 2, Table 1.8-2 for consideration of other Seismic Category I structures consideration.

3.8.4.6 *Conclusions*

The staff reviewed FSAR Tier 2, Section 3.8.4, to determine whether the design, fabrication, construction, testing, and ISI of other Seismic Category I structures comply with 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, and GDC 5. The staff concludes that, except for the open and confirmatory items described above, the applicant meets the relevant requirements of 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, and GDC 5. The basis for this conclusion is the following:

The applicant meets the requirements of 10 CFR 50.55a and GDC 1, to ensure that the other Seismic Category I structures are designed, fabricated, erected, constructed, tested, and inspected to quality standards commensurate with the importance of the safety function to be performed, by meeting the guidelines in SRP Section 3.8.4; RG 1.69, RG .124, RG 1.142, RG 1.143, RG 1.160, and RG 1.199; ACI 349 with additional guidance provided by RG 1.142 and RG 1.199; and ANSI/AISC N690-1994, including S2004.

The applicant meets the requirements of GDC 2 to ensure that the other Seismic Category I structures are able to withstand the most severe natural phenomena, by analyzing and designing these structures to withstand the most severe earthquake, wind, tornado, and external flood loads that have been established for the U.S. EPR certified design, in load combinations that include these loads.

The applicant meets the requirements of GDC 4 to ensure that the other Seismic Category I structures are appropriately protected against other dynamic effects, by analyzing and designing these structures, where applicable, to withstand the dynamic effects due to pipe whipping, missiles, and discharging fluids associated with the LOCA.

The applicant meets the requirements of GDC 5 to ensure that SSCs are not shared between units or that sharing will not impair their ability to perform their intended safety functions, by analyzing and designing these structures for the design loads and load combinations described

in SRP Section 3.8.4; ACI 349 with additional guidance provided by RG 1.142 and RG 1.199; and ANSI/AISC N690-1994, including S2004.

The applicant meets the requirements of 10 CFR Part 50, Appendix B, to provide a quality assurance program for the design, construction, and operation of SSCs, by meeting the guidelines in SRP Section 3.8.4; RG 1.69, RG 1.127, RG 1.142, RG 1.143, RG 1.160, and RG 1.199; ACI 349 with additional guidance provided by RG 1.142 and RG 1.199; and ANSI/AISC N690-1994, including S2004.

As mentioned above, the acceptability of the design, fabrication, construction, testing, and ISI of other Seismic Category I structures is predicated on the adequate resolution of the open and confirmatory items discussed in this section.

3.8.5 Foundations

3.8.5.1 *Introduction*

This section of the report provides the staff's review and evaluations of FSAR Tier 2, Section 3.8.5, "Foundations," which describes the foundations for Seismic Category I structures including the NI foundation basemat and the independent foundations for the EPGB and the ESWB.

3.8.5.2 *Summary of Application*

FSAR Tier 1: FSAR Tier 1 information associated with this section is found in FSAR Tier 1, Sections 2.1.1, "Nuclear Island," 2.1.2, "Emergency Power Generating Building," and 2.1.5, "Essential Service Water Building."

FSAR Tier 1, Section 2.1.1 provides a physical and functional description of the NI foundation basemat. The NI foundation basemat is a heavily reinforced concrete slab, approximately 110 m by 110 m (360 ft by 360 ft) in plan by 3.0 m (10 ft) in thickness, which supports all NI structures including the RCB, RBIS, RSB, FB, and the Safeguard Buildings. The NI foundation basemat acts together with the RCB to maintain an essentially leak-tight barrier against the uncontrolled release of radioactivity to the environment and to maintain containment design conditions important to safety so that they are not exceeded for as long as postulated accident conditions require.

FSAR Tier 1, Section 2.1.2 provides a physical and functional description of the EPGB foundation basemats. FSAR Tier 1, Section 2.1.2 describes the arrangement and key design features of the foundation. Key mechanical design features, including seismic classifications, are also specified.

FSAR Tier 1, Section 2.1.5 provides a physical and functional description of the ESWB foundation basemats. FSAR Tier 1, Section 2.1.5 describes the arrangement and key design features of the foundation. Key mechanical design features, including seismic classifications, are also specified.

FSAR Tier 2: The applicant has provided an FSAR Tier 2 description of foundations in Section 3.8.5, summarized as follows:

Description of the Foundations

FSAR Tier 2, Section 3.8.5.1, “Description of the Foundations,” describes the following Seismic Category I foundations: The NI foundation basemat; the EPGB foundation basemats; and the ESWB foundation basemats. FSAR Tier 2, Figure 3B-1, “Dimensional Arrangement Reference Plant Building Location,” shows the outline of these foundations, as well as their respective locations.

FSAR Tier 2, Section 3.8.5.1, identifies that (1) foundations for buried items are addressed in FSAR Tier 2, Section 3.8.4, “Other Seismic Category I Structures”; (2) structures described within this section are not shared with any other power plant units; (3) FSAR Tier 2, Section 3.7.2, “Seismic System Analysis,” addresses design requirements for non-safety-related structures in order to preclude adverse interaction effects on Seismic Category I structures; and (4) a COL applicant that references the U.S. EPR design certification will address site-specific foundations for Seismic Category I structures that are not described in this section.

FSAR Tier 2, Section 3.8.5.1.1, “Nuclear Island Common Basemat Structure Foundation Basemat,” provides details for the NI foundation basemat, which is a heavily reinforced concrete slab that supports the RCB and the RSB located near its center, the FB, and the four Safeguard Buildings. The basemat has a cruciform shape in plan that is embedded into the supporting media approximately 12.20 m (40 ft). The NI basemat also provides anchorage for the vertical post-tensioning tendons of the RCB. This portion of the NI basemat is designed in accordance with the ASME Code, Section III, Division 2, Subsection CC. A circular gallery is provided beneath the NI basemat for maintenance access to the bottom of the vertical post-tensioning tendons. The tendon access gallery is approximately 6.10 m (20 ft) wide by 5.50 m (18 ft) high.

Horizontal shear loads are transferred from the NI basemat to the underlying media by friction between the bottom of the basemat, mud mat, and the supporting media, and by passive earth pressure on the below-grade walls of the supported Seismic Category I structures. In addition, the tendon gallery is classified as a Seismic Category I structure and analyzed as a shear key to transfer loads to the supporting media. The walls and slab of the tendon access gallery are designed according to ACI 349-01. FSAR Tier 2, Section 2.5.4.2, “Properties of Subsurface Materials,” describes the angle of internal friction of the supporting media assumed in the U.S. EPR design.

FSAR Tier 2, Section 3.8.5.1.2, “Emergency Power Generating Buildings Foundation Basemats,” provides details for the EPGB foundation basemats. Each EPGB basemat supports the superstructure and associated equipment. At the basemat interface, heavily reinforced concrete shear walls transfer loads from floors and the roof. Each basemat is embedded approximately 1.50 m (5 ft) in the supporting media; the overall dimensions are approximately 54.20 m (178 ft) long by 28.80 m (94.5 ft) wide by 1.80 m (6 ft) thick. There is also a system of shear keys embedded in the supporting media under the basemat.

FSAR Tier 2, Section 3.8.5.1.3, “Essential Service Water Buildings Foundation Basemats,” provides details for the ESWB foundation basemats. Each ESWB basemat supports the superstructure and water basin. At the basemat interface, heavily reinforced concrete shear walls transfer loads from the floors and the roof. Each basemat is embedded approximately 6.40 m (21 ft) into the supporting media; the overall dimensions are approximately 50.0 m (164 ft) long by 32.90 m (108 ft) wide by 1.80 m (6 ft) thick.

Applicable Codes, Standards, and Specifications

FSAR Tier 2, Section 3.8.5.2, “Applicable Codes, Standards, and Specifications,” identifies that codes, standards, specifications, design criteria, regulations, and regulatory guides applicable to the design, fabrication, construction, testing, and ISI of Seismic Category I foundations are the same as those specified in FSAR Tier 2, Section 3.8.4.2, “Applicable Codes, Standards, and Specifications.” The portion of the NI foundation basemat supporting the RCB/RSB is designed in accordance with ASME Code, Section III, Division 2, Subsection CC.

Loads and Load Combinations

FSAR Tier 2, Section 3.8.5.3, “Loads and Load Combinations,” describes the loads and load combinations for Seismic Category I foundations. The loads and load combinations are the same as those defined in FSAR Tier 2, Section 3.8.4.3, “Loads and Load Combinations.” In addition, the NI foundation basemat is designed for the loads and load combinations for the RCB, as described in FSAR Tier 2, Section 3.8.1.3, “Loads and Load Combinations.” The NI foundation basemat provides anchorage for the RCB vertical post-tensioning tendons. The portion of the basemat under the RCB/RSB is designed to accommodate loads from containment. The FSAR further states that the load combinations identified in SRP Section 3.8.5, “Foundations,” Revision 2, do not include certain independent loadings which should be considered to account for potential structure-to-structure effects on the NI common foundation basemat. To account for this, the load combinations in SRP Section 3.8.5 are modified to include the necessary additional independent loadings.

FSAR Tier 2, Section 3.8.5.3 also identifies that the following additional load combinations are applied for Seismic Category I foundations, to consider sliding and overturning due to earthquakes, winds, and tornados and against flotation due to floods:

D + H + W + F + Fb

D + H + E’+ F + Fb

D + H + Wt + F + Fb

D + Fb + F

where D is the dead load, H represents the soil loads and lateral earth pressure loads that result from soil bearing pressures applied to buried exterior walls and structures up to the finished grade elevation of the surrounding soil, W represents the wind loads, F represents the hydrostatic loads due to fluids stored in pools and tanks, Fb is the buoyant force of the design basis flood at maximum site water level, E’ represents the SSE loads, and Wt represents the tornado loads.

Seismic Category I foundations are also designed for the effects of short term and long term settlements. FSAR Tier 2, Section 2.5, “Geology, Seismology, and Geotechnical Engineering,” provides the settlement limits considered for the U.S. EPR design.

Design and Analysis Procedures

FSAR Tier 2, Section 3.8.5.4, “Design and Analysis Procedures,” contains a very detailed discussion of the design and analysis procedures for the Seismic Category I foundation basemats. FSAR Tier 2, Section 3.8.5.4.1, “General Procedures Applicable to Seismic

Category I Foundations," describes design and analysis procedures applicable to all Seismic Category I foundation basemats. FSAR Tier 2, Section 3.8.5.4.2, "Nuclear Island Common Basemat Structure Foundation Basemat," Section 3.8.5.4.3, "Emergency Power Generating Buildings Foundation Basemats," and Section 3.8.5.4.4, "Essential Service Water Building Foundation Basemats," describe specific analysis procedures applicable to the NI foundation basemat, the EPGB foundation basemats, and the ESWB foundation basemats, respectively.

FSAR Tier 2, Section 3.8.5.4.1 indicates that concrete foundation basemats for Seismic Category I structures are analyzed as thick slabs on elastic supports to represent the supporting media. Loads are applied to the foundation basemats by the connecting walls and columns that comprise the building structures being supported, and by the equipment supported directly on the foundations. The intersecting walls also stiffen the basemat slabs which increase resistance to bending moments resulting from bearing pressures under the slabs. Foundations are analyzed for the loads and load combinations identified in FSAR Tier 2, Section 3.8.5.3.

Seismic Category I foundation basemats transfer vertical loads from the buildings to the soil by direct bearing of the basemat on the soil. Horizontal shear forces, which include wind, tornados, and earthquakes, are transferred to the soil by friction along the bottom of the foundation basemat, shear key, or by passive earth pressure.

The design and analysis procedures for the foundations are the same as those described in FSAR Tier 2, Section 3.8.1.4, "Design and Analysis Procedures," Sections 3.8.3.4, "Design and Analysis Procedures," and 3.8.4.4, "Design and Analysis Procedures," for the structures that are supported by the respective foundations. Foundation design is performed for the various soil cases described in FSAR Tier 2, Section 3.7.1, "Seismic Design Parameters." The soil and seismic parameters used for the analysis and design of Seismic Category I structures and their foundations are described in FSAR Tier 2, Section 2.5 and Section 3.7, "Seismic Design." FSAR Tier 2, Section 2.5.4.2 identifies the soil parameters used to determine soil loads and lateral earth pressures.

Loads for normal lateral earth pressure consider saturated soil up to a groundwater elevation of - 1.0 m (-3.3 ft) relative to site finished grade. For external floods, the lateral soil loads consider saturated soil up to elevation -0.30 m (-1.0 ft) relative to the site finished grade. Seismic loads due to the three perpendicular components of the earthquake motion are combined using the SRSS method or time-history analysis methods in conformance with the guidance provided in RG 1.92, "Combining Modal Responses and Spatial Components in Seismic Response Analysis," July 2006, Revision 2. The components of soil loads due to the SSE are determined using densities corresponding to saturated soil in order to account for the weight of the soil plus the weight of either normal or flood water levels.

The minimum factors of safety required to prevent sliding and overturning are presented in FSAR Tier 2, Table 3.8-11, "Minimum Required Factors of Safety Against Overturning, Sliding, and Flotation for Foundations." Where passive pressure is assumed to act on embedded structures, in the stability analysis against sliding and overturning, the walls of such structures are designed to withstand the passive pressure.

When the effects of vertical seismic acceleration are included in the stability analysis against sliding, the unfactored dead weight of the structure is used to calculate the resistance to sliding due to friction. The effects of buoyancy of saturated soil due to a groundwater level of elevation -1.0 m (-3.3 ft) below finished grade or to a flood water level of elevation -0.30 m (-1.0 ft) below finished grade are considered in the sliding and overturning analyses. For uplift analyses, of

flotation and seismic overturning, the dead load includes the weight of water which is permanently stored in pools and tanks.

Differential foundation settlements effects and dead load are considered to act concurrently using the same load factors. In addition, the effects of varying settlements between adjacent foundations are considered for the design of mechanical and electrical systems such as piping, duct banks, and conduit that are routed between the structures which are supported on separate basemats. The analysis and design procedures for Seismic Category I buried items that interface with structures on separate foundations are described in FSAR Tier 2, Section 3.8.4.4.5, "Buried Conduit and Duct Banks, and Buried Pipe and Pipe Ducts."

FSAR Tier 2, Section 3.8.5.4.2 indicates that the NI foundation basemat is analyzed and designed using the overall ANSYS static finite element model, which is described in FSAR Tier 2, Section 3.8.1.4.1, "Computer Programs." This model includes the RCB, RBIS, RSB, FB, SBs, as well as the NI foundation basemat.

ANSYS SOLID45 solid elements are used to model the NI foundation basemat in the static FE model. Three to five layers of SOLID45 elements are used, depending on the thickness of the basemat; four elements are used on average in the typical 3.0 m (10 ft) thick basemat areas (see FSAR Tier 2, Figure 3.8-103, "Nuclear Island Common Basemat Structure Foundation Basemat ANSYS Model").

Springs are used in the static FE model to represent the supporting media. Spring constants vary for each soil case based on the soil properties. To address potential dishing effects, the spring constants also vary depending on the location under the basemat, with a stiffer constant on the outer edges and a softer constant in the middle of the basemat. An elliptical distribution is used to represent this latter variation.

A 3-D FE dynamic model used in the seismic SSI analysis of the NI structures was also used to evaluate dynamic soil bearing pressures, as well as sliding and overturning due to seismic loads. This model explicitly represents the transient nature of the seismic loads, the properties of the soils, and the dynamic characteristics of the NI structure. This approach is able to more realistically capture the response of the structures to seismic loads than the static FE model.

Detailed design procedures for the NI foundation basemat are described with the critical section designs presented in FSAR Tier 2, Appendix 3E, "Design Details and Critical Sections for Safety-Related Category I Structures." FSAR Tier 2, Section 3.8.3 provides a description of the design procedures for the RBIS basemat, which is located above the containment liner plate and bears directly on the NI basemat.

FSAR Tier 2, Section 3.8.5.4.3 indicates that horizontal shear loads are transferred from the EPGB foundation basemat to the underlying soil (1) by friction between the bottom of the basemat, the mud mat, and the supporting medium; and (2) by passive earth pressure on the embedded perimeter of the basemat. FSAR Tier 2, Section 3.8.5.4.4 indicates that horizontal shear loads are transferred from the ESWB foundation basemat to the underlying soil (1) by friction between the bottom of the basemat, the mud mat, and the supporting medium; and (2) by dynamic soil pressure and passive earth pressure on the below-grade walls, which are embedded to a nominal depth of 6.4 m (21 ft).

The EPGB and ESWB foundation basemats are analyzed and designed using the GT STRUDL finite element analysis code. The corresponding FE models contain both the building superstructure and the foundation basemat. The EPGB and ESWB are analyzed for all

applicable design loads and design load combinations described in FSAR Tier 2, Section 3.8.4.3. FSAR Tier 2, Figures 3.8-104, “Emergency Power Generating Building Foundation Basemat Model,” and 3.8-105, “Essential Service Water Building Foundation Basemat Model,” show the foundation basemat portions of the EPGB and ESWB FE models, respectively. The finite element models consist of SBHQ6 rectangular elements capable of capturing both in-plane and out-of-plane behavior. Elastic soil spring boundary conditions are used to simulate the stiffness of the supporting medium. Detailed design procedures for the EPGB and ESWB are described with the critical section designs presented in FSAR Tier 2, Appendix 3E.

Structural Acceptance Criteria

FSAR Tier 2, Section 3.8.5.5, “Structural Acceptance Criteria,” indicates that the allowable stresses, strains, deformations, and other design criteria for Seismic Category I concrete foundations are in accordance with ACI 349-01 and its appendices, including the exceptions specified in RG 1.142, “Safety-Related Concrete Structures for Nuclear Power Plants (Other than Reactor Vessels and Containments),” November 2001, Revision 2. The portion of the NI foundation basemat that supports the RCB/RSB is designed in accordance with the ASME Code and RG 1.136, “Design Limits, Loading Combinations, Materials, Construction, and Testing of Concrete Containments,” March 2007, Revision 3, for containment loadings, as described in FSAR Tier 2, Section 3.8.1, “Concrete Containment.” The allowable stresses, strains, deformations, and other design criteria for structural steel elements of Seismic Category I foundations are in accordance with ANSI/AISC N690-1994, including S2004.

The FSAR indicates that Seismic Category I foundations must satisfy the factors of safety against overturning, sliding, and flotation defined in FSAR Tier 2, Table 3.8-11, and that the calculated minimum factors of safety for the NI basemat are provided in FSAR Tier 2, Table 3.8-12, “Minimum Factors of Safety Against Overturning, Sliding, and Flotation for Foundations – NI Common Basemat Structure.” In addition, acceptance criteria for the media supporting Seismic Category I foundations, and acceptance criteria for settlement for Seismic Category I foundations are addressed in FSAR Tier 2, Section 2.5.

The FSAR further indicates that deviations from the approved design will be summarized in an as-built report, and it will be confirmed that the as-built Seismic Category I foundations are capable of withstanding the design basis loads described in FSAR Tier 2, Section 3.8.5.3.

The FSAR also specifies that a COL applicant that references the U.S. EPR design certification will evaluate site-specific methods for shear transfer between the foundation basemats and the supporting media, soil or rock, for site-specific characteristics that are not within the envelope of the parameters specified in FSAR Tier 2, Section 2.5.4.2.

Materials, Quality Control, and Special Construction Techniques

FSAR Tier 2, Section 3.8.5.6, “Materials, Quality Control, and Special Construction Techniques,” describes the materials, quality control, and special construction techniques applicable to Seismic Category I foundations.

FSAR Tier 2, Section 3.8.5.6.1, “Materials,” indicates that concrete, reinforcing steel, and structural steel materials for Seismic Category I foundations are the same as described in FSAR Tier 2, Section 3.8.3.6, “Materials, Quality Control, and Special Construction Techniques,” with the some additional requirements which include:

- Materials for the portion of the NI foundation basemat supporting the RCB/RSB are described in FSAR Tier 2, Section 3.8.1.6, "Materials, Quality Control, and Special Construction Techniques."

- Concrete used in the construction of Seismic Category I foundations has a minimum compressive strength (f'c) equal to 27.6 MPa (4,000 psi).

- Waterproofing and dampproofing systems are addressed in FSAR Tier 2, Section 3.4.2, "External Flood Protection."

- Concrete exposed to aggressive environments, as defined in ACI 349-01, Chapter 4, shall meet the durability provisions of ACI 349-01, Chapter 4, or ASME B&PV Code Section III, Division 2, Subarticle CC-2231.7, as applicable

- Waterproofing and dampproofing systems of all below-grade Seismic Category I structures subject to aggressive environments, as defined in ACI 349-01, Chapter 4, shall be evaluated for use in such environments,

The FSAR identifies that the waterproofing system will provide adequate frictional characteristics at its interface with concrete, as specified in FSAR Tier 2, Table 2.1-1, "U.S. EPR Site Design Envelope," and that this will be demonstrated by vendor testing. The contact surface between the waterproofing system and the concrete will be finished in accordance with manufacturer recommendations.

The FSAR further identifies that a COL applicant that references the U.S. EPR design certification will (1) evaluate the use of epoxy coated steel reinforcement for foundations subjected to aggressive environments, as defined in ACI 349-01, Chapter 4; and (2) evaluate the applicability of the waterproofing system for all Seismic Category I foundations for use in aggressive environments. The concrete of Seismic Category I foundations which are exposed to aggressive environments will meet the durability provisions of ACI 349-01, Chapter 4 or ASME, Section III, Division 2, Subarticle CC-2231.7, as applicable.

FSAR Tier 2, Section 3.8.5.6.2, "Quality Control," refers to FSAR Tier 2, Section 3.8.3.6 for quality control procedures for other Seismic Category I structures.

FSAR Tier 2, Section 3.8.5.6.3, "Special Construction Techniques," indicates that construction of Seismic Category I foundations uses proven methods and that no special or unique construction techniques are used. Modular construction is used for prefabricating portions of reinforcing and concrete formwork, to the extent practical; these methods have been used extensively in the construction industry.

<u>Testing and Inservice Surveillance Requirements</u>

FSAR Tier 2, Section 3.8.5.7, "Testing and Inservice Inspection Requirements," summarizes the testing and ISI requirements for Seismic Category I foundations. Monitoring and maintenance of Seismic Category I foundations are performed in accordance with 10 CFR 50.65, "Requirements for monitoring the effectiveness of maintenance at nuclear power plants," as supplemented by the guidance in RG 1.160, "Monitoring the Effectiveness of Maintenance at Nuclear Power Plants," Revision 2. For the portion of the NI foundation basemat that supports the RCB/RSB, additional testing and ISI requirements are described in FSAR Tier 2,

Section 3.8.1.7.2, “Long-Term Surveillance.” Physical access is provided for ISI of exposed portions of Seismic Category I foundations.

The FSAR indicates that a COL applicant that references the U.S. EPR design certification will (1) identify whether site-specific settlement monitoring is required for Seismic Category I foundations, based on site-specific conditions; and (2) describe the programs to examine inaccessible portions of below-grade concrete structures for degradation, and to monitor groundwater chemistry.

ITAAC: There are no ITAAC items for this area of review.

3.8.5.3 *Regulatory Basis*

The relevant requirements of NRC regulations for this area, and the associated acceptance criteria, are given in NUREG-0800, Section 3.8.5, “Foundations,” Revision 2, and are summarized below. NUREG-0800, Section 3.8.5 also identifies review interfaces with other SRP sections.

1. GDC 1, “Quality Standards and Records,” as it relates to Seismic Category I structures being designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety function to be performed.

2. GDC 2, “Design Bases for Protection against Natural Phenomena,” as it relates to the capability of Seismic Category I structures to perform their safety function, to withstand the effects of natural phenomena, such as earthquakes, tornadoes, floods, and the appropriate combination of all loads.

3. GDC 4, “Environmental and Dynamic Effects Design Bases,” as it relates to the protection of Seismic Category I structures against dynamic effects, including the effects of missiles, pipe whipping, and discharging fluids, that may result from equipment failures and from events and conditions outside the nuclear power unit.

4. GDC 5, “Sharing of Structures, Systems, and Components,” as it relates to safety-related structures not being shared among nuclear power units, unless it can be shown that such sharing will not significantly impair their ability to perform their safety functions.

5. 10 CFR Part 50, “Domestic Licensing of Production and Utilization Facilities,” specifically, 10 CFR 50.55a, “Codes and Standards,” as they relate to codes and standards.

6. 10 CFR Part 50, Appendix B “Quality Assurance Criteria for Nuclear Power Plants and Fuel Processing Plants,” as it relates to the quality assurance criteria for nuclear power plants.

Acceptance criteria adequate to meet the above requirements include:

1. RG 1.127, “Inspection of Water-Control Structures Associated with Nuclear Power Plants,” Revision 1, March 1978, as it relates to the inservice inspection and surveillance program for water-controled structures associated with emergency cooling water systems or flood protection of nuclear power plants.

2. RG 1.136, “Design Limits, Loading Combinations, Materials, Construction, and Testing of Concrete Containments,” Revision 3, March 2007, as it relates to the design requirements for concrete containments.

3. RG 1.142, “Safety-Related Concrete Structures for Nuclear Power Plants (Other Than Reactor Vessels and Containments),” Revision 2, November 2001, as it relates to the design requirements for safety-related concrete structures, excluding concrete containments.

4. RG 1.160, “Monitoring the Effectiveness of Maintenance at Nuclear Power Plants,” Revision 2, March 1997, as it relates to the monitoring and maintenance of safety-related structures.

5. RG 1.199, “Anchoring Components and Structural Supports in Concrete,” November 2003, as it relates to the design, evaluation and quality assurance of anchors (steel embedments) used for component and structural supports on concrete structures.

3.8.5.4 *Technical Evaluation*

The staff’s evaluation is based on a complete review of FSAR Tier 2, Sections 3.8.5.1 through 3.8.5.7, including all subsections.

This section describes the staff’s review and evaluation of FSAR Tier 2, Section 3.8.5, against the guidance of SRP Section 3.8.5 to ensure that the applicant has adequately addressed identified technical issues.

<u>Description of the Structures</u>

The staff reviewed the description of the Seismic Category I foundations to ensure that it contains sufficient information to define the primary structural aspects and elements that are relied upon to perform the safety-related functions of these foundations. The primary function of a foundation is to transmit the loads imposed by the superstructure to the underlying supporting media, rock, or soil. With the exceptions described below, the staff finds that the description meets the applicable regulatory requirements and regulatory guidance in SRP Acceptance Criteria 3.8.5.II.1.

In FSAR Tier 2, Section 3.8.5.1.1, the applicant described the tendon gallery beneath the NI foundation basemat. The tendon gallery is a concrete structure provided for maintenance access to the bottom of the vertical post-tensioning tendons of the RCB, which are anchored at the underside of the NI basemat. The applicant stated that the connection between the tendon gallery and the NI basemat allows for relative differential movement. However, no design details for this connection were provided in the FSAR. The staff could not determine whether the connection design would effectively minimize the transfer of seismic base shear, and static and dynamic bearing pressures between the NI basemat and the tendon gallery. An inadequate connection design could result in the unintended transfer of loads to the tendon gallery, for which the latter is not designed, and could ultimately lead to structural damage of the tendon gallery and the NI basemat.

In addition, the staff could not determine whether the connection design would prevent water infiltration into the tendon gallery space. The staff also noted that accumulation of water into this space could inhibit inspection procedures and possibly lead to corrosion of the tendon anchorages, which are a fundamental component of the post-tensioning system that ensures

the structural integrity of the containment. Therefore, in RAI 155, Question 03.08.05-2, the staff requested that the applicant provide design details for this connection and clarify how it is designed to minimize load transfer and prevent water infiltration.

In a May 29, 2009 response to RAI 155, Question 03.08.05-2, the applicant provided a conceptual detail which showed waterstops and a separation joint filled with compressible material that allowed for differential movement between the two concrete structures. However, subsequent to the May 29, 2009, RAI response, and as part of the resolution of the technical issues associated with the seismic stability of the NI foundation basemat (see below under “Design and Analysis Procedures”), the applicant modified the design for the NI foundation basemat to consider the tendon gallery as a shear key; that is, a structural element integrally connected to the NI foundation basemat and designed to transfer a portion of the seismic base shear to the surrounding media, soil or rock. Therefore, in follow-up RAI 354, Question 03.08.05-19, the staff requested that the applicant (1) update the FSAR so that it is consistent with the modified design; (2) confirm that the tendon gallery is classified as a Seismic Category I structure; (3) clarify whether a maintenance program in accordance with 10 CFR 50.65 will be implemented for the tendon gallery; and (4) provide design information comparable to other Seismic Category I structures.

In an April 15, 2010, response to RAI 354, Question 03.08.05-19, the applicant provided proposed revisions to the FSAR that updated the description of the tendon gallery and confirmed that it is a Seismic Category I structure, designed in accordance with the methodology described in FSAR Tier 2, Section 3.8.5, and monitored and maintained in accordance with 10 CFR 50.65. The staff finds this information acceptable, because it complies with SRP Acceptance Criteria 3.8.5.II.1 and 3.8.5.II.7.

During the April 26-30, 2010, and February 14-17, 2011, onsite audits, the applicant stated that the tendon gallery had been selected as a critical section; therefore, the requested design details and design information would be provided upon completion of the critical section design.

The staff verified that the changes proposed by the applicant were incorporated into FSAR Tier 2, Revision 2. Therefore, based on the above discussion, the staff considers RAI 155, Question 03.08.05-2 and RAI 354, Question 03.08.05-19 resolved. Further evaluation of the critical section design is described in the discussion of RAI 155, Questions 03.08.01-24, and 03.08.04-6 in Section 3.8.1.4 of this report, under “Design and Analysis Procedures.”

In FSAR Tier 2, Section 3.8.5.1.1, the applicant described the NI foundation basemat as a cruciform shape with outline dimensions of approximately 110 m (360 ft) by 110 m (360 ft) by 3.0 m (10 ft) thick; FSAR Tier 2, Section 3.8.5.1.1 also states that a foundation basemat of lesser thickness would be considered for rock sites. However, the staff could not identify in FSAR Tier 2, Section 3.8.5 or Appendix 3E any design details for a basemat of reduced thickness. Therefore, in RAI 155, Question 03.08.05-1, the staff requested that the applicant clarify whether a basemat of reduced thickness would be considered for rock sites, provide complete design details for such alternate designs, and explain the criteria for determining when such alternate designs would be applicable.

In a July 31, 2009 response to RAI 155, Question 03.08.05-1, the applicant stated that the reference to an alternate basemat design of reduced thickness for rock sites will be deleted from the FSAR. The staff also verified that the changes proposed by the applicant were incorporated into FSAR Tier 2, Revision 2. On this basis, the staff considers RAI 155, Question 03.08.05-1 resolved.

The review of FSAR Tier 2, Revision 0, Appendix 3E, identified several design details for the NI, EPGB, and ESWB foundation basemats that were either missing or inconsistent. First,:the specified horizontal reinforcement configurations for the EPGB and ESWB basemats did not indicate whether this reinforcement was in the top or bottom of the basemat slab, nor was there any specification of vertical (shear) reinforcement (FSAR Tier 2, Figure 3E.2-3, "Reinforcement Sketch for EPGB Basemat Foundation," for EPGB, and Figure 3E.3-3, " Reinforcement Sketch for ESWB Basemat Foundation," for ESWB). Additionally, there was inconsistency in the specified reinforcement for the NI basemat between FSAR Tier 2, Table 3E.1-37, "Reinforcement Summary for the NI Foundation Basemat and RB Internal Structures Base Slab," and Figure 3E.1-75, "Reinforcement Pattern for RB Internal Structures Base Slab - Elevation -25'-7" to -7'-6 1/2"." Therefore, in RAI 155, Question 03.08.05-17 and RAI 306, Question 03.08.01-43, the staff requested that the applicant clarify these inconsistencies and provide additional figures in FSAR Tier 2, Appendix 3E, that show key cross sections and clearly identify size, location, and spacing of top and bottom reinforcement, as well as vertical reinforcement, for the NI, EPGB, and ESWB foundation basemats.

In a March 31, 2009, response to RAI 155, Question 03.08.05-17, the applicant provided limited additional information for the EPGB and ESWB foundation basemats, but did not include the requested cross section details in the FSAR, nor did it address the inconsistency in the design information for the NI basemat. The staff determined that the applicant's response did not comply with SRP Acceptance Criteria 3.8.5.II.1, which requires that sufficient information be provided to define the primary structural aspects and elements of Seismic Category I foundations.

In a March 12, 2010, response to RAI 306, Question 03.08.01-43, the applicant stated that the design information provided in the March 31, 2009, response to RAI 155, Question 03.08.05-17 was superseded. Due to design changes being implemented for the NI, EPGB, and ESWB, the requested design details and design information would be provided upon completion of the critical section design. In particular, FSAR Tier 2, Appendix 3E, including content, tables, and figures, would be revised to reflect the completed critical section design and would be provided in the response to RAI 155, Question 03.08.04-6. The staff finds acceptable the applicant's commitment to submit the critical section design information, including FSAR Tier 2, Appendix 3E content, tables, and figures. This information, when submitted, will be reviewed by the staff to verify compliance with SRP Acceptance Criteria 3.8.5.II.1.

The staff notes that based on the above responses, RAI 155, Question 03.08.05-17 and RAI 306, Question 03.08.01-43 are superseded by **RAI 155, Question 03.08.04-6, which is being tracked as an open item** pending the completion of the critical section design. The response to RAI 155, Question 03.08.04-6, to be submitted by the applicant upon the completion of the critical section design, will be reviewed by the staff to verify that the applicant fulfills the commitments as discussed above. The resolution of the technical issues associated with critical section design is described in the discussion of RAI 155, Questions 03.08.01-24, and 03.08.04-6 in Section 3.8.1.4 of this report under, "Design and Analysis Procedures."

Applicable Codes, Standards, and Specifications

To ensure that the Seismic Category I foundations can perform their intended safety function, the staff verified that the applicable codes, standards, and specifications used in the design, fabrication, construction, testing, and ISI meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, and GDC 5; and are in accordance with SRP Acceptance Criteria 3.8.5.II.2. With the exceptions described below, the staff finds

that the applicable codes, standards, and specifications meet the regulatory requirements and regulatory guidance cited above.

The staff reviewed FSAR Tier 2, Section 3.8.5, and could not conclude whether the design and installation of concrete anchors were in accordance with (1) ACI 349-01, Appendix B, subject to the conditions and limitations specified in RG 1.199, and (2) IE Bulletin 79-02, Revision 2; or complied with SRP Acceptance Criteria 3.8.1.II.2 and 3.8.4.II.2. The staff was concerned whether the applicant's criteria resulted in the adequate design of structural connections that utilize these anchors, with the appropriate safety margins. The same issues were also raised in the review of FSAR Tier 2, Section 3.8.3, "Concrete and Steel Internal Structures of Concrete Containment," Therefore, in RAI 155, Question 03.08.03-2 and RAI 283, Question 03.08.03-20, the staff requested that the applicant clarify the criteria used for design and installation of concrete anchors. The resolution of these RAIs is discussed in Section 3.8.3.4 of this report under "Applicable Codes, Standards, and Specifications," where the staff concluded that these questions are resolved.

In FSAR Tier 2, Section 3.8.5.4, the applicant identified ACI 349-01 as the code used for the design and construction of reinforced concrete foundations outside the containment. However, RG 1.142 endorses ACI 349-97, with certain regulatory positions. Since ACI 349-01 is not endorsed by RG 1.142, the staff was concerned with the use of a later edition of the ACI 349 code that has not been reviewed for applicability to nuclear power plants. To ensure the safety of the reinforced concrete foundations, the staff would need to review the application of ACI 349-01 on a case-by-case basis. The same issues were also raised in the review of FSAR Tier 2, Section 3.8.3.2, "Applicable Codes, Standards, and Specifications." Therefore, in RAI 155, Question 03.08.03-3 and RAI 376, Question 03.08.03-21, the staff requested that the applicant (1) identify the differences between ACI 349-01 and ACI 349-97; (2) identify which differences were relaxed provisions; (3) provide the technical basis for using any relaxed provisions; and (4) provide the technical basis for using an isolated provision from ACI 349-06: the shear strength reduction factor of 0.85, which is different from either ACI 349-01 or ACI 349-97. The details of the resolution to these RAIs are discussed in Section 3.8.3.4 of this report under "Applicable Codes, Standards, and Specifications" where the staff concluded that these questions are resolved.

Loads and Load Combinations

To ensure that the Seismic Category I foundations can perform their intended safety function, the staff verified that the loads and load combinations used in the design meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, and GDC 5; and are in accordance with SRP Acceptance Criteria 3.8.5.II.3. With the exceptions described below, the staff finds that the loads and load combinations meet the regulatory requirements and regulatory guidance cited above.

The staff reviewed FSAR Tier 2, Section 3.8.1.3.1, "Design Loads," and FSAR Tier 2, Section 3.8.3.3.1, "Description of the Internal Structures," and determined that certain loads described, did not provide sufficient detail to determine whether they complied with SRP Acceptance Criteria 3.8.1.II.3 and 3.8.3.II.3. These included dead loads associated with the weight of miscellaneous components (e.g., cable tray systems, conduit systems, HVAC systems), uniformly distributed dead loads (floor, platform, and wall), uniformly distributed live loads (floor, platform, and roof), and hydrostatic loads. It was unclear to the staff whether these loads were adequately applied to the finite element model of the NI foundation basemat, which supports the RCB and RBIS. In addition, it was unclear to the staff whether the masses

corresponding to all applicable dead loads and a percentage of live loads were included in the masses used in the seismic analysis. The staff was concerned that the applicant was not taking into consideration all applicable gravity and seismic inertial loads in the design and analysis of the NI foundation basemat. Therefore, in RAI 155, Question 03.08.01-5, the staff requested that the applicant provide the magnitude of the gravity and seismic inertial loads, and confirm that this information was included in the structural design as applicable. The resolution of this RAI is discussed in Section 3.8.1.4 of this report under "Loads and Load Combinations," where the staff concluded that it is resolved.

Design and Analysis Procedures

The staff reviewed the design and analysis procedures used for the Seismic Category I foundations to ensure that the procedures meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, and GDC 5; and are in accordance with SRP Acceptance Criteria 3.8.5.II.4. The staff also reviewed the FSAR to establish that the design is essentially complete, as required by 10 CFR 52.47(c). With the exceptions described below, the staff finds that the design and analysis procedures meet the regulatory requirements and regulatory guidance cited above.

The staff reviewed the maximum static and dynamic bearing pressures specified for the U.S. EPR standard design reported in FSAR Tier 1, Table 5.0-1, "Site Parameters for the U.S. EPR Design," and FSAR Tier 2, Table 2.1-1, as well as in FSAR Tier 2, Section 2.5.4.10.1, "Bearing Capacity," and FSAR Tier 2, Section 3.8.5.4.1, to verify that these FSAR sections comply with SRP Acceptance Criteria 3.8.5.II.4. The maximum static bearing pressure is the maximum pressure imposed by the foundations on the underlying supporting media, soil or rock, from gravity loads. The maximum dynamic bearing pressure is the maximum pressure imposed by the foundations on the supporting media, typically at the toe of the foundations, from seismic and other dynamic loads. This information is important, because it is used to determine whether the supporting media at a particular site has sufficient bearing capacity to withstand the pressures imposed by the foundations with an adequate safety margin.

The staff reviewed FSAR Tier 1, Table 5.0-1 and FSAR Tier 2, Table 2.1-1, and concluded that a number of site parameters used in the analysis and design of Seismic Category I structures and foundations were not included in these tables. The tables included the static bearing capacities for EPGB and ESWB basemats, the dynamic bearing capacities for all basemat foundations, angle of internal friction of the soil and backfill, total settlements beneath each Seismic Category I basemat foundation, and other key parameters that were relied upon in the design of the foundation walls and evaluations for foundation stability associated with FSAR Tier 2, Section 3.8.5. To meet the requirements in 10 CFR 52.47(a)(1), in RAI 155, Question 03.08.05-16 and RAI 354, Question 03.08.05-23, the staff requested that the applicant identify the set of properties of the supporting media, soil or rock, used in the various analyses and designs of Seismic Category I structures and foundations, and include these parameters in the aforementioned FSAR tables.

In a June 30, 2009, response to RAI 155, Question 03.08.05-16, the applicant identified the sections in the FSAR that describe the properties of the supporting media, soil or rock, used in the various analyses and designs of Seismic Category I structures and foundations. However, from the review of this information, the staff could not determine whether all of the key parameters were included in these FSAR sections. Furthermore, this information was not incorporated in FSAR Tier 1, Table 5.0-1 and FSAR Tier 2, Table 2.1-1, as requested by the staff. The staff was concerned that without a complete set of key information and parameters

clearly identified in FSAR Tier 1, Table 5.0-1 and FSAR Tier 2, Table 2.1-1, it would not be possible for a COL applicant to reconcile all site-specific parameters with those postulated for the design of the foundations, as required by 10 CFR 52.47(a)(1).

In a May 4, 2011, response to RAI 354, Question 03.08.05-23, the applicant provided a revised version of FSAR Tier 1, Table 5.0-1, which included the following key information and parameters: (1) The minimum angle of internal friction is 26.6 degrees (associated with the development of minimum coefficient of static friction in the in-situ soil and backfill); (2) the minimum shear wave velocity (low strain best estimate average value at bottom of basemat) is 305 m/s (1,000 ft/s); (3) the maximum static bearing pressure is equal to 1,054 kPa (22,000 psf), which must be less than the ultimate static bearing capacity divided by a factor of safety equal to 3.0, and is conservatively assumed to be the same for the NI, EPGB, and ESWB foundations; (4) the maximum dynamic bearing pressure is equal to 1,677 kPa (35,000 psf) at the toe of the basemat, which must be less than the ultimate dynamic bearing capacity divided by a factor of safety equal to 2.0, and is conservatively assumed to be the same for the NI, EPGB, and ESWB foundations; (5) there is no potential for liquefaction; (6) the maximum assumed groundwater level is 1.0 m (3.3 ft) below grade; (7) the maximum tilt settlement is 8 mm in 10 m (0.5 in. in 50 ft) in any direction; and (8) there is no potential for slope failure.

In addition, a revised version of FSAR Tier 2, Table 2.1-1 provided in the May 4, 2011, response to RAI 354, Question 03.08.05-23 included the following key information and parameters: (1) The maximum static bearing pressure is equal to 1,054 kPa (22,000 psf), which must be less than the ultimate static bearing capacity divided by a factor of safety equal to 3.0, and is conservatively assumed to be the same for the NI, EPGB, and ESWB foundations; (2) the maximum dynamic bearing pressure is equal to 1,677 kPa (35,000 psf) at the toe of the basemat, which must be less than the ultimate dynamic bearing capacity divided by a factor of safety equal to 2.0, and is conservatively assumed to be the same for the NI, EPGB, and ESWB foundations; (3) the minimum shear wave velocity (low strain best estimate average value at bottom of basemat) is 305 m/s (1,000 ft/s); (4) there is no potential for liquefaction; (5) the maximum differential settlement is in accordance with the settlement contours in FSAR Tier 2, Figures 3.8-124, "Relative Differential Settlement Step 1, NI Basemat," through 3.8-136, "Total Differential Settlement, ESWB" (see discussion below on differential settlements); (6) the maximum tilt settlement is 8 mm in 10 m (0.5 in. in 50 ft) in any direction; (7) there is no potential for slope failure; (8) the minimum angle of internal friction is 26.6 degrees (associated with development of minimum coefficient of static friction in the in-situ soil and backfill); (9) the maximum angle of internal friction is 30 degrees (associated with maximum assumed passive pressure coefficient in the in-situ soil and backfill); (10) the range of in-situ soil and backfill densities between 1762 kg/m^3 (110 lb/ft^3) and 2146 kg/m^3 (134 lb/ft^3) (associated with the structural design of basemats and below-grade walls, not with seismic SSI analyses); (11) the maximum assumed groundwater level is 1.0 m (3.3 ft) below grade; and (12) the minimum coefficient of static friction at all interfaces between the basemat and supporting media, soil or rock, is equal to 0.5.

The May 4, 2011, response to RAI 354, Question 03.08.05-23, also clarified that total settlements beneath each Seismic Category I basemat foundation are not included as key parameters, because they are only relevant to the design of umbilicals between structures (e.g., piping, conduit, duct banks, etc.), which lie outside the scope of the design certification.

The staff finds the above response technically acceptable, because it complies with 10 CFR 52.47(a)(1), which requires that the FSAR include all site parameters postulated in the design. In particular, FSAR Tier 2, Table 1.8-2, COL Information Item 2.0-1 ensures that the

site parameters in FSAR Tier 2, Table 2.1-1 will be verified by COL applicants at a particular site in order to reference the U.S. EPR certified design. The staff also noted that the numerical values included in FSAR Tier 1, Table 5.0-1 and FSAR Tier 2, Table 2.1-1 may be subject to changes due to the resolution of other technical issues relating to the design of Seismic Category I structures and foundations. On this basis, the staff concluded that RAI 155, Question 03.08.05-16 is resolved, while **RAI 354, Question 03.08.05-23 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed changes.

Subsequent to the resolution of RAI 155, Question 03.08.05-16 and RAI 354, Question 03.08.05-23 discussed above, the applicant added two items to FSAR Tier 2, Table 2.1-1: (1) The coefficient of friction between the NAB basemat and supporting media is greater or equal to 0.5 but less than or equal to 0.7 (June 17, 2011, interim response to RAI 370, Question 03.07.02-64); and (2) the minimum coefficient of friction between side walls of the EPGB and surrounding media is equal to 0.36 (July 22, 2011, interim response to RAI 376, Question 03.08.05-31). This additional information was added as part of the resolution of **RAI 370, Question 03.07.02-64** (see Section 3.7.2 of this report) and **RAI 376, Question 03.08.05-31** (see below), **are being tracked as open items** pending completion of the NAB and ESWB stability analyses. Therefore, the numerical values included in these two items may be subject to changes due to the resolution of these RAIs.

The staff initially reviewed the applicant's design and analysis procedures for the NI structures, described in FSAR Tier 2, Section 3.8.1, Section 3.8.2, "Steel Containment," Section 3.8.3, Section 3.8.4, and Section 3.8.5, and identified a number of technical issues pertaining to the design and analysis of the NI foundation basemat, which are discussed in detail in the subsequent paragraphs.

To address several of these issues, the applicant decided to undertake a number of significant modifications to its design and analysis procedures; in particular, the modification of the static finite element model of the NI used in the structural design of the superstructure and foundation basemat. Initially, the applicant used a single static FE model with springs and contact elements at the soil-foundation interface to simulate uplift of the structures under seismic loading conditions. This was later modified to consider two FE models: (1) a fixed-base FE model of the NI used to design the NI superstructure; and (2) an FE model of the NI with springs and contact/sliding elements at the soil-foundation interface used to design the NI basemat. These modifications were discussed with the staff during the April 26-30, 2010, and February 14-17, 2011, onsite audits. The staff expected that implementing these modifications would address several of the technical issues identified during the initial review.

As indicated above, the staff noted that the original design of the NI structures attempted to include possible uplift of the foundation basemat as part of the design process. Since the typical structural design process considers loads and load combinations are based on the principle of superposition (linear design process), it is difficult to incorporate nonlinear effects such as foundation uplift to this process. The proposed modifications effectively separate the nonlinear aspects such as foundation uplift and stability from the design of the NI structures, thereby maintaining the integrity of the linear design process. Foundation uplift and stability are now treated as evaluations of the NI design, which can be performed with linear or nonlinear methods.

In a June 24, 2011, interim response to RAI 371, Question 03.07.02-66, the applicant provided a detailed description of the FE model used to determine the design loads for the NI basemat utilizing dynamic time-history methods and the ANSYS computer code to determine seismic

demands. The staff's evaluation of this model and corresponding dynamic analyses are discussed in Section 3.7.2 of this report. Based on the staff's review, the model and analyses are considered an acceptable methodology to estimate seismic demands for the design of the NI basemat.

The staff reviewed the applicant's methodology for seismic stability analyses of Seismic Category I structures reported in FSAR Tier 2, Section 3.8.5, to verify compliance with SRP Acceptance Criteria 3.8.5.II.4. Stability analyses are performed to ensure that Seismic Category I structures have sufficient capacity to resist sliding and overturning under seismic loads with an adequate safety margin. The safety margin is given in terms of factors of safety, which should be equal to or greater than the minimum values of 1.1 specified in SRP Acceptance Criteria 3.8.5.II.5.

The staff identified the following technical issues:

1. The methodology for the seismic stability analysis of NI structures was based on nonlinear time-history analyses which indicated that a certain amount of sliding would occur. This appeared to not comply with SRP Acceptance Criteria 3.8.5.II.4 and 3.8.5.II.5 and required additional technical justification. In addition, the applicant indicated that sliding and overturning factors of safety were smaller than the minimum values specified in SRP Acceptance Criteria 3.8.5.II.5 but were not reported in the FSAR.

2. A minimum friction coefficient equal to 0.7 was used in the stability analyses. This coefficient was assumed to be the same for both static and dynamic friction, even though sliding was found to occur. Also, the value of 0.7 appeared to be relatively high with respect to what typical site conditions could accommodate. The staff noted that an overestimation of the friction resistance may lead to an overestimation of the sliding factor of safety; therefore, the staff could not determine if the stability analysis for the foundation design is in conformance with SRP Acceptance Criteria 3.8.5.II.4 and 3.8.5.II.5.

3. Stability analyses for the EPGB and ESWB appeared to be incomplete. Sliding and overturning factors of safety were not reported in the FSAR.

4. The FSAR did not clearly identify the methodology to compute dynamic soil pressures on below-grade walls due to seismic loads, used in the design of the walls, which should be consistent with the assumptions of the seismic stability analyses, to comply with SRP Acceptance Criteria 3.8.4.II.4.H. Furthermore, the staff could not identify the dynamic soil pressures on below-grade walls and embedded portions of the foundations that were assumed in the stability analyses.

Therefore, in RAI 155, Questions 03.08.05-8 and 03.08.05-12, RAI 371, Question 03.07.02-69, and RAI 376, Questions 03.08.05-28 and 03.08.05-31, the staff requested that the applicant address these technical issues.

The above issues were discussed with the applicant in January 26-30, 2009; April 26-30, 2010; February 14-17, 2011; and June 6-10, 2011, onsite audits. To address the staff's concerns, the applicant proposed several modifications to its analysis and design procedures. These included: (1) Modification of the methodology for seismic stability analysis of the NI, EPGB, and ESWB; (2) modification of the NI design to incorporate the tendon gallery under the basemat as a shear key, to facilitate stability against sliding; (3) modification of the EPGB design to

incorporate a shear key under the basemat to facilitate stability against sliding; (4) modification of the ESWB design to increase the basemat dimensions to facilitate stability against sliding and overturning; (5) modification of the methodology for computing dynamic soil pressures on below-grade walls due to seismic loads, used in the design of these walls; and (6) modification of the methodology for computing static and dynamic soil bearing pressures, which would be based on SSI analyses using the SASSI computer code. The staff expected that implementing these modifications would address the technical issues identified during the initial review.

The revised methodology for seismic stability analysis of the NI structures, EPGB, and ESWB would be based on the seismic SSI analyses to compute seismic demands and time-dependent factors of safety, and a separate analysis to determine the displacement-dependent passive pressure capacity of the soil. A friction coefficient equal to 0.5 would be assumed in the stability analyses. The staff evaluation of this revised methodology is discussed below.

The revised methodology for computing dynamic soil pressures on below-grade walls due to seismic loads, used in the design of these walls, would consider: (1) Dynamic soil pressures computed from the seismic SSI analyses that are scaled to have peak values corresponding to, at least, passive soil pressures with passive pressure coefficient K_p=3.0; (2) dynamic soil pressures computed using the methodology described in ASCE 4-98 Section 3.5.3.2 (Wood's elastic solution); and (3) passive pressure capacity of the soil (displacement-dependent) assumed in the seismic stability analyses. The staff finds this revised methodology acceptable, because it complies with SRP Acceptance Criteria 3.8.4.II.4.H.

During the February 14-17, 2011, onsite audit, the applicant provided preliminary calculations for the seismic stability analysis of the NI, as well as dynamic soil pressures on below-grade walls due to seismic loads. The applicant indicated that detailed results of these calculations would be provided in their responses to RAI 371, Question 03.07.02-69, and RAI 376, Questions 03.08.05-28 and 03.08.05-31. RAI 371, Question 03.07.02-69 would provide a detailed description of the revised methodology for seismic stability analysis of the NI, which would also be applied to the EPGB and ESWB; RAI 376, Question 03.08.05-28 would provide the analysis results and corresponding factors of safety, as well as detailed information on dynamic soil pressures applicable to below-grade walls for the NI; RAI 376, Question 03.08.05-31 would provide similar information for the EPGB and ESWB.

In a June 24, 2011, response to RAI 371, Question 03.07.02-69, the applicant provided a detailed description of the revised methodology for seismic stability analysis of the NI, which is also applicable to the EPGB and ESWB. In this methodology, time-dependent factors of safety were computed based on seismic demands determined from the seismic SSI analyses (linear elastic assumptions). Separate nonlinear FE analyses were performed to determine the displacement-dependent passive pressure capacity of the soils using the ADINA computer code. The nonlinear analyses consisted of a rigid model of the basemat and sidewall that were quasi-statically "pushed" into the surrounding soil media that was characterized by constitutive models typical of granular soils. Relationships between wall displacements and passive soil pressures mobilized by these displacements were provided with the RAI response. A friction coefficient equal to 0.5 was assumed at the interface between the bottom of the basemat and the supporting media. Friction at the side walls of the NI and ESWB structures was not necessary to demonstrate stability; however, a friction coefficient equal to 0.36 at the side walls of the EPGB was found to be necessary.

The staff evaluation of the revised methodology for seismic stability analysis of the NI, EPGB, and ESWB is discussed in Section 3.7.2 of this report. Based on the staff's review, this

methodology was considered acceptable to demonstrate seismic stability of the NI, EPGB, and ESWB. The staff emphasized that this determination was based on the finding that computed wall displacements from the seismic SSI analyses were small. This validated the use of the nonlinear analyses to estimate the corresponding passive pressure capacity. Otherwise, there would be an incompatibility between the linear methods used to compute the seismic demand and the nonlinear methods used to compute the capacity.

In a July 29, 2011, interim response to RAI 376, Question 03.08.05-28, the applicant reported the minimum factors of safety computed in their stability analyses of the NI structures. The lowest factor of safety against sliding was 1.1 for soil cases 5ae-h, 4ue-m, and 1n5ae-h. The lowest factor of safety against overturning was 1.7 for soil cases 5ae-h, 4ue-m, and 1n5ae-h. The staff finds this information acceptable, because the factors of safety were equal to or greater than the minimum values of 1.1 specified in SRP Acceptance Criteria 3.8.5.II.5.

The July 29, 2011, interim response to RAI 376, Question 03.08.05-28, also provided a detailed description and results of the revised methodology to compute dynamic pressures applicable to the design of below-grade walls for the NI, for all soil and rock cases. This information included plots of the total applied pressures (static with surcharge plus seismic dynamic) for all the NI below-grade walls with the exception of the shear key. The following effects were differentiated in the plots: (1) Scaled dynamic pressures computed from the seismic SSI analyses; (2) pressures computed using Wood's elastic solution; and (3) passive pressure capacity of the soils (displacement-dependent) assumed in the seismic stability analyses. The information demonstrated that, in most cases, the wall design will be governed by the scaled pressures from the seismic SSI analyses. The staff finds this information acceptable, because it complies with SRP Acceptance Criteria 3.8.4.II.4.H.

The applicant indicated the final response to RAI 376, Question 03.08.05-28 will provide the following additional information: (1) Dynamic pressures applicable to the design of the NI shear key (tendon gallery); and (2) static and dynamic soil bearing pressures for the NI based on SSI analyses using the SASSI computer code.

In a July 22, 2011, interim response to RAI 376, Question 03.08.05-31, the applicant indicated that the minimum factors of safety computed in their stability analyses of the EPGB were 1.1 for sliding and overturning. The staff finds this information acceptable, because the factors of safety were equal to or greater than the minimum values of 1.1 specified in SRP Acceptance Criteria 3.8.5.II.5.

The applicant indicated the final July 22, 2011, response to RAI 376, Question 03.08.05-31 would provide the following additional information: (1) Minimum factors of safety computed in their stability analyses of the ESWB, (2) dynamic soil pressures applicable to the design of below-grade walls for the EPGB and ESWB, comparable to the information provided for the NI; and (3) static and dynamic soil bearing pressures for the EPGB and ESWB based on SSI analyses using the SASSI computer code.

Based on the above discussion, the staff notes that the technical issues associated with RAI 155, Questions 03.08.05-8 and 03.08.05-12 are superseded by **RAI 376, Questions 03.08.05-28 and 03.08.05-31, which are being tracked as open items**. The final response to RAI 376, Questions 03.08.05-28 and 03.08.05-31, to be submitted by the applicant upon the completion of the seismic stability analyses for the ESWB, will be reviewed by the staff to verify that the applicant fulfills its commitments as discussed above.

In FSAR Tier 2, Section 3.8.5.4.2, the applicant stated that the NI foundation basemat was analyzed and designed using the overall ANSYS static FE model of the NI structures, which is described in FSAR Tier 2, Section 3.8.1.4.1. In this FE model, the stiffness of the underlying supporting media was represented by linear or tri-linear springs with frequency-independent stiffness parameters, for the different soil and rock cases considered in the design certification. To simulate uplift under seismic loading conditions, contact elements were used to limit the transfer of tension loads to the springs. The applicant also indicated that this FE model was used to determine static bearing pressures on the supporting media.

The staff's review of the FSAR could not identify a description of the development of the linear spring stiffness parameters for the various soil and rock cases. Since linear frequency-independent spring models are simplified representations of soil-structure interaction effects, additional information was needed to determine the adequacy of the chosen representation; in particular, additional information regarding the impact of the chosen spring stiffness parameters on the structural response of the NI foundation basemat and on the bearing pressures imposed on the supporting media. The staff was concerned that an inadequate selection of the spring stiffness parameters could result in an inaccurate computation of bearing pressures, as well as inaccurate forces and moments in the basemat, which could lead to a non-conservative structural design of the latter or to the overstress of the supporting media.

In addition, since the NI foundation basemat has a cruciform shape, there are areas of the foundation that may be susceptible to large bending moments. These areas may be even more susceptible if stiff and soft spots occur beneath the foundation due to the non-homogeneous nature of the supporting media, soil or rock. The staff could not identify whether the applicant had performed studies to assess the effects of stiff and soft spots on the structural response of the NI foundation, and to determine limiting criteria for horizontal variation in soil or rock properties that would limit the effects of these stiff and soft spots, in conformance with SRP Acceptance Criteria 3.8.5.II.4. The staff was concerned that ignoring the effects of stiff and soft spots could lead to a non-conservative structural design, because potential increases in forces and moments in the NI basemat due to these effects would not be taken into account.

Finally, the staff could not identify the technical basis for using tri-linear spring models for two specific soil cases instead of the conventional linear spring models.

Therefore, in RAI 155, Question 03.08.05-5 and RAI 376, Question 03.08.05-25, the staff requested that the applicant address the concerns regarding the selection of linear spring stiffness parameters and the effects of stiff and soft spots under the foundations. The staff's evaluation of the tri-linear spring models is discussed below in RAI 155, Question 03.08.05-7 and RAI 376, Question 03.08.05-25.

In an April 30, 2009, response to RAI 155, Question 03.08.05-5, the applicant indicated that the linear spring parameters used to represent the stiffness of the supporting media are based on the reference G. Gazetas, "Foundation Vibrations," Chapter 15, pp. 553-593; in H.Y. Fang, Editor, "Foundation Engineering Handbook," 2nd Edition, Van Nostrand, 1991. The resulting parameters were compared to those obtained using the Wong-Luco formulation (H.L. Wong and J.E. Luco, "Tables of impedance functions for square foundations on layered media," Soil Dynamics and Earthquake Engineering, Vol.4, No. 2, pp 64-81, 1985) and found to be similar. In the Gazetas formulation, the spring parameters are based on the soil or rock shear modulus (G) of the various layers of supporting media. In the equivalent-static seismic analysis, a value of G corresponding to low strain was used (Gmax). In the analysis for static conditions, a

reduced value of G was conservatively set to 50 percent of Gmax. The applicant further clarified that, given the basic spring parameters obtained using the Gazetas formulation, the spring parameters also vary depending on the location under the basemat footprint, with a stiffer parameter on the outer edges and a softer parameter in the middle of the basemat. An elliptical distribution is used to represent this latter variation. Upon review of the reference provided by the applicant, the staff finds the use of the Gazetas formulation acceptable, because it complies with SRP Acceptance Criteria 3.8.5.II.4 and yields similar results to other formulations (e.g., Wong-Luco).

Regarding the issue of soft and stiff spots under the foundations, the applicant's April 30, 2009, response to RAI 155, Question 03.08.05-5, indicated that basemat reinforcement is uniform and is designed based on the most critical location for bending and shear, for the softest soil case considered in the design certification. Furthermore, the applicant noted that FSAR Tier 2, Table 1.8-2, COL Information Item 2.5-10 requires COL applicants to determine the uniformity of underlying layers of site-specific soil or rock conditions under the foundations, including horizontal variation of soil or rock properties. The applicant's response, however, did not address this issue because COL Information Item 2.5-10 refers to FSAR Tier 2, Section 2.5.4.10.3, "Uniformity and Variability of Foundation Support Media," which does not discuss the extent to which the horizontal uniformity of soil or rock properties should be defined or the acceptable variability relative to its effect on foundation design. Therefore, the staff could not determine whether the applicant's foundation design would be sensitive to potential soft and stiff spots under the foundations.

Subsequent to the RAI response, the applicant modified its analysis and design methodology for the NI foundation basemat. As discussed during the April 26-30, 2010, February 14-17, 2011, and June 6-10, 2011, onsite audits, the applicant would determine the static and dynamic bearing pressures from SSI analysis of the NI structures using the SASSI computer code. In addition, the structural design of the NI basemat for seismic loads would be based on time-history analyses and not on the static FE model of the NI structures described in FSAR Tier 2, Section 3.8.1.4.1.

In a June 9, 2011, response to RAI 376, Question 03.08.05-25, the applicant confirmed that the static FE model of the NI structures with springs to represent the supporting media was only used to determine forces and moments in the foundation basemat for structural design under static loads (i.e., non-seismic loads), and as a check for the static bearing pressures. The spring parameters used for these static analyses correspond to the parameters developed for static loads (i.e., G taken as 50 percent of Gmax). The applicant also indicated that the static FE models of the EPGB, and ESWB were updated to include springs based on a similar formulation as the NI. Additional spring parameters for the EPGB and ESWB would be added to the FSAR, comparable to the information provided for the NI.

The staff finds the above approach acceptable, because, as previously stated, the application of the Gazetas formulation to static loading conditions was found acceptable and complies with SRP Acceptance Criteria 3.8.5.II.4.

Regarding the issue of soft and stiff spots under the foundations, the applicant's June 9, 2011, response to RAI 376, Question 03.08.05-25, explained that consideration of site-specific stiff and soft spots under the foundations is included in the verification of soil settlement effects that will be performed by COL applicants as part of the resolution of RAI 354, Question 03.08.05-22.

The staff finds the June 9, 2011, response to RAI 376, Question 03.08.05-25, acceptable, because any site-specific stiff and soft spots under the foundations would be reflected in the

site-specific settlement profiles that will be calculated by COL applicants. COL applicants will perform a verification to determine whether such site-specific settlement profiles are bounded by the settlement profiles established for the certified design, as discussed in the resolution of RAI 354, Question 03.08.05-22 (see below under "Structural Acceptance Criteria," RAI 155, Questions 03.08.05-11 and 03.08.05-15, RAI 354, Question 03.08.05-22, and RAI 376, Question 03.08.05-30). If the verification determines that site-specific settlement profiles are indeed bounded by the settlement profiles established for the certified design, then any additional forces and moments induced in the basemat by the stiff and soft spots under the foundations would be bounded by forces and moments accounted for in the certified design; otherwise, additional site-specific analyses would be required.

Based on the above discussion, the staff concluded that RAI 155, Question 03.08.05-5 is resolved, while **RAI 376, Question 03.08.05-25 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed changes.

In FSAR Tier 2, Section 3.8.5.4.2, the applicant described the use of tri-linear springs, with strain softening, to represent the soil stiffness in the equivalent-static seismic analysis of the NI foundation basemat for soil cases 4u and 2sn4u. However, the staff could not identify the technical basis for the tri-linear properties used or the reason why these tri-linear springs were only used for two specific soil cases and not for the others. Therefore, in RAI 155, Question 03.08.05-7 and RAI 376, Question 03.08.05-27, the staff requested that the applicant clarify (1) the technical basis for using tri-linear instead of linear springs to represent soil stiffness; (2) the technical basis for determining the properties of these tri-linear springs; and (3) the rationale for using these tri-linear springs for only two specific soil cases.

In a June 30, 2009, response to RAI 155, Question 03.08.05-7, the applicant stated that tri-linear springs were used in the static FE model of the NI structures to estimate soil bearing pressures due to seismic loads using the equivalent-static method. Tri-linear springs with strain softening were used for the two soil cases identified above, because using linear springs the bearing pressures obtained for these two cases were judged to be unrealistically high. The properties of the tri-linear springs were determined based on dynamic soil shear modulus reduction curves consistent with the associated strains.

The staff did not agree with the applicant's assumption that the behavior of soils beneath and near the edge of a foundation can be represented by shear modulus reduction curves, presumably associated with simple one-dimensional site response analyses (i.e., as used in seismic SSI analysis). The staff noted that if the soil to the side of an embedded foundation is confined then strain stiffening rather than strain softening would be the more likely scenario.

Subsequent to the June 30, 2009, response to RAI 155, Question 03.08.04-7, the applicant modified its analysis and design methodology for the NI foundation basemat. As discussed during the April 26-30, 2010, onsite audit, the applicant would determine the dynamic bearing pressures from the seismic SSI analysis of the NI structures. During the audit, the applicant indicated that tri-linear soil springs would no longer be considered.

In a November 22, 2010, response to RAI 376, Question 03.08.05-27, the applicant confirmed that tri-linear soil springs were no longer considered in the analysis and design of the NI basemat for soil cases 4u and 2sn4u. The RAI response also provided revisions to FSAR Tier 2, Section 3.8.5.4.2 and FSAR Tier 2, Table 3.8-14, "Tri-Linear Subgrade Modulus vs. Bearing Pressures," which eliminated references to the use of tri-linear soil springs. On this basis, the staff concluded that RAI 155, Question 03.08.05-7 is resolved, while **RAI 376,**

Question 03.08.05-27 is being tracked as a confirmatory item pending revision of the FSAR to incorporate the proposed changes.

During the January 26-30, 2009, onsite audit, the staff could not locate documented criteria for the selection of critical sections corresponding to Seismic Category I foundations. The identification, analysis, and design of critical sections provides reasonable assurance that the structural design of Seismic Category I structures is essentially complete, as required by 10 CFR 52.47(c). The same issues were also identified by the staff in a review of FSAR Tier 2, Sections 3.8.1, 3.8.3, and 3.8.4. Therefore, in RAI 155, Questions 03.08.01-20, 03.08.01-24, and 03.08.04-6, the staff requested that the applicant address these issues. RAI 155, Question 03.08.01-20 is resolved. **RAI 155, Questions 03.08.01-24, and 03.08.04-6 are being tracked as open items** pending completion of the critical section design. The details of the resolution to these RAIs are discussed in Section 3.8.1.4 of this report under "Design and Analysis Procedures."

In FSAR Tier 2, Section 3.8.5.4.1, Revision 0, the applicant stated that the three components of seismic loads are combined using the SRSS method or the 100-40-40 percent rule described in ASCE 4-98. As described in RG 1.92, Revision 2, the SRSS method and the 100-40-40 percent rule are two different acceptable methodologies that can be used to combine the seismic analysis responses of the structure from each of the three orthogonal directions. The staff notes that the guidance provided in RG 1.92 only refers to a single design parameter. For a design involving multiple parameters, the implementation of the 100-40-40 rule is rather complex, and the FSAR description was unclear to the staff with respect to how the multiple design parameters were considered in the application of the 100-40-40 rule. The same issue was also raised in the review of FSAR Tier 2, Section 3.8.3.4.4, "Seismic and Other Dynamic Analyses and Design." Therefore, in RAI 155, Question 03.08.03-10 and RAI 376, Question 03.08.03-24, the staff requested that the applicant provide a detailed explanation of its implementation of the 100-40-40 rule, including quantitative demonstration which shows that its implementation conforms to the guidance described in RG 1.92, as well as its applicability to nonlinear analysis procedures. **RAI 155, Question 03.08.03-10 and RAI 376, Question 03.08.03-24 are being tracked as confirmatory items.** The resolution of these RAI issues is discussed in Section 3.8.3.4 of this report under, "Design and Analysis Procedures," where the staff concluded that these questions are resolved.

Structural Acceptance Criteria

To ensure that the Seismic Category I foundations, as designed, can perform their intended safety function, the staff verified that the structural design acceptance criteria meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, and GDC 5; and are in accordance with SRP Acceptance Criteria 3.8.5.II.5. With the exceptions described below, the staff finds that the structural acceptance criteria meet the regulatory requirements and regulatory guidance cited above.

The bearing pressures imposed by a Seismic Category I foundation on the surface of the underlying soil media introduce stresses in the soil that cause the soil to deform, which ultimately leads to settlement of the foundation and superstructure. Since soils are non-homogeneous media and loads are not applied uniformly on the foundation footprint, the resulting settlements are not uniformly distributed, which means that certain areas of the foundation settle more or less than others. This is known as differential settlement. Differential settlement induces additional stresses on the foundation and superstructure that need to be evaluated and accounted for in the structural design. In addition, settlement does not occur

instantaneously but is recognized as a time-dependent process. For certain types of soils (e.g., sandy soils), most of the settlement occurs during and shortly after construction due to soil compaction; however, for other types of soils (e.g., clay soils), settlement may continue for significantly longer periods of time due to soil consolidation. The design needs to account for these time-dependent effects.

Differential settlement effects, especially during the construction process, can significantly affect the integrity and performance of Seismic Category I structures critical to safety. To this end, SRP Acceptance Criteria 3.8.5.II.4 describes that differential settlement effects should be considered in the structural design.

The staff reviewed FSAR Tier 2, Section 3.8.5, and could not determine whether the Seismic Category I structures (foundations and superstructures) were being adequately designed to take into account the additional stresses induced by the differential settlements, as well as by the effects of the construction sequence, at all potential sites considered in the design certification, and as described in SRP Acceptance Criteria 3.8.5.II.4.

In particular, the staff determined that the settlement criterion described in FSAR Tier 2, Section 3.8.5.5.1, "Nuclear Island Common Basemat Structure Foundation Basemat," and FSAR Tier 2, Section 2.5.4.10.2, "Settlement," Revision 1, which refer to a maximum differential settlement of 8 mm in 10 m (0.5 in. in 50 ft) in any direction, was not appropriate to address the above design considerations, because it was unclear to the staff how it would be applied to the design process. For example, a rigid-body tilt of 8 mm in 10 m (0.5 in. in 50 ft) in any direction would not impose significant additional stresses on the structures; however, a vertical shear distortion of the same magnitude would certainly impose significant additional stresses.
The fact that the same criterion could be interpreted and applied in two different ways, yielding significantly different outcomes, indicated to the staff that the criterion was not appropriate and did not meet SRP Acceptance Criteria 3.8.5.II.4.

Furthermore, the staff could not identify the interface requirements established by the FSAR on the COL applicant to ensure that additional stresses induced by differential settlements and construction sequence at a particular site, predicted or measured, would be bounded by those considered in the certified design.

Finally, the staff was concerned that without a clear identification of design methodology, interface requirements, or acceptance criteria for the COL applicant, the applicant has not demonstrated a means of assuring that the construction conforms to the design, as required by 10 CFR 52.47, "Contents of applications; technical information."

Therefore, in RAI 155, Questions 03.08.05-11 and 03.08.05-15, RAI 354, Question 03.08.05-22, and RAI 376, Question 03.08.05-30, the staff requested that applicant address the technical issues identified above.

In an April 30, 2009, response to RAI 155, Question 03.08.05-11, the applicant referred to a study performed in which a tilt settlement of 8 mm in 10 m (0.5 in. in 50 ft) was applied to the NI structures from the FB to the SB2/3, for the softest soil case considered in the certified design. The study found negligible effects in terms of additional bearing pressures and induced stresses on the NI basemat. The applicant's response was consistent with the staff's initial assessment that, for the stated tilt settlement values, the rigid-body tilt would not impose significant additional stresses on the structures. However, the applicant's response did not address issues related to possible foundation deformations due to differential settlements during and after the construction, and the effect of the additional stresses induced by differential

settlements, as well as the effect of construction sequences on the design of the foundation basemat.

In a June 30, 2009, response to RAI 155, Question 03.08.05-15, the applicant stated that no limitations are imposed by the FSAR on the construction sequence of the NI basemat structure. The fact that the RCB and RSB (the largest and heaviest NI structures) are at the center of the NI basemat imposes an inherent construction sequence that promotes concentric, evenly distributed loading. Also, the cruciform design of the basemat further minimizes potential differential settlement associated with construction. The applicant concluded that control of the construction sequence of the NI basemat structures is not necessary for the U.S. EPR design and that settlement control criteria are sufficient to implement design and construction considerations. However, the staff determined that a construction sequence for the NI basemat was implicit in the applicant's response. In particular, the applicant's indication that the largest mass would be placed at the center of the basemat early in construction was equivalent to imposing a sequence on construction, although this is not explicitly stated in the FSAR. Therefore, the foundation design described in the FSAR has implicitly assumed certain sequences for the construction, which the staff believes should be clearly described in the instruction to the COL applicants.

In a November 22, 2010, response to RAI 376, Question 03.08.05-30, the applicant indicated that the issues associated with differential settlements were being revised and would be addressed under RAI 354, Question 03.08.05-22. The applicant confirmed that a rigid body-type evaluation would not be used, which the staff finds acceptable.

During the April 26-30, 2010, onsite audit, the staff stated that, to investigate the effect of soil settlements on the structural performance of the NI foundation basemat and connecting superstructure walls, it is necessary to distinguish between the settlements that are expected to occur during construction, and post-construction settlements caused by soil consolidation and compaction. The modeling of soil stiffness for these two cases should be differentiated.

The staff also emphasized that the certified design should adequately address the effects of the construction sequence and soil settlements to ensure safety. However, the staff recognized that these issues are site-specific *per se*. Therefore, the applicant should address an interface requirement that allows for the certified design to account for construction sequence and settlement loads and also permits the COL applicant to verify that these loads are not exceeded during or after the construction.

In response to the staff's questions and discussions during the April 26-30, 2010, onsite audit, the applicant proposed a design methodology, interface requirements, and acceptance criteria for the COL applicant, to address the issue of differential settlements and construction sequence. The design methodology and acceptance criteria are summarized below.

The applicant would consider in the foundation design (1) a postulated set of soil stiffness parameters (soil spring constants) for the construction phase, (2) a postulated set of soil stiffness parameters for the post-construction phase, and (3) a postulated construction sequence and corresponding set of construction loads. To account for the construction sequence and settlement loads in the design, the applicant would perform a detailed, sequential, FE analysis of the NI foundation and superstructure with realistic modeling of the supporting soil stiffness and construction sequence, including anticipated effects during the operating life of the NPP. The sequential FE analysis would be based on the postulated soil conditions and validated with geotechnical soil settlement analyses for these postulated conditions. The envelope of forces and moments computed during the sequential FE analysis

would then be compared with corresponding forces and moments obtained from a "reference" FE analysis that does not include construction sequence or soil settlement effects. From this comparison, any difference in forces and moments would be taken as a separate construction sequence/soil settlement load case, and incorporated to the structural design of the foundation and superstructure in addition to all other load cases, in accordance with ACI 349-01, Section 9.2.2. The settlement profiles at all stages of the sequential FE analysis would also be computed and would be incorporated to the FSAR, as described below, to be used by the COL applicant for verification purposes.

Since the differential settlement effect on the foundation design is more pronounced for a soft soil rather than for a stiff soil, the applicant postulated that soft soil conditions in the certified design bound other sites.

The construction sequence postulated in the certified design would consist of 11 steps for the NI structures: beginning with the basemat only and progressing to intermediate elevations of -4.90 m (-16 ft), 16.80 m (55 ft), 29.30 m (96 ft), 43.90 m (144 ft), and finally the remainder of the structure. This construction sequence would be described in the FSAR. If the COL applicant were not to follow this postulated construction sequence, the COL applicant would need to provide site-specific justifications to demonstrate that the certified design is adequate for the proposed alternate site-specific construction sequence.

In the above discussion, the applicant assumed that the total magnitude of soil settlements is not a design consideration during either construction or post-construction phases. The staff noted that it is not so much the magnitude of the settlements that affects the structural performance of the NI foundation and superstructure; rather, it is the relative shape of the settlement profile in terms of slope and curvature. Such an assumption is valid for the design certification, because total soil settlements are only relevant to the design of umbilicals between structures (e.g., piping, conduit, duct banks, etc.), which will be addressed for site-specific plants by COL applicants.

Given the settlement profiles established in the certified design, to be included in the FSAR, the COL applicant would be required to perform site-specific geotechnical investigations to determine predicted settlement profiles during construction and post-construction phases, based on the actual construction sequence to be used. If the predicted settlement profiles compare favorably to the settlement profiles in the FSAR—in terms of slope and curvature, not necessarily in absolute magnitude—then it is inferred that the forces and moments induced by the predicted settlements would be bounded by the forces and moments considered in the certified design. This comparison would be specified in terms of the "angular distortion" concept, as described in U.S. Army Corps of Engineers Manual No. 1110-1-1904, "Engineering and Design: Settlement Analysis," issued 1990. In addition to the predictive calculations, the COL applicant would be required to establish a settlement monitoring program to verify whether measured settlements are consistent with predicted settlements during the operating life of the NPP.

The staff finds the design methodology and acceptance criteria proposed by the applicant acceptable because: (1) The foundation design would adequately account for additional stresses induced by postulated soil settlements and construction sequence and comply with SRP Acceptance Criteria 3.8.5.II.4; (2) the soil characteristics and construction sequence assumed in the certified design are clearly described; and (3) a verification by the COL applicant can be performed to establish whether the site-specific settlement profiles, predicted and measured, are bounded by the settlement profiles established for the certified design.

Therefore, the staff concluded that based on this approach, the applicant can demonstrate a clear means for assuring the construction conforms to the design, as required by 10 CFR 52.47.

During the February 14-17, 2011, onsite audit the staff reviewed the applicant's preliminary calculations, which implemented the design methodology summarized above, as well as the resulting settlement profiles for the NI, EPGB and ESWB. The applicant explained that the design methodology for the EPGB and ESWB would be a simplified version of the methodology for the NI, in which the construction sequence would consist of only one step, and the soil stiffness parameters for the post-construction phase would be conservatively set to one-half the value of the soil stiffness parameters for the construction phase. The staff finds the latter approach acceptable because of the relative structural simplicity and smaller size of these structures.

In an April 18, 2011, response to RAI 354, Question 03.08.05-22, the applicant provided a detailed documentation of the design methodology and acceptance criteria summarized above. In particular, the response provided revised FSAR Tier 2, Table 1.8-2, COL Information Items 2.5-7, 2.5-12, 3.8-13, 3.8-18, 3.8-19, and 3.8-20, which describe the verification process to be performed by COL applicants, and FSAR Tier 2, Figures 3.8-124 through 3.8-136, which show the settlement profiles established for the NI, EPGB, and ESWB.

Based on the above discussion, the staff considers RAI 155, Questions 03.08.05-11 and 03.08.05-15 resolved, while **RAI 354, Question 03.08.05-22 and RAI 376, Question 03.08.05-30 are being tracked as confirmatory items** pending revision of the FSAR to incorporate the proposed changes.

Materials, Quality Control, and Special Construction Techniques

The staff reviewed the materials, quality control, and special construction techniques used for the Seismic Category I foundations to ensure that they meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, and GDC 5; and are in accordance with SRP Acceptance Criteria 3.8.5.II.6. With the exceptions described below, the staff finds that the materials, quality control, and special construction techniques meet the regulatory requirements and regulatory guidance cited above.

The staff reviewed FSAR Tier 2, Section 3.8.5.6.1, and could not identify the criteria to be used by a COL applicant to address potential aggressive environments; in particular, the criteria to address potential concrete corrosion due to aggressive groundwater or soil, and the requirements for utilizing epoxy coated steel reinforcement. The staff was concerned about the potential deterioration and corrosion of foundations in aggressive environments, which could occur if the latter could not be clearly identified. Therefore, in RAI 155, Question 03.08.05-13 and RAI 354, Question 03.08.05-20, the staff requested that the applicant provide these criteria and incorporate them into the relevant FSAR sections. The staff also requested that the applicant clarify whether permanent dewatering systems were being considered to protect Seismic Category I SSCs. The staff noted that if such dewatering systems were provided to meet assumed design conditions, then they should also be designated as Seismic Category I unless further technical justification is provided to justify otherwise.

In a June 30, 2009, response to RAI 155, Question 03.08.05-13, the applicant provided the following information: (1) The criteria for identifying groundwater and soil chemistry points at which aggressive conditions commence, which is based on ACI 349-01, Tables 4.3.1 and 4.4.1; (2) confirmation that Seismic Category I foundations and below-grade concrete structures exposed to aggressive environments will meet applicable requirements of ACI 349-01,

Chapter 4 “Durability Requirements,” or ASME, Section III, Division 2, Subarticle CC-2231.7 “Durability,” including the use of special cement types, maximum water-to-cement ratios, and minimum compressive strengths, in addition to epoxy coated reinforcement; and (3) clarification that COL applicants are required by FSAR Tier 2, Table 1.8-2 to perform site-specific evaluations to determine if waterproofing membranes and epoxy coated reinforcement should be used (COL Information Item 3.8-11), and to examine inaccessible portions of Seismic Category I concrete structures below grade for degradation and monitoring of groundwater chemistry (COL Information Item 3.8-12).

In a July 7, 2010, response to RAI 354, Question 03.08.05-20, the applicant further clarified that: (1) COL applicants shall evaluate aggressive environments in accordance to ACI 349-01, Chapter 4 or ASME Code, Section III, Division 2, Subarticle CC-2231.7, as applicable (modified COL Information Item 3.8-11); (2) epoxy coated reinforcement used in Seismic Category I foundations and below-grade concrete structures exposed to aggressive environments shall be in accordance with ACI 349-01 specifications, including increased splice lengths; (3) waterproofing systems of Seismic Category I foundations and below-grade concrete structures exposed to aggressive environments shall be evaluated for use in such environments; and (4) the use of dewatering systems to mitigate potentially aggressive groundwater effects shall not be relied upon as an alternative for lowering existing groundwater levels to meet assumed design conditions. The applicant also indicated that this additional information would be added to the FSAR.

The staff finds the applicant’s responses technically acceptable, because they comply with SRP Acceptance Criteria 3.8.1.II.6, 3.8.5.II.6, and 3.8.5.II.7. In particular, the applicant ensures that Seismic Category I foundations and below-grade concrete structures exposed to aggressive environments will meet the applicable provisions of ACI 349-01, Chapter 4, or ASME, Section III, Division 2, Article CC-2231.7. The staff notes that both codes are essentially equivalent in this regard and require the use of special cement types, maximum water-to-cement ratios, and minimum compressive strengths in case of exposure to aggressive environments. Both codes specify similar chemical criteria for identifying these aggressive environments. The issue of waterproofing for below-grade concrete structures is further discussed below in RAI 155, Question 03.08.05-14 and RAI 354, Question 03.08.05-21.

The staff verified that the changes proposed by the applicant were incorporated into FSAR Tier 2, Revision 2. Therefore, based on the above discussion, the staff considers RAI 155, Question 03.08.05-13 and RAI 354, Question 03.08.05-20 resolved.

In FSAR Tier 2, Section 3.8.5.6.1, the applicant stated that a textured geosynthetic material would be considered on a site-specific basis for use as a waterproofing membrane for sites with a high groundwater level. However, the staff could not determine the reason why waterproofing was not always required for Seismic Category I foundations and below-grade concrete structures, regardless of groundwater level. Based on past operating plant experience, the staff noted that most soils can attract water to significant heights above the free groundwater level by capillarity effects; therefore, perched groundwater conditions are always a potential condition at soil sites, which could lead to concrete deterioration if no waterproofing is provided. The use of waterproofing systems has always been recognized as good engineering practice to prevent degradation of foundations and below-grade concrete structures. In addition, it was unclear to the staff how the waterproofing membrane, described as “double-textured,” would achieve the minimum coefficient of friction assumed in the design; and also how the construction sequence and presence of a concrete mud mat between the soil and the structural concrete affects this

coefficient of friction. Therefore, in RAI 155, Question 03.08.05-14 and RAI 354, Question 03.08.05-21, the staff requested that the applicant address these issues.

In a June 30, 2009, response to RAI 155, Question 03.08.05-14, the applicant indicated that (1) waterproofing membrane testing will be performed according to procurement specifications to evaluate membrane ductility, durability, and other mechanical properties such as tensile strength, tear resistance, impact resistance, puncture resistance, to reasonably assure global integrity; (2) the waterproofing membrane will be subject to vendor testing to evaluate friction characteristics; (3) the waterproofing membrane will be installed according to manufacturer's recommendations and configured to achieve the required design properties, including the required coefficient of friction; and (4) the concrete mud mat surface finish will be in accordance with ACI 301, Section 5.3.4.2d, which specifies the surface to be roughened to a minimum of 6 mm (0.25 in.) in two perpendicular directions.

In an October 7, 2010, response to RAI 354, Question 03.08.05-21, the applicant stated that the use of a geosynthetic membrane embedded within a mud mat would no longer be specified as the only form of waterproofing and references to waterproofing membranes would be removed from FSAR Tier 2, Section 2.5.4, "Stability of Subsurface Materials and Foundations," and FSAR Tier 2, Section 3.8.5. Instead, the applicant stated that all Seismic Category I foundations and below-grade concrete structures will require waterproofing and damp-proofing systems, and that these systems shall be applied in accordance with the International Building Code (IBC), Sections 1805.2 and 1805.3, 2009 Edition. IBC Section 1805.2 specifies damp-proofing systems where hydrostatic pressure does not occur. IBC Section 1805.3 specifies waterproofing systems where hydrostatic pressure exists up to 0.30 m (12 in.) above the maximum elevation of the groundwater table. In addition, waterproofing and damp-proofing materials selected for use in horizontal applications shall have the physical properties to achieve the minimum coefficient of friction assumed in the design. Finally, waterproofing and damp-proofing systems subjected to aggressive environments shall be evaluated for use in such environments by the COL applicant.

The staff finds the above response technically acceptable, because it is ensured that waterproofing and damp-proofing systems will be provided for all Seismic Category I foundations and below-grade concrete structures in accordance with established industry standards (e.g., IBC, Sections 1805.2 and 1805.3, 2009 Edition), regardless of groundwater level; and that these systems will achieve the minimum coefficient of friction assumed in the design, to comply with SRP Acceptance Criteria 3.8.5.II.4.

Based on the above discussion, the staff considers RAI 155, Question 03.08.05-14 resolved, while **RAI 354, Question 03.08.05-21 is being tracked as a confirmatory item** pending revision of the FSAR to incorporate the proposed changes.

In FSAR Tier 2, Section 3.8.5.6.3, the applicant stated that no special construction techniques are considered for the Seismic Category I foundations. However, the staff noted that the construction process for pouring heavy foundation sections, such as those for the NI basemat, needs to be carefully considered to ensure that differential settlements, particularly at softer soil sites, will not cause segmental cracking or any other distress to the structural system. Based on past precedent with other NPP certified designs, applicants have typically performed studies to provide limitations to the construction process to ensure relatively uniform loads over the plan area. Although some mention is made in FSAR Tier 2, Revision 0, Section 3.8.5.5.1 of such differential settlement questions, the staff could not determine whether such studies had been

performed for the U.S. EPR. Therefore, in RAI 155, Question 03.08.05-15, the staff requested that the applicant provide details of such studies.

In a June 30, 2009, response to RAI 155, Question 03.08.05-15, the applicant indicated that no limitations are imposed by the FSAR on the construction sequence of the NI basemat structure. The fact that the RCB and RSB (the largest and heaviest NI structures) are at the center of the NI basemat imposes an inherent construction sequence that promotes concentric, evenly distributed loading. Finally, the cruciform design of the basemat further minimizes potential differential settlement associated with construction. The applicant concluded that control of the construction sequence of the NI basemat structures is not necessary for the U.S. EPR design and that settlement control criteria are sufficient to implement design and construction considerations.

The staff determined that a construction sequence for the NI basemat was implicit in the applicant's response. In particular, the applicant's indication that the largest mass would be placed at the center of the basemat early in construction was equivalent to imposing a sequence on construction, although this is not explicitly stated in the FSAR. The resolution of this RAI is discussed together with the technical issues associated with differential settlements (RAI 155, Question 03.08.05-11, RAI 354, Question 03.08.05-22, and RAI 376, Question 03.08.05-30), under "Structural Acceptance Criteria." The staff considers RAI 155, Questions 03.08.05-11 and 03.08.05-15 resolved, while, **RAI 354, Question 03.08.05-22 and RAI 376, Question 03.08.05-30 are being tracked as confirmatory items**.

In FSAR Tier 2, Section 3.8.5.6.3, the applicant provided a brief description of modular construction methods and composite-type structural members used in the U.S. EPR standard design. The staff was concerned about the structural design, fabrication, configuration, layout, and connections of designated modules because of past experience with other NPP certified designs (e.g., AP 1000), which utilized composite-type modules and structural units that were new to U.S. nuclear power plants and required extensive case-by-case review. The staff identified the same issue in the review of FSAR Tier 2, Section 3.8.3.6.5, "Special Construction Techniques." Therefore, in RAI 155, Question 03.08.03-13, the staff requested that the applicant provide a more detailed description of each specific type of module or composite member used in the U.S. EPR standard design, and provide a description of the analysis and design approach used for each type of module and composite member. The staff considers RAI 155, Question 03.08.03-13 resolved. The resolution of this RAI is discussed in Section 3.8.3.4 of this report, under "Materials, Quality Control, and Special Construction Techniques."

<u>Testing and Inservice Surveillance Requirements</u>

To ensure that the Seismic Category I foundations can perform their intended safety function, the staff verified that the testing and ISI surveillance requirements meet the relevant requirements in 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, and GDC 5; and are in accordance with SRP Acceptance Criteria 3.8.5.II.7. With the exception described below, the staff finds that the testing and ISI surveillance requirements meet the regulatory requirements and regulatory guidance cited above.

In FSAR Tier 2, Section 3.8.5.7, the applicant stated that monitoring and maintenance of structures is performed in accordance with RG 1.160, without reference to 10 CFR 50.65. The same issue was raised in the staff's review of FSAR Tier 2, Section 3.8.3.7. Therefore, in RAI 155, Question 03.08.03-14, the staff requested that the applicant clarify why monitoring and maintenance of structures is not performed in accordance with the requirements of

10 CFR 50.65 and supplemented with the guidance in RG 1.160. The resolution of this RAI is discussed in Section 3.8.3.4 of this report under "Testing and Inservice Surveillance Requirements," where the staff concluded that this question is resolved.

Site Parameters

The staff has reviewed the following site parameter that will be addressed in COL applications: Item No. 3-4, Site-specific loads that lie within the standard plant design envelope for Seismic Category I structures, in FSAR Tier 2, Chapter 1, Table 1.8-1, and finds it acceptable for this area of review.

3.8.5.5 *Combined License Information Items*

Table 3.8.5-1 provides a list of foundations related COL information item numbers and descriptions from FSAR Tier 2, Table 1.8-2:

Table 3.8.5-1 U.S. EPR Combined License Information Items

Item No.	Description	FSAR Tier 2 Section
3.8-9	A COL applicant that references the U.S. EPR design certification will describe site-specific foundations for Seismic Category I structures that are not described in this section.	3.8.5.1
3.8-10	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 site-specific soil characteristics that are not within the envelope of the soil parameters specified in Section 2.5.4.2.	3.8.5.5
3.8-11	A COL applicant that references the U.S. EPR design certification will evaluate the use of epoxy coated rebar for foundations subjected to aggressive environments, as defined in ACI 349-01, Chapter 4. In addition, the waterproofing system of Seismic Category I foundations subjected to aggressive environments will be evaluated for use in aggressive environments. Also, the concrete of Seismic Category I foundations subjected to aggressive environments will meet the durability requirements of ACI 349-01, Chapter 4 or ASME, Section III, Division 2, Article CC-2231.7, as applicable.	3.8.5.6.1
3.8-12	A COL applicant that references the U.S. EPR design certification will describe the program to examine inaccessible portions of below-grade concrete structures for degradation and monitoring of groundwater chemistry.	3.8.5.7
3.8-13	A COL applicant that references the U.S. EPR design certification will identify if any site-specific settlement monitoring requirements are required for Seismic Category I foundations based on site-specific soil conditions.	3.8.5.7

Item No.	Description	FSAR Tier 2 Section
3E-1	A COL applicant that references the U.S. EPR design certification will address critical sections relevant to site-specific Seismic Category I structures.	3E

With the exception of COL Information Items 3.8-11, 3.8-12, and 3.8-13, which have been revised as part of the resolution of RAI 354, Questions 03.08.05-20 and 03.08.05-22, and COL Information Items 3.8-18, 3.8-19, and 3.8-20, which have been added as part of the resolution of RAI 354, Question 03.08.05-22, the staff finds the items in the above table consistent with FSAR Tier 2, Section 3.8.5, and therefore acceptable and complete. The next revision of FSAR Tier 2, Table 1.8-2, needs to be reviewed by the staff to confirm that the applicant has incorporated the proposed changes. Also, as revised, the list adequately describes actions necessary for the COL applicant. The staff notes that no additional COL information items need to be included in FSAR Tier 2, Table 1.8-2 for foundation design.

3.8.5.6 *Conclusions*

The staff reviewed FSAR Tier 2, Section 3.8.5, Revision 2, to determine whether the design, fabrication, construction, testing, and ISI of Seismic Category I foundations comply with 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, and GDC 5. The staff concludes that, except for the open and confirmatory items described above, the applicant meets the relevant requirements of 10 CFR 50.55a; 10 CFR Part 50, Appendix B; and GDC 1, GDC 2, GDC 4, and GDC 5. The basis for this conclusion is the following.

The applicant meets the requirements of 10 CFR 50.55a and GDC 1, to ensure that the Seismic Category I foundations are designed, fabricated, erected, constructed, tested, and inspected to quality standards commensurate with the importance of the safety functions to be performed, by meeting the guidelines in SRP Section 3.8.5; RG 1.127, RG 1.136, RG 1.142, RG 1.160, and RG 1.199; ASME Code, Section III, Division 2, Subsection CC, and Section XI, Subsections IWE and IWL; and ACI 349 with additional guidance provided by RG 1.142 and RG 1.199.

The applicant meets the requirements of GDC 2 to ensure that the Seismic Category I foundations are able to withstand the most severe natural phenomena, by analyzing and designing these structures to withstand the most severe earthquake, wind, tornado, and external flood loads that have been established for the U.S. EPR certified design, in load combinations that include these loads.

The applicant meets the requirements of GDC 4 to ensure that the Seismic Category I foundations are appropriately protected against other dynamic effects, by analyzing and designing these structures, where applicable, to withstand the dynamic effects due to pipe whipping, missiles, and discharging fluids associated with the LOCA.

The applicant meets the requirements of GDC 5 to ensure that SSCs are not shared between units or that sharing will not impair their ability to perform their intended safety functions, by analyzing and designing these structures for the design loads and load combinations described in SRP Section 3.8.5; ASME Code, Section III, Division 2, Subsection CC with additional

guidance provided by RG 1.136; and ACI 349 with additional guidance provided by RG 1.142 and RG 1.199.

The applicant meets the requirements of 10 CFR Part 50, Appendix B, to provide a quality assurance program for the design, construction, and operation of SSCs, by meeting the guidelines in SRP Section 3.8.5; RG 1.127, RG 1.136, RG 1.142, RG 1.160, and RG 1.199; ASME Code, Section III, Division 2, Subsection CC, and Section XI, Subsections IWE and IWL; and ACI 349 with additional guidance provided by RG 1.142 and RG 1.199.

As mentioned above, the acceptability of the design, fabrication, construction, testing, and ISI of Seismic Category I foundations is predicated on the adequate resolution of the open and confirmatory items discussed in this section.

3.9 Mechanical Systems and Components

3.9.1 Special Topics for Mechanical Components

3.9.2 Dynamic Testing and Analysis of Systems, Components, and Equipment

3.9.2.1 *Introduction*

Specific U.S. EPR systems and components are designed to retain their structural and functional integrity during normal operation, plant transients, and external events. This includes dynamic loads from pump and valve operation, flow-induced vibration, and seismic events. The adequacy of systems, components, and equipment design is confirmed through analyses and startup testing, which verify that regulatory requirements for dynamic loads are met. This section verifies that the FSAR describes the criteria, testing procedures, and dynamic analysis employed to ensure the structural and functional integrity of piping systems, mechanical equipment, and reactor internals and their supports under dynamic and vibratory loads, including those due to fluid flow and postulated seismic events.

3.9.2.2 *Summary of Application*

FSAR Tier 1: The applicant has provided a non-system based description of the U.S. EPR initial test program (ITP) in FSAR Tier 1, Section 3.3, “Initial Test Program.”

FSAR Tier 2: The applicant has provided a description of the dynamic testing and analysis of systems, components, and equipment in FSAR Tier 2, Section 3.9.2, “Dynamic Testing and Analysis of Systems, Components, and Equipment,” and FSAR Tier 2, Section 14.2, “Initial Plant Test Program,” along with FSAR Tier 2, Appendix 3A, “Criteria for Distribution System Analysis and Support,” and FSAR Tier 2, Appendix 3C, “Reactor Coolant System Structural Analysis Methods.” The applicant described the U.S. EPR tests and analysis required for reactor internals, specified high-energy and moderate-energy piping, and associated piping supports and restraints to demonstrate that structural integrity and functionality is maintained when subjected to static and dynamic loads that can occur during normal operation, plant transients, and seismic events.

AREVA Topical Report ANP-10264-NP-A, “U.S. EPR Piping Analysis and Pipe Support Design Topical Report,” presents the U.S. EPR acceptance criteria, analysis methods, and modeling

techniques for ASME Class 1, 2, and 3 piping and pipe supports acceptable to the NRC. In FSAR Tier 2, Section 3.9.2, the applicant lists the TR report as Reference 2.

The details of methods of analysis, experimental modeling, and correlation of results to assure the integrity of reactor vessel internals with respect to FIV as recommended by RG 1.20, “Comprehensive Vibration Assessment Program for Reactor Internals During Preoperational and Startup Testing,” Revision 3, are provided in TR ANP-10306P, “Comprehensive Vibration Assessment Program for U.S. EPR Reactor Internals Technical Report,” (Comprehensive Vibration Assessment Program (CVAP) Report).

ITAAC: FSAR Tier 1 provides ITAAC for the reactor internals FIV analyses and tests and ITAAC by system for the dynamic testing of piping and components.

3.9.2.3 *Regulatory Basis*

The relevant requirements of NRC regulations for this area of review, and the associated acceptance criteria, are given in NUREG-0800, Section 3.9.2, “Dynamic Testing and Analysis of Systems, Structures, and Components,” and are summarized below. NUREG-0800, Section 3.9.2 also identifies review interfaces with other SRP sections.

1. GDC 2, “Design Bases for Protection Against Natural Phenomena,” and 10 CFR Part 50, Appendix S, “Earthquake Engineering Criteria for Nuclear Power Plants,” as they relate to systems, structures, and components important to safety designed to withstand appropriate combinations of the effects of normal and accident conditions with the effects of natural phenomena.

2. GDC 4, “Environmental and Dynamic Effects Design Bases,” as it relates to systems, structures, and components important to safety appropriately protected against the dynamic effects of discharging fluids.

3. GDC 14, “Reactor Coolant Pressure Boundary,” as it relates to designing systems, structures, and components of the reactor coolant pressure boundary to have an extremely low probability of rapidly propagating failure and of gross rupture.

4. GDC 15, “Reactor Coolant System Design,” as it relates to designing the reactor coolant system with sufficient margin to assure that the reactor coolant pressure boundary is not exceeded during normal operating conditions, including anticipated operational occurrences.

5. 10 CFR Part 50, Appendix B, “Quality Assurance Criteria for Nuclear Power Plants and Fuel Reprocessing Plants,” as it relates to quality assurance in the dynamic testing and analysis of systems, structures, and components.

6. 10 CFR 50.55a, “Codes and standards,” and GDC 1, “Quality Standards and Records” to 10 CFR Part 50, “Domestic Licensing of Production and Utilization Facilities,” Appendix A, “General Design Criteria for Nuclear Power Plants,” as they relate to the testing of systems and components to quality standards commensurate with the importance of the safety function to be performed.

Acceptance criteria adequate to meet the above requirements include, but are not limited to, guidance from:

1. RG 1.20, “Comprehensive Vibration Assessment Program for Reactor Internals During Preoperational and Initial Startup Testing,” March 2007, Revision 3.

2. RG 1.29, “Seismic Design Classification,” March 2007, Revision 4.

3. RG 1.61, “Damping Values for Seismic Design of Nuclear Power Plants,” March 2007, Revision 1.

4. RG 1.68, “Initial Test Programs for Water-Cooled Nuclear Power Plants,” March 2007, Revision 3.

5. RG 1.92, “Combining Modal Responses and Spatial Components in Seismic Response Analysis,” July 2006, Revision 2.

6. ASME OM-S/G-2000, “Standards and Guides For Operation of Nuclear Power Plants,” Part 3, “Requirements for Preoperational and Initial Start-Up Vibration Testing of Nuclear Power Plant Piping Systems,” and Part 7, “Requirements for Thermal Expansion Testing of Nuclear Power Plant Piping Systems.”

3.9.2.4 *Technical Evaluation*

3.9.2.4.1 Piping Vibration, Thermal Expansion, and Dynamic Effects

In FSAR Tier 2, Section 3.9.2.1, “Piping Vibration, Thermal Expansion, and Dynamic Effects,” the applicant described the pre-operational testing, initial fuel loading, and pre-critical testing performed to verify that piping systems, including snubbers, restraints, and supports, satisfy design requirements for vibration and thermal deflection, and dynamic loading effects during routine operational and transient events (e.g., valve closures and flow instabilities). In FSAR Tier 2, Section 3.9.2.1, the applicant made a commitment to conduct a piping steady-state vibration, thermal expansion, and operational transient test program. The test program is also described by the applicant in FSAR Tier 2, Section 14.2.

The staff reviewed the applicant’s description of analyses and startup testing provided in FSAR Tier 2, Sections 3.9.2.1 and 14.2 to determine if the testing program will demonstrate that:

- Applicable piping systems, restraints, components, and supports have been adequately designed, fabricated, and installed to withstand static and dynamic loadings under steady-state and transient conditions.

- Piping systems can thermally expand consistent with the stress analysis.

In FSAR Tier 2, Section 3.9.2.1, the applicant stated that testing is performed on the following piping systems and identified in FSAR Tier 2, Table 3.2.2-1, “Classification Summary”:

- ASME Code, Section III, Class 1, 2, and 3 piping systems

- High-energy piping systems inside Seismic Category I structures

- High-energy piping systems whose failure would reduce the safety level of a Seismic Category I SSC

- Seismic Category I portions of moderate-energy piping systems located outside of containment

The staff reviewed FSAR Tier 2, Table 3.2.2-1, but did not find where the applicant identified the high and moderate piping systems recommended in SRP Section 3.9.2, Subsections I.1.B, C, and D. In addition, the staff was unable to determine if all ASME piping systems are included in the applicant's testing program. Therefore, in RAI 160, Question 03.09.02-10, the staff requested that the applicant provide this information.

In a February 23, 2009, response to RAI 160, Question 03.09.02-10, the applicant stated that FSAR Tier 2, Table 3.2.2-1 contains all the ASME Code, Section III, Class 1, 2, and 3 piping systems for the U.S. EPR, as recommended by SRP Section 3.9.2, Subsection I.1.A. FSAR Tier 2, Table 3.6.1-1, "High-Energy and Moderate-Energy Fluid Systems Considered for Protection of Essential Systems," contains a listing of the high-energy piping systems inside Seismic Category I structures for the U.S. EPR, as recommended by SRP Section 3.9.2, Subsection I.1.B. In accordance with the guidance of SRP Section 3.9.2, Subsection I.1.C, FSAR Tier 2, Table 3.6.1-3, "Building, Room, Break, Target, and Protection Required," lists those high-energy systems in which a failure of a portion (i.e., terminal end breaks) could potentially reduce the functioning of some Seismic Category I feature to an unacceptable safety level. Seismic Category I moderate-energy piping systems provided in FSAR Tier 2, Table 3.6.1-1 are among the ASME Code Class 2 and 3 piping systems provided in FSAR Tier 2, Table 3.2.2-1. Therefore, the staff finds that FSAR Tier 2, Table 3.2.2-1 along with FSAR Tier 2, Table 3.6.1-3 satisfy the guidance of SRP Section 3.9.2, Subsection I.1.D. In addition to the information provided above, as part of FSAR Tier 2, Section 3.9.2.1, the SSCs for which startup testing is performed are identified in FSAR Tier 2, Section 14.2.1, "Summary of Test Program and Objectives," which includes ASME Code Class 1, 2, and 3 piping systems. FSAR Tier 2, Section 3.9.2.1 will be revised to add references to FSAR Tier 2, Tables 3.6.1-1 and 3.6.1-3.

The staff reviewed the applicant's February 23, 2009, response to RAI 160, Question 03.09.02-10, and finds that the applicant has included all necessary ASME Code Class 1, 2, and 3, high- and moderate-energy piping in the ITP, therefore satisfying the guidance of the SRP. FSAR Tier 2, Section 3.9.2.1, Revision 1 was revised; therefore, the staff considers RAI 160, Question 03.09.02-10 resolved.

In FSAR Tier 2, Section 3.9.2.1, the applicant states that performance tests are run on systems to verify operation and to check the performance of critical pumps, valves, controls, and auxiliary equipment. This testing includes transients to identify unacceptable movement, noise, vibration, and damage caused by rapid valve opening and closing, safety valve discharge, pump operation, and other operational transients. The U.S. EPR transients to be simulated during testing are based on RG 1.68. During transient testing, the piping and restraints are monitored for vibration and expansion, and automatic safety devices, control devices, and other major equipment are observed for indications of overstress, excessive vibration, overheating, and noise. The staff finds the applicant's description of transient testing and the use of RG 1.68 acceptable.

The staff reviewed FSAR Tier 2, Section 3.9.2.1 and determined that the applicant did not provide a list of selected locations in the piping system at which visual inspections and measurements (as needed) will be performed during testing as recommended in SRP Section 3.9.2, Subsection II.1, Acceptance Criterion C. Therefore, in RAI 160, Question 03.09.02-12, the staff requested that the applicant provide a list of selected locations

in the piping system for which visual inspections and measurements as needed will be performed (as needed) during testing.

In a February 23, 2009, response to RAI 160, Question 03.09.02-12, the applicant stated SRP Section 3.9.2, Subsection II.1 Acceptance Criterion C states that an acceptable test program will include a list of selected locations in a piping system at which visual inspections and measurements will be performed during the tests. These locations will be at pipe supports, particularly supports with allowances for free thermal movements (e.g., spring and snubber supports). The criteria for determining these locations are described in FSAR Tier 2, Section 3.9.2.1. Additionally, this FSAR section states: "Specific information concerning the locations where visual inspection or measurements are to be taken is also addressed in the applicable test procedures."

The staff reviewed the applicant's February 23, 2009, response to RAI 160, Question 03.09.02-12, but could not identify the locations in test procedures cited by the applicant. The staff determined that a list of the selected locations as required had not been provided. Therefore, the staff closed RAI 160, Question 03.09.02-12, and in follow-up RAI 245, Question 03.09.02-40, the staff requested that the applicant provide the selected locations where visual inspections or measurements will be taken.

In an August 12, 2009, response to RAI 245, Question 03.09.02-40, the applicant stated that there are several tests that will require monitoring of vibration, thermal expansion, and dynamic effects as part of the ITP (e.g., FSAR Tier 2, Section 14.2, Test Nos. 035, 164, 165, etc.). The ITP plan will include a list of locations in the specific piping systems that are selected for visual inspection and other measurements during the vibration, thermal expansion, and dynamic effects testing program, as recommended by SRP Section 3.9.2. This is consistent with SRP Acceptance Criterion 1.

In addition, the ITP plan will include acceptance criteria for the deflection, pressure, and/or other appropriate criteria to be obtained during the tests to determine if the stress and fatigue limits are within design levels. To clarify that the recommendations of SRP Section 3.9.2 are incorporated in the ITP plan, the applicant will revise FSAR Tier 2, Section 3.9.2.1 to state that the list of locations for visual inspection and other measurements, as well as acceptance criteria, are part of the ITP plan.

The staff notes that the ITP vibration, thermal expansion, and dynamic effects testing for piping systems described in FSAR Tier 2, Section 14.2 is limited to Balance of Plant (BOP) Piping Thermal Expansion Measurement (Test No. 034), BOP Piping Vibration Measurement (Test No. 035), and Pre–Core RCS Expansion Measurements (Test No. 165). The staff was unable to determine if the proposed ITP meets the requirements of GDC 14 and GDC 15 to perform vibration, thermal expansion, and dynamic effects testing during startup functional testing for specified high- and moderate-energy piping and their supports and restraints. Therefore, the staff closed RAI 245, Question 03.09.02-40, and in follow-up RAI 331, Question 03.09.02-62, the staff requested that the applicant clarify if all piping systems identified in FSAR Tier 2, Table 3.2.2-1, Table 3.6.1-1, and Table 3.6.1-3 are included in the ITP.

In a March 2, 2010, response to RAI 331, Question 03.09.02-62, the applicant stated that the first bullet in FSAR Tier 2, Section 3.9.2.1 states that piping systems tested include ASME Code, Section III, Class 1, 2, and 3 piping systems as shown in FSAR Tier 2, Table 3.2.2-1. All such ASME piping systems for the U.S. EPR will be Seismic Category I, and will, therefore, be safety-related. Safety-related piping systems are included in the ITP, and to clarify this, the applicant will modify FSAR Tier 2, Section 14.2.1.1.2, "Phase I - Preoperational Testing," to

indicate that ASME Class 1, 2, and 3 piping systems as described in FSAR Tier 2, Section 3.9.2.1 are included in the ITP.

In FSAR Tier 2, Section 3.9.2.1, the applicant indicates that the second bullet states that the piping systems tested include high-energy piping systems inside Seismic Category I structures, or those whose failure would reduce the safety level of Seismic Category I SSCs, as shown in FSAR Tier 2, Tables 3.6.1-1 and 3.6.1-3. Most high-energy piping inside Seismic Category I structures will be safety-related and, therefore, included in the testing described in the first paragraph of this response. However, the applicant will modify FSAR Tier 2, Section 14.2.1.1.2 to clarify that any non-safety-related portions of high-energy piping systems in Seismic Category I structures are also included in the scope of the ITP.

Lastly, the applicant stated that the high-energy piping provided in FSAR Tier 2, Table 3.6.1-3 is a subset of the piping listed in FSAR Tier 2, Table 3.6.1-1. The third bullet in FSAR Tier 2, Section 3.9.2.1 states that the piping systems tested include Seismic Category I portions of moderate-energy piping systems located outside of containment, as shown in FSAR Tier 2, Tables 3.2.2-1 and 3.6.1-1. As described previously, all Seismic Category I piping is ASME piping for the U.S. EPR. Therefore, this Seismic Category I moderate-energy piping is included in the ITP.

The staff finds that the applicant’s responses satisfy the recommendations of the SRP and are therefore acceptable. **RAI 331, Question 03.09.02-62 is being tracked as a confirmatory item.**

FSAR Tier 2, Table 1.8-2, “U.S. EPR Combined License Information Items,” Combined License Information Item 3.9-12, states that the list of snubbers will be included by a COL applicant. The staff finds the provision that the list of snubbers is the responsibility of the COL applicant acceptable, since the piping design will not be complete at the design certification stage.

In FSAR Tier 2, Section 3.9.2.1, the applicant states that piping system design is validated through a series of checks, inspections, and tests during construction and initial plant heat up to normal operating temperatures. The applicant stated that pipe and equipment supports are checked for proper installation and their initial “cold” positions are recorded during construction. During initial plant heat up to normal operating temperatures, piping systems are checked for damage and unacceptable thermal displacement. Clearances are monitored during both thermal expansion and contraction to identify if unanalyzed contact between piping and restraints occurs. Snubber and spring hanger travel are also monitored to confirm that they will allow the required thermal expansion.

In FSAR Tier 2, Section 3.9.2.1.2, “Piping Thermal Expansion Details,” the applicant states that thermal expansion testing verifies proper piping system design for thermal contraction and expansion during normal and upset transient events, and that the component supports (including spring hangers, snubbers, and struts) can accommodate the expansion of the piping. The applicant stated that if unacceptable movement is identified during thermal expansion testing, they will modify the piping layout or restraints, and retest the systems. FSAR Tier 2, Section 14.2.12, “Individual Test Descriptions,” provides descriptions of selected planned piping thermal expansion measurement tests.

Upon review of FSAR Tier 2, Section 3.9.2.1, the staff determined that the applicant did not provide a description of the thermal motion monitoring program acceptance criteria or how motion will be measured as recommended in SRP Section 3.9.2, Subsection II.1, Acceptance Criterion E. Therefore, in RAI 160, Question 03.09.02-14, the staff requested that the applicant

describe the thermal motion monitoring program acceptance criteria and how motion will be measured.

In a February 23, 2009, response to RAI 160, Question 03.09.02-14, the applicant stated that ASME OM-S/G, Part 7 describes the scope of piping systems to be monitored for thermal expansion during pre-operational and initial start-up testing and the monitoring techniques that would satisfy the minimum requirements for testing and acceptance criteria. The applicant stated that FSAR Tier 2, Section 3.9.2.1.2 will be revised to indicate that monitoring the thermal expansion of piping systems during pre-operational and startup testing, including measurement of snubber movement and verification of adequate clearance and gaps, will be done in accordance with ASME OM-S/G, Part 7. The staff finds this response acceptable, because the applicant has committed to ASME OM-S/G, which provides acceptable guidance to preservice and inservice testing to assess thermal expansions of piping systems. The staff has confirmed that the applicant revised FSAR Tier 2, Section 3.9.2.1.2 in this manner and, therefore, considers RAI 160, Question 03.09.02-14 resolved.

In FSAR Tier 2, Section 3.9.2.1.1, "Piping Vibration Details," the applicant described piping vibration testing. Testing is performed to demonstrate that piping systems withstand vibrations arising from Normal (Level A) loads and Upset (Level B) loads.

In FSAR Tier 2, Section 3.9.2.1.1, the applicant states that if excessive vibration levels are detected during testing, consideration is given to modifying the design specification to re-verify applicable code conformance using the measured vibration as input. If testing and subsequent analysis reveal that additional restraints are needed to reduce stresses to acceptable levels, they will be installed. However, the staff determined that the applicant did not provide a description of steady state or transient vibration analysis or analytical methods for Level A or Level B system vibration loads in the FSAR or in TR ANP-10264NP-A. Therefore, in RAI 160, Question 03.09.02-15, the staff requested that the applicant provide this explanation.

In a February 23, 2009, response to RAI 160, Question 03.09.02-15, the applicant stated that the vibration monitoring evaluation method VMG-2, as described in Reference 3 of FSAR Tier 2, Section 3.9.2.7, "References," is used to evaluate the Level A and Level B vibrations in the U.S. EPR piping systems. The method VMG-2 involves beam calculations of the piping to develop conservative criteria for vibration velocity and displacement based on limiting the stress to the fatigue stress limit. As stated in FSAR Tier 2, Section 3.9.2.1.1, in the event that vibrations arising from Level A or Level B loads in Phase I and Phase II tests are observed to be excessive when compared to those computed using the VMG-2 method, more detailed analyses based on VMG-1 methodology may be performed to demonstrate the acceptability of measured vibrations. If unacceptable results are obtained, appropriate corrective actions will be performed and included in the results of the CVAP, which is the responsibility of the COL applicant as noted in FSAR Tier 2, Table 1.8-2.

The staff reviewed the applicant's February 23, 2009, response to RAI 160, Question 03.09.02-15, and was unable to determine if the applicant will perform additional testing after corrective action is taken. In addition, a reference to a CVAP that includes piping vibration assessment was not identified in FSAR Tier 2, Table 1.8-2. Therefore, the staff closed RAI 160, Question 03.09.02-15 and in follow-up RAI 245, Question 03.09.02-41, the staff requested that the applicant provide further information regarding additional testing after corrective action is taken.

In an August 12, 2009, response to RAI 245, Question 03.09.02-41, the applicant stated that if unacceptable results are obtained, corrective actions will be performed and included in the

results of the CVAP for piping, and additional testing will be performed after corrective action is taken. The staff agrees that the applicant's commitment to perform additional testing after corrective action is performed meets the recommendations of SRP Section 3.9.2. However, the staff could not locate the FSAR section in FSAR Tier 2, Table 1.8-2 that references a CVAP or a list that demonstrates how the COL information items have been addressed for all piping systems specified in FSAR Tier 2, Section 3.9.2.1. Therefore, the staff closed RAI 245, Question 03.09.02-41 and in RAI 331, Question 03.09.02-63, the staff requested that the applicant identify the applicable FSAR sections.

In an April 8, 2009, response to RAI 331, Question 03.09.02-63, the applicant stated that COL Information Item 3.9-1 in FSAR Tier 2, Table 1.8-2 will be modified to address piping systems specified in FSAR Tier 2, Section 3.9.2.1 to address the CVAP. The staff finds the applicant's response acceptable because the COL information item now clearly specifies a CVAP to be completed during the pre-operational testing phase and that it includes all piping systems specified in FSAR Tier 2, Section 3.9.2.1. The staff has confirmed that the applicant revised FSAR Tier 2, Table 1.8-2 and, therefore, considers RAI 331, Question 03.09.02-63 closed.

As stated in FSAR Tier 2, Section 3.9.2.1, piping systems to be tested include ASME instrumentation lines up to the first support in each of the three orthogonal directions from the process pipe or equipment connection point. The staff agrees that this is an acceptable decoupling point for the testing boundary based on standard industry practice. Therefore, the staff finds this acceptable.

3.9.2.4.2 Seismic Analysis and Qualification of Seismic Category I Systems, Components, and Equipment

The applicant provided a description of the seismic analysis and qualification of Seismic Category I systems, components, and equipment in FSAR Tier 2, Section 3.9.2.2 and FSAR Tier 2, Appendix 3A, which describes the design criteria for piping, cable trays, and ventilation ducts and FSAR Tier 2, Appendix 3C which describes the seismic qualification of reactor vessel internals for SSE loadings. A list of Seismic Category I equipment is provided in FSAR Tier 2, Section 3.2, "Classification of Structures, Systems, and Components." Seismic analysis of the ASME Code Class 1, 2, and 3 piping due to seismic events is also described in FSAR Tier 2, Section 3.12, "ASME Code Class 1, 2, and 3 Piping Systems, Piping Components, and their Associated Supports." The applicant described methods used to confirm the ability of all Seismic Category I systems and components and their supports to function as needed during and after an earthquake.

The applicant stated that the U.S. EPR design uses both dynamic and equivalent static load analysis methods for all ASME Code Class 1, 2, and 3 pressure retaining components and their supports. The equivalent static load method is used for those piping systems that can be modeled by a simple single degree of freedom model. These methods are described in the TR ANP-10264-NP-A and the Final Safety Evaluation Report documents from the staff's review of the TR. The review of the TR is further addressed in FSAR Tier 2, Section 3.12.

The staff performed a review of FSAR Tier 2, Appendix 3A. In FSAR Tier 2, Section 3A.2.4.4, "Seismic Analysis," the applicant states that the methods for the seismic analysis of HVAC ductwork and supports are provided in FSAR Tier 2, Section 3.7, "Seismic Design." A similar statement is also made in FSAR Tier 2, Section 3A.3.6, "Seismic Analysis," for cable tray, conduit, and supports. In FSAR Tier 2, Section 3.7, the applicant provided the methods and criteria for the seismic analysis of HVAC ductwork and cable tray, conduit, and their supports.

FSAR Tier 2, Sections 3A.2.4.4 and 3A.3.6 provide the pertinent methods and criteria from FSAR Tier 2, Section 3.7.3, "Seismic Subsystem Analysis," for seismic analysis of HVAC ductwork, cable tray, conduit, and their supports. In addition, FSAR Tier 2, Sections 3A.2.1, "Codes and Standards"; 3A.3.5, "Damping"; 3A.4, "References"; 3.7.1.2, "Percentage of Critical Damping Values"; 3.7.1.4, "References"; 3.7.3.9, "Multiply-Supported Equipment and Components with Distinct Inputs"; and FSAR Tier 2, Figure 3.7.1-16, "Damping Values for Cable Tray Systems," provide further clarification related to the topics of FSAR Tier 2, Appendix 3A. The staff finds that all codes and standards (including the editions) provided in FSAR Tier 2, Section 3A.4 have been accepted for the design analysis of HVAC ductwork and cable tray, conduit, and their supports. These codes and standards have been used in previous applications.

The staff notes that FSAR Tier 2, Tables 3A-1, "HVAC Ductwork Load Combinations," and 3A-2, "HVAC Support and Restraint Load Combinations," list the loading combinations for the HVAC ductwork and the HVAC supports, respectively. In RAI 52, Question 03.09.02-4, the staff requested that the applicant clarify why loading and loading combinations for Service Level B are not required. For Service Level D, the staff requested that the applicant clarify why it is not required unless a design pressure differential (DPD) load is applicable. The staff also requested that the applicant discuss the required loadings and loading combinations to be considered with DPD and the methods of combining the dynamic loads (including seismic loads), and the bases of the combinations.

In a September 15, 2008, response to RAI 52, Question 03.09.02-4, the applicant stated that the information in the two load combination tables, FSAR Tier 2, Table 3A-1, "HVAC Ductwork Load Combinations," and FSAR Tier 2, Table 3A-2, "HVAC Support and Restraint Load Combinations," is similar to the information in FSAR Tier 2, Tables SA-4212 and ASME AG-1-2003 Code, SA-4216. The applicant stated that it has formally asked the ASME Code committee for a clarification regarding reasons for not requiring load combinations for Level B or for Level D unless a DPD is applicable and the combination that would be utilized for Level D if DPD was applicable. In addition, the applicant stated that a note will be added to both FSAR Tier 2, Tables 3A-1 and 3A-2 to address time phasing when combining dynamic load events. The applicant stated that FSAR Tier 2, Section 3A.4 will also be revised to add the reference NUREG-0484, "Methodology for Combining Dynamic Responses." The staff finds this acceptable, because the reference provides an acceptable method of load combination. The staff verified that the applicant revised the FSAR and, therefore, considers RAI 52, Question 03.09.02-4 resolved.

FSAR Tier 2, Tables 3A-1 and 3A-2 do not include load combination for DPD. Therefore, in RAI 287, Question 03.09.02-61, the staff requested that the applicant clarify whether the ductwork support design includes DPD. If DPD is included in the ductwork support design, the staff requested that the applicant explain how the loads are combined for Service Level D. The staff also requested that the applicant clarify the stress allowable for the different service levels.

In an August 31, 2009, response to RAI 287, Question 03.09.02-61, the applicant stated that as a result of the ASME response for ductwork stress allowable limits, FSAR Tier 2, Section 3A.2.4.1, "Allowable Stress Criteria," will be revised to show the combined membrane, and bending stress allowable for Level C will be revised to 1.8x0.6 Fy to be consistent with ASME AG-1, 2003, Table AA-4321. Also as a result of the ASME response for load combinations for ductwork and ductwork supports, FSAR Tier 2, Tables 3A-1 and 3A-2 will be revised to align with combinations defined in ASME AG-1, 2003, AA-4300. In addition, FSAR Tier 2, Section 3A.2.2.1, "HVAC Ductwork Loads," will be revised to align the HVAC

ductwork loads with ASME AG-1, 2003, Paragraph AA-4211, and to clarify that seismic loads are only used for SSE events.

DPD loads will be further described and considered in the Level D combination, where ductwork exists in a pipe break affected area. Also, the applicant will revise FSAR Tier 2, Section 3A2.4.1 to add stress allowable limits for those service levels due to the addition of load combinations for Levels B and D.

The staff closed RAI 287, Question 03.09.02-61, and in RAI 331, Question 03.09.02-67, the staff requested that the applicant clarify how normal loads will be determined if a duct system may be designed using one pressure value that envelops normal operating pressure differential (NOPD) and system operational transient (SOPT), and clarify why fluid momentum load (FML) is not included in the normal load combination for support design. In a March 2, 2010, response to RAI 331, Question 03.09.02-67, the applicant stated that use of an envelope of NOPD and SOPT is allowed in HVAC duct design per ASME AG-1, Section SA, and an option for normal loads will be added for this case to FSAR Tier 2, Section 3A.2.2.1 and Section 3A.2.3.1, "HVAC Support and Restraint Loads." The applicant stated that FSAR Tier 2, Section 3A.2.3.1 will be changed to include FML loads in the normal loads for the supports. The staff finds that the normal load calculation option and load combination for Level D comply with ASME AG-1 Section AA. The staff has confirmed that the applicant revised the FSAR and, therefore, considers RAI 331, Question 03.09.02-67 resolved.

3.9.2.4.3 Dynamic Response Analysis of Reactor Internals under Steady State and Operational Flow Transient Conditions

As described below, the response of the reactor pressure vessel internals to flow-induced excitation mechanisms will be addressed during pre-operational testing in accordance with the guidance of RG 1.20. In FSAR Tier 2, Section 3.9.2.3, "Dynamic Response Analysis of Reactor Internals Under Operational Flow Transients and Steady-State Conditions," the applicant stated that the vibration response of the U.S. EPR reactor internals is determined by a combination of evaluation and testing. Analyses based on scale models, confirmed by scale model testing, are used to verify that the analytical methods and design inputs used for the full-scale design analysis are valid.

In FSAR Tier 2, Sections 3.9.2.3 and 3.9.2.4, "Preoperational Flow-Induced Vibration Testing of Reactor Internals," the applicant stated that the RPV internals are currently classified as a prototype with the intent to reclassify the RPV internals as a non-prototype design by using the Olkiluoto-3 reactor as prototype. This reclassification is pursuant to the completion of hot functional testing (HFT) and inspection of the Olkiluoto-3 RPV internals together with meeting other RG 1.20 provisions for prototype designation. In FSAR Tier 2, Section 3.9.2.4, the applicant stated that either an extensive measurement program or a complete inspection of the U.S. EPR RPV internals will be performed during hot functional testing in accordance with RG 1.20 for non-prototype Category I designation. In RAI 245, Question 03.09.02-45, the staff requested that the applicant provide details of the pre-operational vibration and test program, details of prototype testing and design, or the justification for classification of the U.S. EPR as non-prototype.

In a December 16, 2009, response to RAI 245, Question 03.09.02-45, the applicant stated that CVAP Report, Section 3.0 through Section 6.0 provide a description of the test program and that the first U.S. EPR will have the classification of a "prototype" design. Reference to the CVAP Report will be added to the FSAR Tier 2, Sections 3.9.2.1.1, 3.9.2.3, 3.9.2.4, and 3.9.2.7 and Table 1.6-1, "Reports Referenced." FSAR Tier 2, Sections 3.9.2.3 and 3.9.2.4 will be

revised to state that the U.S. EPR reactor vessel internals will be classified “prototype” and to remove references to the Olkiluoto-3 RV internals as being the “prototype” for the RV internals. FSAR Tier 2, Tables 3.9.2-1 through 3.9.2-5 will be deleted.

The staff agrees with the classification of the U.S. EPR as a prototype and the applicant’s proposed revisions to remove the Olikuoto-3 subject material. The applicant made the change to the FSAR. However, the staff reviewed the CVAP Report sections and noted that the applicant did not provide a discussion of the details of the pre-operational vibration and test program to demonstrate the adequacy of the reactor internals. The staff also noted that neither the CVAP Report, nor FSAR discusses the provisions for vibration predictions, test acceptance criteria and bases, nor permissible deviations from the criteria required before testing, as recommended in SRP Section 3.9.2. The staff further determined that the proposed CVAP test plan did not explain why all recommendations of SRP Section 3.9.2, Subsection II Acceptance Criteria (4) are not applicable, and consequently, did not satisfy GDC 1 and GDC 4. Therefore, the staff closed RAI 245, Question 03.09.02-45, and in RAI 422, Question 03.09.02-87, the staff requested that the applicant clarify how the CVAP meets all the recommendations of SRP Section 3.9.2, Subsection II Acceptance Criteria (4) or why an exception is taken. **RAI 422, Question 03.09.02-87 is being tracked as an open item.**

The staff notes that since the applicant has affirmed the U.S. EPR as a prototype, the provisions of the SRP and of RG 1.20 which apply to a prototype must be provided. One of these provisions recommends the inclusion of dynamic analysis and testing of steam generator (SG) internals for PWRs and other non-safety-related components that must retain structural integrity to avoid production of loose parts which could adversely affect the capabilities of the other plant equipment to perform their safety functions. In FSAR Tier 2, Section 3.9.2.4, the applicant stated that the SG upper internals and flow conditions experienced by SG upper internals are similar to existing and currently operating SGs in the U.S. and Europe. However, the staff notes that the U.S. EPR feed water and steam flow rates are greater than those in currently operating SGs in the U.S. In FSAR Tier 2, Section 3.9.2.4, the applicant concluded that based on operational experience, U.S. EPR SG components will not be subject to excessive vibration, and, therefore, no FIV analyses or startup testing is planned.

However, changes in the U.S. EPR design due to increased power level could introduce differences that may challenge the applicant’s premise that FIV analyses or startup testing is not required. In RAI 160, Question 03.09.02-25, the staff requested that the applicant identify differences between the SG upper internals and flow conditions in the U.S. EPR design and those in the “similar” plants cited by the applicant and clarify why these differences will result in similar, and problem-free vibration response such that no FIV analyses or startup testing is needed. The staff also requested that when FIV response results from other reactors are used to predict U.S. EPR component responses, the applicant provide complete justifications for the structural and flow similarities between the U.S. EPR and the other reactors for each U.S. EPR reactor component.

In a May 6, 2009, response to RAI 160, Question 03.09.02-25, the applicant stated that excessive vibrations due to acoustic resonances from flow in attached piping systems are eliminated by verifying that the piping systems are screened for this phenomenon in the design phase. The staff determined that the information in this response was incomplete; therefore, in follow-up RAI 245, Question 03.09.02-58, the staff requested that the applicant clarify the following:

1. the methodology used in screening the U.S. EPR steam system design for potential flow-excited and structural resonances

2. the results of its implementation of the methodology for the U.S. EPR design

In an August 12, 2009, response to follow-up RAI 245, Question 03.09.02-58, the applicant stated the following:

1. The methodology used in screening for sources of acoustic resonance in the U.S. EPR is described in R.M. Baldwin and H.R. Simmons, "Flow-Induced Vibration in Safety Relief Valves," ASME Journal of Pressure Vessel Technology, Volume 108/267, August 1986. The applicant's [date] response to RAI 245, Question 03.09.02-43 is relevant**,** as that response expands on the design criteria described in Reference 1 to provide an overview of the methodology to be incorporated into the design criteria for the RCS piping, as well as the design of piping attached to the SG. This design objective and its evaluation will be included in the CVAP for the SG and applicable piping systems (RCS, main steam system (MSS), and main feed water system (MFWS)). Implementation of the methodology will be performed later in the design process.

2. The screening methodology provided in the response to RAI 245, Question 03.09.02-43 states that the methodology is based on testing of 40 inservice valves and standoff branch lines and is the method typically followed by the industry when screening for this source of excitation.

The applicant stated that the described methodology is intended to prevent the possibility of vortex shedding over stand pipes, branch lines and cavities from coupling into acoustic and structural resonances of piping. The applicable systems are RCS, MSS, and MFWS. The methodology outlined by the applicant follows that described in the referenced ASME journal by Baldwin and Simmons (1986) and that piping systems that may require corrective action have a Strouhal number in the range between 0.3 and 0.6. The applicant further stated that if standoff pipes are found to be susceptible to acoustic resonance, as determined by a Strouhal number between 0.3 and 0.6, through the entire operating range of flow, then measures will be taken to redesign the piping so that acoustic resonance will not occur. The staff noted that more recent studies such as the Ziada and Shine study, "Strouhal Numbers of Flow-Excited Acoustic Resonance of Closed Side Branches" (Journal of Fluids and Structures, Volume 13, Issue 1, January 1999), indicate that there are piping system configurations for which the Strouhal number range should be expanded to between 0.3 and 0.63. The staff closed RAI 245, Question 03.09.02-58, and in RAI 331, Question 03.09.02-65, the staff requested that the applicant justify why an upper Strouhal number of 0.6 is used in the piping screening criteria as opposed to 0.63.

In an April 8, 2010, response to RAI 331, Question 03.09.02-65, the applicant stated that the CVAP Report will be revised to require that screening for acoustic resonance will be performed in accordance with a Strouhal number range of 0.3 to 0.63. **RAI 331, Question 3.09.02-65 is being tracked as a confirmatory item until the CVAP report is revised.**

Lastly, in the applicant's May 6, 2009, response to RAI 160, Question 03.09.02-25, the applicant stated that a FIV analysis for the U.S. EPR steam separator design determined that the steam separators are not subjected to excessive vibration. Since the applicant did not provide the information requested the staff closed RAI 160, Question 03.09.02-25, and in RAI 245, Question 03.09.02-59 and in RAI 422, Question 03.09.02-88, the staff requested that the applicant provide a comparison of the structural capability of the U.S. EPR SG internals with

similar plants. The staff noted that the four European units cited by the applicant that have similar SG upper internals to the U.S. EPR RSG design and are representative of the expected performance of the U.S. EPR are:

1. Doel Unit 4 (79/19T SG) – in service since 1996 (13 years)

2. Tihange Unit 3 (79/19T SG) – in service since 1998 (11 years)

3. CZB1 (73/19TE SG) – in service since 1996 (13 years)

4. CV2 (73/19TE SG) – in service since 1999 (10 years)

In a December 16, 2009, response to RAI 245, Question 03.09.02-59, the applicant stated CVAP Report, Appendix A describes the methods that will be used to screen for sources of acoustic resonances in the piping systems to confirm that this mechanism will not be active. Additionally, the applicant states that the FIV analyses using upper bound FIV inputs to determine flow excitation resulting from turbulence and vortex-shedding were performed for steam separators, dryers, and miscellaneous structures in the steam dome. The results of the analyses concluded that these components are not susceptible to excessive vibrations. In a July 29, 2011, response to RAI 422, Question 03.09.02-88, the applicant stated that although the mass flow rate is 17 percent higher in the U.S. EPR than in the comparison plants, the larger surface area of the dryer cells results in the flow velocity being bounded by the four comparison plants. Further, the applicant has stated that the temperature and density within the steam drum is approximately the same as for the comparison plants. Lastly, the steam dryer should have similar modal frequencies and mode shapes as the other plants. After reviewing these responses, the staff considered RAI 245, Question 03.09.02-59 closed. The staff agrees that the applicant has adequately addressed the effects of the higher mass flow rate on the comparison of the U.S. EPR with the four comparison plants. The staff also finds that the applicant has demonstrated that the four comparison plants are similar in design and operating conditions. Except for the change to the CVAP Report to include the information, the staff considers RAI 422, Question 03.09.02-88 is resolved.

The applicant described in the CVAP Report that the primary steam separators are a new design and an FIV analysis to demonstrate the integrity of these separators has been performed. The upper frequency limit of this analysis is 350 Hz. In RAI 422, Question 03.09.02-90, the staff requested that the applicant provide the basis for the 350 Hz upper frequency limit used in the FIV analysis.

In a May 20, 2011, response to RAI 422, Question 03.09.02-90, the applicant stated that the first seven modes of the steam separators were analyzed. The highest frequency mode was below 250 Hz. The applicant further stated that the response to turbulence decreased by five orders of magnitude for the frequency range within the first seven modes, and concluded that this magnitude reduction coupled with the fact that turbulent pressures decrease rapidly with frequency demonstrated that an analysis of these first seven modes was sufficient to capture any significant response of the steam separators to turbulence. The staff reviewed this response and noted the reasons expressed for going to 350 Hz are based upon the calculated responses and the trends of the magnitude of the displacement response. The staff further noted that the calculated response is dependent upon the forcing function provided by the applicant in the CVAP Report. The staff finds extending the turbulence forcing function to 250 Hz acceptable, because the 250 Hz frequency range covers the first seven modes of the response. These modes are sufficient to capture the significant response of the steam separators for the reasons discussed above. Further, the applicant will update the CVAP

Report to include Figure 03.09.02-90.1 which contains the power spectral density (PSD) over the 250 Hz frequency range. **RAI 422, Question 03.09.02-90 is being tracked as a confirmatory item.**

The applicant provided an analysis of the steam separators that is similar to the analysis performed for the flow distribution device (FDD). The damping applied to the model is briefly summarized in CVAP Report, Section B.3.1. The total damping consists of the following:

1. structural damping associated with hysteresis of the material

2. structural damping created by the nonlinear interactions of the lateral supports

3. hydrodynamic damping of steam mixture

The total damping ratio of less than one percent viscous, due to the structural damping associated with hysteresis of the material, was applied to the steam separators. The applicant neglected the additional damping of the other two mechanisms. The staff found the viscous damping value of less than one percent used in the analysis to be conservative and acceptable.

The applicant included CVAP Report, Figure B-6 from Au-Yang (Flow-Induced Vibration of Power and Process Plant Components, New York, 2001) in discussing the forcing function for the steam separators. This forcing function shows peaks that exceed the upper bound curve the applicant used in the analysis. These are at a non-dimensional frequency of F=2.5 (+3 dB) and F=3.0 (+30 dB). This pressure PSD data was presented in Nuclear and Engineering Design, Volume 58, Issue 1, May 1980, in the article titled, "Dynamic Pressure Inside a PWR -- A Study Based on Laboratory and Field Test Data," by M.K. Au-Yang and K.B. Jordan.
In reviewing the references, the staff noted that the pressure PSDs were measured in the downcomer and that the referenced peaks are due to the RCP rotational tones. Since the staff does not expect the RCP tones to be present in the RSG outside of the primary cooling loop, the PSD levels due to the RCP tones and shown to be above the upper bound curve are acceptable. Nevertheless, the staff cannot ascertain if there are other acoustic sources in the piping systems attached to the RSG which could manifest as pressure levels above the upper bound PSD. Therefore, in RAI 422, Question 03.09.02-92, the staff requested that the applicant provide clarification.

In a March 30, 2011, response to RAI 422, Question 03.09.02-92, the applicant stated that on the main steam line (MSL), there are no pumps upstream of the turbine and that the MSL piping will be subject to the same design criteria as those used to prevent vortex shedding excitation. On the feed water system, there are pumps that may put acoustic tones into the fluid column, but the conditions (fluid to steam mixture) do not favor coupling of this acoustic source to the RSG volume. Further, the applicant has committed to monitoring the RSG and the associated piping systems during startup testing and during the operating life of the U.S. EPR for evidence of acoustic excitation as documented in the CVAP Report. If these sources are detected, corrective actions, including piping system redesign, will be taken. With this clarification, the staff finds that monitoring the RSG and the associated piping during startup testing can detect adverse acoustic sources. Therefore, the staff considers RAI 422, Question 03.09.02-92 resolved.

The applicant provided a comparison of the steam separator response to both the displacement and fatigue criteria and the frequency response functions (FRFs) and mode shapes for the primary steam separator in CVAP Report, Section B.3.3. As shown in the CVAP Report, Tables B-2 and B-3, the acceptance criteria are met with very large margins, which the staff

finds acceptable. However, the applicant did not provide a discussion of bias error or uncertainty for this analysis as recommended in SRP Section 3.9.2. Therefore, in RAI 422, Question 03.09.02-95, the staff requested that the applicant clarify the effects of bias error and uncertainty on the results of the analysis shown in CVAP Report, Tables B-2 and B-3.

In a March 30, 2011, response to RAI 422, Question 03.09.02-95, the applicant provided a general discussion of the effects of both mass (through the effects of dimensional tolerance) and stiffness (through material properties) on the analysis of the RSG. The applicant stated that the effect of uncertainties on the response is +/-5 percent of the vibration amplitude. The very large margins to the criteria predicted by the applicant are not affected by this magnitude of uncertainty. The applicant further stated that the conservatism introduced into the analysis through the turbulent pressure PSDs, the large correlation lengths, the conservative convective velocity, and low damping values provide upper bound responses. Further, the analysis incorporated additional conservatism via the fatigue strength reduction factor (FSRF) into the limiting case responses. Lastly, the applicant will revise CVAP Report, Appendix B to include this information. With these clarifications and discussions of conservatism, the staff finds that the issues of uncertainties in frequency and the level of the response have been adequately addressed. **RAI 422, Question 03.09.02-95 is being tracked as a confirmatory item** pending the CVAP Report revision.

In CVAP Report, Section B.3.3.2, the applicant discusses the effects of axial leakage flow on the vibration of the primary steam separator in the RSG. The staff agrees that restricted channels or tight clearances are a requirement for this type of excitation and, therefore, the primary steam separator will not be affected by this mechanism. The staff noted that acoustic resonances in the piping system due to flow past valve bodies, branch lines, etc., can excite the structures of the RSG. The applicant is using the methods described in CVAP Report, Appendix A, to ensure that this mechanism is not present in the design of the piping system. The staff agrees that the resonance condition in piping systems due to this shear wave mechanism is unlikely as long as this design criterion is met. The staff was unable to locate where the applicant addressed the likelihood of any other sources of acoustic excitation in the RSG. Therefore, in RAI 422, Questions 03.09.02-96 and 03.09.02-97, the staff requested that the applicant clarify a statement addressing the magnitude of flow and to identify all sources of acoustic pressure fluctuation and provide a discussion that addresses excitation of the steam separators due to turbulent cross flow.

In a March 30, 2011, response to RAI 422, Questions 03.09.02-96 and 03.09.02-97, the applicant stated that there are two primary sources of acoustic pressure fluctuation, pressure pulsations due to MFWS pumps and acoustic resonances in both the MFWS piping and in the MSL. The staff concurs with the applicant's assertion that the pressure pulsation from the MFWS pumps will not efficiently couple with the RSG volume because of the acoustic impedance mismatch between the water in the MFWS piping and the water steam mixture in the RSG. Additionally, the staff agrees that the geometry of the transmission path for the acoustic pressures from within the MFWS piping to the spray nozzles in the MFWS will attenuate the acoustic pressures significantly. The staff has also reviewed and concurs with the applicant's design methodology for preventing the acoustic resonances within piping system due to flow over the cavities of safety valves, standoff pipes for valves and branch lines as discussed in the CVAP Report. With this clarification, the staff concluded that acoustic excitation of the RSG is unlikely and has been adequately addressed by the applicant. The response further stated that the recirculation flow is approximately 3.5 percent of the feedwater flow to the RSG. Additionally, in CVAP Report, Appendix B, Section B.3.3.2, the applicant stated that the majority of the steam flow is through the steam separators and that only the recirculation flow created by

the steam separators would have the capability of creating a secondary side flow condition to induce a response. However, the staff concluded that because the majority of the recirculation flow exits the steam separators in the first stage, below the entrance to the separators and passes over the external surfaces of the piping and is spread over a large surface area, and because the total recirculation flow is about 3.5 percent of the feedwater flow, that the recirculation flow within the steam separators is too low to induce a significant response of the steam separators from turbulent cross flow. With the clarification of the magnitude of the recirculation flow, the staff considers RAI 422, Question 03.09.02-96 resolved. The staff also considers RAI 422, Question 03.09.02-97, resolved because the applicant clarified the potential sources of acoustic excitation, and the staff concluded from the discussion of the mechanisms above, that acoustic excitation of the RSG volume and structures will not occur and that turbulent cross flow excitation of the steam separator is unlikely to occur.

SRP Section 3.9.2 recommends that the RSG be instrumented during the pre-operational test program. The staff reviewed the applicant’s description of the instrumentation for the piping systems external to the RSG, but have a contact with RSG (e.g., the MSS, MFWS water, and the RCS) as outlined in CVAP Report, Appendix A.3 and shown in CVAP Report, Figures A-2 and A-3. In RAI 422, Question 03.09.02-98, the staff requested that the applicant provide the details of the instrumentation, including types, locations, specifications/accuracy, and ability to detect acoustic resonances that affect the RSG.

In a July 7, 2011, response to RAI 422, Question 03.09.02-98, the applicant stated that the MSL and MFWL instrumentation plans include both temporary and permanent instrumentation. The applicant also provided figures from a similar U.S. EPR plant to show the general instrumentation scheme that will be followed. However, the applicant also stated that the details of the instrumentation, including types, locations, and specifications will be developed later in the design processes as part of the vibration measurement and inspection program in accordance with RG 1.20. The staff reviewed the general instrumentation layout and the generic/typical instrumentation specifications contained within CVAP Report, Appendix A, and find them sufficient at this point in the design. The staff considers RAI 422, Question 03.09.02-98, resolved because the applicant has provided the general details of the instrumentation locations and specifications for the piping systems attached to the RSG. Further, the applicant has committed to providing the full details of the instrumentation prior to the pre-operational test, as part of the vibration measurement and inspection program consistent with the recommendations of RG 1.20. The response provides the clarification that the CVAP Report meets RG 1.20.

The analysis of the RSG lower internals is addressed in FSAR Tier 2, Section 5.4.2.3.1, “Mechanical and Flow-Induced Vibration.” In RAI 384, Question 03.09.02-68, the staff requested that the applicant provide the applicable details or reference documents:

1. to relate to the vibration model tests and operating experience of similar SG designs in France

2. to support the results indicating that fluid-elastic instability will not occur during full-power, steady-state, and normal operating conditions

3. to support the statement that the allowable limits that would prevent high cycle fatigue failure and tube impacts would not be exceeded

4. to support the statement that the effects from acoustic pressure fluctuations resulting from the blade passing frequencies would be negligible

In a March 22, 2011, response to RAI 384, Question 06.09.02-68, the applicant provided an overview of the analyses of the SG tube bundle performed in accordance with the 2004 ASME B&PV Code, Section III, Appendix, N-1300. This overview included analysis for fluid-elastic instability, cross-flow turbulence induced vibration, vortex-induced vibration, and parallel FIV on the primary and secondary side. The applicant described in detail the EVA I, EVA II, and SRM series of tests reported in the Electric Power Research Institute (EPRI) Report NP-4559 and the EPRI Steam Generator Reference Book. Together with the ASME Code and in-air damping measurements, these tests were used to select appropriate values of the Connors' constant of the tube bundle for fluid elastic evaluation. The applicant stated that the value of Connors' constants are conservative relative to the EVA I derived values. The applicant's values are conservative relative to the SRM testing and the ASME Code recommended values as well. Further, for low reduced damping cases (i.e., single phase water flow), the ASME Code recommends a Connors' constant of 3.3. The applicant uses a value which is less than 3.3 and is thus conservative. Based on the conservatism of the values, the staff finds the applicant's values of the Connors' constant acceptable.

For loosely supported tubes, the applicant stated damping values from ASME Code, Appendix N-1331.3 (as much as five percent); "Flow-Induced Vibration of Power and Process Plant Components," M.K. Au Yang stated three percent. The applicant chose a lower value for a loosely supported tube. Further, the applicant credited an additional 2 percent damping for tube sections that experience secondary side fluid void fraction between 60 and 90 percent based on industry literature. The staff finds the damping values conservative and acceptable.

The applicant also set fluid elastic instability acceptance criteria of fluid-elastic instability margin (FSM) to greater than 1.3. The staff notes that this criterion is consistent with all of the fluid elastic instability analysis the applicant performed for the RV upper internals, and finds this criterion acceptable.

The random turbulence excitation and vortex shedding analysis followed the methodology employed for the RV internals. The applicant chose a turbulence pressure PSD forcing function that is representative of a single phase flow, because the PSD is shown to be higher in magnitude than the other measured spectra from industry sources. The applicant applies the forcing function to the entire tube – both single and two phase regions. The staff concurs that the turbulence forcing function is thus demonstrated to be conservative. The staff notes that the applicant is employing a standard percentage of the free-stream fluid velocity for the convection velocity. Additionally, the correlation length is sufficiently large to cause the forcing function to be in phase along the entire length of the tube – which the staff agrees is also conservative. The staff finds the approach employed by the applicant conservative and acceptable.

The acceptance criteria for displacements follow the methodology used for the RV upper internals. The staff finds the criteria to be acceptable in terms of preventing tube impacts. The acceptance criteria for high cycle fatigue is curve "A" for both the straight and the u-bend sections except for the first span, where the applicant has determined that curve "C" from the ASME Code, Section III Code is appropriate. All of the tubes are projected to meet the stress criteria for non-flawed tubes. In fact, the staff notes that with all the noted conservatism, all the tubes still meet the more stringent curve "C" with significant margin.

When the stress analysis is conducted for tubes with 40 percent through wall wear (the applicant stated that 40 percent is chosen, because it is the usual plugging limit), a FSRF of four

together with the ratio of the section moduli are used. The staff found that the stress calculation procedure for turbulence forcing (cross-flow) and the resulting comparison to the criteria is acceptable.

For the axial flow turbulence, the calculation procedure and the acceptance criteria were the same as for cross flow turbulence. The length that resulted in the maximum response was chosen by the applicant for the analysis. The staff finds the procedure used acceptable, because it is conservative. The single phase turbulence pressure PSD developed for the fuel assembly was used for the SG analysis together with a convective velocity of 100 percent of the parallel flow velocity. A constant damping value was used for the entire tube. The effects of two phase flow and flow velocity on damping were ignored. The staff finds this approach conservative. When both the computed stresses and displacements are compared to their criteria, all the tubes meet with large margins.

For vortex-shedding induced vibration, the applicant followed the recommendations of ASME Code, Appendix N-1344, and used the turbulence induced vibration technique with an amplification factor of 20 applied to the PSD. The same technique for Strouhol number and correlation length in CVAP Report, Section 4.5.1.1.4 for RV upper internals is applied. Further, the damping ratio was the same as the value for the turbulence analysis. The applicant states that this approach means that the computed results are only applicable to the first span of the bundle. The acceptance criterion for displacement is one-half of the separation distance between the tubes. The staff notes that the applicant is employing the same criteria as it did for the RV internals analysis, and the staff finds this criterion acceptable. The high cycle fatigue criteria is the ASME fatigue curve "C" for austenitic steel for 10^{13} cycles, because the maximum stress occurs in the first span at the secondary face where residual stress from the hydraulic expansion is present. The smallest displacement margin is one percent of the criteria. The smallest stress margin is two percent of the criteria. The staff finds that these are very large margins to the criteria, and that the analysis and results are acceptable.

The applicant expects the response due to acoustic excitation of the tube bundle from the RCP vane-passing frequencies to be insignificant. The staff concurs with this assessment in the case of the RSG, because the mode shapes of the tubes will couple poorly with the longitudinal waves of the RCP blade passing forcing function. The amplitudes of the RCP pump tones assumed by the applicant are the subject of other questions. The staff finds that the applicant has sufficiently addressed the concerns raised and, therefore, considers RAI 384, Question 03.09.02-68 resolved.

The tabulated information contained within the CVAP Report did not address all the documentation suggested in RG 1.20, including the recommended information on the effects of bias errors and uncertainty on the analyses, and their results have not been included in the tables. Primarily, the applicant has not included discussions of the effects of such bias errors and uncertainties on the analyses conducted for self-excited flow instabilities. It is possible that the effect will be to make such lock-in phenomenon possible. Therefore, in RAI 458, Question 03.09.02-149, the staff requested that the applicant address the following:

1. Verify that the effect of the uncertainties and bias errors on the calculations does not result in a possible range of lock-in occurring for the RPV internals.

2. Provide tables of resonant frequencies, flow rate, shedding frequency (for the upper internals) together with any possible range of lock-in and the effect of uncertainty on the lock-in parameters per the recommendation of RG 1.20.

3. Revise the CVAP Report to include the requested information.

In a July 27, 2011, response to RAI 458, Question 03.09.02-149, the applicant provided in the markup for the CVAP Report, Table 4.19 which states the natural frequencies of the control rod guide assembly (CRGA), the normal column, level monitoring probe (LMP), and instrumentation guide tubes compared to the four criteria from ASME Code Section III, Appendix N-1324.1. The applicant has also provided Figures 03.09.02-149-1 and 03.09.02-149-2 in the RAI response which show the computed velocities over the components of the upper internals as a function of span. From reviewing the curves of velocities over the upper internals, the staff concludes that the velocities are not constant over any large spanwise portion of the columns and that this distribution of velocity makes formation of a coherent vortex shedding along the length of the CRGA, the normal columns, the LMP, and the instrumentation guide tubes less likely. The staff concluded that applicant has included significant conservatisms in the analysis of the upper internals for vortex shedding lock-in which address the concerns that vortex shedding lock-in is possible because of uncertainty in the computed velocity distributions in the upper internals. The staff finds this RAI response acceptable. **RAI 458, Question 03.09.02-149 is being tracked as a confirmatory item** pending revision of the CVAP Report.

With regard to shear wave instabilities in piping systems, the staff notes that the discussions of the design methodology in CVAP Report, Appendix A should be expanded to include the effects of bias and uncertainty. Therefore, in RAI 458, Question 03.09.02-150, the staff requested that the applicant discuss in detail the procedure for handling the bias and uncertainties in the Strouhal number and how it is incorporated into the screening methodology for shear-wave resonance in piping systems discussed in CVAP Report, Appendix A.

In a July 8, 2011, response to RAI 458, Question 03.09.02-150, the applicant provided a discussion of the methodology for incorporating the bias and uncertainties into the piping system design criteria for the Strouhal number range. Uncertainty enters into the Strouhal number through the frequency of the acoustic cavities (wave speed and characteristic length of the cavity), the uncertainties in the free stream velocities, and the uncertainties in the branch opening diameter. The staff finds that the applicant had incorporated into the piping system design criteria (see CVAP Report, Appendix A) uncertainties that would tend to bias the computed Strouhal number downward toward the criteria limit of 0.63 and is conservative. The applicant has committed to treating the uncertainty in the velocity in a pipe segment by adjusting the mass flow upward by 10 percent. Since the Strouhal number is inversely proportional to velocity, the effect of adjusting the velocity upward decreases the computed Strouhal number and is conservative. Additionally, the staff finds that the applicant intends to use the wave speed that corresponds to thermal hydraulic conditions local to the pipe under consideration. This would tend to result in lower wave speeds than would be computed by using plant conditions to calculate the wave speed. Since the wave speed is directly proportional to the Strouhal number, lower wave speed reduces the Strouhal number which the staff notes is conservative. The last parameter is the characteristic length of the piping feature. By incorporating an effective length, based on actual length, cavity diameter, and cavity blend radius (see CVAP Report, Appendix A) into the calculation of the cavity frequency, the Strouhal number can be biased downward, since the cavity frequency is inversely proportional to the effective length and directly proportional to the Strouhal number. The staff finds this response acceptable and, therefore, considers RAI 458, Question 03.09.02-150 resolved.

With regard to the acoustic modes in the RPV, in RAI 458, Question 03.09.02-151, the staff requested that the applicant provide justification for neglecting the tones associated with the

RCP in the analysis of the RPV upper internals. The staff also requested that the applicant revise the CVAP Report to include this information.

In a July 7, 2011, response to RAI 458, Question 03.09.02-151, the applicant responded that CVAP Report, Section 4.5 would be updated with an analysis of the RCP upper internals. The staff reviewed the proposed CVAP Report changes and finds that the applicant had provided the analysis using a zero to peak amplitude for all RCP tones. The staff considered the concern addressed because the applicant provided the analysis and markups for inclusion in the CVAP Report. The staff reviewed the analysis and finds that the applicant incorporated the first three rotational harmonics together with the first three blade passing harmonics. The staff finds that the analysis results in large margin to the high cycle fatigue criteria to 10^{11} cycles. Lastly, the applicant will verify the analysis using the measured RCP tone amplitude per ITAAC 3.8 in FSAR Tier 1, Table 2.2.1-5, "RCS ITAAC." **RAI 458, Question 03.09.02-151 is being tracked as a confirmatory item**.

The CVAP Report does not tabulate the resonant frequencies of the volumes internal to the RPV, the RSG, or for the attached piping systems. Therefore, in RAI 458, Question 03.09.02-152, the staff requested that the applicant revise the CVAP Report to address this information and compare these resonant frequencies to the vortex shedding frequencies for appropriate flow rates within the standard operating range of the reactor through full power to verify that the pressure disturbances generated by vortex shedding will not excite an acoustic resonance.

In an August 11, 2011, response to RAI 458, Question 03.09.02-152, the applicant stated that it will modify the FSAR Tier 1, Table 2.2.1-5, "RCS ITAAC," ITAAC 3.8 to include the requirement to perform an analysis to determine the resonant frequencies of the RCS and then determine the interaction of these resonant frequencies and the flow induced excitation created by both vortex shedding and the blade passing frequencies of the RCP. The analysis is called for in an ITAAC, because information on the piping system design from later in the design process is needed to adequately perform the analysis. The staff concluded that the modified ITAAC provides reasonable assurance that the analysis will address the concerns regarding exciting the volumes of the RCS by acoustic sources within the RCS. **RAI 458, Question 03.09.02-152 is being tracked a confirmatory item.**

CVAP Report, Figure 4-1 indicates that individual slabs representing the heavy reflector (HR) are positioned by a pair of male and female spigots, which prevents radial sliding between any two HR slabs. However, the details of the HR shown in CVAP Report, Figure 2-3 do not show how the spigots can prevent radial sliding. In addition, the staff did not locate an adequate description of the FDD. The staff was unable to determine if the theoretical structural model of the HYDRAVIB mock-up is an accurate representation of the reactor internals. Therefore, in RAI 407, Question 03.09.02-69, the staff requested that the applicant to provide additional information as requested below:

- clarify how the core barrel flange is secured in the reactor vessel
- clarify if the scaling is purely geometric for all components
- clarify the scaling used for the hold down spring
- describe the spigot construction of the HR and how they are scaled

- provide details of the FDD that can be used to confirm the finite element model input

RAI 407, Question 03.09.02-69 is being tracked as an open item.

The staff notes that CVAP Report, Section 2.1.1 does not describe how manufacturing tolerances, which can result in geometric uncertainties, are addressed in the model or in the analysis as called for in RG 1.20. Therefore in RAI 407, Question 03.09.02-71, the staff requested that the applicant provide this detail for the upper and lower skirt plates and the reactor vessel. **RAI 407, Question 03.09.02-71 is being tracked as an open item.**

The staff notes that CVAP Report, Figures 2-1 through 2-5 appear to indicate that bolting is used at the following connections:

- the upper CRGAs and the upper support plates
- the lower control rod guide columns and the upper support plates
- the flow distribution device and the lower support plate

The staff was unable to locate a description of how the bolts are included in the model or the analysis that was performed to demonstrate bolting integrity is maintained for the required design life. Therefore, in RAI 407, Question 03.09.02-72, the staff requested that the applicant provide details of the bolting used in the reactor internals and describe how bolts were addressed in the modeling, including the evaluation of fatigue loading and effects of loss of pre-compression on bolted joint integrity.

In a December 20, 2010, response to RAI 407, Question 03.09.02-72, the applicant clarified that the design preload in the bolting connections is sufficient to prevent separation of the mating members that may be induced by the FIV loadings on these components. The restraint created by the bolted connections was modeled with appropriate translational and rotational boundary conditions. By showing that the reaction loads at the bolt locations generated by the dynamic loads resulting from the flow excitations do not exceed the preload in the bolted connections, ensures that the two mating surfaces will not separate. Further, by showing that the preload exceeds the reaction loads, it is demonstrated that these bolted connections will not experience the cyclic stresses, and thus do not require analysis for high cycle fatigue, thus ensuring structural integrity. The applicant further stated that the detailed stress analysis that will address the integrity of the bolting connections to ASME Code, Section III requirements will consider appropriate load sources and categories, including the effects of loss of pre-compression, and is described in FSAR Tier 1, Table 2.2.1-5, ITAAC 3.16.

With this clarification, the staff finds that the applicant has designed the bolting of the reactor vessel internals appropriately to ensure that the separation of the mating components does not occur and therefore fatigue does not occur. Therefore, the staff considers RAI 407, Question 03.09.02-72 resolved.

To determine the adequacy and correctness of the FEM developed in the CVAP Report, as recommended by RG 1.20, in RAI 407, Question 03.09.02-73, the staff requested that the applicant provide details of the FEM. In a March 23, 2011, response to RAI 407, Question 03.09.02-73, the applicant provided additional details regarding modeling techniques for the following reactor vessel internals: Core Barrel (CB) flange; CB skirt and the hot leg nozzles in the CB; lower support plate (LSP) and connection between CB skirt and LSP; HR;

volume between the CB skirt and RV; and volume between the CB skirt and the HR. The staff reviewed the additional details in the response and determined that the response was acceptable. With the additional clarification, the staff finds the applicant developed the FEM correctly and further provided confirmatory analysis that one fluid element across the representative gap between the HR and the core barrel provides results similar to the theoretical values. Therefore, the staff considers RAI 407, Question 03.09.02-73 resolved.

To verify that the stiffness of the lower internal assembly and its restraint at the CB flange is modeled accurately, in RAI 407, Question 03.09.02-74, the staff requested that the applicant clarify how stiffness in the HYDRAVIB model was measured and discuss the comparison of the measured stiffness value with the stiffness value obtained in the FEM. In a December 20, 2010, response to RAI 407, Question 03.09.02-74, the applicant stated that the stiffness of the restraint at the CB flange and the bending stiffness of the lower internal assembly were determined by laterally displacing the assembly at the LSP and recording the applied load and the displacement. The load-displacement measurements were done for a range of LSP displacements from 0 to 0.051 mm (0 to 0.002 in.) in the two transverse directions to yield a linear relationship between the stiffness and the displacement. The experiment confirmed the absence of any spurious gaps within the mockup. Through the experiment, the applicant determined that the mean stiffness values obtained through FEM calculation agree closely with those obtained through the experimental observations. The staff finds that the applicant has clarified that the method of determining experimental value of stiffness and its verification by finite element analysis (FEA) is satisfactory. Therefore, the staff considers RAI 407, Question 03.09.02-74 resolved.

The staff reviewed CVAP Report, Section 4.2.3.1 where the applicant described the power spectral density and provided graphs of three locations in Figures 4-12, 4-13, and 4-14. The staff could not determine how the applicant input the PSD shown in Figures 4-12, 4-13, and 4-14 into the ANSYS FEM. Therefore, in RAI 407, Question 03.09.02-76, the staff requested that the applicant provide the details of the PSD FEM. In a July 5, 2011, response to RAI 407, Question 03.09.02-76, the applicant provided additional information including the codes used to input the PSD into the model, an example of how the PSD and forcing function were input to ANSYS for a spectrum analysis, how the PSD curves are interpolated by ANSYS, how the analytical computations were performed for the HYDRAVIB mockup, and explanation that the HYDRAVIB flow tests were aimed at determining the PSD for turbulence. The ITAAC for the RPV internals was modified to require an evaluation to justify the response of the internals created by the RCP acoustic pressure fluctuations that will be measured during hot functional testing in accordance with RG 1.20. The staff concluded that the applicant has correctly applied the forcing PSDs obtained by testing of the HYDRAVIB model to the analytical FEM. The staff finds that the modified ITAAC provides reasonable assurance that the reactor internals will be designed against the acoustics generated by the RCPs. **RAI 407, Question 03.09.02-76 is being tracked as a confirmatory item** pending revision of the FSAR.

To verify that the boundary conditions to the FEM have been applied correctly or conservatively, in RAI 407, Question 03.09.02-77, the staff request that the applicant provide details of the boundary conditions. In a December 20, 2010, response to RAI 407, Question 03.09.02-77, the applicant stated that the boundary conditions imposed on the FEM at the CB flange juncture were modeled through a series of one dimensional translational spring elements. The stiffness of the elements was derived through tests on the hold-down spring. To assure accurate behavior of the numerical model, the stiffness values of the spring elements were confirmed by the experiments described in the response to RAI 407, Question 03.09.02-74.
Figure 03.09.02-73-3 of the response shows the FEM and illustrates the boundary conditions

applied to the CB flange to restrain its motion. This information will be included in CVAP Report, Section 4.2.2.3. The staff finds that the applicant's FEA modeling and the application of the boundary conditions have been verified by tests and are conservative with respect to the restraints. **RAI 407, Question 03.09.02-77 is being tracked as a confirmatory item** pending revision of the CVAP Report.

The staff reviewed CVAP Report, Section 4.2.7.2, where the applicant stated that the mean stress in the CB is less than PL+PB+Q stress range of 188 MPa (27.2 ksi) therefore, fatigue curve "A" was selected for evaluating the CB. The staff notes that in accordance with ASME Code, Subsection NG-3222.4 (c), the fatigue curves of CVAP Report, Figure I-9.0, are selected based on meeting specific criteria, including mean stress. Therefore, in RAI 407, Question 03.09.02-78, the staff requested that the applicant clarify the basis for the selection of fatigue curve "A" and how the applicant determined that the mean stress in the CB is less than PL+PB+Q stress range of 188 MPa (27.2 ksi).

In a March 25, 2011, response to RAI 407, Question 03.09.02-78, the applicant stated that referring to the flow chart in ASME Code, Section III, Appendices, Figure I-9.2.3, for an elastic analysis of a stress location that is within three wall thickness of a weld with a (PL+Pb+Q) stress range greater than 188 MPa (27.2 ksi), the more conservative "C" is prescribed. Following the methodology in CVAP Report, Section 4.2.7.2, the endurance limit is estimated for fatigue curve "C" (see CVAP Report, Figure 4-21). This allowable stress is several times greater than the peak stress. Fatigue failure of the CB resulting from random turbulence in the RV downcomer is not predicted to occur. CVAP Report, Section 4.2.7.2, will be revised to refer to fatigue curve "C." The staff concluded that the applicant has correctly changed the applicable fatigue curve from fatigue curve "A" to fatigue curve "C." The staff finds this acceptable, because curve "C" considers the effect of mean stress. **RAI 407, Question 03.09.02-78 is being tracked as a confirmatory item** to verify the change to CVAP Report.

The staff reviewed CVAP Report, Figure 5.4, and noted that the core barrel components have various thicknesses in the core barrel flange at the joint, core barrel upper and lower skirt and the LSP tongue. The staff could not determine from the description of the FEM in CVAP Report, Sections 4.2.2.1 or 4.2.5 or in the ANSYS pictures provided in CVAP Report, Figures 4.6 through 4.9 if different thicknesses have been included in the FEM. Therefore, in RAI 407, Question 03.09.02-80, the staff requested that the applicant clarify how the determination of the maximum stress included the effects of different skirt thicknesses, manufacturing tolerances, stress concentrations from geometric discontinuities, and the effects of welds on the maximum calculated stress.

In a December 20, 2010, response to RAI 407, Question 03.09.02-80, the applicant stated that the FEM of the lower and upper shells of the CB uses the nominal (design) dimensions of each shell. The LSP, which is welded to the bottom of the lower CB, was modeled as a solid circular plate with the density and stiffness adjusted to obtain the same bending modal frequency that would be obtained with a more detailed local model of the LSP. The nominal (design) dimensions of the CB flange were used while modeling the CB. As described in the response to RAI 407, Question 03.09.02-74, the applicant stated that the hold-down spring was modeled as a series of spring elements with properties derived from numerical and experimental simulations to verify accurate behavior of this aspect of the model. The components of the CB were modeled with all the geometric features of the hot leg nozzles and the CB flanges, except for the local region on the CB flange where the alignment pins are located. This detail was not included in the numerical model. The application of a FSRF of 3.0 conservatively accounted for the stress amplification effects of the local structural discontinuities of the hot leg nozzles and

the CB flange juncture. Based on the margin for the most limiting stress in the CB, the manufacturing tolerances for the CB would not impact the overall results or conclusions of the current analytical evaluation.

With this clarification, the staff concluded that the applicant has investigated the effects of different skirt thicknesses, manufacturing tolerances, stress concentrations from geometric discontinuities, and the effects of welds on the maximum calculated stress and finds that due to the very large margin the structural integrity of the CB will not be compromised. The staff finds that the applicant's December 20, 2010, response to RAI 407, Question 03.09.02-80, provides the clarification that the CVAP Report meets RG 1.20. Therefore, the staff considers RAI 407, Question 03.09.02-80 resolved.

3.9.2.4.4 Pre-operational Flow-Induced Vibration Testing of Reactor Internals

In FSAR Tier 2, Section 1.8, "Interfaces with Standard Designs and Early Site Permits," FSAR Tier 2, Table 1.8-2, COL Information Item 3.9-1, the applicant stated, "A COL applicant that references the U.S. EPR design certification will submit the results from the CVAP for the U.S. EPR RPV internals and piping systems specified in FSAR Tier 2, Section 3.9.2.1, in accordance with RG 1.20." In FSAR Tier 1, Section 2.2.1, Table 2.2.1-5, the applicant provided ITAAC 3.8 to verify that the RPV internals will withstand the effects of FIV. The staff finds that the ITAAC provide reasonable assurance that the reactor internals will withstand the FIV without loss of structural integrity.

In FSAR Tier 2, Section 3.9.2.3, the applicant described the required comprehensive analysis of the RPV internals. The analysis is a combination of analytical and testing evaluations that considers various flow excitation mechanisms such as vortex shedding, leakage-flow-induced vibrations, turbulence buffeting, and acoustic sources due to pump operation and loop oscillations. In addition to these mechanisms, SRP Section 3.9.2 recommends incorporating the response of the RPV internals due to the flow excited structural and/or acoustic resonances (self-excited loads) into the analyses of any potential adverse flow conditions that may lead to a self-excited response. The staff was unable to locate where the applicant addressed the response of the RPV internals as called for by the SRP. Therefore, in RAI 245, Question 03.09.02- 46, the staff requested that the applicant provide the details of the assessment of acoustic resonances and self-excited response, along with discussion of the bias errors, uncertainties, and operational experience to be included in the vibration assessment program for the RPV internals.

In a December 16, 2009, response to RAI 245, Question 03.09.02-46, the applicant directed the staff to CVAP Report, Section 4.2.5.2.2, which states the pressure fluctuations associated with loop acoustics are not included in the numerical simulations (lower internals). The applicant stated that as the flow enters the downcomer, it is possible that fluid momentum changes can produce large magnitude and coherent pressure fluctuations that can excite the large structures. The applicant further stated that if a significant response of the RV lower internals to loop acoustics is observed during HFT, then the numerical model will be modified to include this source. The staff closed RAI 145, Question 03.09.02-46, and in RAI 422, Question 03.09.02-99, the staff requested that the applicant either include the loop acoustics in the numerical model evaluation prior to the HFT or justify waiting to revise the numerical model until after a significant response of the RV lower internals to loop acoustics is observed. **RAI 422, Question 03.09.02-99 is being tracked as an open item.**

The FDD (CVAP Report, Section 4.3.2) and the irradiation specimen basket (CVAP Report, Section 4.4) analyses also do not include loop acoustic pressures. The applicant justified not

including this forcing function on the relatively high frequencies of lowest modes of these structures. In RAI 422, Question 03.09.02-100, the staff requested that the applicant provide the anticipated frequency range and amplitude of the loop acoustic source together with the basis of that estimate. **RAI 422, Question 03.09.02-100 is being tracked as an open item.**

For the other structures of the upper internals, in CVAP Report, Sections 4.2.5.2.2 and 4.5.1, the applicant justified not including the loop acoustic source in the numerical model evaluation based upon the premise that acoustic pressures will not pass through the fuel bundle. The staff acknowledges that if viscous losses through the core are high enough, they can dissipate acoustic pressures provided the wavelength (and frequency) is proportioned correctly with the length of the fuel bundle. The staff notes that typical noise control requires that a baffle or absorber to be a minimum of one-fourth of the wavelength of the sound intended to be removed. The wavelengths of sound in the frequency range of interest to the upper internals are very long, and the distance the sound waves pass through the core may not be long enough for sufficient attenuation of the sound pressures over the entire frequency range. Therefore, in RAI 422, Question 03.09.02 101, the staff requested that the applicant address the staff's concern that the acoustic pressures from the loop acoustic source will not be sufficiently attenuated as the pressures propagate through the core resulting in excitation of the upper internals and also clarify why the loop acoustic source is not included in the numerical model. **RAI 422, Question 03.09.02-101 is being tracked as an open item.**

FSAR Tier 2, Section 14.2, Test No. 164 requires that the core be loaded with dummy fuel assemblies (FAs) or suitable alternate flow restriction devices. Since the mass of the FAs is a significant fraction of the mass of the CB and the HR, in RAI 422, Question 03.09.02-103, the staff requested that the applicant provide the mass of the CB and HR and clarify why the additional FA mass will not impact the structural frequency. **RAI 422, Question 03.09.02-103 is being tracked as an open item.**

CVAP Report, Section 4.2.5.3.4 describes viscous damping as the only mechanism found to contribute to the shell modes. The contribution of viscous damping over a frequency band was calculated using formulation by R. J. Gibert. In RAI 422, Question 03.09.02-104, the staff requested that the applicant provide documentation that the methodology is an approved industry approach accepted by the NRC, justification for use of this method, or an alternate source for this calculation. **RAI 422, Question 03.09.02-104 is being tracked as an open item.**

The tie rods are described as eight pre-stressed, long hollow tubes in CVAP Report, Section 4.7.1, which are used to hold the HR slabs together during removal of the lower internals. The applicant stated in the CVAP Report that tie rods are "not absolutely needed from an FIV perspective" during normal operating or transient conditions. In RAI 422, Question 03.09.02-107, the staff requested that the applicant clarify what is meant by this statement and provide the modal frequencies, the mode shapes and the FRFs that form the basis of the tie rod dynamic model, details of the analysis and evaluation of the measurements. **RAI 422, Question 03.09.02-107 is being tracked as an open item.**

CVAP Report, Sections 4.2.2.4 for the lower internals, 4.3.2.1 for the FDD, 4.5.1.1.4 for the upper internals, 4.6.3.2 for the CRGA and rod cluster control assembly (RCCA), and 4.7 for the HR tie rods did not address bias errors and uncertainties associated with the analysis. In RAI 422, Question 03.09.02-108, the staff requested that the applicant discuss in quantitative and definitive terms, the methods for ascertaining and incorporating the bias errors and uncertainties in the calculation of the natural (resonant) frequencies, peak response magnitude,

mean FE response, and stress magnitude, and tabulate the values of these bias errors and uncertainties. **RAI 422, Question 03.09.02-108 is being tracked as an open item.**

CVAP Report, Section 4.2.5.2.1 states that the dynamic similitude between the scale model and full scale is created using Strouhal number scaling of the turbulent pressure spectra, while noting that the Reynolds numbers between model scale and full scale are different by a factor of nearly 100. The staff noted that dynamic similitude between any two scales requires that force ratios be the same between the two scales. Since the Reynolds number is the ratio of viscous to inertial forces, the fact that it is different between scales, indicates that dynamic similitude between the scale model and full scale has not been achieved. The staff noted that partial similitude is sufficient provided it is justified through application of conservatism as indicated in SRP Section 3.9.2, Subsection 3.B.ii.1.a. Therefore, in RAI 422, Question 03.09.02-110, the staff requested that the applicant clarify:

1. The large difference between model and full-scale Reynolds Number

2. If Strouhal number scaling of the pressure spectra results in a conservative estimate of the turbulent pressure spectra that supports the partial similarity between the model scale and full scale

3. The significance of operating the model scale with the same mean flow speed in the downcomer as the full scale

In an August 19, 2011, response to RAI 422, Question 03.09.02-110, the applicant stated that the Reynolds numbers demonstrate that both the HYDRAVIB and the full-scale analysis are conducted in the turbulent flow regime. Being within the turbulent flow regime permits scaling the turbulent pressure PSDs measured in the HYDRAVIB from one-eighth scale to full-scale with sufficient accuracy for the analysis. Further, in Table 03.09.02-110-1 of the response, the applicant provided a comparison between the Strouhal numbers for the computed beam and shell modes of both the HYDRAVIB and full-scale RPV lower internals which indicates agreement between the two scales of within 1.7 percent for the dominant CB pendulum mode and 7 to 9.5 percent for the other modes. The applicant stated that close agreement between the Strouhal numbers for modes indicates the degree of similitude between the two scales. Lastly, the significance of maintaining the same flow velocity in the downcomer for the two scales is to achieve this close agreement between the Strouhal numbers. The staff considers RAI 422, Question 03.09.02-110, resolved since both scales are within the turbulent flow regime, the use of Strouhal number scaling of the pressure PSDs from one-eighth scale to full scale is acceptable. The staff further finds that using the flow velocity to maintain the modes of the full-scale and the model scale structure at nearly the same value of Strouhal number to be acceptable. The response provides the basis that the CVAP Report meets the SRP.

In CVAP Report, Section 4.2.2.3, the applicant provided a comparison of the modal response of the HYDRAVIB test facility to the FEA model of the HYDRAVIB. The techniques employed in modeling the one-eighth scale structure are used in modeling the full scale structure. Thus, the model scale could be used to assess the bias and uncertainty in the FEA modeling technique. Two tests models were developed; one without the effects of water and one with the effects of water. The results indicate that the scale model measured results and the FEA structural model computed results for the lower frequency CB “pendulum” mode are closely matched within 4 percent, but are mismatched (13 to 26 percent) for the other modes occurring at higher frequencies for the “dry” model without the effects of water. This indicates a strong frequency bias in the modeling technique. The staff noted that modal testing was repeated only for the CB “pendulum” mode under “wet” conditions and did not include the other modes occurring at

higher frequencies as was done for the “dry” model. Therefore, in RAI 422, Question 03.09.02-111, the staff requested that the applicant provide a discussion of the rational for the differences in the wet and dry testing.

In a June 21, 2011, response to RAI 422, Question 03.09.02-111, the applicant explained the differences between the wet and dry modal testing of the HYDRAVIB model in regards to the absence of many of the modes in the wet modal testing relative to the dry modal testing. The applicant referenced a July 7, 2011, response to RAI 422, Question 03.09.02-115. The applicant stated that the shell type modes were not detected in the wet HYDRAVIB modal tests. The applicant attributes the absence of the detection of shell modes to high damping resulting from the coupling of the RPV, the CB, and the HR through various water gaps. The applicant calculates the damping due to this mechanism using a methodology found in work by Gibert. Further, the applicant stated that the damping due to the water flow will provide additional damping to these modes. In the numerical model, however, the applicant has only applied a factor four lower viscous damping value to the shell modes; thus the model is more lightly damped than the experimental mockup and thus is capable of predicting the shell modes.

The staff concluded that the applicant has adequately addressed the differences between the wet and dry modal testing of the HYDRAVIB model and demonstrated that the high hydrodynamic damping prevented detection of the shell modes during wet modal testing. The response provides clarification that the results in CVAP Report, Section 4.2.2.3 meet RG 1.20. The staff considers RAI 422, Question 03.09.02-111 resolved.

SRP Section 3.9.2.3 recommends the use of small scale model tests and analyses to produce the full scale prediction of forcing functions and structural response. The applicant is using the one-eighth scale HYDRAVIB test results to compare to an FEA model of the HYDRAVIB mockup to benchmark their analytical techniques and forcing functions. However, in CVAP Report, Section 4.2.2.3, the applicant discussed the comparison of modal testing of the HYDRAVIB with the FEA prediction of the HYDRAVIB modes. CVAP Report, Section 4.2.3.2, Table 4-4 provides only the CB pendulum mode. All other modes were not determined. Since the ability to accurately predict the behavior of the scale model is used to benchmark the analytical/computational methods used to predict the response of the full scale structure, in RAI 422, Question 03.09.02-112, the staff requested that the applicant provide a discussion of the application of a frequency bias and uncertainty in the FEA predictions and the specifics about the locations, models, and reason for placement of transducers for the water filled modal testing to understand the inability of the applicant’s experimental model to predict modes other than the CB pendulum mode in the presence of water. **RAI 422, Question 03.09.02-112 is being tracked an open item.**

The applicant described the modal damping ratio for the HYDRAVIB model in CVAP Report, Section 4.2.3. The scale model was tested to determine the damping, but only the CB beam-type modes were acquired. The result of the CB beam-type mode determined a damping ratio greater than one percent, which is applied throughout the model for both beam-type modes and shell modes. RG 1.20 recommends justification of damping ratios greater than one percent. The staff could not locate justification for the use of such a high modal damping ratio for submerged components in a water environment or the FRFs used to compute the damping. Therefore, in RAI 422, Question 03.09.02-113, the staff requested that the applicant provide a detailed description of the testing that determined the damping ratio, including a discussion of the instrumentation, instrumentation locations and rationale for their placement.

In a July 7, 2011, response to RAI 422, Question 03.09.02-113, the applicant stated that for the CB pendulum mode, the damping ratio was obtained by applying the half-power bandwidth method to measurements from the HYDRAVIB test. The response of the shell modes at HYDRAVIB mockup conditions were insignificant, and the applicant stated that a more realistic value of the damping would be five times higher than the CB beam modes as justification for the damping value being conservative when applied to the shell modes. The staff finds that the applicant was justified in the use of the higher damping ratio (as compared to the one percent damping ratio recommended in the SRP). Further, the staff concluded that the damping ratio was justified through the analytical calculations of damping for the shell modes at mockup conditions stated in the response to RAI 422, Question 03.09.02-115 and discussed below. The staff considers RAI 422, Question 03.09.02-113 resolved.

CVAP Report, Section 4.2.1.2 includes descriptions of pressure transducer locations and the models of the transducers used for developing the turbulent pressure spectra and the input forcing functions in the FEA model. The staff concluded that the placement of the transducers in the HYDRAVIB model should be adequate to measure all of the important areas of turbulent flow forcing the lower internals. The applicant has placed transducers to measure axial and circumferential variations in the turbulent flow due to the flow entering the downcomer, as well as near the lower elevations of the CB away from the entrant jets. However, the applicant has provided only straight line, approximate curves to represent the measured pressure PSDs without indication of whether the provided curves are representative of the mean, the lower bound or the upper bound of the measured pressures for each of the locations. Further, the provided curves do not indicate if tonal content is present in the measurements or if the measured pressures have excursions above the spectral level (non-conservative) that can be seen in, for example, CVAP Report, Figure B-6, as well as in other measurements taken from literature. Therefore, in RAI 422, Question 03.09.02-114, the staff requested that the applicant provide additional detail describing each of the primary forcing locations, inlet jets, RV downcomer and lower support plate to determine if the scale model pressure measurements are acceptable. **RAI 422, Question 03.09.02-114 is being tracked as an open item.**

The applicant provided information on how small scale model and analytical model results are used, specifically with respect to the procedures used for the full scale structure in CVAP Report, Sections 4.2.2.3, 4.2.3.2, and 4.2.4. These sections include a comparison of the numerical and the experimentally derived modal solutions of the HYDRAVIB one-eighth scale model of the RV internals for non-operational modes in air (dry) and in water (wet). These comparisons were performed to verify modeling techniques for the FEM and forcing functions for application to the full scale RV lower internals. In CVAP Report, Section 4.2.2.3, the scale model modal numerical (i.e., FEA) results are considerably higher in frequency than the corresponding experimental results for dry HR modes. In CVAP Report, Section 4.2.2.3, the applicant stated that this indicates that the model of the HR is stiffer than the experimental structure and that no attempt was made to precisely match the experimentally obtained frequencies because these values are only representative of the HR mockup structure. The staff noted that the wet modal results, while matching experimentally determined CB pendulum modes well, have no corresponding comparison to any of the HR modes, because the modes were not identified in the experimental results. Since the comparison modes for the experiment and the FEA contain only one mode, in RAI 422, Question 03.09.02-115, the staff requested that the applicant provide the basis for the validation of the numerical solution.

In a July 7, 2011, response to RAI 422, Question 03.09.02-115, the applicant provided verification that the basis for the comparison of the numerical predictions and the experimental results for the RV lower internals is the CB pendulum mode. As a part of the rationale for using

only the CB pendulum mode for the basis, the applicant stated that the shell modes were not observed in the HYDRAVIB experimental results and that the numerical models employed low values of damping compared to the damping that the applicant calculates as being applicable. This use of lower value results in a conservative prediction of the response. The staff assessed the in air and in water modal prediction of the dominant mode of the HYDRAVIB to demonstrate the modeling methodology employed by the applicant, as well as justify that the applicant did not attempt to match the in air shell modes between experiment and FEA. Further, the wet modal testing indicates the damping applied in the HYDRAVIB model to be conservative. Lastly, the staff finds that the applicant has provided physical justification for the use of the CB pendulum mode as the basis for benchmarking the modeling procedure. Therefore, the staff considers RAI 422, Question 03.09.02-115 resolved.

The staff noted that data in CVAP Report, Section 4.2.2.3 indicates that a large frequency shift by more than a factor of 10 lower occurs from the dry to wet case modes for the HR. The modeling of the HR as a stiffer structure than the experimental model may affect the magnitude of this frequency shift when going from an air backed model to a water backed model. This large shift in modal frequencies between the wet and the dry may be indicative of an error in the modeling approach, one that could affect the full scale FEA accuracy. Therefore, in RAI 422, Question 03.09.02-117, the staff requested that the applicant: (1) Clarify how the lack of a basis for the water coupled HR/CB modes affects the conservatism of modeling procedures and on the conservatism of the full scale FEM; and (2) clarify the difference in the modeling of the HYDRAVIB HR and the modeling of the full-scale HR and why the full scale HR does not have the same higher stiffness indicated by the HYDRAVIB dry and wet modal responses discussed in CVAP Report, Section 4.2.2.3.

In a July 7, 2007, response to RAI 422, Question 03.09.02-117, the applicant stated that the HR is constructed of a series of stacked slabs and, because of this the modes are primarily shell modes. The coupled HR/CB modes are thus shell type in general, and the applicant states that the physical mockup of the HR is sufficiently accurate to characterize the modes and the corresponding behavior of the lower internals. The applicant stated that the rocking mode of the HR is a rigid body mode and thus all of the flexibility is from the shell of the CB. The CB shell is an influential part of the motion of the wet coupled HR/CB. The majority of the strain energy is stored in the shell of the CB. Thus, inaccuracy of the beam stiffness of the HR would not result in a significant change in the motion of the HR. The staff concluded from the discussion of the dynamics, that the HR responses are not strongly dependent on the beam stiffness and the differences between the model scale. Further, the staff concluded that differences between the beam stiffness of the HYDRAVIB model and full scale HR do not affect the conservatism of the prediction of dynamic behavior of the full scale RV lower internals. Therefore, the staff considers RAI 422, Question 03.09.02-117 resolved.

The staff notes that the conclusions discussed in CVAP Report, Section 4.2.4 do not agree with the results tabulated in CVAP Report, Table 4.4. Further, the response curves of CVAP Report, Figures 4.16 and 4.17 do not agree with the values listed in CVAP Report, Tables 4.3 or 4.4, but rather indicate frequencies closer to those listed in CVAP Report, Table 4.2. Therefore, in RAI 422, Question 03.09.02-119, the staff requested that the applicant reconcile the values used for conclusions reached in CVAP Report, Section 4.2.4 with those tabulated in the CVAP Report, Tables 4.2, 4.3, and 4.4, and CVAP Report, Figures 4.16 and 4.17. **RAI 422, Question 03.09.02-119 is being tracked as an open item.**

The staff concluded that results of the HYDRAVIB Mockup flow test in CVAP Report, Section 4.2.3.2 indicate that the dominant response of the lower internals is the global

pendulum mode. In RAI 422, Question 03.09.02-122, the staff requested that the applicant clarify how non-symmetric loading from one, two, and three RCPs in service affect the magnitude of the response and the implications on the full-scale RV lower internals.

In a July 7, 2011, response to RAI 422, Question 03.09.02-122, the applicant stated that the HYDRAVIB one-eighth scale mockup was operated using different combinations of pumps to create non-symmetric loading. In the HYDRAVIB, the highest beam mode response was consistently for four pump operation. The applicant further stated, when considering the broadband response, the largest response was from two pump operation where the two flow loops are oriented side by side. The applicant took the largest value of the LSP response for the two pump case and projected it to full scale with the result that the LSP still met the criteria for high cycle fatigue. The staff concluded that with the LSP meeting the high cycle fatigue criteria even for the unusual two pump case, and since the beam motion of the CB was highest with four pumps operating at full power condition, the applicant's use of the four pump case for operating projections is conservative. Therefore, the staff considers RAI 422, Question 03.09.02-122 resolved.

In CVAP Report, Section 4.2.1, the applicant stated that the HYDRAVIB scale model was designed to make an assessment of the lower internals vibrations induced by flow turbulences in the downcomer and the RV bottom head, and to identify other potential sources of FIV phenomena like vortex shedding (discrete frequency). The staff noted that the applicant only included an assessment of FIV due to turbulence and did not include an assessment of FIV from other sources. Therefore, in RAI 422, Question 03.09.02-123, the staff requested that the applicant provide an assessment of other sources of FIV that could be determined by the HYDRAVIB and the implications of the testing on the full scale RV internals. In a March 28, 2011, response to RAI 422, Question 03.09.02-123, the applicant stated that the purpose of the HYDRAVIB test was to assess the vibrations of the lower internals due to turbulence and vortex shedding. The applicant further stated that only turbulence excitation was observed; no vortex shedding occurred. Based on this clarification that only turbulence excitation was observed and the understanding that turbulence and vortex shedding are the most prevalent FIV phenomenon for the lower internals, the staff finds the response acceptable and, therefore, considers RAI 422, Question 03.09.02-123 resolved.

The applicant described the use of the PCRandom software in CVAP Report, Section 4.3.2 for the flow distribution device. The modal responses of the column supports were determined at full power operating temperatures and conditions using the applicant's CASS structural analysis program. The applicant stated that the limitations and accuracy verification against the classical closed form solutions are documented in the applicant's certification reports for the CASS software. The applicant employs AREVA computer code PCStab2 to perform the fluid-elastic instability analysis. These software codes are internally developed and maintained analysis software programs; therefore, in RAI 422, Question 03.09.02-124, the staff requested that the applicant verify that they have been used in a previously accepted NRC application.

In a February 17, 2011, response to RAI 422, Question 03.09.02-124, the applicant stated that none of these analysis programs have been used in a previously accepted NRC application. The applicant will update the FSAR Tier 2, Appendix 3C to include descriptions of PCRandom and PCStab2. The applicant further stated that the certification reports for the computer programs are available for NRC inspection and audit. With the inclusion of PCRandom and PCStab2 in the FSAR Tier 2, Appendix 3C, the computer codes will be reviewed separately in Section 3.9.1.2 of this report. **RAI 422, Question 03.09.02-124 is being tracked as a confirmatory item.**

In CVAP Report, Section 4.3.2.1, the applicant indicated that the damping for the FDD includes a one percent damping ratio for material properties and ignores the damping due to non-linearity of the bolted connections and viscosity of the fluid. The staff finds this value acceptable, because it is appropriate for this type of structure and is consistent with the SRP.

The staff noted that CVAP Report, Section 4.3.2.1 describes the development of the turbulent pressure forcing function that was the turbulent pressure spectrum prior to performing the HYDRAVIB model scale test. In reviewing the CVAP Report, the staff concluded that CVAP Report, Figure 4.28 demonstrates that the forcing function is conservative over the range of non-dimensional frequencies from 0.5 to 10. Comparing CVAP Report, Figure 4.28 to 4.29 based upon the inflection point at F=2 (25 Hz), indicates that the conservative range of the forcing function covers frequencies from about 6 Hz to 125 Hz. The staff noted that this appears to indicate that the lower frequency limit is acceptable based upon the first modal frequency of the FDD. Therefore, in RAI 422, Question 03.09.02-125, the staff requested that the applicant confirm this comparison and discuss the comparison of the HYDRAVIB and the estimated non-dimensional pressure spectrum below F=0.5. Further, the staff requested that the applicant address the non-conservative nature of the pressure forcing function PSD for the low frequencies where it is below the HYDRAVIB derived forcing function.

In a December 22, 2010, response to RAI 422, Question 03.09.02-125, the applicant clarified that the first mode of the FDD is above these frequencies and that there is sufficient conservatism in the rest of the analysis to compensate for any non-conservatism introduced. The applicant further stated (see the applicant’s March 24, 2011, response to RAI 422, Question 03.09.02-126 addressed below) that the analysis has been updated to use the experimentally derived HYDRAVIB forcing function. The staff concurs with the applicant’s clarification of the effect of the non-conservative region in the forcing function, because the first resonant frequency of the FDD is predicted to be above the non-conservative region in the forcing function curve and in the absence of a resonance in the frequency region, the forced response of the FDD will be lower than if it were at the resonance. With the conservatism the applicant has incorporated into the overall analysis, the effect of the non-conservative region in the forcing function on the overall response of the FDD has been shown by the applicant to be within the acceptance criteria of the FDD. The responses provide clarification that the CVAP Report meets RG 1.20. Therefore, the staff considers RAI 422, Question 03.09.02-125 resolved.

Based on the upper limit curve depicted in CVAP Report, Figure 4.29 and discussed in CVAP Report, Section 4.3.2.1, in RAI 422, Question 03.09.02-126, the staff requested that the applicant expand CVAP Report, Figure 4-28 to include the full range of frequencies as shown in CVAP Report, Figure 4.29 and covered by the analysis of the FDD so that the conservatism of the turbulent pressure spectrum can be assessed.

In a March 24, 2011, response to RAI 422, Question 03.09.02-126, the applicant supplied a chart showing the PSD for frequencies covering the full analysis band. The chart demonstrated that the PSD was non-conservative for frequencies above 155 Hz. The applicant has performed an analysis for the FDD using a turbulent pressure PSD developed during the HYDRAVIB test program for the lower plenum. This PSD is more conservative for frequencies below 5 Hz and above 155 Hz than the previously used “upper bound” curve. The applicant will update CVAP Report, Section 4.3.2, Figure 4-29, Section 4.3.4, and Table 4-13 through Table 4-16. The CVAP Report markup for this response details the results for the FDD using the PSD developed from HYDRAVIB testing. The analysis details remain the same for the turbulence-induced vibration analysis, except that the acoustic excitation analysis was carried out separately. In

reviewing the results, the staff notes that the change in the forcing function has resulted in a reduction of the margin to the criteria, from a factor of four to a factor of two. The staff finds that the response addressed the concern, because the final forcing function provided by the applicant and used in the analysis of the FDD response to turbulent flow excitation is conservative, and the resulting analysis shows the FDD response to fall below the acceptance criteria for turbulence excitation of the FDD. **RAI 422, Question 03.09.02-126 is being tracked as a confirmatory item.**

In CVAP Report, Section 4.3.2.1, the applicant provided an analysis of the FDD to the RCP shaft rate and blade rate acoustic pressures as a part of the response prediction for the FDD to turbulent flow. The staff was unable to locate where the applicant discussed the basis of the estimated RCP shaft rate and blade rate acoustic pressure amplitudes. Therefore, in RAI 422, Question 03.09.02-127, the staff requested that the applicant provide the basis for determining the RCP shaft rate and blade rate acoustic pressure amplitudes so that the conservatism of the levels can be ascertained.

In a July 7, 2011, response to RAI 422, Question 03.09.02-127, the applicant stated the analysis uses a conservative value for the RCP tones. The applicant further stated that there are as of yet no test data or analytical methods for determining the tone amplitudes for the RCP and that in the applicant's experience, the RCP tone amplitudes are dependent on the RCP, the installation, and piping system layout. The applicant plans to measure the tone amplitudes during the HFT and states that the instrumentation layout is located in CVAP Report, Section 5.0 and that corrective action will be taken if the acceptance criteria of CVAP Report, Section 5.5 are not met. Lastly, the applicant referenced their July 7, 2011, response to RAI 407, Question 03.09.02-76(e) for an updated FSAR Tier 1, Table 2.2.1-5, ITAAC 3.8 which requires the performance of an analysis of the effects of the RCP acoustic frequencies on the internals. The acceptance criteria for ITAAC 3.8 conclude that the internals stresses which result from the RCP tones are within the ASME code limits. The staff closed RAI 422, Question 03.09.02-127, because the applicant has performed an analysis based on the RCP acoustic pressure from the applicant's experience. Secondly, the applicant will measure the RCP acoustic pressure during HFT. Finally, the COL applicant will validate the reactor internals stress analysis using the measured RCP acoustic pressure as required by FSAR Tier 1, Table 2.2.1-5, ITAAC 3.8.

In CVAP Report, Section 4.3.2, the applicant used the estimated bandwidth of the RCP tone acoustic signal based upon the viscous damping of the fluid. The staff notes that the frequency of the shaft rate tone and its harmonics will be governed by the stability of the revolutions per minute (rpm) of the RCP. Variations in the RCP revolutions per minute will cause corresponding variations in the frequency of the shaft rate tone. A 10 percent RCP over speed will increase the shaft rate frequency of the RCP which results in the center frequency of the blade passing frequency (BPF) increasing to a frequency which is very close to the plate mode frequency of the FDD. The staff notes that the bandwidth of the RCP shaft rate harmonics used in the analysis by the applicant are not wide enough to cover these contingencies. Therefore, in RAI 422, Question 03.09.02-128, the staff requested that the applicant discuss how the analysis accounts for frequency bias and uncertainty in the modeling and in the forcing function which includes these potential effects, especially in light of the closeness of the BPF and the resonance frequency in the FDD.

In a December 22, 2010, response to RAI 422, Question 03.09.02-128, the applicant provided two analyses; one with the BPF at its nominal frequency and one with the BPF lining directly up with the FDD resonant frequency. The staff notes that the comparison of the resulting stresses

with the 60 effective full-power year (EFPY) cycle fatigue criteria for resonant frequency case shows that the FDD meets the criteria under this higher loading. Further, the staff notes that the applicant has affirmed their commitment to perform corrective actions if a resonant condition is detected during HFT that could endanger the performance and the structural integrity of the FDD. The staff determined the applicant's methodology will result in conservatively capturing the effects of the pump blade passing tones aligning with the nearby FDD resonances, the applicant has examined the effect of the BPF on the FDD at its nominal operating frequency, and the applicant has committed to performing measurements during HFT which would identify such a resonant response and to take actions if it should occur and the response is severe enough to cause damage. **RAI 422, Question 03.09.02-128 is being tracked as a confirmatory item until the CVAP Report is revised**.

SRP Section 3.9.2 recommends that the RPV upper internals be analyzed for vortex excitation, turbulence buffeting, and fluid elastic instability. The applicant detailed the analysis procedure for fluid-elastic instability in CVAP Report, Section 4.5.1.1.3. The applicant states that the fluid-elastic calculations will be performed for the CRGA, but fluid-elastic analysis was not performed for the normal columns, the LMP, and the instrumentation guide tube. Therefore, in RAI 422, Question 03.09.02-131, the staff requested that the applicant provide justification for not performing the fluid-elastic instability analysis for these components. In a July 29, 2011, response to RAI 422, Question 03.09.02-131, the applicant provided new analysis of the LMP, the normal columns, the instrumentation guide tube and the CRGA for turbulence buffeting, vortex shedding and fluid-elastic instability using the results of a three-dimensional CFD calculation of the thermal hydraulic conditions in the RPV upper internals. This information will be included in the revised CVAP Report. The staff finds that the updated calculations provided the analyses of the upper internals following the accepted analysis procedures. Further, these analyses indicate that the components of the upper internals will meet the criteria for high cycle fatigue (ASME Curve C, 19 (MPa) (2,800 psi), rms), fatigue stress limits for vortex shedding (93 Mpa (13,600 psi), 0-peak) and meet design criteria indicating that vortex shedding lock-in and fluid-elastic instability are unlikely to occur. **RAI 422, Question 03.09.02-131 is being tracked as a confirmatory item.**

The FIV analysis of the RPV upper internals reported by the applicant in CVAP Report, Section 4.5.3 utilized thermal hydraulic conditions determined from one-dimensional analysis. The results of this analysis indicate that several of the components would fail both the high cycle fatigue criteria (19 MPa (2,800 psi), rms) and the vortex shedding stress criteria (93 MPa (13,600 psi), 0-peak) as reported in the CVAP Report, Table 4-20 markup accompanying the July 29, 2011, response to RAI 422, Question 03.09.02-131. The applicant has performed a three-dimensional CFD analysis of the U.S. EPR and has used the results to update the thermal hydraulic conditions employed in the RPV upper internals FIV analysis. The applicant has stated in both CVAP Report, Section 4.5 and in the response to RAI 422, Question 03.09.02-131 that the CFD approach has been benchmarked against the ROMEO one-fifth scale flow testing, but the applicant has provided no information from that analysis. Further, the updated predictions substantially reduce the predicted stress for the RPV upper internals, in some cases by more than an order of magnitude, resulting in all of the upper internals meeting the stress criteria by wide margins. Therefore, in RAI 508, Question 03.09.02-169, the staff requested that the applicant provide the discussion of the CFD models and the ROMEO tests which address both the procedure used to validate the CFD model on a system reflecting the degree of complexity of the RPV upper internals and the sensitivity analysis performed to ensure that the grid size of the model is sufficiently small. **RAI 508, Question 03.09.02-169 is being tracked as an open item.**

In CVAP Report, Section 4.5.1.1.3, the applicant discussed the design criterion employed in the design of the upper internals to prevent the onset of fluid elastic instability. The applicant's design criterion is to ensure that the FSM is greater than 1.3. The staff concluded that the approach is acceptable, since fluid elastic instabilities are avoided for structures with velocities less than the critical velocity. However, the degree of certainty that fluid elastic instability has been avoided is a function of how well the thermal hydraulic conditions are known. The staff noted that in CVAP Report, Section 4.5.1.1.3, the applicant uses a factor of safety for this ratio of 30 percent to provide margin in the design to the critical velocity and the onset of fluid elastic instability. Therefore, in RAI 422, Question 03.09.02-132, the staff requested that the applicant discuss the ramifications of the 30 percent factor of safety present in the FSM criteria of greater than 1.3 in relation to the bias, errors and uncertainty inherent in the estimation of the pitch velocity.

In a December 22, 2010, response to RAI 422, Question 03.09.02-132, the applicant described the worst FSM (for the CRGA column support) calculated in the analysis and the factor of safety it represents. The staff concluded that a sufficiently large margin exists relative to the onset of fluid elastic instability to make the analysis acceptable. The applicant further refers to conservatism in the Connors' constant and the damping ratio as providing additional margin to the analysis. The staff concurs that conservative values of the Connors' constant were used in the analysis from reviewing CVAP Report, Section 4.5.1.1.3. Further, the applicant stated that the analysis is being updated to reflect the 3-D calculations of the thermal hydraulic conditions as discussed above in relation to RAI 422, Question 03.09.02-131. The CFD computational fluid dynamics computed thermal hydraulic density and flow velocity variations along the span of the CRGA were used by the applicant to compute a new FSM for the CRGA. The applicant's comparison of the velocity distribution to the assumed value used in the original calculation is given in Figure 03.09.02-132-1, attached to the response. The staff concludes that this velocity comparison for CRGA S6 appears to indicate that the original analysis is conservative and further that the FSM calculated from the CFD generated updated conditions would be larger than the previously disclosed value and thus less conservative, but the more realistic computation of the fluid conditions indicates fluid elastic instability is less likely. The applicant provided the new thermal conditions for the S6 location, and as this CRGA is located on the outer edge of the bundle on the centerline of one of the outlet pipes for the upper internals, the staff concurs that this is likely to be one of the most conservative locations to assess in terms of the magnitude of the flow velocities. The staff resolved RAI 422, Question 03.09.02-132, as acceptable because the applicant clarified that the much larger FSM was computed for the worst CRGA location, which then indicates a much larger margin to the onset of fluid elastic instability than the criteria value of 1.3. Further, the original analysis resulted in a smaller FSM, but still met the criteria. The additional margin indicates that fluid elastic instability is less likely to occur when a more accurate velocity / density is considered. Finally, FSAR Tier 1, Table 2.2.1-5, ITAAC 3.8 provides reasonable assurance that the updated analysis results will meet the FSM criteria.

The applicant detailed the analysis procedure for the upper plenum internals random turbulence induced vibrations in CVAP Report, Section 4.5.1.1.4. The methodology is standard and follows the work of Pettigrew and Gorman, which is referenced in the ASME Code, 2004. Further, the applicant discussed the mean square vibratory amplitude in terms of a single sided random force PSD. In RAI 422, Question 03.09.02-133, the staff requested that the applicant provide additional details of the derivation of this force PSD from the pressure PSD obtained from the random lift coefficient of CVAP Report, Figure 4-34.

In a December 22, 2010, response to RAI 422, Question 03.09.02-133, the applicant explained that the force PSD is computed from the product of the pressure PSD and the area the pressure is acting on. The area is determined as the sum of one half the area of each of the FEs adjacent the node the pressure is applied to. With this clarification, the staff concurs with this approach for computing the force from the product of the pressure and the area it's acting on as being acceptable and, therefore, considers RAI 422, Question 03.09.02-133 resolved.

In CVAP Report, Section 4.5.1.1.4, the staff notes that the applicant uses values for the correlation length which are typical for turbulent flow over structures of this type and an acceptable structural damping coefficient (one percent) for the instrumentation guide tube. In RAI 422, Question 03.09.02-135, the staff requested that the applicant provide additional details and discussion of the use of the correlation length in the analysis procedure and the calculation of the joint acceptance integral.

In a December 22, 2010, response to RAI 422, Question 03.09.02-135, the applicant verified that the cross diameter correlation length is the full diameter of the instrumentation guide tube and twice the instrumentation guide tube diameter for the spanwise (lengthwise) direction. The applicant has provided the details of the use of the correlation length in the applicant's procedure. The staff closed RAI 422 Question 03.09.02-135, because the verification of the joint acceptance integral and correlation length were provided. The staff finds that with the definitions used, the turbulence is correlated across the diameter and spanwise. Outside of these dimensions, the pressure PSD would become uncorrelated and the pressure is not considered. The applicant applies the technique using a two-dimensional FE formulation, and the discretized equations replace the double integrals. The formulation applies half of the node-to-node spacing to each node for computation of the applicable force at each node. The staff finds the approach in the CVAP Report standard and acceptable, and considers RAI 422, Question 03.09.02-135 resolved.

The internal components of the CRGA and which portions of the structure will be susceptible to FIV are described in CVAP Report, Section 4.6.3. In RAI 422, Question 03.09.02-136, the staff requested that the applicant provide a diagram of the CRGA showing the relationship of the various components, the position relative to the upper core plate (UCP), and the upper support plate (USP), as well as which components are exposed to the flow. Such a diagram is required to adequately assess the statements in the CVAP Report regarding which components are not evaluated for FIV.

In a July 7, 2011, response to RAI 422, Question 03.09.02-136, the applicant produced two figures showing the components of the CRGA, the relationship of the components to the UCP and USP, and documenting the flow speeds used in each area of the CRGA. In addition to the figures, the applicant documented the flow conditions through the CRGA in a table which will also be added to the CVAP Report. The staff considered that the figures and tables demonstrate that the main flange, the guide plates, and the RCCA spider are in areas of the upper internals where FIV excitation is unlikely. Further, the documentation of flow speeds throughout the CRGA indicates a degree of conservatism is present in the analysis because of the conservative velocities the applicant has chosen to employ in several sections of the CRGA. **RAI 422, Question 03.09.02-136 is being tracked as a confirmatory item** pending update of the CVAP Report.

In CVAP Report, Table 4-27, the applicant provides values for damping of the HR tie rod which exceed those recommended by RG 1.20. In RAI 458, Question 03.09.02-148, the staff requested that the applicant provide a discussion of the measurement methodology and the

measurements which support the high damping values tabulated in CVAP Report, Table 4-27 for the HR tie rod.

In a June 30, 2011, response to RAI 458, Question 03.09.02-148, the applicant stated that the damping values used for the HR tie rod were obtained from measurements. The tie rod was submerged and tested at two preloads. The tie rod was driven by an electrodynamic shaker and subjected to flow rates up to twice the design flow rate. Measurements were made by six accelerometers mounted along the length of the tie rod and the damping computed using the logarithmic decrement technique. The applicant will revise the CVAP Report to contain the equation for logarithmic decrement in the discussion of the tie rod damping measurement. The staff finds the approach of determining the damping of the tie rod to follow standard techniques. Further, the staff finds the values of damping reported in CVAP Report, Section 4.7, and the values of damping for the tie rod reported in CVAP Report, Table 4-27 acceptable. **RAI 458, Question 03.09.02-148 is being tracked as a confirmatory item.**

In CVAP Report, Section 4.6.3.1, the applicant discussed the damping applied to the CRGA internals. The discussion is unclear to the staff as to the total damping employed in the analysis. In RAI 458, Question 03.09.02-147, the staff requested that the applicant clarify the total damping employed for the CRGA analysis and further, the applicant is requested to discuss the damping mechanisms included in the total damping and the contribution of each mechanism to the total. In a July 26, 2011, response to RAI 458, Question 03.09.02-147, the applicant stated the damping values used in the analysis for the CRGA internals and will include the information in the revised CVAP Report. The staff considered the structural damping value of one percent structural used for predicting the response of the CRGA internals to random turbulence is typical for these types of structures. Further, the staff reviewed the analyses provided by the applicant that demonstrate that the CRGA internals are not susceptible to vortex shedding and concur that the methodology employed and damping values used are consistent with ASME Code Section III, Appendix N-1324.1. **RAI 458, Question 03.09.02-147 is being tracked as a confirmatory item.**

In FSAR Tier 2, Section 3.9.2.3, the applicant stated that the RPV internal vibrations are induced by flow turbulence, vortex shedding and leakage-flow induced vibration and acoustic sources such as the RCPs (high-frequency) and loop oscillation related low-frequency acoustics sources. In the lower RPV internals, the applicant stated the dominant sources to be turbulence (especially in the downcomer) and the low frequency acoustic excitation due to loop oscillations. In the upper internals, the applicant stated vortex shedding and fluid-elastic instability to be the potential sources of strong vibrations. The applicant stated that the RPV internals are evaluated for primary flow turbulence and fluid structure interaction vibration mechanisms at a full-power, steady-state normal operating condition. The staff noted that acoustic excitation and vortex shedding functions are not necessarily the largest amplitude at the highest flow rates.

In FSAR Tier 2, Section 3.9.2.4, the applicant stated that during HFT the internals are subjected to a total operating time at normal full-flow or higher flow conditions for a minimum of 240 hrs, including cyclic loading of greater than 10^6 vibration cycles on the structural elements of the RPV internals. The applicant also noted that the RPV internals vibratory response is mainly a function of flow conditions, and that large scale temperature transients could influence the vibratory response. The applicant stated that because plant heat up and cooldown transients are slow, the transients can be treated as a series of steady state conditions. The staff did not find how the applicant determined which operating condition is the bounding case for the RPV internals vibratory response. Therefore, in RAI 160, Question 03.09.02-30, the staff requested that the applicant justify why full-power, steady-state normal operating conditions are the most

conservative to examine and justify the selection of the temperature conditions relative to anticipated operational modes of the plant. The staff also requested that the applicant address any flow-excited acoustic and structural resonances or other self-excited response to vortex-induced vibration, turbulence and turbulence buffeting, flow separation, reattachment, and impinging flow instabilities.

In a January 23, 2009, response to RAI 160, Question 03.09.02-30, the applicant stated that from a high-cycle fatigue perspective, the full power steady state normal operating condition would be the most conservative condition to evaluate for the RPV internals, since this condition combines high flow rates and high temperature conditions over a long period of time. During plant heat up and cooldown transient conditions when different combinations of RCP operation are occurring, the response of the RPV internals to turbulence may exceed the full power steady state values but still be within the acceptable limits of base excitation. The relatively short duration of these operating configurations does not contribute significantly to the high cycle fatigue. In accordance with the guidance of RG 1.20, another RPV transient condition for consideration is a 10 percent over speed increase of the RCP. This momentary increase in reactor coolant flow also increases the response of the RPV internals but does not have a significant effect on the high cycle fatigue. The 10 percent over speed increase for the RCPs was considered in the FIV analysis of the upper internals.

The staff concurs that in terms of high cycle fatigue, the forcing functions are largest at the highest temperatures for the longest period of time at full-power steady state operating conditions. The applicant stated that because transient evaluation of the RPV lower internals to transient conditions will occur during HFT, no analytical evaluation of these transient conditions is planned. The staff position is that analysis and testing portions of the CVAP are intended to compliment, not supplant, each other. The staff closed RAI 160, Question 03.09.02-30, and in RAI 245, Question 03.09.02-49, the staff requested that the applicant provide justification for relying solely on the HFT to determine the safety of the plant response to transients and to explain in detail why transient analysis is not performed and the details of their plans to ensure that other adverse flow conditions are identified and mitigated.

In a December 3, 2009, response to RAI 245, Question 03.09.02-49, the applicant referred to CVAP Report, Sections 4.2.8 and 4.3.5 which indicate that a transient analysis is not warranted, because the primary loading for the lower internals is high cycle fatigue. The staff finds the applicant's approach to bounding the transient response of the lower internals by evaluating the RV internals to a 10 percent RCP over speed for 60 EFPY acceptable. The analysis is performed by considering the effect of increasing the coolant flow through the reactor by 10 percent and scaling the turbulent forcing functions on the increased dynamic pressure which results in an increase in the turbulent forcing functions of $(1.1)^2$. The staff notes that this scaling is based on the relationship between dynamic pressure and flow velocity and the linear response of the system. The applicant treats this response in a conservative fashion by comparing the response to the displacement and fatigue criteria and assuming a 100 percent duty cycle for the 60 EFPY of plant operation; in effect considering the short duration 10 percent increase in flow velocity as a steady state event occurring over the 60 years of the plant.

The description for the upper RPV internals in CVAP Report, Sections 4.5.4 and 4.6.6 states that the flow passage through the downcomer, the FDD, and the core should even out the flow. Therefore, the applicant concludes that the 10 percent over speed for four RCPs should be the maximum condition for turbulent excitation.

The staff has concluded that the applicant has demonstrated through a conservative estimation technique that the 10 percent RCP transient condition can be met by the RV internals. The applicant also demonstrated compliance to the steady state criteria for an estimated 10 percent RCP over speed condition for the components of the CRGA and RCCA structures in CVAP Report, Section 4.6.6.

In CVAP Report, Section 4.5.1, the applicant stated that the four RCP line-up 10 percent over speed condition may not be the highest transient. The applicant suggests that the two pump operation will result in higher response by the upper internals. It is unclear to the staff in CVAP Report, Sections 4.2.8 and 4.3.5, if the 10 percent over speed for all four pumps is believed to be the upper bound case for the RCP operations. Therefore, the staff closed RAI 245, Question 03.09.02-49, and in RAI 422, Question 03.09.02-137, the staff requested that the applicant discuss the higher response by the upper internals and clarify why the two RCP operation is not the bounding case to justify not performing an explicit transient analysis of the RV lower internals.

In a December 22, 2010, response to RAI 422, Question 03.09.02-137, the applicant stated that the use of four pumps operating at 110 percent of the full power condition for 60 EFPY is a more limiting case for high cycle fatigue than short duration transient operation of four or fewer pumps. The other pump line ups may produce higher levels of stress during the transient operation of one, two, or three pumps for short durations of time, but will have less of an effect on the high cycle fatigue of components than the case of four pumps operating at 110 percent. The staff considers RAI 422, Question 03.09.02-137, resolved because the fatigue damage from four pump operation at 110 percent for 60 EFPY is more limiting than that from transient operation of one, two, or three pumps for a short duration of time. Further, in the response to RAI 422, Question 03.09.02-122, the applicant discussed measurements from the HYDRAVIB test program that demonstrated that four pumps operating simultaneously was more conservative than the operation of any other line ups for the CB beam mode analysis.

In FSAR Tier 2, Section 3.9.2.3, the applicant stated that vibration of the RPV upper internals may result from vortex shedding and fluid-elastic instability. The instrument guide tubes (and therefore the upper internals) are subjected to turbulence from flow through the FA outlet nozzles. The staff noted that the described turbulent flow may also excite vibrations in other components and that flow excited structural and acoustic resonances do not necessarily occur at the highest flow rates. Lock-in may occur at any operating condition where the flow conditions coincide with structural or acoustic response and, once it occurs, can continue to persist over a range of conditions with resulting responses that may be far in excess of the response at the normal operating condition. Therefore, in RAI 160, Question 03.09.02-31, the staff requested that the applicant provide the FIV analyses for the mechanisms as indicated in SRP Section 3.9.2.

In a February 23, 2009, response to RAI 160, Question 03.09.02-31, the applicant described the analytical acceptance criteria employed with the FIV analysis of the RPV upper internal components. The criterion for preventing the onset of fluid-elastic instability in the upper internals is a FSM greater than 1.3. For random turbulence induced vibration, the applicant developed both a stress criterion for each component and a displacement criterion. The stress criterion is based on the fatigue curves found in the ASME Code, Section III Code. The displacement criterion is based on one-fifth of half the distance between the nearest two columns and represents a five sigma confidence that the displacement will not be exceeded. The applicant stated that the analysis and criteria for preventing vortex shedding lock-in follow the guidance of ASME Code, Section III, Appendix N-1300 to avoid a resonance

condition for an array of cylinders. The applicant stated the acceptance criteria for the off resonant response to vortex shedding of the support columns as the high cycle fatigue resulting from this FIV mechanism is acceptable, and the mid-span displacements of the support columns are small enough such that impact with adjacent support columns is avoided. The applicant stated that the CVAP final report will provide the results of the HFT and the final analytical results with documentation of any adjustments required to obtain agreement with the test results. The staff concurred with the criteria described above for the FIV analysis of the RPV upper internals and that the acceptance criteria for the random turbulence-induced vibration and for vortex-shedding induced vibrations were acceptable. Therefore, the staff considers RAI 160, Question 03.09.02-31 resolved.

The description of instrumentation for the CRGA is found in CVAP Report, Section 5.2.2.3 and is not intended to be a permanent installation. CVAP Report, Section 5.0 description of the set of instrumentation specified for determining the response of the CRGA column and internals is sufficient to obtain the upper bound response provided there are limited instrumentation failures during the HFT testing. A CRGA located at S6, on the outer periphery, near the center outlet jet port for loop 4 is described as located in a region anticipated to offer the highest cross flows. This CRGA column is instrumented with four strain gauges near the connection to the USP to measure the reaction of the CRGA column. If two strain gauges fail on opposite sides of the column, one of the principle directions will be lost. One of the guide plates, located at mid-span of the CRGA at S6, has been instrumented with two accelerometers, oriented to measure in the horizontal plane at 90 degrees to each other. The guide plate of a second CRGA, located at J9, in the center of the CSP, is instrumented with two radial accelerometers, oriented at 90 degrees to each other. The staff noted that in this configuration, if either accelerometer fails, one of the principal directions will be lost. Therefore, in RAI 422, Question 03.09.02-139, the staff requested that the applicant:

- Confirm that the instrumentation on the CRGA at S6 is intended to be oriented in and transverse to the mean flow direction.

- Discuss the impact of failing transducers and the potential negative impact to achieving the stated HFT goal of characterizing the behavior of the CRGAs and columns (see CVAP Report, Section 5.2.2.3).

- Discuss options or plans for increasing the number of transducers installed or replacing failed transducers during the test phase.

- Discuss the methodology, analysis, or rationale used for selecting the guide plate in these two CRGAs, one at the periphery and the second in the center.

In a June 24, 2011, response to RAI 422, Question 03.09.02-139, the applicant stated that to achieve greater redundancy, the strain gauges are oriented at 45 and 135 degrees to the flow field. The staff concluded that this is acceptable, because the strain gauges will be oriented at a known angle to the flow, that the applicant can resolve the load experienced by the CRGA into cross-flow and mean flow directions. Further, the staff concluded that orientation of the gauges at a known angle to the flow will provide the redundancy the applicant has stated, because this orientation will permit the applicant to determine the load in the CRGA in the cross–flow and mean flow directions through simple geometric relationships in the event that one or the other gauge fails. Lastly, the staff concludes that orientation of the gauges at 90 degrees to each other provides the applicant the ability to resolve the CRGA loading even if the flow is oriented in line with one of the gauges.

The applicant indicated that orientation of the transducers on the CRGA provides some degree of redundancy for failed transducers and that failure of CRGA instrumentation following the first eighteen tests identified in CVAP Report, Table 5.4 would not have a negative impact on the understanding of the behavior of the CRGA. Sufficient data to confirm the turbulent flow forcing function can be obtained at the 60 °C (140 °F) and 2,690 kPa (390 psia) test conditions. The staff finds this explanation acceptable, because the applicant has indicated which tests and conditions are necessary for characterizing the CRGA and because the applicant has incorporated redundancy into the instrumentation for these components.

The applicant stated that the number of transducers cannot be increased because of the limitations in the size of the RV head penetrations. The applicant focused any redundancy in instrumentation in characterizing the CB pendulum mode, since the applicant has assessed that mode as being the most important to the dynamics of the RPV lower internals. Further, the applicant indicated that if the testing objective is not accomplished, it may be necessary to suspend the HFT to either replace failed transducers or relocate the transducers at CRGA location J9 to S6. The staff accepts the discussion, because some degree of redundancy has been incorporated to capture the modes of the lower internals which analysis and model scale testing have indicated are most important. Further, the applicant indicated some of the conditions which could cause the HFT to be suspended while instrumentation is replaced or relocated.

The applicant stated that the CRGA guide plate modal characterization was performed separately from the HFT and that these tests provided the justification of the CRGA structural integrity to high cycle fatigue. The results of the HFT should be bounded by the tests to validate the CRGA design for the full scale thermal hydraulic conditions in the RV at normal and transient operating conditions. The staff accepts that the CRGA guide plate design was characterized in separate testing and accepts the applicant's explanation that the instrumentation placed on the guide plates for HFT is located to validate the guide plate design for the thermal hydraulic conditions that exist in full scale operation. The staff considers RAI 422, Question 03.09.02-139 resolved because the applicant has discussed the degree of redundancy which may be achieved with instrumentation for the HFT and has identified priorities in terms of the RV internal response characterization for allocating those redundant transducers. Further, the applicant has indicated contingencies that may result in suspension of the HFT due to the failure of instrumentation. The response provides clarification that the CVAP Report meets RG 1.20.

The staff notes that the applicant describes instrumentation for the pre-operational testing in CVAP Report, Sections 5.2 and 5.3 as stipulated by the SRP. These sections describe the instrumentation locations, types of instrumentation, and some of the phenomena each transducer is intended to measure. The RV internals are instrumented with accelerometers, displacement gauges, strain gauges, and dynamic pressure transducers. The instrumentation is selected to measure the turbulence pressures in the downcomer at three locations, the low frequency motion of both the core barrel and the HR, both the pendulum mode and the shell modes, and the behavior of the columns and CRGAs in the upper plenum. In examining the sensor suite, it is apparent to the staff that the transducer suite is sufficient to capture simple modes such as the pendulum mode directly. However, the transducer suite discussed appears to consist of a minimal set of transducers. Failure of only a few transducers will place the measurement of key responses at risk. In RAI 458, Question 03.09.02-153, the staff requested that the applicant provide additional details and specifically discuss the RG 1.20 recommendations regarding the sufficient number and placement of transducers to monitor significant lateral, vertical, and torsional structural motions resulting from shell, beam, and rigid

body vibrations of major RPV internal components, as well as being useful for confirming input forcing functions.

In a July 8, 2011, response to RAI 458, Question 03.09.02-153, the applicant stated that the HYDRAVIB testing demonstrated that the CB pendulum mode dominates the response of the lower internals and that the instrumentation planned will measure this mode. In CVAP Report, Section 5.5, the applicant stated that the CB pendulum mode has the largest impact on the fuel bundle response, and that the shell modes do not add significant stress to the CB or significantly excite the fuel bundle. For this reason, the instrumentation is focused on accurately capturing the CB pendulum motion. The staff accepts that the CB motion is predicted to dominate the response of the lower internals and that the applicant will focus the instrumentation plans on capturing this mode.

The applicant references the June 24, 2011, response to RAI 422, Question 03.09.02-139 to address plans for transducer replacement. The applicant states that there was not sufficient penetration to add any additional (and therefore no redundant) transducers. Further, the applicant stated that if the objectives of the testing outlined were not met, it may be necessary to suspend the HFT so that instrumentation could be replaced or relocated. The applicant stated that the priorities and contingencies cannot be determined until the failed transducers are identified. The staff considers RAI 458, Question 03.09.02-153, resolved because the staff concurs that the HFT effort should be focused on adequately capturing the modes that are predicted to dominate the response of the lower internals. The staff notes that CVAP Report, Section 4.2 states that the shell modes of the CB and HR are predicted to be significantly less important than the CB pendulum mode and, therefore, it is appropriate that the applicant focus on adequately characterizing the CB pendulum mode. Furthermore, the staff notes that in the above response, the applicant indicates that some instrumentation is dedicated to detecting the CB shell modes. The staff finds that the response provides clarification that the CVAP Report meets RG 1.20.

CVAP Report, Table 5.4 provides a series of pre-operational tests. These tests are specified in terms of number of operating pumps, temperature, and pressure. The staff was unable to locate the recommended description of hold points that meet the guidance in RG 1.20. Therefore, in RAI 422, Question 03.09.02-141 and RAI 458, Question 03.09.02-154, the staff requested that the applicant provide the information recommended in RG 1.20.

In a July 8, 2011, response to RAI 458, Question 03.09.02-154, the applicant provided the hold points and stated that the duration of the hold points cannot be determined at this stage in the design. The applicant has also referenced several tests from FSAR Tier 2, Section 14.2 which provide the activities during the test program to measure the response of the piping system, the RPV internals, and the vibration monitoring system. The staff reviewed the referenced tests and finds the tests provided objectives, methods, and acceptance criteria that are consistent with the test objectives. The applicant stated (consistent with RG 1.20) that the details of the acceptance criteria and actions to be taken if the acceptance criteria are not met will be provided at least 60 calendar days prior to the start of testing. The applicant also referenced the detailed December 22, 2010 response to RAI 422, Question 03.09.02-141 to address the points raised in RAI 458, Question 03.09.02-154. In FSAR Tier 2, Section 14.2.5, “Review, Evaluation, and Approval of Test Results,” the applicant provided the required discussion of the review, evaluation, approval of test results, and the communication with the NRC. The staff closed RAI 458, Question 03.09.02-154, as resolved because the staff concluded that information stated in RG 1.20 has been addressed in the CVAP Report.

CVAP Report, Section 6.0 states the inspection results of the RV and the RV internals will be considered acceptable if there is no indication of abnormally large vibration amplitudes or excessive wear. The staff noted that HFT is required to subject the RV internals to a minimum fatigue loading of 10^6 cycles. In CVAP Report, Section 6.1, the applicant stated that nondestructive testing is not planned for the inspection of the RV or the RV internals. The staff noted that RG 1.20 recommends that an applicant provide a description of the inspection procedure, including the method of examination, method of documentation, provisions for access to the reactor internals, and specialized equipment to be employed during the inspections to detect and quantify evidence of the effects of vibration. Therefore, in RAI 422, Question 03.09.02-140, the staff requested that the applicant clarify why nondestructive testing was excluded when it is known that visual inspections will not be sufficient to reveal cracks that might have initiated during the HFT.

In a July 29, 2011, response to RAI 422, Question 03.09.02-140, the applicant stated that the inspection plan for the pre-operational HFT will meet the guidelines and requirements of ASME Code, Section III, Paragraph NG-5111, 2004 and the methods defined in ASME Code, Section V, Article 9. The applicant stated the visual inspections VT-1 and VT-3 required by ASME Code, Section XI, Subsection IWB-2500, Table IWB-2500-1 for the examination categories of the RV and core support structures would be followed in order to meet RG 1.20. The acceptance criteria for these nondestructive surface examinations defined in ASME Code, Section XI, Subsection IWB-2500, Table IWB-2500-1 will be used to inspect the surfaces and welds of the components identified in CVAP Report, Table 6-1 through Table 6-5. Table 03.09.02-140-1 in the RAI response provides a summary of the visual inspection plan. The staff finds that the applicant has incorporated adequate inspection methods in their inspection plan after the HFTs to meet the recommendations of the RG 1.20. **RAI 422, Question 03.09.02-140 is being tracked as a confirmatory item** pending revision of the CVAP report.

In FSAR Tier 2, Section 3.9.2.4, the applicant stated representative trains of piping attached to the RCS, MSS and MFWS are monitored during startup testing. In RAI 160, Question 03.09.02-17, the staff requested that the applicant justify the use of representative trains instead of all lines encompassing the RCS in the assessment of flow-excited acoustic and structural resonances or other self-excited responses given that flow-excited acoustic and structural resonances are sensitive to small changes in the construction of even supposedly identical systems.

In a January 23, 2009, response to RAI 160, Question 03.09.02-17, the applicant stated that acoustic resonances, such as those caused by flow past a dead leg of closed relief valves, have the characteristic that the pressure-flow oscillations travel through the entire affected piping system with little attenuation. SRP Section 3.9.2 states that instabilities in these flow fields can couple with acoustic and structural resonance causing high dynamic loads throughout the steam system and RPV. Therefore, the vibrations from acoustic resonances are readily identifiable throughout an affected piping system. This is a basis for monitoring only representative trains of piping systems. In FSAR Tier 2, Section 3.9.2.4, the applicant described that the RCS, MSS, and MFWS are measured for vibration during initial start-up testing. The applicant stated that the MSS and MFWS will also be instrumented with permanent sensors during the operating life of the plant. The applicant stated that in accordance with RG 1.20, details of the vibration measurement, including use of the test results to compute loads on safety-related structures, will be included in the CVAP final report.

The staff was unable to determine how a resonance problem found during testing will be localized. The staff closed RAI 160, Question 03.09.02-17, as unresolved and in follow-up RAI 245, Question 03.09.02-43, the staff requested that the applicant clarify how a problem area will be localized so that corrective action can be taken. In addition, the staff noted that flow excited acoustic resonances are sensitive to small changes in plant construction and operating conditions. The applicant only indicated plans to test “representative” piping trains at full-power conditions in FSAR Tier 2, Section 3.9.2.4. This introduces a concern to the staff that, due to the acoustic resonances sensitivity to small changes, representative piping trains will be difficult to identify. Additionally, at conditions just below and just above that range of conditions where the lock-in occurs, local vibratory response can be high without system-wide coupling. The staff noted that measurement schemes that depend upon system-wide response to determine unacceptably high response may not detect locally high levels. In addition, the applicant was requested to describe plans for assuring that measuring representative piping systems will capture excessive vibration in the remaining piping systems.

In a July 10, 2009, response to RAI 245, Question 03.09.02-43, the applicant described the screening methodology for flow excited acoustic resonance in the design of the RCS and attached piping. To prevent the RCS from experiencing unacceptable dynamic loads, piping systems attached to the RCS will be designed so that the sources of flow excitation generated by cavities located in safety relief valves, standoff pipes for valves, and branch lines are not possible. The applicant described the evaluation of acoustic resonances caused by shear wave resonance of valve standpipes (i.e., dead leg of closed relief valve) in the RCS and piping systems attached to RCS components (i.e., components such as the RV, SGs, and pressurizer). The applicant will use a screening methodology in the design of the piping systems intended to preclude the possibility of the shear wave resonance from occurring. The applicant stated that an accepted method of predicting conditions conducive to acoustic resonance in piping systems and an explanation of these phenomena is presented by R. M. Baldwin and H. R. Simmons, “Flow-Induced Vibration in Safety Relief Valves,” ASME Journal of Pressure Vessel Technology, Volume 108/267, August 1986. As noted in the R. M. Baldwin and H. R. Simmons paper, flow past a standoff pipe or relief valves will separate near the leading edge forming a shear layer downstream. The applicant states that when the piping legs are designed, the Strouhal number in each part of the piping will not fall between 0.3 and 0.6. The applicant stated that final piping design of the MSS, MFWS, and the other piping systems attached to the RCS is not complete. Therefore, the evaluation of standoff pipes for the valves, standoff branch lines, or other cavities that have the potential to create acoustic resonance will be evaluated later in the design process for these piping systems.

The staff closed RAI 245, Question 03.09.02-43, and in RAI 331, Question 03.09.02-64, the staff requested that the applicant clarify how problem areas will be localized so that corrective action can be taken, or describe how measuring representative piping systems will determine that excessive vibration is not occurring in non-instrumented piping systems.

In a November 30, 2009, response to RAI 331, Question 03.09.02-64, the applicant stated that representative piping systems may be regarded as such only if they are all part of the same overall plant system (e.g., MSS), and have the same components and similar piping routing. To be considered representative piping systems, the main piping needs to be the same diameter and have essentially the same flow conditions. The staff agrees that if an acoustic resonance occurs on one line, the same is expected at nearly the same flow conditions in other representative lines. In the event that a resonant response is measured, the piping can be identified by determining the characteristic dimensions, for example diameter and length, required for the resonant frequency to occur. These characteristic parameters will be used to

identify the piping that is in resonance. The staff closed RAI 331, Question 03.09.02-64, and in RAI 422, Question 03.09.02-144, the staff requested that the applicant provide the criteria and/or metrics employed to determine when piping meets the "essentially the same" criteria used in determining distance to the upstream elbow, distance between standpipe and flow conditions, and clarify if elbows are required to have the same orientation relative to the branch line.

In a June 1, 2011, response to RAI 422, Question 03.09.02-144, the applicant stated that the orientations of the piping elbows are not important for the acoustic waves or the effects on acoustic reflection and so are not required for determining the similarity of piping systems. In contrast, the distance between piping fixtures, and the diameters of pipes are important for determination of similarity and must be within the tolerances specified for the construction drawings and pipe specifications. Further, the applicant stated that the piping leg flow conditions are considered "essentially the same" when the flow conditions are within two percent of the tolerances for the piping system. The staff accepts that the methodology outlined in CVAP Report, Appendix A together with the clarifications contained in the responses to RAIs related to this issue should result in the selection of appropriate piping trains. Therefore, the staff considers RAI 422, Question 03.09.02-144 resolved.

In FSAR Tier 2, Section 3.9.2.4, the applicant stated that there is a history of excessive vibration of standoff branch lines in the MSL when a plant uprates to higher power levels and, therefore, proposes to pay greater scrutiny to these piping configurations. The applicant stated that critical piping systems such as the MSS and MFWS will be permanently instrumented to monitor accelerations in each translational direction during the life of the plant. The staff noted that placement of accelerometers on the MSL is consistent with the SRP recommendation to determine the presence and magnitude of any flow excited acoustic resonance or vibration. The staff accepts that if the MSS and MFWS are permanently instrumented, then the MSS and MFWS should be capable of identifying acoustic resonances throughout the affected system. In RAI 160, Question 03.09.02-17, the staff requested that the applicant clarify how pressure fluctuations are to be measured and analyzed to determine loads on any safety-related or critical structures.

In a January 23, 2009, response to RAI 160, Question 03.09.02-17, the applicant stated that details of the vibration measurement, including use of the test results to compute loads on safety-related structures, will be included in the results from the CVAP Report, which is the responsibility of the COL applicant as noted in FSAR Tier 2, Table 1.8-2. The staff noted that a discussion of the planned pressure instrumentation and the plans for analyzing the pressures to compute loads are not dependent upon the results from the CVAP Report referenced in FSAR Tier 2, Table 1.8-2, COL Information Item 3.9-1. Therefore, the staff determined that the applicant is required to provide additional information to complete the review of how pressure fluctuations would be measured and analyzed. The staff closed RAI 160, Question 03.09.02-17, and in RAI 245, Question 03.09.02-44, the staff requested additional information on the measurement and analysis of pressure fluctuations.

In a July 10, 2009, response to RAI 245, Question 03.09.02-44, the applicant explained that during initial startup, strain gauges will be placed on the MSS and MFWS at certain stations to measure pressure oscillations. At each station, strain gauges will be placed around the pipe in a symmetric pattern, and each strain gauge will be oriented circumferentially to provide indication of pressure oscillations inside the piping. Distances between stations among piping lines in each system will be varied so that, if there is an acoustic resonance, it will not be possible for the distances between stations to have the same acoustic half-wavelength. The

two-microphone method will be used to develop the amplitude and phasing of the acoustics, as well as the frequency. This approach is expected to provide the required sensitivity to indicate acoustic resonance, should that occur. Acoustic resonance would be identified by a high Q factor peak in the power PSD at the frequency of resonance, with high coherence among strain gauge signals at that frequency. From this information, if acoustic resonance occurs, pressure oscillations observed at components would be computed for evaluation at the appropriate frequency.

The staff noted that the SRP recommends placing a minimum of four strain gauges equidistant around the pipe circumference to measure the hoop strain. The SRP recommends logarithmic spacing of a minimum of three measurement stations along the pipes. The staff closed RAI 245, Question 03.09.02-44, and in RAI 422, Question 03.09.02-84, the staff requested that the applicant provide a discussion of how this pressure measurement is performed, including the equations for the two microphone method and in RAI 422, Question 03.09.02-85, the staff requested that the applicant clarify the numbers of strain gauges and the circumferential spacing, demonstrate how SRP recommendations are met, discuss how the strain gauges will be calibrated for the pipe on which they will be mounted, and add the information to the CVAP Report. In a July 19, 2011, response to RAI 422, Question 03.09.02-84, the applicant referenced the response to RAI 422, Question 03.09.02-85. The applicant stated that they are employing methodology from two papers:

> "A Wave Decomposition Method for the Active Control of Sound Absorption: Theory and First Experimental Results," Marco Norambuena, Andre Jakob, Michael Moser, Institut fur Stromungsmechanik and Technische Akustik, Technische Universitat Berlin, Einsteinufer 25, D-10587

> "Error Analysis of Two-Microphones Measurements in Ducts with Flow," Mats Abom and Hans Boden Department of Technical Acoustic, Royal Institute of Technology, S-10044 Stockholm, Sweden

The staff considers RAI 422, Question 03.09.02-84, resolved because the applicant has provided the equations used to compute the pressure waves in the fluid from measurements of strain in the pipe walls together with the derivations necessary to verify correct employment of the method. Further, the applicant has included a correction to the calculation of the pressures from the measured strain in the pipe walls that is necessary to account for the effects of axial stresses on the measured hoop stress.

In an August 19, 2011, response to RAI 422, Question 03.09.02-85, the applicant stated that the method is based on hoop strain measurement using four strain gauges oriented at 90 degrees to each other around the circumference of the pipe. The applicant stated that there are two such measurement stations associated with each measurement. The staff considers the response to RAI 422, Question 03.09.02-85 acceptable because the applicant has described a placement of strain gauges that should enable the applicant to detect acoustic pressure waves propagating within the pipes and detect from the measurements the existence of acoustic resonant conditions within the RPV volume or the RSG volume attached to the pipe being measured. **RAI 422, Question 03.09.02-85 is being tracked as a confirmatory item** until the CVAP Report is revised.

The staff finds the responses to RAI 384, Question 03.09.02-68; RAI 407, Question 03.09.02-74; RAI 422, Questions 03.09.02-88, 03.09.02-96, 03.09.02-113, 03.09.02-115, 03.09.02-117, 03.09.02-122, 03.09.02-123, and 03.09.02-144; and RAI 458, Question 03.09.02-150 are acceptable. However, in RAI 522, Question 03.09.02-170, the staff

requested that the applicant include a summary of the responses in the CVAP Report or the FSAR. **RAI 522, Question 03.09.02-170 is being tracked as an open item.**

3.9.2.4.5 Dynamic Analysis of Reactor Internals under Faulted Condition

In FSAR Tier 2, Section 3.9.2.5, "Dynamic System Analysis of the Reactor Internals Under Faulted Conditions," the applicant stated that an analysis of the reactor internals has been performed that considers the effects of SSE and blowdown loads resulting from guillotine breaks in safety injection line nozzles on the hot and cold leg piping. The applicant stated that breaks are not considered in the main coolant loop piping (RCS cross, hot, and cold legs), pressurizer surge line, and MSL piping (from the SGs to the first anchor point location) due to the application of leak before break methodology to these lines, as described in FSAR Tier 2, Section 3.6.3. The staff concurs that postulated blowdown loads from these locations are not required in the dynamic system analysis methods and procedures used to confirm the structural design adequacy of the reactor internals under a simultaneous occurrence of a LOCA and an SSE.

The staff reviewed the information presented in FSAR Tier 2, Section 3.9.2.5 and FSAR Tier 2, Appendix 3C. SRP Section 3.9.2.III.5 recommends that the staff review the detailed information on whether adequate analysis has been made of the reactor internal structures to withstand dynamic loads from the most severe LOCA in combination with the SSE. Therefore, in RAI 467, Question 03.09.02-155, the staff requested that the applicant clarify the analytical methods to determine stability of the CB and other compressive elements of reactor internals subject to combined SSE and LOCA loads. In a July 29, 2011, response to RAI 467, Question 03.09.02-155, the applicant clarified that the CB and other core support structures will be analyzed (including buckling analysis) to meet the requirements of ASME Code, Section III, Subsection NG-3000. FSAR Tier 1, Table 2.2.1-5, ITAAC 3.16 requires that the reactor internals are designed in accordance with the ASME Code. The staff finds that conducting the buckling analysis according to ASME Code NG-3000 is acceptable. The ITAAC provides reasonable assurance that the reactor internals will be designed to withstand the combined SSE and LOCA loads. Therefore, the staff considers RAI 467, Question 03.09.02-155 resolved.

The staff reviewed FSAR Tier 2, Appendix 3C.1.1, "Reactor Coolant System Four Loop Hydraulic Model," and FSAR Tier 2, Appendix 3C.1.2, "Steam Generator Secondary Side Hydraulic Model," where the applicant stated that the "SG model is based on zero percent tube plugging." In RAI 467, Question 03.09.02-157, the staff requested that the applicant clarify why zero percent tube plugging is conservative over the life of the plant. In an August 5, 2011, response to RAI 467, Question 03.09.02-157, the applicant clarified that the maximum pressure in the hot leg is controlled and maintained constant by the pressurizer. This pressure is maintained constant throughout the life of the plant, irrespective of degree of tube plugging. Tube plugging increases the pressure drops across the SG and decreases the mass flow. The staff reviewed the clarification and finds the evaluation of the pipe break event with zero percent tube plugging, which yield the highest system pressures and maximum mass flow rates, conservative and acceptable. Therefore, the staff considers RAI 467, Question 03.09.02-157 resolved.

FSAR Tier 2, Appendix 3C, Section 3C.2.1.1 states that the RPV internals and fuel mass are lumped at the appropriate center of gravity locations. In RAI 467, Question 03.09.02-161, the staff stated:

- The fuel mass consists of a very large number of loose fuel pellets, constituting the active fuel length inside the fuel rod. Due to the high density of the fuel pellets, the fuel pellets contribute the most significant mass to the FA. The staff requested that the applicant clarify that under combined SSE and LOCA loads, the fuel pellets will behave dynamically like a rigid single lumped mass, instead of multiple masses.

- The FAs themselves are supported at the bottom by the bottom nozzle, while the top end is clamped down by the hold down spring. The staff requested that the applicant clarify that with the presence of the hold down spring, the FA will behave like a rigid single lumped mass, instead of preloaded multiple masses, under combined SSE and LOCA loads.

RAI 467, Question 03.09.02-161 is being tracked as an open item.

FSAR Tier 2, Appendix 3C, Figure 3C-8 illustrates the modeling of the RPV internals. Physically, the CB is supported in the spigot of the reactor vessel shell at the CB flange. The flange is supported at its bottom surface by the reactor vessel spigot, while on the top surface the CB flange is loaded by the hold down spring. Further, the flange of the USP rests on the hold down spring. The staff determined that the applicant has not provided details of modeling of the hold down spring. This is to determine the upward and rotational (roll and pitch) stability about the two horizontal axes for the CB and USP assembly under depressurization pressure loads and SSE acceleration loads. Therefore, in RAI 467, Question 03.09.02-162, the staff requested that the applicant provide additional details of modeling of the hold down spring, particularly related to the possibilities of CB flange and the upper support flange partially (due to rotation about the two horizontal axes) or fully separating from their contacting surfaces, or the spring.

In a July 29, 2011, response to RAI 467, Question 03.09.02-162, the applicant clarified the FE structural model of the RPV and its internal structures, includes springs representing the local stiffness of the CB flange, upper USP flange and hold-down spring assembly, in the vertical direction, and as well, about the two horizontal axes (roll and pitch direction). Further, in the structural model, rotation of the CB about the horizontal axes is supported by a rotational spring at the CB flange in addition to lateral springs between the CB and the RPV. The lateral springs at the LSP provide additional rotational support as they act at a distance from the flange/RPV joint. Rotation of the USP about the horizontal axes is restrained by the rotational spring at the USP flange. The rotations of the RPV internals about the two horizontal axes occur primarily due to lateral seismic accelerations. The dynamic analysis is performed up to and beyond the hold down spring going solid. The applicant stated that analysis has shown that due to the preload on the spring, separation of the joint does not occur.

The staff reviewed the additional details in the response and finds that the applicant has fully clarified modeling of the hold down spring and, consequently, the results obtained from full seismic analysis showing that separation of the component flange contacting faces does not occur, and, that the modeling of the RPV internals is acceptable. The response provides clarification that the FSAR meets the SRP. Therefore, the staff considers RAI 467, Question 03.09.02-162 resolved.

In FSAR Tier 2, Section 3.9.2, the applicant stated that the forcing functions obtained from hydraulic analysis of the safety injection line breaks are defined at points in the RPV internals where changes in cross-section or direction of flow occur such that differential loads are generated during the blowdown transient. Additional details of the structural analysis of the

RPV isolated model for LOCA loading are given in FSAR Tier 2, Appendix 3C. The staff reviewed FSAR Tier 2, Appendix 3C.2.2, "Reactor Pressure Vessel Isolated Structural Model," and determined that the RPV isolated structural model consists of representations of the RPV pressure boundary, control rod drive mechanism (CRDM), CRDM nozzles, closure head equipment (CHE), lower internals, upper internals, and FAs. The staff determined that the list may not include all safety-related RPV internals. Therefore, in RAI 503, Question 03.09.02-168, the staff requested that the applicant: (1) List and clarify all components that the applicant has included in their definition of reactor internals; (2) determine whether the list above includes all components within the reactor vessel or any components have been excluded; and (3) verify whether the developed forcing functions, the analysis, and the interpretation of results by the applicant's thermal-hydraulic modeling and analysis correctly determine the necessary dynamic parameters to confirm the structural design adequacy and the ability to perform the function of all RPV internal components. The staff also requested that the applicant include this information in the FSAR. **RAI 503, Question 03.09.02-168 is being tracked as an open item**.

3.9.2.4.6 Correlation of Reactor Internal Vibration Tests

The staff reviewed FSAR Tier 2, Section 3.9.2.6, "Correlations of Reactor Internals Vibration Tests with the Analytical Results," and the CVAP Report, to determine if the applicant described the method to be used for correlating the results from the reactor internals pre-operational vibration test with the analytical results derived from dynamic analyses of reactor internals under operational flow transients and steady-state conditions. In reviewing CVAP Report, Sections 5.4 and 5.5, the staff concluded that the results of the vibration testing will be screened to determine if the responses are in the acceptable limits. The staff noted, however, that the applicant did not define the acceptable limits. The applicant stated that the natural frequencies will be determined via spectrum analysis for comparisons to the pre-operational CVAP. However, the staff noted that the applicant did not clarify what constitutes acceptable agreement between the measured and predicted frequencies nor explain how the appropriate node within the analytical model is determined for comparison with the appropriate measurement location from the HFT. In CVAP Report, Section 5.4, the applicant discussed how bias and random errors from transducers and the measurement system affect the results. Prior to the HFT, the total instrument error is calculated using a root sum of squares method. The staff recognizes that the accuracy of the transducers will not be known until they are selected; however, the required accuracy for both the transducers in the RV and on the piping systems should be included in the CVAP Report. Therefore, in RAI 422, Question 03.09.02-141, the staff requested that the applicant provide explanations to address the concerns.

In a December 22, 2010, response to RAI 422, Question 03.09.02-141, the applicant stated that the general acceptance criterion for a match between experimentally measured frequencies and analytically determined frequencies is 10 percent. The applicant further stated that the difference for some RPV components whose behavior is critical to the vibratory behavior of the internals will be more stringent than 10 percent. The applicant provided the pendulum mode of the CB as an example of a mode that must agree to better than 10 percent between HFT experimental results and analytical results for acceptance. The applicant stated that, if necessary, the analysis will be modified with forcing functions measured during the HFT to obtain better agreement between the HFT and the analysis. The applicant reiterated statements from CVAP Report, Section 5.4.4 concerning the combination of the HFT errors from sensors, signal conditioning, and data acquisition using the "square root sum of squares" methodology. The staff finds the explanation of acceptable agreement between frequencies of 10 percent (or better) acceptable. Further, the staff notes that modification of the analysis with HFT measured forcing functions, if necessary, is consistent with the SRP. The staff notes that

the information provided is consistent with RG 1.20 and that more detailed information will be provided with the CVAP prior to the HFT. Therefore, the staff considers RAI 422, Question 03.09.02-141 resolved.

3.9.2.5 *Combined License Information Items*

Table 3.9.2-1 provides a list of dynamic testing and analysis of systems, components, and equipment related COL information item numbers and descriptions from FSAR Tier 2, Table 1.8-2:

Table 3.9.2-1 U.S. EPR Combined License Information Items

Item No.	Description	FSAR Tier 2 Section
3.9-1	A COL applicant that references the U.S. EPR design certification will submit the results from the vibration assessment program for the U.S. EPR RPV internals and piping systems specified in U.S. EPR FSAR Tier 2, Section 3.9.2.1, in accordance with RG 1.20.	3.9.2.4

The staff finds the above listing to be complete. Also, the list adequately describes actions necessary for the COL applicant. No additional COL information items need to be included in FSAR Tier 2, Table 1.8-2 for dynamic testing and analysis of systems, components, and equipment consideration.

3.9.2.6 *Conclusions*

Except for the confirmatory item discussed in Section 3.9.2.4.1 of this report, the staff concludes that the pre-operational testing, initial fuel loading, and pre-critical testing to be performed to verify that vibration and thermal expansion and contraction of piping systems will meet the relevant requirements of 10 CFR Part 50, Appendix A, GDC 14 and GDC 15 for the design and testing of the reactor coolant pressure boundary to ensure that a low probability of rapidly propagating failure and gross rupture will not occur and to ensure that the design conditions are not exceeded.

The staff concludes that the U.S. EPR seismic analysis and design methodology meets the relevant requirements of 10 CFR Part 50, Appendix A, GDC 2 for demonstrating design adequacy of all Seismic Category I systems, components, equipment, and their supports to withstand an earthquake.

Except for the open items discussed in Sections 3.9.2.4.3 and 3.9.2.4.4 of this report, the staff concludes that the U.S. EPR reactor internals design meets the requirements of 10 CFR Part 50, Appendix A, GDC 1 and GDC 4 for design and testing of reactor internals with the potential to generate loose parts to quality standards commensurate with the importance of the safety functions performed with appropriate protection against dynamic effects, including FIV and acoustic resonance. The staff also concludes that the comprehensive vibration assessment program for the prototype reactor, in accordance with the regulatory positions of RG 1.20, provides an acceptable basis for design adequacy of the reactor internals under test loading conditions comparable to those experienced during operation. The integrity of the reactor internals in service is essential to proper positioning of reactor fuel assemblies and

unimpaired operation of the control rod assemblies for safe reactor operation and shutdown. The combination of tests, predictive analysis, and post-test inspection provide adequate assurance that the reactor internals will, during their service lifetime, withstand the FIV and acoustic resonance of reactor operation without loss of structural integrity.

Except for the open items discussed in Section 3.9.2.4.5 of this report, the staff concludes that the dynamic system analyses have been performed to confirm that the structural design of the reactor internals is able to withstand the dynamic loadings of the most severe LOCA in combination with the SSE, with no loss of function. The staff also concludes that the methods and procedures for dynamic systems analyses, the considerations in defining the mathematical models, the descriptions of the forcing functions and the acceptance criteria, and the interpretation of the analytical results are in compliance with the relevant requirements of 10 CFR Part 50, Appendix A, GDC 2 and GDC 4.

The staff concludes that the applicant meets the relevant requirements of 10 CFR Part 50, Appendix A, GDC 1 with regard to the internals of a prototype reactor being tested to quality standards commensurate with the importance of the safety functions being performed by the proposed program to correlate the test measurements with the analysis results. The staff also concludes that the program provides an acceptable basis for demonstrating the compatibility of the results from tests and analyses, the consistency among mathematical models used for different loadings, and the validity of the interpretation of the test and analysis results.

3.9.3 ASME Code Class 1, 2, and 3 Components, Component Supports, and Core Support Structures

3.9.4 Control Rod Drive System

3.9.5 Reactor Pressure Vessel Internals

3.9.5.1 *Introduction*

This section verifies that the FSAR describes the arrangement of the reactor pressure vessel (RPV) internals, their specific functions, the flow path through the reactor vessel, and the applicant’s design criteria. The RPV internals serve several functions. They provide support and alignment for the reactor core, provide a flow path that directs and distributes the flow of reactor coolant through the nuclear fuel under all design conditions, and shield the RPV from neutron impingement.

The objectives of the staff’s review are to confirm the following:

- The RPV internals have been designed and tested to appropriate quality standards.
- The portions of the internals that provide structural support for the core meet the applicable requirements of ASME B&PV Code, Section III.
- The appropriate design transients and loading combinations have been specified.
- The internals mechanical stresses, deflections and deformations will not result in a loss of structural integrity or impairment of function.

3.9.5.2 *Summary of Application*

FSAR Tier 1/ITAAC: The system description and ITAAC for the reactor internals are discussed in FSAR Tier 1, Section 2.2.

FSAR Tier 2: FSAR Tier 2 describes the arrangement of the RPV internals and the flowpath of reactor coolant through the reactor vessel, including core bypass flows, from the point where the coolant enters the RPV through the cold leg nozzles and exits through the hot leg nozzles. The RPV internals are divided into two main categories: The lower internals and the upper internals. The lower internals function to vertically support the reactor core and the heavy reflector that functions as the neutron shield and to align the nuclear fuel bundles. The function of shielding the RPV from neutron impingement is performed by the heavy reflector. As the name implies, this component functions to reflect neutrons back into the reactor core, which thereby increases neutron economy and minimizes the effects of neutron-induced embrittlement on the RPV. The lower internals also direct and distribute the reactor coolant flow through the core. The principle components that comprise the reactor vessel lower internals are the core barrel, lower core support plate, and flow distribution device. The upper internals function to provide support and alignment of the fuel assemblies and the RCCAs. The upper internals also direct the core flow exiting the reactor core to the RPV outlet nozzles. The principle components of the upper RPV internals are the upper support plate, upper core plate, control rod guide assembly (CRGA) columns, and other columns, to transfer the flow-induced core uplift load to the RPV closure head.

- The applicant states that the RPV internals classified as performing a core support function are designed, analyzed, fabricated, and inspected in accordance with the requirements of Section III of the ASME B&PV Code, Division 1, Subsection NG. The applicant further states that the remaining RPV internals, not classified as core support structures, are designed and constructed to not adversely affect the core support structures as required by ASME Code, Section III, Subparagraph NG-1122(c). The applicant also states that the RPV internals meet the requirements of 10 CFR Part 50, Appendix A, GDC 10, “Reactor Design,” such that specified acceptable fuel design limits are maintained during normal operation and anticipated abnormal occurrences. It is also stated in the application that environmental effects and fatigue (cyclic deflection and loads) have been considered in the design of the RPV internals, as have flow-induced vibration (FIV) and acoustic resonance.

3.9.5.3 *Regulatory Basis*

The relevant requirements of NRC regulations for this area of review, and the associated acceptance criteria, are given in NUREG-0800, Section 3.9.5, “Reactor Pressure Vessel Internals,” and are summarized below. NUREG-0800, Section 3.9.5 also identifies review interfaces with other SRP sections.

1. 10 CFR Part 50, Appendix A, GDC 1, “Quality Standards and Records,” 10 CFR Part 52. “Licenses, Certifications, and Approvals for Nuclear Power Plants,” and 10 CFR 50.55a, “Codes and Standards,” as they relate to the requirement that components important to safety be designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety function to be performed.

2. 10 CFR Part 50, Appendix A, GDC 2, “Design Bases for Protection Against Natural Phenomena,” as it relates to the requirement that components important to safety be

designed to withstand the effects of natural phenomena, such as earthquakes, without loss of capability to perform their safety function.

3. 10 CFR Part 50, Appendix A, GDC 4, "Environmental and Dynamic Effects Design Bases," as it relates to the requirement that components important to safety be designed to accommodate the effects of, and be compatible with, the environmental conditions associated with normal operation, maintenance, testing, and postulated accidents, including LOCAs.

4. 10 CFR Part 50, Appendix A, GDC 10, "Reactor Design," as it relates to the requirement that the reactor core and associated coolant, control, and protection systems shall be designed with appropriate margin to assure that specified acceptable fuel design limits are not exceeded during any condition of normal operation, including the effects of anticipated operational occurrences.

Acceptance criteria adequate to meet the above requirements include:

- ASME B&PV Code, Section III, Paragraph NG-3000 contains guidelines that should be met for the design criteria, loading conditions, and analyses that provide the bases for the design of reactor internals other than the core support structures. Reactor internals other than core support structures should be constructed such that the integrity of the core support structures is not adversely affected, as specified in ASME B&PV Code, subparagraph NG-1122.

3.9.5.4 *Technical Evaluation*

In accordance with review guidelines in SRP Section 3.9.5, the term, "reactor internals," used in this section includes core support structures (CSS) and other internal structures (IS), and refers to all structural and mechanical elements inside the RPV, except the following:

- Reactor fuel elements and the reactivity control elements up to the coupling interface with the drive units, except the structural aspects of the reactor fuel assemblies

- Control rod drive elements, except the guide tubes

- In-core thermocouple instrumentation, except the in-core and thermocouple instrumentation supports, including the in-flux detector support

The staff reviewed the following six specific areas: (1) Codes and standards; (2) IITAAC; (3) reactor internals physical or design arrangements; (4) design-basis loading conditions for all service conditions; (5) design bases for the mechanical design of the reactor internals; and (6) potential adverse flow effects on RPV internals.

3.9.5.4.1 Codes and Standards

To comply with 10 CFR Part 50.55a requirements for codes and standards, the applicant has given the RCS mechanical design features, including the industry code to be used for design and testing in FSAR Tier 1, Table 2.2.1-1, "RCS Equipment Mechanical Design (9 Sheets)."

A review of FSAR Tier 1, Table 2.2.1-1 by the staff indicates that the RPV is classified as ASME Code, Section III, Class 1 component. However, FSAR Tier 1, Table 2.2.1-1 does not specify or give code requirements for RPV CSS. ASME B&PV Code, incorporated by reference in

10 CFR 50.55a, provides specific requirements for code construction of CSS which are different and separate from the requirements for the RPV. Therefore, in RAI 149, Question 03.09.05-1, the staff requested that the applicant add RPV CSS along with construction requirements as a separate item in FSAR Tier 1, Table 2.2.1-1. In a January 29, 2009, response to RAI 149, Question 03.09.05-1, the applicant stated that FSAR Tier 1, Section 2.2.1, "Reactor Coolant System," will be revised to add the RPV internals classified as ASME Code, Section III to FSAR Tier 1, Table 2.2.1-1. The staff finds the addition of the RPV/CSS to FSAR Tier 1, Table 2.2.1-1 acceptable, because the ASME B&PV Code has specific requirements for RPV CSS, which meet regulatory requirements. Additionally, the staff verified the proposed changes were incorporated in Revision 2 of the U.S. EPR FSAR; therefore, the staff considers RAI 149, Question 03.09.05-1 resolved.

The RPV internal structure components stated in FSAR Tier 2, Table 3.2.2-1, "Classification Summary," are classified as safety classification "NS-AQ," quality group (QG) Classification "D." The "Comments/Commercial Code" column identifies these same components as ASME Code, Class CS, which implies that their basis for construction complies with ASME Code, Section III, Subsection NG rules. Therefore, in RAI 149, Question 03.09.05-5, the staff requested that the applicant clarify the extent to which these non-safety, QG D classified components are designed and constructed to the requirements of ASME Code, Section III, Subsection NG.

In a January 29, 2009, response to RAI 149, Question 03.09.05-5, the applicant stated that the components classified as core supports are designed and constructed in accordance with the requirements in ASME Code, Section III, Subsection NG. The components classified as internal structures are designed such that they meet the requirements of ASME Code Subsections NG-3000 and NG-1122(c). Additionally, the applicant stated that FSAR Tier 2, Section 3.9.5.2, "Loading Conditions," would be revised as described in the response. The staff finds that the IS designed to the requirements of ASME Code, Section III, Subsection NG-3000 is an acceptable approach to meeting the intent of ASME Code, Section III, Subsection NG-1122(c). The staff verified the proposed changes were incorporated in FSAR Revision 2 and, therefore, considers RAI 149, Question 03.09.05-5 resolved.

3.9.5.4.2 ITAAC

The ITAAC for the RCS components are given in FSAR Tier 1, Table 2.2.1-5, "RCS ITAAC." Item 3.1 in this table is written to address ASME Code design or pressure boundary components. CSS are not pressure boundary components but do require design and construction in accordance with ASME Code, Section III, Subsection NG. Therefore, in RAI 149, Question 03.09.05-2, the staff requested that the applicant provide a separate ITAAC appropriate for a non-pressure boundary, safety-related CSS be added to FSAR Tier 1, Table 2.2.1-5.

In a January 29, 2009, response to RAI 149, Question 03.09.05-2, the applicant stated that ITAAC for RPV CSS will be added to FSAR Tier 1, Section 2.2.1. The staff verified that the applicant added ITAAC for reactor internals components in FSAR Tier 1, Table 2.2.1-5, Revision 1 of the U.S. EPR FSAR. Since the ITAAC to inspect the certified ASME stress report will ensure that the calculated stresses meet code allowables, the staff finds this approach acceptable and considers RAI 149, Question 03.09.05-2 resolved.

The RPV internals ITAAC criteria in FSAR Tier 1, Table 2.2.1-5 state that the RPV internals can withstand the effects of FIV. The staff determined this is an inappropriate acceptance criterion, because it does not provide a basis for documenting that the design commitment has been met. Therefore, in RAI 149, Question 03.09.05-3, the staff requested that the applicant revise the

acceptance criteria such that it will provide for a comparison of documented results to any established standard and can be used for inspection verification. In a January 29, 2009, response to RAI 149, Question 03.09.05-3, the applicant revised FSAR Tier 1 Section 2.2.1, acceptance criteria to include ASME Code stress limits and to discover no loose parts in inspections. The staff finds this response acceptable, because the ITAAC was revised to provide an appropriate acceptance criterion for documenting the design commitment. The staff confirmed that Revision 2 of the FSAR contains the changes committed to in the RAI response. Accordingly, the staff finds that the applicant has adequately addressed this issue and, therefore, the staff considers RAI 149, Question 03.09.05-3 resolved.

The design commitment and acceptance criteria in FSAR Tier 1, ITAAC, Table 2.2.1-5 for CSS (Item 3.15), are exactly the same. The staff review finds that this is an unacceptable ITAAC, because it does not identify a specific design commitment, does not provide a basis for documenting that the design commitment has been met, and does not provide any means for construction verification. Therefore, in RAI 149, Question 03.09.05-4, the staff requested that the applicant provide a design commitment which indicates that the design bases for RPV CSS conform to the industry code requirements specified in 10 CFR Part 50.55a. In addition, the staff requested that the applicant provide acceptance criteria that references documentation required to demonstrate that CSS have been constructed to the requirements of the appropriate industry design code incorporated by reference in 10 CFR Part 50.55a. In a January 29, 2009, response to RAI 149, Question 03.09.05-4, the applicant stated that FSAR Tier 1, Table 2.2.1-5, Item 3.15 will be deleted and replaced with Item 3.16. The acceptance criterion for FSAR Tier 1, Table 2.2.1-5, Item 3.16 in the FSAR was updated to state that an ASME Code Section III, Subsection NG stress report exists for each RPV internal component. The certified ASME stress report will ensure that the calculated stresses meet code allowables. The staff confirmed that Revision 2 of the FSAR contains the changes committed to in the RAI response. Accordingly, the staff finds that the applicant has adequately addressed this issue and, therefore, the staff considers RAI 149, Question 03.09.05-4 resolved.

3.9.5.4.2.1 *Design Arrangement*

In FSAR Tier 2, Section 3.9.5.1, “Design Arrangements,” the applicant described the physical or design arrangements of all reactor internals structures, components, and assemblies, including the manner of positioning and securing of such items, providing for axial and lateral retention and support, and the accommodation of dimensional changes due to thermal and other effects. The basic design of the U.S. EPR evolved from existing 4-loop PWR plants. The reactor internals of the U.S. EPR consist of CSS, threaded structural fasteners, and IS.

The RPV internals consist of two primary sections: the upper internals and the lower internals. In FSAR Tier 2, Section 3.9.5.1, the applicant provided a description of the major subassemblies and components of the upper and lower reactor internals including the functional requirements of each component. The reactor coolant’s flow path through different components of the RPV internals is also described in FSAR Tier 2, Section 3.9.5.1.

The review of the RPV internals design arrangements was performed in accordance with SRP Section 3.9.5 to ensure compliance with GDC 1, GDC 2, GDC 4, and GDC 10. The specific area of review included the physical or design arrangements of all reactor internals structures, components, and assemblies, including: (1) Reactor coolant flow path through the different components of the RPV internals; (2) the positioning and securing of different components and assemblies within the RPV; (3) the provision for axial and lateral retention and

support of the internals assemblies and components; and (4) the accommodation of dimensional changes due to thermal and other effects.

3.9.5.4.2.2 *Flow Path*

The reactor coolant flow path for the reactor internals is described in FSAR Tier 2, Sections 3.9.5.1.1, "Flow Path"; 3.9.5.1.2.4, "Lower Support Plate"; and 3.9.5.1.2.6, "Flow Distribution Device." The applicant states that the main coolant flow enters the bottom of the reactor vessel and turns upward, flowing past the flow distribution device and directed through the lower support plate (LSP). The LSP has inlet holes under each fuel assembly location which have an orifice at their base to equalize the flow rates at the fuel assembly inlets. The flow distribution device provides a homogenous flow distribution among the LSP holes.

The staff notes that it was difficult to follow the description of the flow path without additional information and figures. Therefore, in RAI 149, Question 03.09.05-7, the staff requested that the applicant clarify and supplement the discussion of the flow path in the FSAR to illustrate the directions of cooling flow within the RPV showing both the main core cooling flow and the core bypass flow paths. The staff also requested that the applicant provide further identification of the locations of RPV dome spray nozzles, and the orifices in the upper support plate which allow core bypass flow to travel from the RPV upper dome down to the upper plenum. Additionally, the staff requested that the applicant characterize in the FSAR the volume and velocity of the core bypass flow, and discuss whether the bypass flow is sufficient to cause vortex shedding and FIV in the internals components located in the RPV upper dome above the upper support plate.

In a January 29, 2009, response to RAI 149, Question 03.09.05-7, the applicant provided a figure to illustrate the flow paths in the reactor internals and location of the RPV dome spray nozzle. The staffs finds Figure 03.09.05-7-1 provided as part of this RAI response acceptable, because this figure illustrates the directions of cooling flow within the RPV, including the main core cooling flow and the core bypass flow path. Figure 03.09.05-7-1 also identifies the locations of RPV upper dome spray nozzles and the flow through the CRGA columns from the RPV upper dome down to the upper plenum. However, the applicant did not provide information on the core bypass flow volume or velocity including flow through the orifices. Therefore, the staff closed RAI 149, Question 03.09.05-7, and in follow-up RAI 291, Questions 03.09.05-23 and 03.09.05-24, the staff requested that the applicant provide information on the core bypass flow volume and velocity including the flow through the orifices.

Additionally, in the January 29, 2009, response to RAI 149, Question 03.09.05-7, the applicant stated that SRP Section 3.9.5 does not request provision of the flow volume and velocity, and provided no information on the core bypass flow volume or velocity. The staff determined this response unacceptable and in follow-up RAI 291, Question 03.09.05-23, the staff requested that the applicant provide a qualitative assessment of the bypass flow since volume and velocity are contributors to FIV and vortex shedding. In a February 11, 2010, response to RAI 291, Question 03.09.05-23, the applicant stated that calculations performed by AREVA demonstrate that the CRGA support columns will not experience excessive vibrations due to flow conditions in the upper plenum. These results are presented in TR ANP-10306P, "Comprehensive Vibration Assessment Program for US-EPR Reactor Internals Technical Report." The staff finds this acceptable, since this report is addressed and found acceptable in Section 3.9.2 of this report. Therefore, the staff considers RAI 291, Question 03.09.05-23 resolved.

In RAI 291, Question 03.09.05-24, the staff requested that the applicant provide plan views of the reactor internals in the FSAR to show the flow hole layout of the flow distribution plate,

upper core plate, and upper support plate. In a February 11, 2010, response to RAI 291, Question 03.09.05-24, the applicant provided revisions to FSAR Tier 2, Figure 3.9.5-2, “Lower Reactor Internals,” and FSAR Tier 2, Figure 3.9.5-4, “Reactor Pressure Vessel Upper Internals.” The staff finds this acceptable, because the revised figures include the layout of openings in the flow distribution plate, upper core plate, and upper support plant. The applicant also provided three additional figures. The staff finds this acceptable, since FSAR Tier 2, Figure 3.9.5-5, “Section A-A Flow Distribution Device”; Figure 3.9.5-6, “Section B-B Upper Support Plate”; and Figure 3.9.5-7, “Section C-C Upper Core Plate,” illustrate detailed views of the three plates. The staff confirmed that Revision 2 of the U.S. EPR FSAR contains the changes committed to in the RAI response. Accordingly, the staff finds the applicant has adequately addressed this issue and, therefore, considers RAI 291, Question 03.09.05-24 resolved.

3.9.5.4.2.3 *Lower Internals*

According to FSAR Tier 2, Section 3.9.5.1.2, “Lower Internals,” the RPV lower internals consist of:

- core barrel flange
- core barrel cylinder
- irradiation specimen (capsule) baskets
- LSP
- radial key inserts
- flow distribution device
- heavy reflector

The staff’s review of the different components of the lower internals is presented below.

<u>Core Barrel Flange and Cylinder</u>

FSAR Tier 2, Section 3.9.5.1.2.1, “Core Barrel Flange,” states that the core barrel flange is welded to the core barrel upper shell, rests on a ledge machined in the RPV flange, and transmits core and lower internal loads to the RPV. The core barrel outer diameter is customized to the corresponding RPV dimension in order to control the radial gap between the flanges.

The core barrel cylinder is composed of two cylindrical shells welded together. The upper section of the barrel has four integrated outlet nozzles located opposite the RPV outlet nozzles. The core barrel cylinder provides the passageway for the reactor coolant from the core to the RPV outlet nozzles. The core barrel outlet nozzle external radius is customized to the corresponding RPV nozzles in order to control the radial gap. The radial gap restricts the bypass flow between the RPV inlet and outlet nozzles. However, the applicant did not assess the potential of the FIV due to the core barrel nozzle leakage.

FSAR Tier 2, Table 4.1-1, “Summary of U.S. EPR Reactor Design and Performance Characteristics,” states that the U.S. EPR reactor has 241 fuel assemblies. This is approximately 25 percent more that than the number of fuel assemblies in existing 4-loop

U.S. PWR reactors. The larger number of fuel assemblies requires a larger reactor core barrel diameter. The larger diameter of the reactor will result in lower stiffness and natural frequency of vibration, and higher leakage flow rate as compared to the currently operating U.S. PWR reactors. Therefore, in RAI 184, Question 03.09.05-22, the staff requested that the applicant discuss the potential of the core barrel flange to FIV caused by the leakage (or bypass) flow between the outlet nozzle of the core barrel flange and the RV exit nozzle. In addition, the staff requested that the applicant provide evidence to show that the leakage flow between the outlet nozzle of the core barrel flange, and the RV exit nozzle will not cause excessive vibration of the core barrel flange.

In a February 27, 2009, response to RAI 184, Question 03.09.05-22, the applicant indicated that leakage FIV is associated with flow through narrow passages which are not rigidly constrained, and the mechanical vibration of the passage walls may couple with the flow field causing instability. The structural boundaries for the nozzle are not flexible. Also, the shell frequencies of the nozzle are very high due to its wall thickness and radius, both of which contribute to the stiffness of the nozzle. Therefore, considering the pressure drop across this interface and stiffness of the nozzle design, the core barrel outlet nozzles are not conducive to leakage flow instability. The applicant also stated that hot functional testing that will be performed as part of the comprehensive vibration assessment program for the U.S. EPR RPV will identify leakage flow instability of the core barrel outlet nozzle. If evidence of excessive vibration is identified during hot functional testing, appropriate corrective actions will be taken to eliminate or minimize the source of excitation. The staff finds that the rigidity of the core barrel outlet nozzle design and the comprehensive vibration assessment program provide reasonable assurance that the nozzle bypass flow will not cause adverse FIV; therefore, the staff considers RAI 184, Question 03.09.05-22 resolved.

3.9.5.4.2.3.1 *Radial Key Inserts*

According to FSAR Tier 2, Section 3.9.5.1.2.5, “Radial Key Inserts,” the lower internals are centered within the RPV by radial support keys and grooves machined in the LSP. The radial support keys are welded to and integral with the RPV. These radial keys and LSP grooves have radial key inserts to maintain tight lateral clearances. The radial key inserts are pinned and bolted in the LSP grooves. The radial keys also provide a secondary support function by limiting the consequences of the postulated failure of the lower internals. The energy that would be absorbed by the radial keys is limited by the controlled vertical gaps. In RAI 149, Question 03.09.05-9, the staff requested that the applicant clarify the following:

- The postulated failure of the lower internals does not identify the failure mechanism postulated for the internal structure.
- The analytical methodology addressing this accident postulation, including applicable design codes, load definition, and allowable stress and deformation criteria.
- Whether or not this is a design-basis accident considered in FSAR Tier 2, Chapter 15.

In a January 29, 2009, response to RAI 149, Question 03.09.05-9, the applicant stated that the failure mechanism is the postulated failure of the core barrel flange which is a beyond-design-basis event. According to FSAR Tier 2, Table 3.2.2-1, the radial key inserts are designed to ASME Code, Section III, Subsection NG requirements. Designing to the ASME Code requirements will ensure that the radial keys will provide adequate support to the reactor internals in the unlikely event that the core barrel flange fails. The staff finds that the designing

to ASME Code, Section III, Subsection NG requirements will provide reasonable assurance that the radial key inserts will provide the core support function discussed in FSAR Tier 2, Section 3.9.5.1.2.5. Therefore, the staff considers RAI 149, Question 03.09.05-9 resolved.

However, in FSAR Tier 2, Table 3.2.2-1, the radial keys are not listed. Therefore, in RAI 291, Question 03.09.05-25, the staff requested that the applicant correct this apparent omission. In a February 11, 2010, response to RAI 291, Question 03.09.05-25, the applicant stated that FSAR Tier 2, Table 3.2.2-1 will be revised to include an entry for RPV radial keys. As discussed in FSAR Tier 2, Section 3.9.5.1.2.5, radial keys are welded to and integral with the RPV. The radial keys are specified as part of the RPV rather than the RPV internals. The FSAR Tier 2, Table 3.2.2-1 entry will be revised to be applicable to the RPV pressure boundary, and a second entry will be provided specific to the radial keys. The staff finds this addition to FSAR Tier 2, Table 3.2.2-1 acceptable which classifies the RPV radial keys separately and designed to ASME Class CS. The staff confirmed that Revision 2 of the U.S. EPR FSAR contains the changes committed to in the RAI response. Accordingly, the staff finds the applicant has adequately addressed this issue and, therefore, considers RAI 291, Question 03.09.05-25 resolved.

3.9.5.4.2.3.2 *Heavy Reflector*

According to FSAR Tier 2, Section 3.9.5.1.2.7, “Heavy Reflector,” the heavy reflector is located inside the core barrel between the core and core barrel cylinder. The heavy reflector increases neutron efficiency due to its neutron reflective properties, protects the RPV from radiation-induced embrittlement, and improves the long term mechanical behavior of the lower internals. To avoid any welded or bolted connections close to the core, the heavy reflector consists of stacked slabs positioned one above the other. The top slab is fitted with alignment pins that extend through the upper core plate to provide proper alignment. The heavy reflector also provides lateral support to the core.

FSAR Tier 2, Table 3.2.2-1 lists the following components of the heavy reflector assembly as safety-related:

- heavy reflector centering pins
- heavy reflector normal and centering rings
- heavy reflector slabs
- heavy reflector upper core plate guide pins

However, heavy reflector positioning keys and tie rods are classified as NS-AQ (non-safety-related augmented quality, that is, with pertinent 10 CFR Part 50, Appendix B quality assurance requirements applied) and QG D components in FSAR Tier2, Table 3.2.2-1. The staff noted that it is unclear in the FSAR why these components are not classified as safety-related. The heavy reflector slabs are connected together by the tie rods which presumably perform a safety function. Therefore, in RAI 149, Question 03.09.05-6, the staff requested that the applicant justify the different safety and commercial code classifications stated in FSAR Tier 2, Table 3.2.2-1 for the various components of the heavy reflector. In a January 29, 2009, response to RAI 149, Question 03.09.05-6, the applicant stated that the tie rods are reclassified as safety-related components and the next revision of the FSAR will be revised accordingly. The applicant also indicated that the heavy reflector-positioning keys do not serve a core support function. The heavy reflector-positioning keys assist alignment of the

lower slab during installation of the heavy reflector into the core barrel. Therefore, the heavy reflector-positioning keys classification will remain as NS-AQ. The staff confirmed that the tie rods are reclassified as safety-related in FSAR Revision 1. The staff finds that the revised safety classification and design requirements will provide reasonable assurance that the heavy reflector assembly will provide the core support function described in FSAR Tier 2, Section 3.9.5.1.2.7 and, therefore, considers RAI 149, Question 03.09.05-6 resolved.

3.9.5.4.2.4 *Upper Internals*

- According to FSAR Tier 2, Section 3.9.5.1.3, "Upper Internals," the upper internals consist of:
- upper support assembly (including the flange, shell, and upper support plate)
- upper core plate
- CRGAs
- columns

The staff's review of these components is presented below.

3.9.5.4.2.4.1 *Upper Support Assembly*

In FSAR Tier 2, Section 3.9.5.1.3.1, "Upper Support Assembly," the applicant stated that the upper support assembly flange rests on the hold-down spring, which rests on the core barrel flange, which in turn is supported on the ledge machined in the RPV flange. The upper support assembly flange is held in place and preloaded by the RPV closure head flange. Its outer diameter is customized to the corresponding vessel dimension in order to control the radial gap between flanges. The radial gap controls lateral displacements in normal and faulted conditions.

The staff notes that the applicant did not discuss in the FSAR how the horizontal loads on the upper core support assembly due to flow, vibration, and seismic and pipe rupture events are transmitted from the upper core support flange to the RV head and hold-down spring by friction or direct contact with the RV flange. It is also unclear to the staff if the head and vessel alignment pins also transmit some of the horizontal loads. In addition, the FSAR does not address the potential loss of preload in the hold-down spring due to stress relaxation during service and its potential effect on the functional and structural integrity of the upper core support assembly. Therefore, in RAI 184, Question 03.09.05-13, the staff requested that the applicant provide an assessment of the potential loss of preload of the hold-down spring due to stress relaxation during the design lifetime of these components, and discuss its effect on the horizontal and vertical restraint of the upper core support and core barrel assemblies. In a February 27, 2009, response to RAI 184, Question 03.09.05-13, the applicant clarified that the horizontal loads on the upper core support assembly due to flow, vibration, seismic and piping rupture events are transmitted from the upper core support flange to the RPV head and hold-down spring by direct contact. The head and vessel alignment pins are not credited with transmitting any of the horizontal loads. The applicant also stated that the stress and FIV analyses of the upper core support assembly consider preload relaxation in the hold-down spring. Initial preload is applied to the hold-down spring so that adequate final preloads exist to vertically and horizontally restrain the upper core support and core barrel assemblies. The staff finds that the consideration of preload relaxation will provide reasonable assurance that the

hold-down spring will provide core support function during operational transients and design basis events and, therefore, considers RAI 184, Question 03.09.05-13 resolved.

3.9.5.4.2.4.2 *Upper Core Plate*

In FSAR Tier 2, Section 3.9.5.1.3.2, "Upper Core Plate," the applicant provided a description of the U.S. EPR upper reactor internals upper core plate with holes located opposite the fuel assemblies for core coolant outlet flow which are designed to equilibrate the outlet flow from the core. The upper core plate contains fuel alignment pins at each fuel assembly location that position, align, and restrain the fuel assemblies.

The staff reviewed FSAR Tier 2, Section 3.9.5.1.3.2; FSAR Tier 2, Figure 3.9.5-1, "Reactor Pressure Vessel General Arrangement"; FSAR Tier 2, Figure 3.9.5-2; FSAR Tier 2, Figure 3.9.5-3, "Reactor Pressure Vessel Heavy Reflector"; and FSAR Tier 2, Figure 3.9.5-4, and determined that the applicant did not provide sufficient information to allow the review of the upper core plate design and its interfaces with other reactor internals components. Therefore, in RAI 184, Question 03.09.05-14, the staff requested that the applicant provide sufficient details regarding the design of the upper core plate and its interface with the fuel assemblies, core barrel, upper support columns, and lower guide tubes. Also, the staff requested that the applicant clarify how these component assemblies are evaluated against possible excitation mechanisms of FIV. In a February 27, 2009, response to RAI 184, Question 03.09.05-14, the applicant stated that the upper core plate is not susceptible to excitation from turbulence or any other FIV mechanisms. This is due to the high fundamental frequency that this component exhibits because of the rigidity provided by the support columns and the plate thickness. The applicant also provided additional justification for the following four potential sources of upper core plate flow excitation: random turbulence resulting from parallel flow excitation, acoustic type loading, vibration of the core barrel, and dynamics of the fuel bundle. The applicant did not propose to revise FSAR Tier 2, Section 3.9.5, "Reactor Pressure Vessel Internals," to include details of the design of the upper core plate and its interfaces and the evaluation against possible excitation mechanisms of FIV. Therefore, the staff considers RAI 184, Question 03.09.05-14 resolved, and in follow-up RAI 510, Question 03.09.05-29, the staff requested that the applicant include sufficient information in FSAR Tier 2, Section 3.9.5 regarding the design arrangement of the upper core plate and associated internal components including a discussion of the evaluation of the potential adverse effects of FIV and vortex shedding. Additionally, the staff requested that the the applicant make the technical basis for these mechanisms available for staff review. **RAI 510, Question 03.09.05-29 is being tracked as an open item.**

3.9.5.4.2.4.3 *Control Rod Guide Assemblies (CRGAs)*

In FSAR Tier 2, Section 3.9.5.1.3.3, "Control Rod Guide Assemblies," the applicant stated that the CRGAs consist of guide tubes held together with support plates and tie rods. The guide tube assemblies provide a straight, low-friction channel to insert, withdraw, and drop the control rod drive mechanism drive shafts and attached RCCAs. The guide tube assemblies are located inside housings and columns. The housings are attached to the top of the USP, and the columns are attached to the bottom of the USP and also to the UCP. The housings and columns also protect the RCCAs from static and hydraulic loads and other mechanical loads.

The staff reviewed FSAR Tier 2, Figure 3.9.5-1 and determined that the applicant did not provide sufficient information regarding the horizontal support plates and tie rods inside the CRGAs. Therefore, in RAI 184, Question 03.09.05-15, the staff requested that the applicant provide design details together with relevant FIV analysis for these support plates. In particular,

the staff requested that the applicant provide drawing/sketches of the control rod guide, to clarify any differences of this design from that of other currently operating PWR reactors and their impact on potential flow excitation mechanisms, and to revise FSAR Tier 2, Section 3.9.5. In a February 27, 2009, response to RAI 184, Question 03.09.05-15, the applicant stated that the CRGA design details are shown in upper internals drawings which are available for NRC inspection. The applicant stated that the support plates are not susceptible to excitation resulting from the vertical flow through the CRGA columns for the reasons cited for the upper core plate. The applicant also referred to several sections in FSAR Tier 2, Section 3.9.2, "Dynamic Testing and Analysis of Systems, Components, and Equipment," for FIV for CRGA components and similar plant information. Since the CRGAs are evaluated for fluid-elastic instability, random turbulence, and vortex-shedding excitation and information for currently operating PWR reactors in Section 3.9.2 of this report, the staff refers to that section for further discussion of this topic. The staff finds this response acceptable and, therefore, considers RAI 184, Question 03.09.05-15 resolved. **RAI 510, Question 03.09.05-30 is being tracked as an open item** until the CRGA detail drawings are reviewed and FSAR Tier 2, Section 3.9.5 is revised.

3.9.5.4.2.4.4 *Columns*

FSAR Tier 2, Section 3.9.5.1.3.4, "Columns," states that the upper support plate is attached to the upper core plate with columns. The columns are bolted to the bottom of the upper support plate and transmit vertical forces to the RPV closure head. The columns are of three types and perform the following functions:

- The CRGA columns serve as housings for the CRGAs. The CRGA Columns are located above those fuel assemblies that are equipped with RCCAs, and they also serve to support the instrument guide tubes for the incore instrumentation lances where the CRGA columns penetrate the upper plenum.

- The level monitoring probe (LMP) columns that are located around the edge of the USP and protect the LMPs in the upper plenum.

- The normal columns that provide support at the USP edge, including when the upper internals are on the refueling cavity storage stand.

SRP Section 3.9.5 recommends that incore instrumentation support structures be reviewed with the reactor internals. The brief description of the LMP columns' location and function as described in FSAR Tier 2, Section 3.9.5.1.3.4 and noted above, and a list of instrument support components (LMP columns, guide tubes for instrumentation, LMP upper housing) in FSAR Tier 2, Table 3.9.5-1 are not sufficient to allow the staff to review the supporting structures' design and their susceptibility to potential adverse flow effects. Therefore, in RAI 184, Question 03.09.05-12, the staff requested that the applicant provide more details of the instrumentation supporting structures (e.g., thermocouple, water level sensor, incore nuclear instrumentation system (ICIS), CRDA), as well as the relevant FIV analysis for these structures. In addition, the staff requested that the applicant explicitly state whether these structures and their operating environment are similar to the other currently operating PWR internals and supporting structures.

In a February 27, 2009, response to RAI 184, Question 03.09.05-12, the applicant refered to FSAR Tier 2, Section 3.9.2.3, "Dynamic Response Analysis of Reactor Internals Under Operational Flow Transients and Steady-State Conditions," and FSAR Tier 2. Section 3.9.2.4,

"Preoperational Flow-Induced Vibration Testing of Reactor Internals," and associated RAI responses. In FSAR Tier 2, Section 3.9.2.3, the applicant stated that the support column design similar to those in the U.S. EPR have proven operating experience in the German Konvoï plants. The incore instrumentation guide tubes attached to the column supports in the upper internals are somewhat shielded from the coolant flow in the upper plenum, but nonetheless are evaluated for FIV. The instrumentation guide tubes are subjected to fuel assembly outlet nozzle turbulence, which is judged to be less than the inlet nozzle turbulence to which incore instrumentation thimbles in previous plants are subjected. The CRGA columns, normal columns, LMP columns, and instrument guide tubes were evaluated for fluid-elastic instability, random turbulence, and vortex-shedding excitations. The top mounted instrument guide tube should be exposed to less turbulent flow than the bottom mounted instrument tubes in operating plants due to the pressure drop through the fuel bundles. Additionally, if unanticipated FIV or acoustic loadings are identified during preoperational or hot functional testing, corrective actions will be taken to eliminate these loadings in accordance with the comprehensive vibration assessment program discussed in FSAR Tier 2, Section 3.9.2.4. In Section 3.9.2 of this report, the FIV is extensively reviewed. However, the applicant did not provide design details of the instrumentation system and did not revise the FSAR Tier 2, Section 3.9.5 to contain details of the design and the relevant FIV. Therefore, the staff considers RAI 184, Question 03.09.05-12 resolved, and in RAI 510, Question 03.09.02-28, the staff requested that the applicant provide details of the instrumentation system and the relevant FIV. **RAI 510, Question 03.09.05-28 is being tracked as an open item.**

3.9.5.4.3 Loading Conditions

FSAR Tier 2, Section 3.9.5.2 states, in part, that the design, analyses, fabrication, and non-destructive examination of the RPV internals, Class CS CSS, is in accordance with the ASME B&PV Code, Section III, Division 1, Subsection NG. Those RPV internals components not designed as ASME Code, Section III, Class CS CSS are designated as IS in accordance with ASME Code, Section III, Subsection NG-1122. In accordance with ASME Code, Section III, Subsection NG-1122(c), the IS are designed and constructed such that they do not adversely affect the integrity of the CSS.

The staff reviewed FSAR Tier 2, Section 3.9.5 and determined that the applicant has not provided the design criteria actually used for internal structure components which are not designated CS for those components classified as IS in FSAR Tier 2, Table 3.9.5-1. Therefore, in RAI 149, Question 03.09.05-10, the staff requested that the applicant provide in the FSAR, a discussion specifying the industry design code, or specific sections of the applicable design code, used as the design bases for identification of design loadings, loading combinations, material stress limits, and allowable stress and fatigue usage limit criteria. In a January 29, 2009, response to RAI 149, Question 03.09.04-10, the applicant stated that the IS are designed such that they meet the guidelines of ASME Code, Section III, Subsection NG-3000 in accordance with SRP Section 3.9.5 and ASME Code, Section III, Subsection NG-1122(c). The applicant revised FSAR Tier 2, Section 3.9.5.2 to reference ASME Code, Section III, Subsection NG-3000. The staff notes that designing IS to ASME Code, Section III, Subsection NG-3000 requirements meets the SRP Section 3.9.5 guidance, which the staff finds acceptable. Therefore, the staff considers RAI 149, Question 03.09.05-10 resolved.

3.9.5.4.4 Design Basis

3.9.5.4.4.1 *Allowable Design/Service Limits*

FSAR Tier 2, Section 3.9.5.3, “Design Bases,” states that the combinations of design and service loadings accounted for in the design of the RPV internals, and the method of combining loads for normal, upset, emergency, and faulted service conditions, are addressed in FSAR Tier 2, Section 3.9.3, “ASME Code Class 1, 2, and 3 Components, Component Supports, and Core Support Structures.” The allowable design or service limits to be applied to the RPV internals and the effects of service environments, deflection, cycling, and fatigue limits—along with a summary of the maximum calculated total stress, deformation, and cumulative usage factor for each designated design or service limit—are addressed in FSAR Tier 2, Section 3.9.3.1, “Loading Combinations, System Operating Transients, and Stress Limits.”

In RAI 404, Question 03.09.03-24, the staff requested that the applicant provide the design specifications for risk-significant mechanical components for staff audit to ensure that the components are ready for procurement, and that the FSAR design methodologies and criteria are adequately reflected in associated component design specifications. **RAI 404, Question 03.09.03-24 is being tracked as an open item** until successful completion of the audit of the reactor internals design specification.

In a review of FSAR Tier 2, Section 3.9.3.1, including FSAR Tier 2, Table 3.9.3-4, “Load Combinations and Acceptance Criteria for ASME Class 1, 2, and 3 Component Supports,” the staff notes that a summary of the allowable or maximum calculated total stress, deformation, and cumulative usage factor is not provided in the FSAR. Therefore, in RAI 149, Question 03.09.05-11, the staff requested that the applicant provide a tabular summary in the FSAR, including each component of the reactor internals and CSS, giving the maximum calculated total stress, deformation, and cumulative usage factor for each designated design and service limit defined in ASME Code, Section III, Subsection NG. In a January 29, 2009, response to RAI 149, Question 03.09.05-11, the applicant stated that providing a summary of maximum total stress, deformation, and cumulative usage factor values is the responsibility of the COL applicant. However, FSAR Tier 2, Table 1.8-2, COL Information Item 3.9-11 is only for ASME Code Class 1 components. The calculated total stress, deformation, cumulative usage factor values for reactor internals have not been addressed by the applicant. The staff considers RAI 149, Question 03.09.05-11 was resolved and in follow-up RAI 291, Question 03.09.05-26, the staff requested that the applicant provide the information, and revise COL Information Item 3.9-11 to include reactor IS and CSS, or add a separate COL information item.

In a January 29, 2009, response to RAI 149, Question 03.09.05-26, the applicant agreed to change FSAR Tier 2, Table 1.8-2, “U.S. EPR Combined License Information Items,” to add a new COL Information Item 3.9-14. The new COL information item states that a COL applicant that references the U.S. EPR design certification will provide a summary of reactor core support structure maximum total stress, deformation, and cumulative usage factor values for each component and each operating condition in compliance with ASME Code, Section III Subsection NG. Additionally, FSAR Tier 2, Section 3.9.5.2 will be revised to describe COL Information Item 3.9-14. The staff finds this additional COL information item acceptable, since it meets ASME Code requirements. The staff also verified that FSAR Tier 2, Table 1.8-2, and FSAR Tier 2, Section 3.9.5.2 were revised. Therefore, the staff considers RAI 291, Question 03.09.05-26 resolved.

3.9.5.4.4.2 *Deformation Limits*

The load or stress component, and displacement limits for the reactor internals that affect the safety and operability of the ASME CSS are summarized in FSAR Tier 2, Table 3.9.3-3, "Load Combinations and Acceptance Criteria for ASME Core Support Structures." However, the staff notes that the FSAR does not provide any details on how the deformation limits will be determined in the design specification or provide the technical basis for these deformation limits. As stated in SRP Section 3.9.5, deformation limits for reactor internals should be established by the applicant and presented in the safety analysis report, and the basis for these limits should be included. Also, the stresses for these displacements should not exceed the specified limits. Therefore in RAI 184, Question 03.09.05-19, the staff requested that the applicant provide deformation limits and their technical basis in the appropriate sub-sections of FSAR Tier 2, Section 3.9.5. Alternately, the staff requested that the applicant provide a reference document where this information is available. Additionally, the staff requested that the applicant revise the FSAR to either include the requested information or provide in FSAR Tier 2, Section 3.9.5, a reference where this information is available. In a May 1, 2009, response to RAI 184, Question 03.09.05-19, the applicant provided a markup of the FSAR which includes the displacement limit for the functionality of the CRGA. The displacement limit is based on full scale CRGA loss of function testing, which determines the empirical deformation limits causing temporary and permanent losses of function. The staff finds that the displacement limit will provide reasonable assurance that the shutdown function of the control rod assemblies will not be impacted. The staff confirmed that Revision 2 of the FSAR contains the changes committed to in the RAI response. Accordingly, the staff finds that the applicant has adequately addressed this issue and, therefore, considers RAI 184. Question 03.09.05-19 resolved.

3.9.5.4.5 Interface Cold Gaps

FSAR Tier 2, Section 3.9.5.3.1, "Interface Cold Gaps," provides a description of the interface cold gaps between the internals and RPV and between the main parts of the internals. The cold gaps are of two types. The functional cold gaps are used for alignment of the equipment and to limit the core bypass flows under normal and upset conditions. The controlled cold gaps are used to reduce the relative displacements between the top of the internals and the RPV under normal, faulted, and beyond-design conditions.

FSAR Tier 2, Section 3.9.5.3.1.2, "Controlled Cold Gaps," states that the radial keys used to limit gaps between the RPV and the IS are fabricated both with and without lateral adjustments. The radial gap at key ends is limited, as is the vertical gap between keys and the LSP. The staff notes that the FSAR does not provide additional details about the function and fabrication of the radial keys. Therefore, in RAI 149, Question 03.09.05-8, to the staff requested that the applicant provide the following information:

- Additional discussion in the appropriate FSAR section clarifying the support function of the radial keys, and further explaining what is meant by the statement, "…fabricated both with and without lateral adjustments"
- Additional figure in the FSAR illustrating the interface among the radial keys, the radial key inserts, the LSP, and the reactor vessel shell
- Clarify the support load path for these components, discussed above, identifying the directions of load restraint (translation and rotation) of the LSP with respect to the principal orthogonal coordinate system

- Quantify the “tight lateral clearances” attributed to the radial key inserts described in FSAR Tier 2, Section 3.9.5.1.2.5

In a January 29, 2009, response to RAI 149, Question 03.09.05-8, the applicant stated that there are radial keys welded to the inside surface of the RPV, which fit into grooves located in the LSP. The LSP grooves and the radial keys have plates installed; which are radial key inserts. The radial keys and the corresponding inserts limit the lateral movement (i.e., swaying) of the core barrel and establish clearances in the circumferential direction to limit core barrel rotation. The keys and corresponding inserts are also designed to support the core barrel in the unlikely event of core barrel failure, a beyond design basis event. In this case, the load on the radial keys is vertical. The four keys that prevent rotation of the core barrel would have a horizontal lateral load imposed on the side of the keys. The applicant also provided in the response Figure 03.09.05-8-1 that illustrates the clearance among the radial keys, radial key inserts, the LSP, and the reactor vessel shell. The applicant proposed to revise FSAR Tier 2, Sections 3.9.5.3.1.2 and 3.9.5.1.2.5 to clarify the support functions of the radial keys and include Figure 03.09.05-8-1. The staff finds that FSAR update and the RAI response provide sufficient description of the function and load paths of the radial keys to provide reasonable assurance that SRP Section 3.9.5 is met. Therefore, the staff considers RAI 149, Question 03.09.05-8 resolved.

However, the applicant did not discuss placement of Figure 03.09.05-8-1 in the next revision of the FSAR. Therefore, in RAI 522, Question 03.09.05-31, the staff requested that the applicant include Figure 03.09.05-8-1 in the FSAR. Additionally, the staff requested that the applicant provide details quantifying the radial key insert clearances on the drawings that can be reviewed during future NRC audits. **RAI 522, Question 03.09.05-31 is being tracked as an open item**.

In Section 3.9.2, “Dynamic Testomg amd Analysis of Systems, Components, and Equipment,” of this report, the staff discussed FIV and acoustic resonance of core support and internals components, and concluded that the applicant adequately addressed the potential adverse flow effects. Additionally, COL Information Item 3.9-1 will ensure that, if adverse FIV is detected during the comprehensive vibration assessment program, the causes will be identified and resolved. The commitments in FSAR Tier 2, Sections 3.9.2 and 3.9.5, and COL Information Item 3.9-1 will provide reasonable assurance that the core support and internals structures are designed to avoid potential adverse flow effects.

3.9.5.5 *Combined License Information Items*

Table 3.9.5-1 provides a list of reactor pressure vessel internals related COL information item numbers and descriptions from FSAR Tier 2, Table 1.8-2:

Table 3.9.5-1 U.S. EPR Combined License Information Items

Item No.	Description	FSAR Tier 2 Section
3.9-14	A COL applicant that references the U.S. EPR design certification will provide a summary of reactor core support structure maximum total stress, deformation, and cumulative usage factor values for each component and each operating condition in conformance with ASME Code, Section III	3.9.5.2

	Subsection NG.	

The staff finds the above listing to be complete. Also, the list adequately describes actions necessary for the COL applicant. No additional COL information items need to be included in FSAR Tier 2, Table 1.8-2 for reactor pressure vessel internals.

3.9.5.6 *Conclusions*

Except for the open items discussed above, the staff concludes that the design of the reactor internals meets the requirements GDC 1, GDC 2, GDC 4, GDC 10, 10 CFR Part 50.55a, and 10 CFR Part 52. This conclusion is based on the following findings:

- The quality standards used for the design are commensurate with the importance of the safety functions to be performed and comply with GDC 1.

- The physical or design arrangements of all reactor internals structures, components, assemblies, and systems, including the manner of positioning and securing of such items, providing for axial and lateral retention and support of the reactor core fuel assemblies, and the accommodation of dimensional changes due to thermal and other effects comply with GDC 2, GDC 4, and GDC 10.

- The reactor internals have been designed in accordance with ASME Code, Section III, Subsection NG which complies with the requirements of 10 CFR Part 50.55a. The loading conditions of normal operation, anticipated operational occurrences, potential adverse flow effects of FIV and acoustic resonances, postulated accidents, and seismic events, as well as the reactor internals design and service limits and displacement limits, that have been considered in the design of the U.S. EPR core support components and IS, the distribution of the design and service loadings acting on the internal components and structures comply with the relevant requirements of 10 CFR Part 50, Appendix A, GDC 2, GDC 4, and GDC 10.

- As required by SRP Appendix A, Section 3.9.5, the applicant has considered the potential adverse affects of FIV and acoustic resonances on reactor internals, including piping and components of plant systems.

3.9.6 Functional Design, Qualification, and Inservice Testing Programs for Pumps, Valves, and Dynamic Restraints

3.10 Seismic and Dynamic Qualification of Mechanical and Electrical Equipment

3.10.1 Introduction

This section documents findings from the staff's review and evaluation of the information provided by AREVA for the U.S. EPR design that is employed to ensure the structural integrity and the functionality of mechanical and electrical equipment, including instrumentation and controls, under the full range of normal and accident loadings (including seismically induced loadings). The review addresses mechanical and electrical equipment associated with systems that are essential to emergency reactor shutdown, containment isolation, reactor core cooling,

and containment and reactor heat removal or are otherwise essential in preventing significant release of radioactive material to the environment. It also addresses instrumentation that is needed to assess plant and environmental conditions during and after an accident.

3.10.2 Summary of Application

FSAR Tier 1: Seismic qualification of equipment is addressed in the system descriptions for Seismic Category I equipment, and ITAAC are included to verify that Seismic Category I equipment will withstand design basis loads. FSAR Tier 1, Section 2.4.7, "Seismic Monitoring System," provides information which indicates that a seismic monitoring system is incorporated into the U.S. EPR design.

FSAR Tier 2: The applicant has provided a description in FSAR Tier 2, Section 3.10, summarized here, in part, as follows:

The applicant has made the following assumptions in determining which plant equipment needed to be seismically qualified: (1) Offsite power may not be available for up to 72 hours following an earthquake; (2) no other extreme events or accidents are postulated to occur, other than the safe-shutdown earthquake and loss of offsite power; and (3) the single failure criterion is applied. Based on these assumptions, the following classes of equipment required seismic qualification:

- Active mechanical equipment which operates or changes state to accomplish safe shutdown
- Active equipment in systems which support the operation of identified safe-shutdown equipment (e.g., power supplies, control systems, cooling systems)
- Instrumentation needed to confirm that the safe-shutdown functions have been achieved and are being maintained
- Instrumentation needed to operate the safe-shutdown equipment
- Tanks and heat exchangers used to reach and maintain safe shutdown
- Cable and conduit raceways which support electrical cable for the selected safe-shutdown equipment
- Post-accident monitoring instrumentation described in RG 1.97 "Criteria for Accident Monitoring Instrumentation for Nuclear Power Plants"

The applicant stated that its program to ensure the seismic and dynamic qualification of mechanical and electrical equipment conforms to the guidance in RG 1.100, "Seismic Qualification of Electric and Mechanical Equipment for Nuclear Power Plants," Revision 3, September 2009. The U.S. EPR design will use IEEE Std 344-2004, "IEEE Recommended Practice for Seismic Qualification of Class 1E Equipment for Nuclear Power Generating Stations." IEEE Std 344-2004 requires the users to: (1) Predict equipment performance by analysis; (2) test the equipment under simulated seismic conditions; or (3) qualify the equipment by a combination of analysis and testing. The applicant will not use earthquake experience data alone to seismically qualify mechanical and electrical equipment for the U.S. EPR design. Mechanical and electrical equipment for the U.S. EPR is qualified only for the case of the SSE.

Consideration of an operating basis earthquake (OBE) is not a design requirement for the U.S. EPR. However, low-level seismic effects (fatigue) required by IEEE Std 344-2004 to qualify mechanical and electrical equipment are considered using the equivalent of five OBEs followed by one SSE event (with 10 maximum stress cycles per event). FSAR Tier 2, Table 3.10-1, “List of Seismically and Dynamically Qualified Mechanical and Electrical Equipment,” provides a comprehensive list (in that it includes all Seismic Category I and II components as well as post-accident monitoring equipment discussed in RG 1.97) of seismically and dynamically qualified equipment in the U.S. EPR design. There will be a seismic qualification data package (SQDP) for each piece of equipment (or class of equipment) listed in FSAR Tier 2, Table 3.10-1, which includes the specification of performance requirements of active components that must be satisfied during and after a seismic event. The equipment qualification file (within SQDP) includes qualification summary data sheets for each mechanical and electrical component.

ITAAC: The applicant provided the information concerning ITAAC with three column format including, “Commitment Wording,” “Inspections, Tests, Analyses,” and, “Acceptance Criteria.” Each design commitment in the left-hand column of the ITAAC tables has an associated requirement for inspections, tests, or analyses (ITA) specified in the middle column of the tables.

With the table format described above, the applicant provided the system-based design descriptions of ITAAC tables for the plant design including Structures, Nuclear Island Systems, Severe Accident Systems, Instrumentation and Control Systems, Electrical Power, HVAC Systems, Support Systems, Steam and Power Conversion Systems, Radioactive Waste Management, and other systems.

The applicant noted in FSAR Tier 2, Appendix 3D, “Methodology for Qualifying Safety-Related Electrical and Mechanical Equipment,” Attachment E, “Seismic Qualification Techniques,” Subsection E.3.4, “Qualification by Experience,” that the seismic qualification based on experience is not utilized by the applicant. This does not prevent the use of applicable test data from previous qualification of similar equipment.

3.10.3 Regulatory Basis

The relevant requirements of NRC regulations for this area of review, as provided in the SRP (NUREG-0800), are summarized below.

1. GDC 1, “Quality Standards and Records,” and GDC 30, “Quality of Reactor Coolant Pressure Boundary,” as they relate to qualifying equipment to appropriate quality standards commensurate with the importance of the safety functions to be performed.

2. GDC 2, “Design Bases for Protection against Natural Phenomena,” and 10 CFR Part 50, Appendix S, “Earthquake Engineering Criteria for Nuclear Power Plants,” as they relate to designing equipment to withstand the effects of natural phenomena such as earthquakes.

3. GDC 4, “Environmental and Dynamic Effects Design Bases,” as it relates to qualifying equipment as capable of withstanding the dynamic effects associated with external missiles and internally generated missiles, pipe whip, and jet impingement forces.

4. GDC 14, “Reactor Coolant Pressure Boundary,” as it relates to qualifying equipment associated with the reactor coolant boundary so that there is an extremely low probability of abnormal leakage, of rapidly propagating failure, and of gross rupture.

5. 10 CFR Part 50, Appendix B, “Quality Assurance Criteria for Nuclear Power Plants and Fuel Reprocessing Plants,” as it relates to qualifying equipment using the quality assurance criteria provided.

6. 10 CFR Part 50, Appendix B, Section III, “Design Control,” as it relates to verifying and checking the adequacy of design, such as by the performance of a suitable test program, among other things, and which specifically requires that a test program used to verify the adequacy of a specific design feature shall include suitable qualifications testing of a prototype unit under the most adverse design conditions.

Acceptance criteria adequate to meet the above regulatory requirements are found in the following guidance documents:

1. NUREG-0800, Section 3.10, “Seismic and Dynamic Qualification of Mechanical and Electrical Equipment,” Revision 3, March 2007. Review interfaces with other SRP sections can also be found in this SRP section.

2. RG 1.61, “Damping Values for Seismic Design of Nuclear Power Plants,” Revision 1, March 2007, as it relates to the selection of damping values for equipment to be qualified.

3. RG 1.92, “Combining Modal Responses and Spatial Components in Seismic Response Analysis,” Revision 2, July 2006, as it relates to the use of multimodal and multidirectional responses for qualification.

4. RG 1.97, as it relates to accident monitoring equipment.

5. RG 1.100 as it relates to the seismic and dynamic qualification of equipment, which includes the endorsemen of equipmentt, with conditions.

6. IEEE Std No. 344-2004 and ASME QME-1-2007, “Qualification of Active Mechanical Equipment Used in Nuclear Power Plants,” as they relate to the seismic qualification of equipment used in nuclear power plants.

3.10.4 Technical Evaluation

The seismic qualification of components must demonstrate that the equipment is capable of performing its safety-related functions while subjected to normal operating loads or the maximum expected seismic loads (e.g., the SSE loads) at the location of the equipment. The required response spectra (RRS) for a component at a specific location is based on the in-structure response spectra (ISRS) of the building or subsystem. The RRS reflect the additional amplification of the ISRS due to the flexibility of the equipment supporting structure. The RRS define the minimum seismic input motion (or loads) for the qualification of the component. The seismic loads are then added to other applicable loads, such as normal and transient operating and accident loads. Non-active mechanical components are only required to maintain their structural and pressure boundary integrity during and after the required seismic event. Permanent deformation of component supports and structures is acceptable, provided it does not impair the ability of the component to perform its safety-related functions. The seismic

qualification of components can be demonstrated by testing, analysis, or a combination of both. Multi-frequency and multi-axis testing is the preferred method of qualifying equipment. Qualification testing should conservatively simulate and envelop the required seismic motion at the location of the equipment. Qualification by analysis alone is only permitted under the following circumstances: (1) When maintaining the structural and pressure boundary integrity is sufficient to perform the equipment's safety-related functions; (2) when the equipment is structurally simple and its behavior can be predicted by a conservative analytical approach; (3) when the equipment is too large or heavy to obtain a representative test input at existing test facilities; or (4) when interfaces, such as interconnecting cables to a cabinet, cannot be conservatively considered in the testing.

The staff reviewed the applicant's FSAR Tier 1 documentation for information having relevance to seismic qualification of equipment.

Design descriptions: The FSAR Tier 1 information contains relevant information pertaining to system design descriptions, and indicates that a seismic monitoring system is incorporated into the U.S. EPR design. A seismic monitoring system can serve as the basis to initiate SCRAM (a reactor trip signal) in the event of an earthquake, and thus, could include components that need to be addressed in the scope of seismic qualification consistent with the guidance in NUREG-0800, Section 3.10. Therefore, in RAI 161, Question 3.10-16, the staff requested that the applicant provide additional information concerning the objective of the seismic monitoring system for the U.S. EPR design. In a February 27, 2009, response to RAI 161, Question 3.10-16, the applicant clarified that the U.S. EPR design does not include an automatic seismic SCRAM capability. Accordingly, with this response the staff considers RAI 161, Question 3.10-16 resolved.

ITAAC: In accordance with SRP Section 3.10, the staff reviewed the applicant's FSAR Tier 1 information concerning proposed ITAAC that are to provide reasonable assurance that a plant incorporating the U.S. EPR design certification is built and will operate in accordance with the design certification and NRC regulations. Relevant to seismic qualification, the staff's evaluation of the proposed ITAAC focused on the seismic performance, as qualified, and preservation of the safety functions of electrical, instrumentation and control (I&C), and mechanical equipment under seismic loads. Additional information on ITAAC, provided by the applicant in FSAR Tier 2, Section 14.3.3, and the staff's evaluation is provided in Section 14.3.3 of this report.

Evaluation approach: The staff performed its review and evaluation in accordance with the criteria and procedures delineated in SRP Section 3.10 (Revision 3, March 2007).

During the course of the review, the staff raised many technical issues through RAIs. The technical issues of minor or no safety significance have been resolved to the staff's satisfaction without having to revise the FSAR. This report addresses only those RAIs involving more significant safety issues and those which resulted in revision of the FSAR.

In FSAR Tier 2, Section 3.10.1.3, "Acceptance Criteria," the applicant indicates, as one acceptance criterion, that seismic qualification should demonstrate that the equipment is capable of performing its safety-related functions when subjected to normal operating loads or the maximum expected seismic loads (e.g., the SSE loads). 10 CFR Part 50, Appendix A, "General Design Criteria for Nuclear Power Plants," GDC 2, requires that design bases for equipment shall reflect appropriate combinations of the effects of normal and accident conditions together with the effects of natural phenomena (without loss of capability to perform their safety functions). 10 CFR Part 50, Appendix B, Section III, requires that a testing program

shall include suitable qualifications testing of a prototype unit under the most adverse design conditions. To meet these requirements, SRP Section 3.10 indicates that seismic qualification should consider the full range of normal and accident loadings. The staff finds that the information contained in FSAR Tier 2, Sections 3.10 and 3.9.3, "ASME Code Class 1, 2, and 3 Components, Component Supports, and Core Support Structures," does not discuss in enough detail combining of seismic loads with loads from other accident conditions and normal operating conditions. Therefore, in RAI 161, Question 3.10-2, the staff requested that the applicant revise the submittal to provide a specific description of the combined load cases involving seismic loads, and to clearly explain how these combined load effects can be suitably addressed in seismic qualification tests and/or analyses for the various categories of mechanical and electrical equipment.

In a February 27, 2009, response to RAI 161, Question 03.10-2, the applicant partially addressed the question, stating that FSAR Tier 2, Section 3.10.1.3 will be revised to add accident load conditions as part of the acceptance criteria for seismic qualification of electrical and mechanical component. However, the staff maintains that the information provided in FSAR Tier 2, Section 3.9.3 related only to pressure-retaining components, their supports, and the core-support structures (i.e., components in the ASME B&PV Code), whereas, guidance in SRP Section 3.10 specifies that full load combinations be considered in seismic qualification testing and analysis for all mechanical and electrical equipment. Therefore, in RAI 291, Question 03.10-24, the staff requested that the applicant provide supplemental information that describes explicitly the load combinations to be considered for all Seismic Category I, mechanical (non-pressure-retaining components), electrical, and I&C equipment not covered by ASME B&PV Code and IEEE Std 344.

In a May 12, 2010, response to RAI 291, Question 03.10-24, the applicant stated that FSAR Tier 2, Section 3.10, identifies sections that interface with it and specifically provides a reference to FSAR Tier 2, Section 3.9.3, regarding the definition of design and service-loading combinations for mechanical and electrical equipment. FSAR Tier 2, Section 3.10.1.3 was revised to add accident load conditions as part of the acceptance criteria for seismic qualification of electrical, instrumentation, and mechanical components. In addition, the applicant stated, that non-pressure retaining mechanical, electrical, and I&C equipment and their supports will be subject to the same applicable loads and load combinations as the pressure retaining equipment as identified in FSAR Tier 2, Section 3.9.3. FSAR Tier 2, Section 3.10, was revised to provide this clarification. The staff considers the applicant's responses to RAI 161, Question 3.10-2 and RAI 291, Question 3.10-24 acceptable. The staff confirmed that Revision 2 of the U.S. EPR FSAR, dated August 31, 2010, contains the changes committed to in the RAI responses. Accordingly, the staff finds that the applicant has adequately addressed this issue and, therefore, the staff considers RAI 161, Question 3.10-2 and RAI 291, Question 3.10-24 resolved.

FSAR Tier 2, Section 3.10, states that, aside from loss of offsite power, no other extraordinary events or accidents (including LOCAs, high-energy line breaks, and other events) are postulated to occur together with the SSE. The submittal also cites NUREG-1030, "Seismic Qualification of Equipment," and European Utility Requirements as bases for excluding consideration of the simultaneous occurrence of a LOCA with a seismic event. Such approaches are not in accordance with NRC regulations in 10 CFR Part 50, Appendix A, GDC 2 and GDC 4. To meet these regulations, SRP Section 3.10 guidance describes a LOCA and other appropriate accident conditions, to be considered in combination with a seismic event. Therefore, the staff does not find the applicant's approach of excluding the occurrence of LOCAs and other postulated accident conditions in combination with an SSE event to be

justified. Therefore, in RAI 161, Question 03.10-4, the staff requested that the applicant revise FSAR Tier 2, Section 3.10, to provide a description of procedures for addressing LOCAs and other accident conditions in combination with seismic events, or alternatively, provide additional information that clearly demonstrates justifiable basis for excluding consideration of LOCAs and other appropriate accident conditions in combination with seismic events.

In a February 27, 2009, response to RAI 161, Question 03.10-4, the applicant cited NUREG-1030 and European Utility Requirements as the basis for excluding consideration of the simultaneous occurrence of a LOCA with the SSE. The staff observed that NUREG-1030 is for operating plants only and that neither reference is applicable to new nuclear power plants. In addition, the applicant cited several occasions where the NRC had accepted the conditions that simultaneous occurrence of a LOCA and a seismic event was not required. The staff determined that the examples provided were for operating plants and a result of special considerations, and were not applicable to new reactors. For new reactor applications, SRP Section 3.10 Acceptance Criteria (1)(B), Design Adequacy of Supports, Item (ii) indicates that the combined stresses of the support structures should be in accordance with the criteria specified in SRP Section 3.9.3, and Item 7 in Table I, "Allowable Service Stress Limits for Specified Service Loading Combinations for ASME Code, Section III, Class 1, 2, and 3 Components, Component Supports, and Core Support Structures," of SRP Section 3.9.3 clearly shows that the Faulted Condition (LOCA+SSE) must be satisfied. Therefore, in follow-up RAI 291, Question 03.10-26, the staff requested that the applicant conform to the SRP acceptance criteria mentioned above.

In a May 12, 2010, response to RAI 291, Question 03.10-26, the applicant revised FSAR Tier 2, Section 3.10, to indicate that the information addressing combined stresses (LOCA+SSE) of support structures comply with the criteria specified in SRP Section 3.9.3, Table I, Item 7. The staff considers the applicant's responses to RAI 161, Question 3.10-4 and RAI 291, Question 3.10-26 acceptable. The staff confirmed that Revision 2 of the U.S. EPR FSAR, dated August 31, 2010, contains the changes committed to in the RAI response. Accordingly, the staff finds that the applicant has adequately addressed these issues and, therefore, the staff considers RAI 161, Question 3.10-4 and its follow up RAI 291, Question 3.10-26 resolved.

In FSAR Tier 2, Section 3.10.1.3, the applicant states that "some" permanent deformation of component supports and structures is acceptable in seismic qualification. The staff considered this statement to be overly vague and potentially inconsistent with NRC regulations and guidance. Therefore, in RAI 161 Question 3.10-5, the staff requested that the applicant clarify and justify this statement. In a February 27, 2009, response to RAI 161 Question 03.10-5, the applicant revised FSAR Tier 2, Section 3.10.1.3, deleting the reference to permanent deformation of component supports and structures. The staff confirmed that Revision 2 of the U.S. EPR FSAR, dated August 31, 2010, contains the changes committed to in the RAI response. Accordingly, the staff finds that the applicant has adequately addressed this issue and, therefore, the staff considers RAI 161 Question 3.10-5 resolved.

The introduction to FSAR Tier 2, Section 3.10, identifies a number of assumptions that appear unclear to the staff or not clearly justified, and are cited by the applicant as the basis for determining the scope of equipment to be included in the seismic qualification program. These assumptions include the following: (1) The single failure criterion is applied; and (2) exclusion of the following equipment types: (i) Equipment which could operate, but does not need to operate, and which, upon loss of offsite power, will fail in the desired position or state; and (ii) self-actuated check valves and manual valves. In RAI 161, Question 3.10-7, the staff requested

that the applicant clarify how the single failure criterion is applied, and provide the basis for excluding those equipment listed.

In a February 27, 2009, response to RAI 161, Question 3.10-7, the applicant stated that the single failure criterion assumption indicates that the safe shutdown list includes more than a single train. The applicant found this clarification unnecessary and deleted it from FSAR Tier 2, Section 3.10. The applicant also deleted the proposed exclusion of equipment types from FSAR Tier 2, Section 3.10. The staff considers the applicant's response acceptable. The staff confirmed that Revision 2 of the U.S. EPR FSAR, dated August 31, 2010, contains the changes committed to in the RAI response. Accordingly, the staff finds that the applicant has adequately addressed these issues and, therefore, the staff considers RAI 161, Question 3.10-7 resolved.

FSAR Tier 2, Table 3.10-1, includes a list of all Seismic Category I and II components in the systems screened for seismic qualification, but FSAR Tier 2, Section 3.10, does not discuss potential Seismic Category II/I issues in terms of influences on scope of equipment. SRP Section 3.10 indicates that equipment whose failure "can prevent the satisfactory accomplishment" of any essential safety function (whether for Seismic Category II/I or other reasons) should also be included in the scope of the seismic and dynamic qualification of electrical and mechanical equipment. Therefore, in RAI 161, Question 03.10-8, the staff requested that applicant provide a list of such components, and in each case, sufficiently describe the potential situation of concern.

In a February 27, 2009, response to RAI 161, Question 03.10-8, the applicant stated that, as indicated in the "Notes" section at the end of FSAR Tier 2, Table 3.10.1-1, components with a designation "SII" are classified as Seismic Category II. As defined in FSAR Tier 2, Section 3.2.1.2, "Seismic Category II," "U.S. EPR SSC classified as Seismic Category II are designed to withstand SSE seismic loads without incurring a structural failure that permits deleterious interaction with any Seismic Category I SSC or that could result in injury to main control room occupants. The seismic design criteria that apply to Seismic Category II SSC are addressed in Section 3.7." Therefore, FSAR Tier 2, Table 3.10.1-1 includes the scope of equipment addressed in the question. The staff finds this response acceptable, and considers RAI 161, Question 3.10-8 resolved.

The applicant's description of fractional SSE events (in Subsections E.4.4, E.5, and E.5.2.3 of Attachment E to Appendix 3D of Revision 0 of the U.S. EPR FSAR, dated December 11, 2007), to address low-cycle fatigue effects, contains apparent discrepancies (or perhaps a typographical mistake). For example, FSAR Tier 2, Section 3.7, "Seismic Design," (referenced from Section 3.10) indicates that earthquake cycles included in the fatigue analysis are composed of five one-third SSE (i.e., five OBEs) events followed by one full SSE event. However, the submittal (Revision 0 of the U.S. EPR FSAR) subsequently states that a number of fractional peak cycles equivalent to the maximum peak cycles for five one-half SSE (i.e., five OBEs) events may be used in accordance with IEEE Std 344-2004 Appendix D when followed by one full SSE event. As a result of the statements mentioned above, the applicant's proposed approach is not clearly and adequately described. Therefore, in RAI 161, Question 03.10-10, the staff requested that the applicant provide a definitive, consistent, and complete statement concerning the proposed treatment of fatigue effects in the seismic qualification of electrical equipment by testing (including instrumentation and control), which accords with appropriate regulatory guidance (i.e., five one-half SSE as delineated in SECY-93-087, section on elimination of OBE).

In a February 27, 2009, response to RAI 161, Question 3.10-10, the applicant revised FSAR Tier 2, Appendix 3D, Attachment E, Sections E.4.4 and E.5, to specify that the fatigue analyses are composed of five one-half SSE events followed by one full SSE event. This is consistent with the response to RAI 108, Question 03.07.03-19, for a corresponding change to FSAR Tier 2, Section 3.7.3.2. FSAR Tier 2, Appendix 3D, Attachment E, Section E.5.2.3, was revised to clarify the sine sweep and sine-beat test requirements. The sine sweep test method is used for the OBE event, which is at two-thirds of the required input motion (RIM) curve or two-thirds SSE level. The sine sweep test is followed by a sine-beat test used for the SSE event, which is at the RIM curve or full SSE level. The staff finds the applicant's response acceptable. The staff confirmed that Revision 2 of the U.S. EPR FSAR, dated August 31, 2010, contains the changes committed to in the RAI response. Accordingly, the staff finds that the applicant has adequately addressed this issue and, therefore, the staff considers RAI 161, Question 3.10-10 resolved.

The staff stated (as captured In RAI 161, Question 3.10-13) that, although FSAR Tier 2, Section 3.10 notes that instrumentation and control equipment are included in the scope of the seismic qualification program, the submittal cites FSAR Tier 2, Sections 7.5, "Information Systems Important to Safety," and 3.11, "Environmental Qualification of Mechanical and Electrical Equipment," for further information regarding I&C. FSAR Tier 2, Section 7.5 indicates that the TELEPERM XS digital I&C system is employed for U.S. EPR, but does not provide adequate seismic qualification approach for the equipment. The staff noticed that the test spectrum used for seismic qualification of the TELEPERM XS digital I&C equipment appears to be inconsistent with the U.S. EPR required seismic spectra for the equipment, in particular, the frequency range of the seismic spectra is inconsistent. Therefore, in RAI 161 Question 3.10-13, the staff requested that the applicant provide more detailed information to justify the use of single axis testing, not considering the potential coupling effects of the equipment axes, and also to justify the overall seismic adequacy of the instrument and control devices in the TELEPERM XS system.

In a February 27, 2009, response to RAI 161, Question 03.10-13, the applicant addressed this issue by identifying NRC Safety Evaluation Report (SER), Section 2.1.2.2 for Topical Report EMF-2110(NP)(A), Revision 1, which provided the results of the staff's review of Topical Report EMF-2110 (NP)(A), Revision 1, "TELEPERM XS: A Digital Reactor Protection System," and accompanying proprietary documents. The Siemens Power Corporation submitted this topical report by letter dated September 1, 1999.

In follow-up RAI 291, Question 3.10-29, the staff further described the issue as follows:
(1) The SER identified that the input excitation for testing the TELEPERM XS equipment was multiple frequency ranging from 5 to 35 Hz, three axes and each staggered by 90 degrees. However, by testing one axis at a time, the results may not be the same as multi-axis testing at the same time due to the potential effect of equipment directional coupling and (2) The SER (for Topical Report EMF-2110(NP)(A), Revision 1) stated that "A US licensee that use the TELEPERM XS system for a safety system application should compare its required seismic qualification level to the Siemens' qualified level, and identify areas requiring further action." The action is necessary to address any potential deficiencies.

The staff determined that the applicant did not adequately respond to the questions as stated above in the February 12, 2010, response to RAI 291, Question 3.10-29 Therefore, in follow-up RAI 384, Question 3.10-31, the staff requested that the applicant provide the following:
(1) As discussed in RAI 291, Question 03.10-29, the staff requested that the applicant submit the seismic qualification report (by Siemens) including the criteria and descriptions of the test

procedures together with the detailed seismic test results, and provide the justification, as required by the criteria delineated in Appendix 3D Subsection E.5.1.1, FSAR Tier 2, Attachment E, for using the single-axis testing one at a time for three times, and (2) The applicant stated in their RAI 291, Question 3.10-29 response that the COL applicant/licensee will demonstrate that the generic qualification bounds the plant-specific condition. However, the staff questions whether the tested spectrum (from 5 to 35 Hz) would envelop the U.S. EPR certified seismic design response spectra. As indicated in SRP Section 3.10, (1)(A)(iv), the tested response spectrum (TRS) should closely resemble and envelop the RRS over the critical frequency range. In a December 18, 2009, response to RAI 248, Question 03.07.02-44, the applicant indicated that the U.S. EPR CSDRS will be revised to include the Bell Bend Nuclear Power Plant (BBNPP). In this response, FSAR Tier 2, Figure 03.07.02-44-1, "Design Response Spectra for EUR Control Motions (hard, medium, soft sites) and Bell Bend Nuclear Power Plant Site – Horizontal and Vertical Directions, 5% Damping," was provided to show the new CSDRS including the higher frequency content of the BBNPP curve. The staff notes that FSAR Tier 2, Figure 03.07.02-44-1, indicates that the new CSDRS has significant frequency content exceeding 35 Hz. Therefore, in follow-up RAI 384, Question 3.10-31, the staff requested that the applicant, address the issue of high frequency input excitation exceedance over the tested limit of 35 Hz in the Topical Report EMF-2110(NP)(A), Revision 1, and accompanying proprietary documents.

In a September 7, 2010, response to RAI 384, Question 3.10-31, the applicant stated that (a) the applicant does not use the European single axis seismic qualification test for U.S. seismic qualification of TELEPERM XS. Seismic qualification of TELEPERM XS uses random multi-frequency, tri-axial testing. This testing is performed to meet U.S. standards. Hardware qualification will be performed for each generation of TELEPERM XS used in a U.S. EPR, in accordance with the process described in FSAR Tier 2, Section 3.10. The requirements for this process include seismic qualification using RRS that at a minimum envelop the specific U.S. EPR sites under consideration, including frequencies beyond 35 Hertz, as applicable, and may also envelop spectra for multiple sites. This process will conform to IEEE Std 344-2004 requirements, as stated in FSAR Tier 2, Section 3.10. (b) The generic qualification documented in Topical Report EMF-2110(NP)(A) does not bound the U.S. EPR-specific conditions for seismic qualification. Plant-specific action Item 1 (specified in the SER for Topical Report EMF-2110) will be satisfied by seismic qualification using RRS that at a minimum envelop the site-specific U.S. EPR, including frequencies beyond 35 Hz, as applicable, and not by demonstrating that the generic qualification bounds U.S. EPR-specific conditions. Additionally, ITAAC exist for seismic qualification (e.g., FSAR Tier 1, Table 2.4.1-7, Item 3.1). Based on the applicant's responses to RAI 161, Question, 3.10-13, RAI 291, Question 3.10-29, and RAI 384, Question 3.10-31, as described above, the staff considers the applicant's responses acceptable, and the RAIs resolved.

Digital I&C equipment qualification involves evaluation of a number of new and unique components and elements not previously encountered in qualification of older analog I&C. Additionally, seismic events present potentially unique challenges to digital I&C systems and components. For instance, accident monitoring and control equipment that support functionality in case of a seismic event will generally include distributed networked sensors and actuators, some of which may include embedded software. Assurance of proper functionality of digital I&C will involve requirements for digital electronic computing hardware; digital sensors, integrated software; human interaction as regards to configuration, maintenance, and intervention (e.g., potential intervention and/or recovery in case of seismic events); integrated performance of components, and other elements. With respect to seismic qualification per guidance in SRP Section 3.10, FSAR Tier 2, Section 3.10 does not include a sufficient delineation of the

components of digital I&C that will be subject to seismic qualification, nor a sufficient description of criteria, for determining successful functionality at the component level. Therefore, in RAI 161, Question 3.10-14, the staff requested that the applicant provide additional information to identify digital I&C components and justify their seismic qualification in sufficient detail to ensure that NRC regulations are met. Furthermore, for each identified digital I&C component including any non-hardware components/elements (whether integrated, embedded, installed, etc.) that are needed to ensure proper functionality of any digital I&C component under seismic conditions, the applicant should provide complete specifications as to the behavioral and state parameters that define proper functionality of the component and associated success criteria for purposes of seismic qualification.

In a February 27, 2009, response to RAI 161, Question 3.10-14, the applicant stated that FSAR Tier 2, Table 3.11-1 identifies digital I&C equipment that will be seismically qualified. For example, FSAR Tier 2, Table 3.11-1, Page 3.11-84 identifies equipment for the protection system (PS) and their seismic qualification level. As shown in FSAR Tier 2, Table 3.11-1, the PS cabinets are seismically qualified which includes the digital I&C components contained in the cabinets. Additionally, ITAAC verify that the safety-related I&C equipment is seismically qualified (see FSAR Tier 1, Sections 2.4.1, 2.4.2, 2.4.4, and 2.4.5). The applicant further noted that, as part of the seismic qualification, a test plan is developed that describes the specifications and acceptance criteria. The acceptance criteria for the ITAAC identified includes the existence of a test report that concludes that the equipment can withstand seismic design basis loads without loss of safety function. The staff considers the applicant's response acceptable, and RAI 161, Question 3.10-14 resolved.

FSAR Tier 2, Section 3.7.1, "Seismic Design Parameters," proposes the use of three control ground motions (EUR control motions) that are representative of common general safety requirements for European site conditions. These motions were not developed according to any NRC regulatory guidance, and the FSAR does not adequately clarify how these three control motions will be used for developing realistic input motions (representing the HF input for CEUS sites) for seismic qualification of U.S. EPR equipment. Additionally, for purposes of certification of a standard design for U.S. EPR, the staff notes that the applicant needs to clarify whether the seismic qualification testing will be done once for an enveloping of the in-structure responses and effects of all three control motions, or will be done three times to address the specific responses and effects for each of the three control motions. Further, the staff finds that the information provided in the FSAR Tier 2, Section 3.7.1, "Seismic Design Parameters," does not adequately demonstrate that the input motions (e.g., time histories or floor response spectra at equipment locations) will suitably represent the character (including high frequency effects) of motions expected at CEUS sites. Therefore, in RAI 161, Questions 03.10-17 and 03.10-18, the staff requested that the applicant fully clarify, in relation to the effects on motions used for seismic qualification, the applicability of the EUR control ground motions to NRC regulations, and how the three control motions of the standard design for the U.S. EPR will be addressed in the applicant's seismic qualification program, including suitable clarification and justification of the development of input motions, or sets of input motions, at equipment mounting locations.

In an October 27, 2009, responses to RAI 161, Questions 03.10-17 and 03.10-18, the applicant stated that the response to RAI 248, Question 03.07.02-44, will address the high frequency ground motions for the U.S. EPR design for the COLA sites in accordance with the recommendation of DC/COL-ISG-1, "Interim Staff Guidance on Seismic Issues of High Frequency Ground Motion." The applicant further stated that the response to RAI 248, Question 03.07.02-44, will also include the revised CSDRS, which will include the addition of ground motions to represent high frequency content at identified COL sites, and will be reflected

in a revision to FSAR Tier 2, Section 3.7.1.1. The applicant also stated that seismic qualification testing will be done once for an envelope of the in-structure response spectra resulting from the entire set of revised certified seismic design response spectra (which will constitute a part of the response to RAI 248 Question, 03.07.02-44) including additional ground motions for the COL sites with high frequency content. The staff notes that FSAR Tier 2, Section 3.10.1, was revised to reflect the information in the response to RAI 161, Questions 03.10-17 and 03.10-18. Therefore, in view of the above responses, the staff considers the applicant's responses acceptable. The staff confirmed that Revision 2 of the U.S. EPR FSAR dated August 31, 2010, contains the changes committed to in the RAI response. Accordingly, the staff finds that the applicant has adequately addressed this issue and, therefore, the staff considers RAI 161, Questions 03.10-17 and 03.10-18 resolved.

The staff notes that FSAR Tier 2, Section 3.10, does not have a sufficiently detailed and complete description of the proposed approach for seismic and dynamic qualification of supports for mechanical and electrical equipment (including I&C), according to the guidance in SRP Section 3.10, Subsection II.1.B. Therefore, in RAI 161, Question 03.10-21, the staff requested that the applicant revise FSAR Tier 2, Section 3.10, to suitably address requirements for design adequacy of supports, in a manner meeting the guidance in SRP Section 3.10 or NRC regulations.

In a February 27, 2009, response to RAI 161, Question 03.10-21, the applicant stated that equipment supports for seismic qualification testing are rigid, where flexibility of the supports can be eliminated. The staff determined that this claim is not realistically possible and cannot even be approximately achievable for many cases (e.g., electrical cabinets and racks). SRP Section 3.10 identifies that for establishing design adequacy of supports, analyses or tests should be performed for all supports of mechanical and electrical equipment to ensure their structural capability. While the applicant provided some information in FSAR Tier 2, Section 3.10.3, about qualification by analysis and testing, the section neither reflected the guidance provided in SRP Section 3.10, Acceptance Criteria (1)(B)(ii) and (iii), nor provided an alternative to demonstrate design adequacy of supports. Therefore, in follow-up RAI 291, Question 03.10-30, the staff requested that the applicant revise FSAR Tier 2, Section 3.10.3, to properly address requirements for design adequacy of supports, in a manner which meets SRP Section 3.10.

In a February 11, 2010, response to RAI 291, Question 03.10-30, the applicant revised FSAR Tier 2, Section 3.10.3, so that rather than generically refer to equipment supports, at least two types are recognized: one type for electrical equipment mounted on flexible structural metal enclosures such as cabinets and racks, and the other type for mechanical equipment and components such as pumps and motors mounted on substantial steel or concrete bases. The applicant stated that the adequacy of equipment supports, either light electrical enclosures or heavy-bases for equipment mounts, will be established by analysis or test in accordance with the guidance in SRP Section 3.10, Acceptance Criteria, (II)(1)(B). Flexible structures will be analyzed or tested as a unit to include the interaction between the supports and the equipment. For non-flexible structures, heavy-bases for equipment mounts may be decoupled from the supported equipment if it can be shown by testing or analysis that it is a rigid base and will not amplify the input motion, in-structure response spectra.

Regarding the adequacy of support design methodology, the applicant further stated in the February 11, 2010, response to RAI 291, Question 03.10-30, that equipment supports will be tested with equipment installed or with a dummy weight simulating the equivalent equipment inertial mass effects and dynamic coupling to the support in accordance with the guidance

provided in SRP Section 3.10, Acceptance Criteria (II)(1)(B)(iii). When performing analysis, the combined stresses of equipment supports will be in accordance with SRP Section 3.9.3. When complete testing is not practicable, simple and passive equipment may be analyzed to confirm their structural integrity under postulated event loadings. However, complex active devices, which are vital to the operation of equipment, should be additionally monitored and/or tested for functionality. For qualification of equipment supports, input motions are represented by ISRS curves at the equipment support mounting locations.

The detailed methodology for derivation of ISRS is discussed in FSAR Tier 2, Section 3.7.2.5, "Development of Floor Response Spectra." The applicant further stated in the February 11, 2010, response to RAI 291, Question 03.10-30, that the RRS which are based on the ISRS, shall include a 1.4 performance-based factor for the critical equipment during severe accident scenarios in accordance with ASCE/SEI 43-05, Section 8.3.2. The 10 percent margin for uncertainties required by IEEE Std 323 in the RRS is included in the 1.4 factor described above. The TRS closely resemble and envelop the RRS. The seismic and dynamic qualification testing for supports performed using single-axis or multi-axes test methods is in accordance with IEEE Std 344-1987. The staff considers the applicant's responses acceptable, and RAI 161, Question 3.10-21 and RAI 291, Question 3.10-30 resolved.

FSAR Tier 2, Section 3.10.2.1.1, indicates that alternative testing methods, such as single frequency and single-axis testing, are permissible in some cases. The staff considers that such testing methods have very limited applicability, and accordingly, the staff believes that it is important to specifically identify and consider such cases. Therefore, in RAI 161, Question 03.10-11, and RAI 291, Question 03.10-28, the staff requested that the applicant provide clarifying information to justify the use of these limited methods. The staff noted that SRP Section 3.10, Acceptance Criteria (1)(A)(vi), indicate that the use of single-axis testing should be justified. The staff also requested the applicant (i) to revise FSAR Appendix 3D, Attachment E, Section E.5.1.1 indicating that justification of using single-axis testing will be provided in the SQDP, and (ii) to modify Tab H of the SQDP in Appendix 3D, Attachment F, to include a note showing that justification will be provided if single-axis testing is selected.

In a February 27, 2009, response to RAI 291, Question 03.10-28, for the first part of the question, the applicant revised FSAR Tier 2, Appendix 3D, Attachment E, and Section E.5.1.1, to indicate that the justification of using single-axis testing is to be included in the SQDP, Tab H. The referenced section is revised as follows: "Single-axis testing can be used when it is demonstrated that a component responds independently in the three orthogonal directions and there is low or no cross coupling between the axes. It can also be used when a device is normally installed on a panel that amplifies motion in one direction only, or when it is restrained to motion in one direction. When single-axis testing is used, justification for its use shall be provided in the SQDP, Tab H." For the second part of the question, the applicant modified Tab H of the SQDP in Appendix 3D, Attachment F, to add Subsection Number H.15 to document the justification when using single-axis testing. The staff considers the applicant's response acceptable. The staff confirmed that Revision 2 of the U.S. EPR FSAR, dated August 31, 2010, contains the changes committed to in the RAI responses. Accordingly, the staff finds that the applicant has adequately addressed these issues and, therefore, the staff considers RAI 161, Question 3.10-11, and RAI 291, Question 3.10-28 resolved.

3.10.5 Combined License Information Items

Table 3.10.5-1 provides a list of Seismic and Dynamic Qualification of Mechanical and Electrical Equipment related COL information item numbers and descriptions from FSAR Tier 2, Table 1.8-2:

Table 3.10.5-1 U.S. EPR Combined License Information Items

Item No.	Description	FSAR Tier 2 Section
3.10-1	A COL applicant that references the U.S. EPR design certification will create and maintain the SQDP file during the equipment selection and procurement phase.	3.10.4
3.10-2	A COL applicant that references the U.S. EPR design certification will identify any additional site-specific components that need to be added to the equipment list in Table 3.10-1.	3.10.1.1
3.10-3	If the seismic and dynamic qualification testing is incomplete at the time of the COL application, a COL applicant that references the U.S. EPR design certification will submit an implementation program, including milestones and completion dates, for NRC review and approval prior to installation of the applicable equipment.	3.10.4

The staff finds the above listing to be complete. Also, the list adequately describes actions necessary for the COL applicant. No additional COL information items need to be included in FSAR Tier 2, Table 1.8-2 for seismic and dynamic qualification of mechanical and electrical equipment.

3.10.6 Conclusions

The staff's review of FSAR Tier 2, Section 3.10, for compliance with NRC regulations for seismic qualification of safety-related mechanical and electrical equipment (including I&C), finds the applicant described the U.S. EPR seismic qualification program and its implementation in conformance to the applicable guidance and acceptance criteria delineated in SRP Section 3.10. The future ITAAC inspection will verify the adequacy of the results of seismic qualification of U.S. EPR TELEPERM XS and other Seismic Category I equipment. The staff concludes that the applicant's description of the U.S. EPR seismic qualification program for Seismic and Dynamic Qualification of Mechanical and Electrical Equipment ensures the design's compliance with the NRC regulations and conforms to relevant acceptance criteria found in the regulatory guides listed in Section 3.10.3, "Regulatory Basis," of this report.