AREVAEPRDCPEm Resource

From: WELLS Russell (AREVA) [Russell.Wells@areva.com]
Sent: Monday, March 07, 2011 10:53 AM
To: Tesfaye, Getachew
Cc: DELANO Karen (AREVA); ROMINE Judy (AREVA); BENNETT Kathy (AREVA); CORNELL Veronica (EXTERNAL AREVA); HALLINGER Pat (EXTERNAL AREVA); WILLIFORD Dennis (AREVA); RYAN Tom (AREVA); COLEMAN Sue (AREVA); BREDEL Daniel (AREVA)
Subject: Draft Response to U.S. EPR Design Certification Application RAI No. 448, FSAR Ch. 3, Questions 03.08.01-50-55
Attachments: RAI 448 Questions 3.8.1-50-55 Response US EPR DC - DRAFT.pdf

Getachew,

Attached are draft responses for RAI No. 448, FSAR Ch 3, Questions 03.08.01-50, 03.08.01-51, 03.08.01-52, 03.08.01-53, 03.08.01-54, 03.08.01-55 in advance of the March 18, 2011 final date. The draft responses address NRC comments from the U.S. EPR FSAR Section 3.8 audit held February 14 – 17, 2011.

Let me know if the staff has questions or if the draft responses can be sent as a final responses.

Sincerely,

Russ Wells
U.S. EPR Design Certification Licensing Manager
AREVA NP, Inc.
3315 Old Forest Road, P.O. Box 10935
Mail Stop OF-57
Lynchburg, VA 24506-0935
Phone: 434-832-3884 (work)
434-942-6375 (cell)
Fax: 434-382-3884
Russell.Wells@Areva.com

From: BRYAN Martin (External RS/NB)
Sent: Friday, February 11, 2011 3:18 PM
To: 'Tesfaye, Getachew'
Cc: DELANO Karen (RS/NB); ROMINE Judy (RS/NB); BENNETT Kathy (RS/NB); CORNELL Veronica (External RS/NB)
Subject: Response to U.S. EPR Design Certification Application RAI No. 448, FSAR Ch. 3, Supplement 1

Getachew,

AREVA NP Inc. (AREVA NP) provided a schedule for a technically correct and complete response to RAI 448 on November 22, 2010. To allow additional time to finalize the responses and interact with NRC staff, the schedule has been revised.

The schedule for a technically correct and complete response to these questions is provided below.

Question #	Response Date
RAI 448 — 03.08.01-49	March 25, 2011
RAI 448 — 03.08.01-50	March 18, 2011
RAI 448 — 03.08.01-51	March 18, 2011

RAI 448 — 03.08.01-52	March 18, 2011
RAI 448 — 03.08.01-53	March 18, 2011
RAI 448 — 03.08.01-54	March 18, 2011
RAI 448 — 03.08.01-55	March 18, 2011

Sincerely,

Martin (Marty) C. Bryan
U.S. EPR Design Certification Licensing Manager
AREVA NP Inc.
Tel: (434) 832-3016
702 561-3528 cell
Martin.Bryan.ext@areva.com

From: BRYAN Martin (External RS/NB)
Sent: Monday, November 22, 2010 10:13 AM
To: 'Tesfaye, Getachew'
Cc: DELANO Karen (RS/NB); ROMINE Judy (RS/NB); BENNETT Kathy (RS/NB); CORNELL Veronica (External RS/NB)
Subject: Response to U.S. EPR Design Certification Application RAI No. 448, FSAR Ch. 3

Getachew,

Attached please find AREVA NP Inc.'s response to the subject request for additional information (RAI). The attached file, "RAI 448 Response US EPR DC.pdf" provides a schedule since a technically correct and complete response to the 7 questions can not be provided at this time.

The following table indicates the respective pages in the response document, "RAI 448 Response US EPR DC.pdf," that contain AREVA NP's response to the subject questions.

Question #	Start Page	End Page
RAI 448 — 03.08.01-49	2	3
RAI 448 — 03.08.01-50	4	5
RAI 448 — 03.08.01-51	6	7
RAI 448 — 03.08.01-52	8	8
RAI 448 — 03.08.01-53	9	9
RAI 448 — 03.08.01-54	10	11
RAI 448 — 03.08.01-55	12	12

A complete answer is not provided for the 7 questions. The schedule for a technically correct and complete response to these questions is provided below.

Question #	Response Date
RAI 448 — 03.08.01-49	February 28, 2011
RAI 448 — 03.08.01-50	February 28, 2011
RAI 448 — 03.08.01-51	February 28, 2011
RAI 448 — 03.08.01-52	February 28, 2011
RAI 448 — 03.08.01-53	February 28, 2011
RAI 448 — 03.08.01-54	February 28, 2011
RAI 448 — 03.08.01-55	February 28, 2011

Sincerely,

Martin (Marty) C. Bryan
U.S. EPR Design Certification Licensing Manager
AREVA NP Inc.
Tel: (434) 832-3016
702 561-3528 cell
Martin.Bryan.ext@areva.com

From: Tesfaye, Getachew [mailto:Getachew.Tesfaye@nrc.gov]
Sent: Monday, October 25, 2010 4:41 PM
To: ZZ-DL-A-USEPR-DL
Cc: Xu, Jim; Hawkins, Kimberly; Miernicki, Michael; Colaccino, Joseph; ArevaEPRDCPEm Resource
Subject: U.S. EPR Design Certification Application RAI No. 448 (4898, 5084),FSAR Ch. 3

Attached please find the subject requests for additional information (RAI). A draft of the RAI was provided to you on September 17, 2010, and discussed with your staff on October 25, 2010. No changes were made to the draft RAI as a result of that discussion. The schedule we have established for review of your application assumes technically correct and complete responses within 30 days of receipt of RAIs. For any RAIs that cannot be answered within 30 days, it is expected that a date for receipt of this information will be provided to the staff within the 30 day period so that the staff can assess how this information will impact the published schedule.

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

Hearing Identifier: AREVA_EPR_DC_RAIs
Email Number: 2661

Mail Envelope Properties (1F1CC1BBDC66B842A46CAC03D6B1CD41040EC4A0)

Subject: Draft Response to U.S. EPR Design Certification Application RAI No. 448, FSAR Ch. 3, Questions 03.08.01-50-55
Sent Date: 3/7/2011 10:53:03 AM
Received Date: 3/7/2011 10:53:27 AM
From: WELLS Russell (AREVA)

Created By: Russell.Wells@areva.com

Recipients:
"DELANO Karen (AREVA)" <Karen.Delano@areva.com>
Tracking Status: None
"ROMINE Judy (AREVA)" <Judy.Romine@areva.com>
Tracking Status: None
"BENNETT Kathy (AREVA)" <Kathy.Bennett@areva.com>
Tracking Status: None
"CORNELL Veronica (EXTERNAL AREVA)" <Veronica.Cornell.ext@areva.com>
Tracking Status: None
"HALLINGER Pat (EXTERNAL AREVA)" <Pat.Hallinger.ext@areva.com>
Tracking Status: None
"WILLIFORD Dennis (AREVA)" <Dennis.Williford@areva.com>
Tracking Status: None
"RYAN Tom (AREVA)" <Tom.Ryan@areva.com>
Tracking Status: None
"COLEMAN Sue (AREVA)" <Sue.Coleman@areva.com>
Tracking Status: None
"BREDEL Daniel (AREVA)" <Daniel.Bredel@areva.com>
Tracking Status: None
"Tesfaye, Getachew" <Getachew.Tesfaye@nrc.gov>
Tracking Status: None

Post Office: AUSLYNCMX02.adom.ad.corp

Files	**Size**	**Date & Time**
MESSAGE	5034	3/7/2011 10:53:27 AM
RAI 448 Questions 3.8.1-50-55 Response US EPR DC - DRAFT.pdf	1881065	

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

Response to

Request for Additional Information No. 448(4898, 5084), Revision 0
Question 03.08.01-50
Question 03.08.01-51
Question 03.08.01-52
Question 03.08.01-53
Question 03.08.01-54
Question 03.08.01-55

10/25/2010

U. S. EPR Standard Design Certification
AREVA NP Inc.
Docket No. 52-020
SRP Section: 03.08.01 - Concrete Containment
Application Section: 3.8.1

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

Question 03.08.01-50:

Follow-up to RAI 155, Question 3.8.1-12

The RAI response has provided additional information regarding the U.S. EPR ISI program. The staff has evaluated the response and determined that the information provided is inadequate with respect to meeting 10 CFR 50.55a and 10 CFR 50, Appendix A, GDC 1, as they relate to concrete containment being designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety function to be performed, and as described in SRP 3.8.1.II.7.D and RG 1.90. The staff requests that the applicant provide clarifications as discussed below:

a. Regarding the criterion for using Pa as ISI test pressure in years 3 and 7, instead of 1.15Pd indicated in RG 1.90, the response states that: (a) using 1.15Pd as initial structural integrity test (ISIT) pressure confirms containment integrity and quality of construction; (b) continued pressurization of the containment to 1.15Pd would induce "unnecessary cyclic loading of the structure;" and (c) using Pa instead of 1.15Pd as ISI test pressure, "will establish a continuous basis for comparison of results, will minimize gradual propagation of cracking during subsequent pressure tests, and will be in compliance with the ISI requirements of ASME BVP Code Subsection IWL, Paragraph IWL-5220."

   The above justification for the exception taken to the ISI test pressures stipulated in RG 1.90 for years 3 and 7 is inadequate for reasons explained in the following.

   The statement of compliance with the ISI requirements of ASME BVP Code Subsection IWL, Paragraph IWL-5220 is not appropriate because Article 5000 of the Code is applicable to pressure testing of containments following repair/replacement activities and not the periodic ISI pressure tests. Also, the response implies that using 1.15Pd as ISI test pressure for years 3 and 7 is unnecessarily conservative and possibly detrimental. However, the staff notes that one of the considerations for using Alternative B (deformation monitoring under pressure tests, also see Item 2 below) given in RG 1.90 is that the design of the containment should be demonstrated with adequate conservatism (i.e., membrane compression stresses maintained and maximum tensile stress in reinforcement limited to one half of the yield strength during ISI pressure tests) so that cracking under repeated ISI pressure tests is minimized. It follows that, whether 1.15Pd or Pa is used as the ISI test pressure, the containment should be designed to minimize cracking under repeated ISI pressure tests in either case. Consequently, the design and pressure testing of the containment should meet the regulatory positions in RG 1.90 or adequate technical justification, preferably supported by quantitative data, should be provided for the exception taken to RG 1.90.

b. Regarding the exception to RG 1.90, by which force monitoring of ungrouted test tendons is not provided, the response states that: (a) ungrouted test tendons are used to evaluate prestress losses due to concrete creep and shrinkage, and tendon steel relaxation; however, since the ungrouted test tendons will be subject to cyclic loading during every ISI, the measured results may not accurately reflect the prestress losses in the containment as a whole, and this has been acknowledged in the past by NRC (Information Notice 99-10, Attachment 3); (b) rather than using ungrouted test tendons for the force monitoring of prestress losses, the U.S. EPR ISI program will implement deformation monitoring of the containment under Pa pressure, and compare results with expected deformation and ISIT deformation; (c) deformation monitoring of the containment during ISI pressure testing has been accepted in the past by NRC (Three Mile Island and Forked River NPPs); and (d) the technical literature reports one instance (Quinshan NPP, China) where monitoring of the prestress level in the containment has been accomplished using overall deformation measurements as an alternative to tendon force measurements.

The response above gives insufficient technical justification for not providing force monitoring of ungrouted test tendons, as prescribed in RG 1.90. According to RG 1.90, the ISI program should consist of three distinct activities: (a) force monitoring of ungrouted test tendons; (b) periodic reading of instrumentation for determining prestress level (Alternative A) or monitoring of deformations under pressure (Alternative B) at preestablished sections; and (c) visual examination. Therefore, the deformation monitoring of the containment (item (b), Alternative B) does not eliminate the requirement to provide force monitoring of ungrouted test tendons (item (a)), but is an additional criterion of RG 1.90. Regarding Item (2)(c) in the above paragraph, explain how the acceptance by the NRC in two old NPPs for deformation monitoring of the containment during ISI tests demonstrates that monitoring of ungrouted tendons is not required. Regarding Item (2)(d) in the above paragraph, although the referenced paper reports an interesting case study where force monitoring was not used, the staff considers that it does not provide a technical basis for the exception taken to RG 1.90. Consequently, provide adequate technical justification, preferably supported by quantitative data, to demonstrate that ungrouted tendons are not needed.

c. FSAR Section 3.8.1.1 states that Pd is equal to 62 psig. The response to RAI 3.8.1-32 states that Pa (as used in the ISI) is set to 55 psig. This information should be added to FSAR Table 3.8-7 “ISI Schedule for the U.S. EPR.”

In addition, revise and update the relevant sections of the FSAR as needed to address the staff’s concerns listed above.

Response to Question 03.08.01-50:

Item a:

The U.S. EPR will use 1.15*P_d as the inservice inspection (ISI) test pressure in years three and seven as indicated in RG 1.90, Revision 1. U.S. EPR FSAR Tier 2, Table 3.8-7 will be revised to match the pressures provided in Figure 2 of RG 1.90, Revision 1.

Item b:

In accordance with RG 1.90, Revision 1, the tendons for the U.S. EPR will be included in an ISI program. The ISI program will consist of three items:

- Force monitoring of ungrouted test tendons.
- Monitoring of deformations under pressure at prescribed locations (Alternative B of RG 1.90, Revision 1).

- Visual inspection of exposed structurally critical areas of the containment and containment prestressing system.

  Visual inspection will be performed of representative areas at structural discontinuities, areas around large penetrations or a cluster of small penetrations and other areas where heavy loads are transferred to the containment structure. Visual inspection of these selected areas will be completed during the pressure tests while the containment is at its maximum test pressure. Samples of the exposed portions of the tendon anchorage assembly hardware will also be included in the visual inspections. The tendon anchorage assemblies utilized for the greased tendons will be representative of the grouted tendons except that provisions will be provided to allow force measurement by lift-off or load cells. The sample size of tendon anchorage assemblies will comply with the requirements of RG 1.90, Revision 1.

  Access to perform the visual inspections is provided from the tendon gallery, annular space between the containment building exterior wall and the reactor shield building wall, and the annular space between the containment building dome and reactor shield building dome shown in U.S. EPR FSAR Tier 1, Figure 3B-1.

The ISI Program will also include a fourth component for the inspection of the test tendons filler grease. The inspection of filler grease will be performed consistent with the guidance in RG 1.35, R3, Regulatory Position 6.

Three greased tendons of each type; vertical, gamma and horizontal hoop will be provided for force monitoring. These test tendons are included in the number of tendons required by design and will be subjected to force measurement by lift-off or load cells to assess the effects of concrete shrinkage and creep and relaxation of the tendon steel. The nine greased tendons are the sample size for load cell or lift-off testing.

In accordance with the Alternative B of RG 1.90, Revision 1, the points to be instrumented for measurement of radial displacements under pressure will be located in six horizontal planes in the cylindrical portion of the shell with a minimum of four points in each plane.

The points to be instrumented for measurement of vertical (or radial) displacements under pressure will be located at the top of the cylinder relative to the base, at a minimum of four approximately equally spaced azimuths. Locations will also be selected at the dome apex and one intermediate point between the apex and the springline on at least three equally spaced azimuths.

U.S EPR FSAR Tier 2, Table 1.9-2 and Table 3.8-7 as well as the text in U.S EPR FSAR Tier 2, Sections 3.8.1.1.2, 3.8.1.2.5, 3.8.1.6.3, 3.8.1.7.2 and Section 5.5 of the Technical Specifications will be updated to include this information.

Item c:

The requested information has been added to the U.S. EPR FSAR Tier 2, Table 3.8-7.

FSAR Impact:

The U.S. EPR FSAR, Tier 2, Tables 1.9-2 and 3.8-7, Sections 3.8.1.1.2, 3.8.1.2.5, 3.8.1.6.3, 3.8.1.7.2 and Section 5.5 of Chapter 16, Technical Specifications will be revised as described in the response and indicated on the enclosed markup.

Question 03.08.01-51:

Follow-up to RAI 155, Question 3.8.1-22

The response to this RAI explains that an FEM analysis of a typical 6-degree 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 FEM model are provided, including the computer code, the loading sequence, and the types of finite elements used in the analyses. Finally, the response indicates that the RCB is the only structure expected to develop a significant thermal gradient across its thickness; therefore, AREVA did not consider thermal loading for the RBIS, EPGB or ESWB.

To ensure compliance with 10 CFR 50, Appendix A, GDC 50, as it relates to the concrete containment being designed with sufficient margin of safety to accommodate appropriate design loads such as thermal loads, and as described in SRP 3.8.1.II.4.C and D, the staff finds that additional information is necessary to determine whether the approach used to reduce the thermal stresses in the RCB is conservative.

a. The RAI response states that the mesh density in the 6-degree slice FEM model is increased to calculate the change in thermal moments due to mesh refinement. Provide a description of this mesh refinement, include a figure of each model, and identify the relative sizes of the original vs. the refined mesh.

b. The RAI response indicates that a thermal modification factor due to mesh refinement was computed. Explain whether a single factor was used for the entire RCB, or multiple factors (e.g., different factor for each element or region) were used. If the latter is the case, also provide representative (max., min.) values of these modification factors and the elements/regions of the RCB to which they apply.

c. The RAI response indicates that thermal moments from the nonlinear FEM model, with concrete cracking included, are compared to the linear FEM model with the refined mesh and no concrete cracking, to determine the thermal modification factor due only to concrete cracking. Explain whether a single factor was used for the entire RCB, or multiple factors were used. If the latter is the case, also provide representative (max., min.) values of these modification factors and the elements/regions of the RCB to which they apply.

d. The final thermal moment reduction factor is calculated as the multiplication of the two thermal moment modification factors described in items 2 and 3 above. Again, explain whether a single factor was used for the entire RCB, or multiple factors were used. If the latter is the case, also provide representative (max., min.) values of these thermal moment reduction factors and the elements/regions of the RCB to which they apply.

e. Since the thermal modification factors are based on a nonlinear analysis (of the coarser-mesh FE model), identify the basis for stating that the final modification factors are simply the product of the thermal modification factors and the mesh refinement factors.

f. Explain how the thermal loads are applied to the nonlinear FEM model. The RAI response simply states that “the model is subjected to accidental pressure loads,” or “the model is subjected to accidental temperature and pressure loads.” However, it is not clear whether the analysis considered the variation of the temperature gradient across the containment thickness at the four critical time points identified in the temperature and pressure transient analysis, or whether the maximum temperature gradient was utilized. Also, it is not clear whether the analysis considered the additional internal pressure due to the thermal expansion of the liner plate.

Response to Question 03.08.01-51:

Items a through e:

The thermal modification factor is no longer used to reduce the thermal moment in the Reactor Containment Building (RCB). The moment in the concrete portion of the RCB as a result of the temperature gradient is calculated separately, consistent with the methodology in Reference 1. The methodology in Reference 1 is based on boundary conditions in which the concrete is restrained from rotation but not from longitudinal expansion. The methodology is applied to the calculation of the thermal moment in the containment wall and dome away from discontinuities, which have these boundary conditions. Additional axial forces resulting from the temperature gradient are not calculated since the boundary condition of unrestrained axial expansion from Reference 1 is applicable for the containment wall and dome away from discontinuities.

The temperature induced moments are determined using a two-step iterative inelastic process.

1. Initial strain and stress due to the applied loads (i.e., non thermal loads) are calculated using linear distribution of strain and non-linear distribution of stress according to Reference 2. The applied loads are the design loads found in U.S. EPR FSAR Tier 2, Section 3.8.1.3.2, with the exception of temperature.
2. The linear temperature gradient for the relevant area is calculated from the heat transfer analysis described in U.S. EPR FSAR Tier 2, Section 3.8.1.4.4. Using the initial strain and stress, the change in curvature due to the applied temperature gradient is calculated using the inelastic analysis described in Reference 1. The thermal moment is determined from the change in curvature of the section and is added to the moment due to non-thermal loads. The thermal moment is dependent on the amount of reinforcement in a given section. The process is iterated to determine the required reinforcement to resist the thermal loads and applied loads. The process is used to calculate the thermal moment for the concrete only. Therefore, additional internal pressure is applied because of the heating and expansion of the liner plate as indicated in U.S. EPR FSAR Tier 2, Section 3.8.1.4.4.

The following are exceptions for when the methodology described above cannot be used to calculate the thermal moment. Forces and moments due to thermal gradient for these conditions are determined using the methodology described in U.S. EPR FSAR Tier 2, Section 3.8.1.4.4.

- The initial strain distribution due to the applied loads, such as non thermal loads, is compressive (i.e. section is uncracked).
- The thermal moment causes the tension face of the section to switch sides from that of the applied loads (i.e., the thermal moment and moment from the applied loads are in opposite directions).
- When complete concrete tension across the face of the section (i.e., complete concrete cracking) results from the applied loads only or from the combination of the applied loads and the thermal moment. This is rare in a post-tensioned concrete containment.

- The boundary conditions for the buttress, ring girder, basemat, and gusset areas do not match the boundary conditions in Reference 1 and therefore cannot be used to calculate the thermal moment in these areas.

Item f:

Transient thermal analysis is performed using containment slice models, where temperature gradients through thickness of the containment are established at 27 time points over a one year period. The slice models are described in the Response to RAI 155, Question 03.08.01-22. As a result of the transient thermal analysis, the temperature distribution through the containment thickness is obtained for all time points. The accidental pressure loads accompanying the accident event are applied to each time step that corresponds to the step pressure magnitudes. From a different structural analysis, equivalent pressure due to liner thermal expansion is calculated at the same time points. Internal pressure resulting from the liner plate thermal expansion is added to the analysis concurrently with the accidental temperature loads.

References:

1. G. Gurfinkel, "Thermal Effects in Walls of Nuclear Containments – Elastic and Inelastic Behavior", Proceedings, First International Conference on Structural Mechanics in Reactor Technology, V. 5-J, pp 277-297, 1971.
2. G. Gurfinkel, and A. Robinson, "Determination of Strain Distribution and Curvature in a Reinforced Concrete Section Subjected to Bending Moment and Longitudinal Load", Journal of the American Concrete Institute, Vol. 64, No.7, July 1967

FSAR Impact:

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

Question 03.08.01-52:

Follow-up to RAI 155, Question 3.8.1-27

The response to this RAI provides additional information on the FEM analysis procedures used to model the thermal and pressure transients from LOCA events. The staff has evaluated the response and determined that the information provided is inadequate with respect to meeting 10 CFR 50, Appendix A, GDC 50, as it relates to the concrete containment being designed with sufficient margin of safety to accommodate appropriate design loads such as thermal and pressure loads, and as described in SRP 3.8.1.II.4.C and D. The staff requests that the applicant provide additional information necessary to determine whether the FEM analysis is conservative, as described below:

a. Item 1 of the RAI response indicates that a six degree slice of the containment is studied for mesh refinement in consideration with thermal moment calculations, presumably as described in the response to RAI 3.8.1-22. Based on this study, AREVA indicates that the existing 4/5 element mesh through the thickness of the RCB overestimates the thermal gradient across the thickness, at the beginning of the accident period, and provides an accurate estimate of the thermal gradient at the later period of the accident, compared to the thermal gradient for a refined mesh. To complete the response to Item 1 of the RAI, provide some representative (max., min.) comparison results determined in this study, for selected elements/regions of the RCB, such that the magnitude of the stated conservatism can be quantified. Since the computed thermal moments are subsequently reduced by “thermal moment reduction factors,” as explained in the response to RAI 3.8.1-22, confirm that this conservatism is actually eliminated from the forces/moments used in the RCB design. Information regarding this issue should be provided in conjunction with the response to the follow-up to RAI 3.8.1-22, Items 1 and 2.

b. Item 4 of the RAI response confirms that ANSYS smeared concrete cracking constitutive models are used to model concrete cracking during thermal loading, presumably as described in the response to RAI 3.8.1-22. To complete the response to Item 4 of the RAI, confirm that the described FEM procedure is used to determine “thermal moment reduction factors,” as explained in the response to RAI 3.8.1-22. Information regarding this issue should be provided in conjunction with the response to the follow-up to RAI 3.8.1-22, Item 3.

Response to Question 03.08.01-52:

Item a:

The U.S. EPR thermal transient analysis considered six-degree slice finite element models (FEMs) with two mesh densities, which are an equivalent slice model mesh density and a refined slice model mesh density (refer the Response to RAI 155, Question 03.08.01-22). The equivalent slice model has similar element thickness and mesh density as the Reactor Containment Building (RCB), while the refined slice model has a refined mesh density. The mesh densities of the containment wall and dome equivalent and refined slice models are provided in Figures 03.08.01-52-1 and 03.08.01-52-2, where the element thickness is shown between the hash-marks. The variation in temperatures though the thickness of the containment wall (at 65.86 feet) and dome (75 degrees from horizontal) at different time points for each one of the two slice models are shown in Figures 03.08.01-52-3 and 03.08.01-52-4. For comparison purposes, the NI static model RCB temperature variations are plotted in Figure 03.08.01-52-3 and 03.08.01-52-4 and labeled as “Full Containment Model”.

The magnitudes of the thermal moments at the containment wall (at 65.86 feet) and dome (75 degrees from horizontal) sections for the equivalent slice model and refined slice model at representative time points are provided in Table 03.08.01-52-1 and Table 03.08.01-52-2, respectively. Thermal moments from the refined model are significantly reduced at the beginning of the accident time period because of better approximation of thermal gradients, while refinement shows an increase in thermal moments at later time periods. The design of containment sections is based on the forces and moments calculated from the applicable critical load combinations, which are eventually governed by the accidental temperature and pressure loads for post-accident period at 1200 seconds.

Thermal moment reduction factors have been eliminated as described in the Response to RAI 448, Question 03.08.01-51. Thermal moment reduction factors are therefore no longer applicable.

Item b:

See response to Item a.

FSAR Impact:

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

Table 03.08.01-52-1—Thermal Moments at 65.86 ft Elevation from Equivalent, Refined and Linear Slice Models

Time Point for Accident Period	Equivalent Slice Model		Refined Slice Model	
	My (Kip-ft/ft)	Mz (Kip-ft/ft)	My (Kip-ft/ft)	Mz (Kip-ft/ft)
Acc-Start	781.5	778.9	183.6	181.3
Acc-20 m	804.6	801.6	313.5	310.4
Acc- 2h	813.2	810.7	597.0	593.3
Acc- 24 h	918.9	919.9	1007.7	1008.1
Acc-110 h	634.0	638.0	689.1	692.9
Acc- 365 d	207.7	209.0	224.2	225.8

Table 03.08.01-52-2—Thermal Moments at 75 Degrees of Dome from Equivalent, Refined and Linear Slice Models

Time Point for Accident Period	Equivalent Slice Model		Refined Slice Model	
	My (Kip-ft/ft)	Mz (Kip-ft/ft)	My (Kip-ft/ft)	Mz (Kip-ft/ft)
Acc-Start	531.0	503.7	156.1	146.9
Acc-20 m	543.3	515.0	248.6	235.0
Acc- 2h	560.6	529.5	454.5	430.8
Acc- 24 h	581.9	527.4	644.3	586.4
Acc-110 h	376.2	307.9	408.3	334.1
Acc- 365 d	132.2	103.8	143.4	112.5

Figure 03.08.01-52-1—Coarse Slice Model Mesh Density (a) Typical RCB Section, (b) Typical RCB Dome, (c) Typical RCB Wall

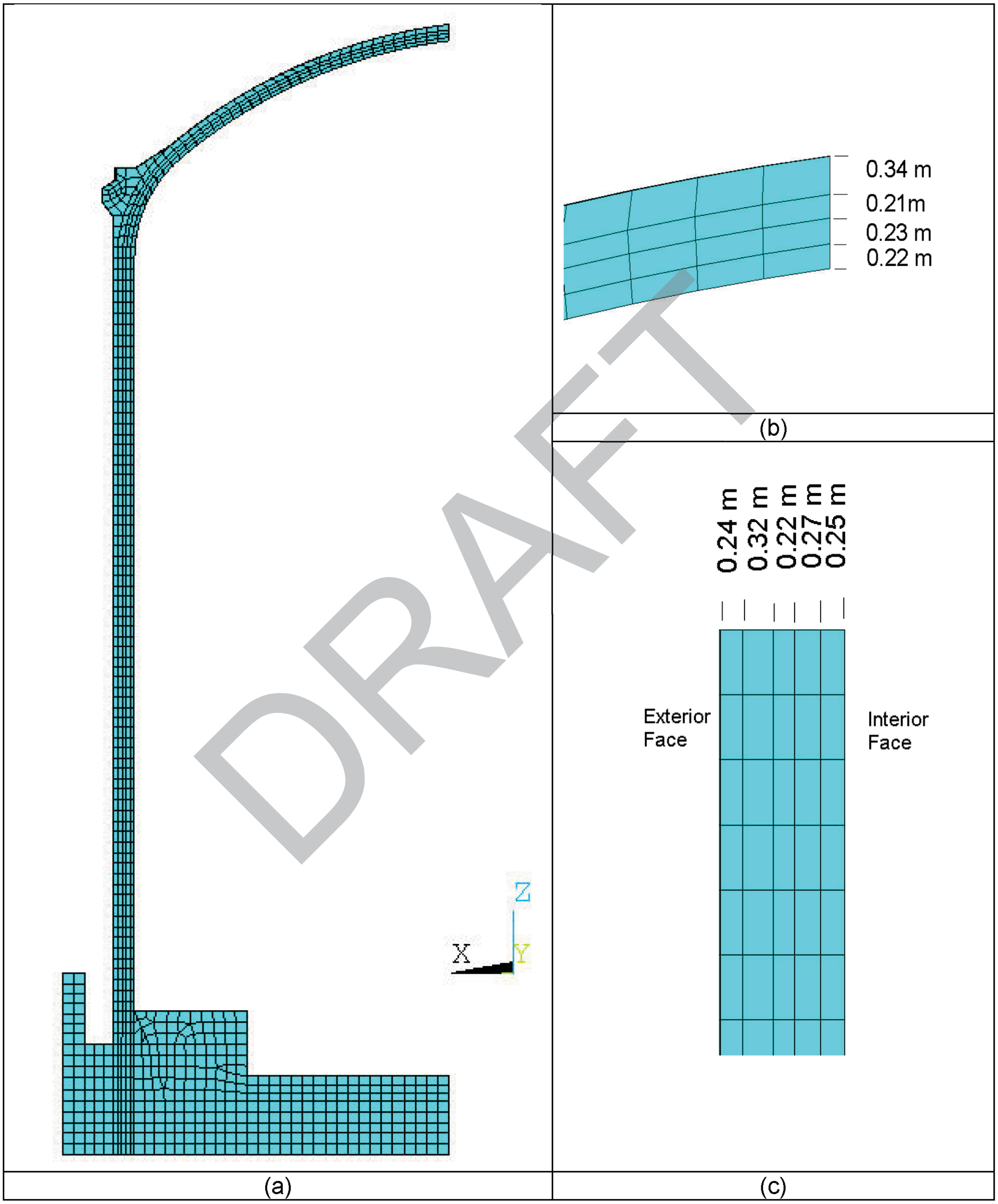


Figure 03.08.01-52-2 Refined Slice Model Mesh Density (a) Typical RCB Section, (b) Typical RCB Dome, (c) Typical RCB Wall

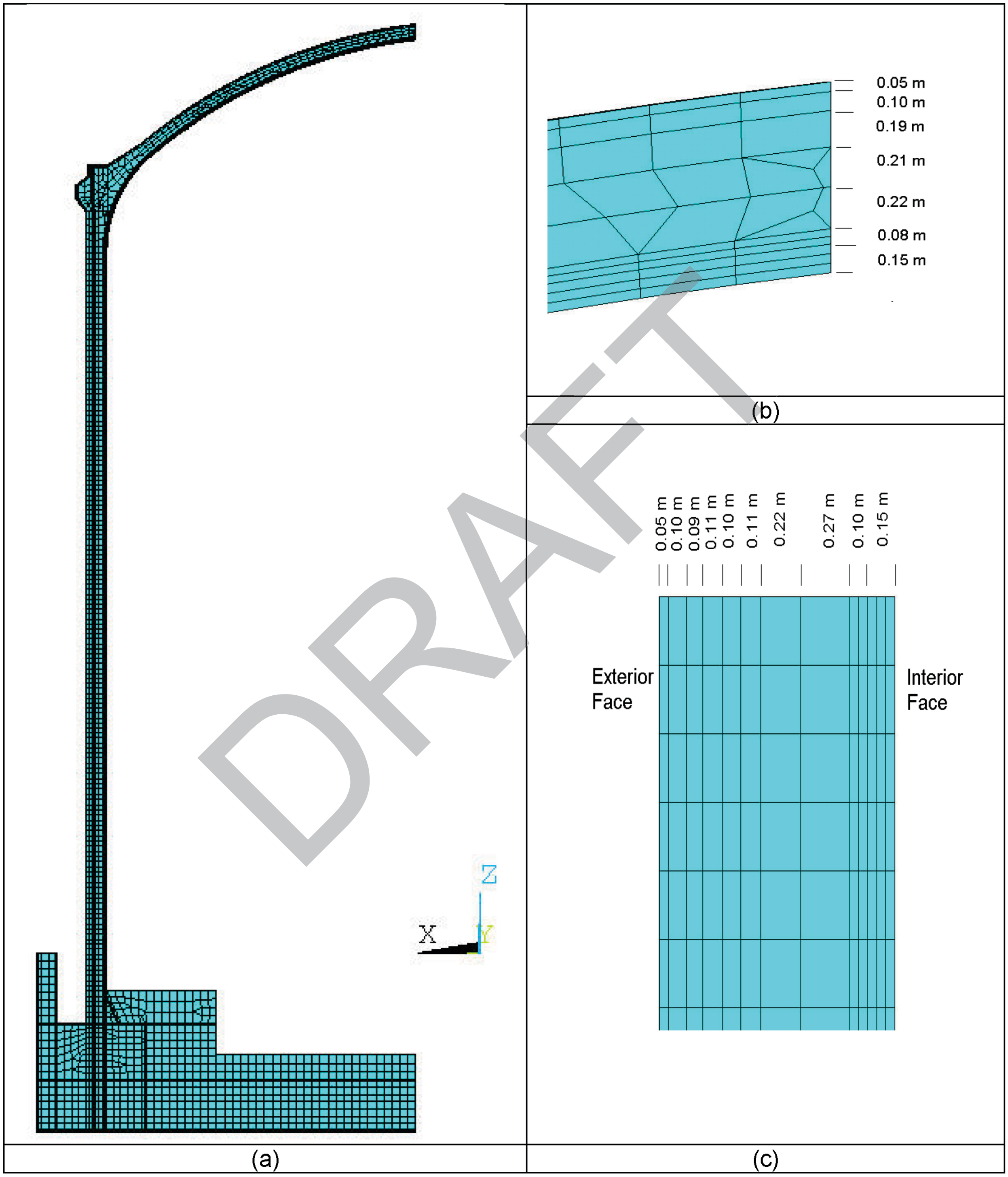


Figure 03.08.01-52-3—Comparison of Temperature Gradients across the Thickness at 65.86 ft Elevation for (a) Steady State Condition, (b) 1200 Seconds, (c) 2 Hours, (d) 24 Hours, (e) 110 Hours, and (f) 1 Year

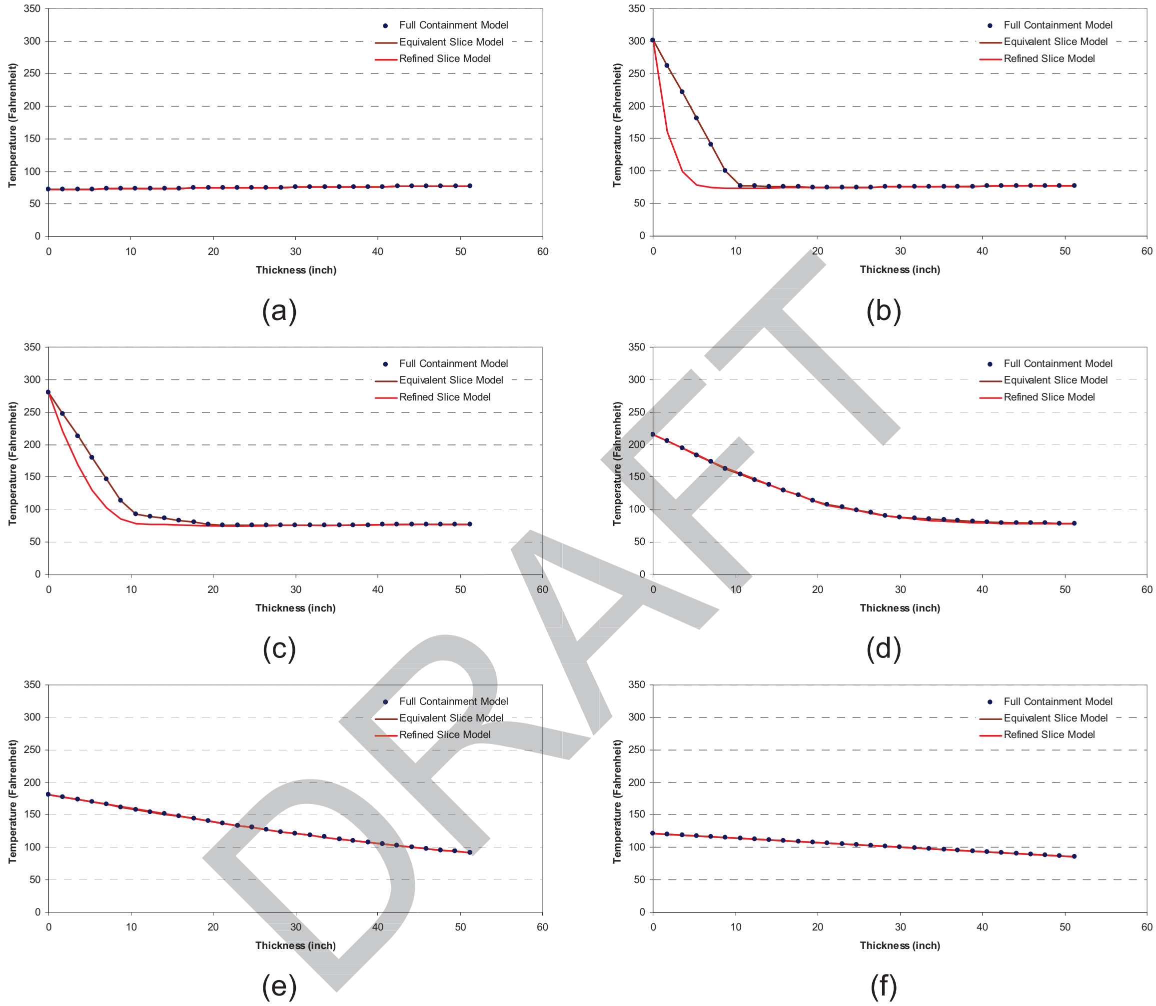
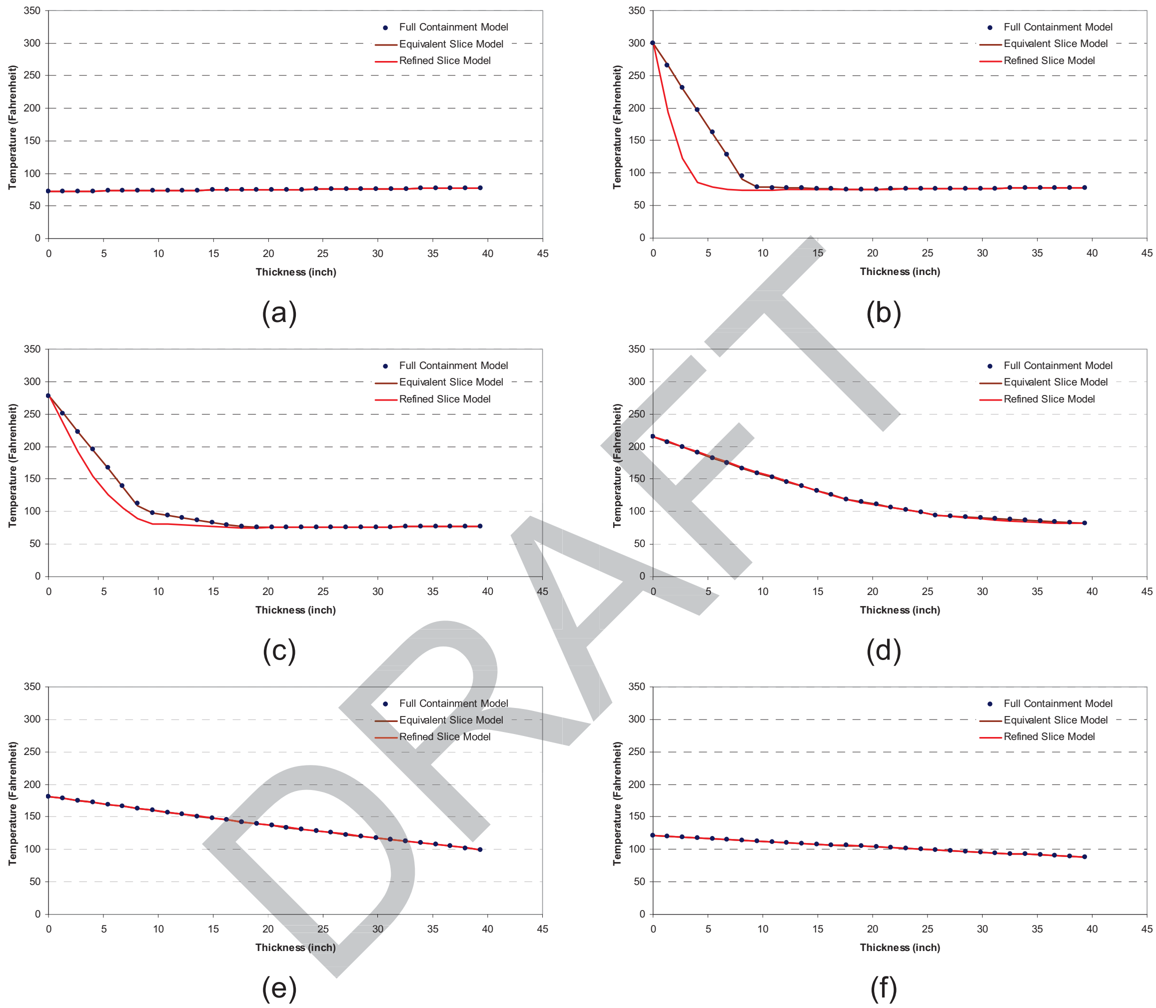


Figure 03.08.01-52-4—Comparison of Temperature Gradients across the Thickness at 75.05° of Dome for (a) Steady State Condition, (b) 1200 Seconds, (c) 2 Hours, (d) 24 Hours, (e) 110 Hours and, (f) 1 Year



Question 03.08.01-53:

Follow-up to RAI 211, Question 3.8.1-31

The RAI response has provided the additional information regarding the U.S. EPR ISI program. The staff has evaluated the response and determined that the information provided is inadequate with respect to meeting 10 CFR 50.55a and 10 CFR 50, Appendix A, GDC 1, as it relates to concrete containment being designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety function to be performed, and as described in SRP 3.8.1.II.7.D and RG 1.90. The staff requests that the applicant provide further clarification as discussed below:

a. Regarding the issue of maximum tensile stresses in the RCB reinforcement under the ISI test pressure, the RAI response appears to contradict subsequent discussions with AREVA. During the meeting on December 14 and 15, 2009, AREVA indicated that the RCB design is consistent with the criterion in RG 1.90, Alternative B, which prescribes a maximum tensile stress of 0.5fy in the reinforcement under the ISI test pressure. The staff notes that one of the considerations for using Alternative B (deformation monitoring under pressure tests) given in RG 1.90 is that the design of the containment should be conservative so that cracking under repeated ISI pressure tests is minimized. It follows that, whether 1.15Pd or Pa is used as the ISI test pressure, the containment should be designed to minimize cracking under repeated ISI pressure tests in either case. Therefore, confirm that the RCB is designed so that, under ISI test pressure, membrane compression stresses are maintained and maximum tensile stresses in the reinforcement are limited to 0.5fy, regardless of whether 1.15Pd or Pa is used as the ISI test pressure. It is emphasized that the issue of ISI pressurization levels is pending resolution under RAI 3.8.1-12.

b. No mention is made in either FSAR Section 3.8.1.7.2 or the RAI response of the visual examination component of the ISI program. The staff notes that according to RG 1.90 the ISI program should consist of three distinct activities: (a) force monitoring of ungrouted test tendons; (b) periodic reading of instrumentation for determining prestress level (Alternative A) or monitoring of deformations under pressure (Alternative B) at preestablished sections; and (c) visual examination. The force monitoring of ungrouted test tendons is pending resolution under RAI 3.8.1-12. However, additional information should be provided on the visual examination component of the ISI program. This information should also be included in the appropriate sections of the FSAR.

In addition, revise and update the relevant sections of the FSAR as needed to address the staff's concerns listed above.

Response to Question 03.08.01-53:

For completeness, also see the Response to Question 03.08-01-50.

Item a:

The U.S. EPR design is based on the use of Alternative B in RG 1.90, Revision 1 for monitoring deformations under pressure. Membrane compression will be maintained and the maximum stress in the tensile reinforcing will be limited to one-half the yield strength of the reinforcing steel (0.5fy) under the peak expected pressure for inservice inspection (ISI) tests.

U.S EPR FSAR Tier 2, Section 3.8.1.1.2 will be revised to include this information.

Item b:

The ISI program and visual inspection of tendons is addressed in the response to RAI 448, Question 03.08.01-50.

FSAR Impact:

The U.S. EPR FSAR Tier 2, Section 3.8.1.1.2 will be revised as described in the response and indicated on the enclosed markup.

Question 03.08.01-54:

Follow-up to RAI 155, Question 03.08.01-8

1. The response to Item 1 of the RAI confirms that a single FE model of the NI, including RCB, RSB, RBIS, SB, FB, and common basemat, has been used for analysis of all loads identified in FSAR Section 3.8.1.3. The response also provides a description of how each of the following loads is applied to the FE model: dead loads (D), live loads (L), soil loads (H), hydrostatic loads (F), thermal loads (To), normal pipe reactions (Ro), tendon loads (J), relief valve loads (G), pressure variant loads (Pv), construction loads, test loads (Pt and Tt), temperature loads (Ta), pressure loads (Pa), accident pipe reactions (Ra), pipe break loads (Rr), and seismic loads (E').

   The staff has evaluated the response and determined that the information provided is inadequate with respect to meeting 10 CFR 50, Appendix A, GDC 2, as it relates to the design of safety-related structures being able to withstand the most severe natural phenomena such as earthquakes, and GDC 50, as it relates to the concrete containment being designed with sufficient margin of safety to accommodate appropriate design loads, and as described in SRP 3.8.1.II.3 and 4. The staff requests that the applicant clarify the response to Item 1 of the RAI as discussed below.

   In addition, the staff finds several inconsistencies between loads described in this RAI response and those described in other RAI responses. Some of these inconsistencies are related to RAIs pending resolution, while others are due to ongoing changes in the analysis methods (e.g., new FEM SSI analysis of the NI, revised set of soil cases). Therefore, to resolve Item 1 of this RAI, reconcile and resubmit the response to reflect the current status of the DC application.

   a. The RAI response indicates that there are no (L), (H) and (F) loads applied to the FE model of the RCB. However, the response to RAI 3.8.1-5 Item 2 states that (L) loads are applied "in the Reactor Building near the equipment hatch (due to staging of equipment during a refueling outage)." Also, the response to RAI 3.8.1-7 Item 2 indicates that (L), (H), and (F) loads are applied to the RCB indirectly through the basemat. Clarify these inconsistencies.

   b. The RAI response states that (Ro) and (Ra) loads are not applied to the FE model of the RCB since they are considered part of the local design. However, the response to RAI 3.8.1-26 indicates that these are independent loads applied to the FE model of the NI. Clarify this inconsistency. In addition, provide additional details of how these loads are applied to the FE model of the NI; especially, a description of how multi-directional effects are considered.

   c. The RAI response provides details on how (J) loads are developed and applied to the FE model of the RCB to account for three-dimensional tendon profiles, geometric and material properties of the tendons and containment, wobble and curvature effects, creep and shrinkage properties of concrete, relaxation of tendon materials, and number of jacking ends, for both a 0-year and a 60-year period. Since the response to RAI 3.8.1-35 states "Bonding between the tendon and surrounding grout is not assumed in RCB design," explain whether the methodology used for determining (J) loads is consistent with the unbounded tendon assumption.

d. The RAI response provides details of how seismic ZPA values in the three principal directions are computed for different elevations of the RCB. The response indicates that these ZPA computations are based on stick models used in the SSI analysis for the various soil types and ground motions considered in the FSAR. However, as indicated in the response to RAI 3.8.5-8, a new FEM SSI analysis of the NI has been performed using fully embedded conditions for a reduced number of soil cases. The stick models have been superseded by this new analysis methodology and are no longer applicable. Clarify this inconsistency.

2. The response to Item 6 of the RAI indicates that the RCB liner is modeled with 4-node SHELL181 elements applied on the inner surface as a pressure load transfer element, smeared over the inner face of the SOLID45 concrete elements. The liner and its anchorages are not considered as structural elements in the structural design of the RCB, so the liner anchorage is not explicitly modeled in the FE model. The stiffness of the liner material is reduced to 1% of its actual value to make the liner structurally inactive in the FE model. Finally, liner anchorage design loads are not determined from FE analysis but are determined using an energy approach described in Bechtel Topical Report BC-TOP-01 Rev. 1 (1971) “Containment Building Liner Plate Design Report.”

   To ensure compliance with 10 CFR 50, Appendix A, GDC 16, as it relates to the capability of the concrete containment to act as a leak-tight membrane, and as described in SRP 3.8.1.II.4.J, explain how the liner plate is designed for “local” loads that are not applied to the FE model of the RCB (e.g., jet impingement loads). Also, provide a description of the energy approach used to determine anchorage design loads (which is stated to follow Bechtel Topical Report BC-TOP-01 Rev. 1), as well as a discussion on how the anchorage design satisfies ASME BVP Code, Section III, Division 2, Article 3810, items (a) through (h). Finally, include a summary of this information in the relevant sections of the FSAR.

Response to Question 03.08.01-54:

Item 1a:

The live (L) loads identified in the Response to RAI 155 Supplement 1, Question 3.8.1-5, Item 2 are applied to the Reactor Building Internal Structure (RBIS) floor area, but not in Reactor Containment Building (RCB) walls. There are no live loads for the RCB wall and dome. The floor access to the equipment hatch in RBIS is subjected to temporary live loads for staging of equipment during the refueling outage (refer to the Response to RAI 155 Question 3.8.1-5, Item 2). The live load is considered in the critical section design for the RBIS.

Soil (H) and hydrostatic (F) loads are not directly applied to the RCB, since the RCB wall is surrounded by other buildings that shield it from these loads. The effects on the RCB from these loads are considered through the Nuclear Island (NI) common basemat structure when analysis is performed for load combinations.

Item 1b:

Accident pipe reaction (R_a) and normal pipe reaction (R_o) loads are not directly applied to the RCB wall of the finite element model (FEM). However, accident pipe reaction and normal pipe reaction loads from nuclear steam supply system (NSSS) components are applied to RBIS of the FEM of NI common basemat structure.

Accident pipe reactions are categorized as:

1. Equipment reactions during accident conditions, and
2. Pipe support reactions during accident conditions.

The only accident pipe reaction loads included in the global FEM are the NSSS system loads, which includes reactions due to the reactor pressure vessel, pressurizer, steam generators, and reactor coolant pumps. Additional accident pipe reaction is considered in critical section design. The accident pipe reaction due to NSSS components are conservatively applied to the NI FEM in the upward (U), downward (D), eastward (E), westward (W), northward (N), and southward (S) in each of the six load input files.

Normal pipe reactions are categorized as:

1. Equipment reactions during normal operating conditions, and
2. Pipe support reactions during normal operating conditions.

The normal pipe reaction loads included in the global FEM are the NSSS system loads, which includes reactions due to the reactor pressure vessel, pressurizer, steam generators, and reactor coolant pumps. Additional normal pipe reaction is considered in critical section design. The normal pipe reactions due to NSSS components are conservatively applied to the NI FEM in the upward (U), downward (D), eastward (E), westward (W), northward (N), and southward (S) directions in each of the six load input files.

In the expansion of load combinations for accident pipe reactions or normal pipe reactions, the horizontal loads (N, S, E, W) and the vertical loads (U, D) result in 4x2=8 variations. These variations are used for final load combinations.

Item 1c:

The tendon loads were calculated based on the ungrouted tendon ducts, such as the hollow ducts, consistent with unbonded tendons, for zero-year and 60-year periods. The duct is grouted after the installation of tendons that forms bonding among tendons, grout, ducts, and concrete. This additional bonding is neglected since it would reduce the creep and relaxation losses. The grout is mainly served as corrosion inhibitor.

Item 1d:

For U.S. EPR design, current zero period acceleration (ZPA)s are calculated based on the embedded soil-structure interaction (SSI) FEM as described in U.S. EPR FSAR Tier 2, Section 3.7. The ZPAs calculated from NI stick model are replaced with the ZPAs calculated from the embedded SSI FEM in analysis of NI common basemat structure. The stick model is used only for obtaining the NSSS loads.

Item 2:

A pipe break hazard analysis performed for the U.S. EPR design indicates that local dynamic impact loads, such as pipe whip, missile impact, and jet impingement are not applicable to the liner plate. COL Information Items 3.6-1 and 3.6-2 require the COL applicant to perform a pipe break hazard analysis that will demonstrate that the liner plate is protected from dynamic loads.

In addition, U.S. EPR FSAR Tier 1, Section 3.8 includes an ITAAC that addresses the dynamic and environmental effects of piping systems.

The method used to determine anchorage loads is outlined as follows:

3. The two-dimensional state of strain is obtained from load combinations in accordance with ASME Section III Division 2. The largest strain is used to determine the uni-axial yield stress that must exist if the strains are to be converted to stress by Hook's law.
4. Load vs. Displacement curves for the anchor and bent plate are determined.
5. The spring constant for the plate relaxation is calculated.
6. The force N' is calculated. This force N' simulates the effects of multiple anchor movements as presented in Section 5.1 (Equation 9) of Bechtel Power Corporation Topical Report, BC-TOP-01, *Containment Building Liner Plate Design Report, Revision 1, December 1972.*
7. The effect of additional internal or external pressure loading can be converted to an equivalent axial load N'' according to Section 5.1 of BC-TOP-01 (Equation 15). N'' is combined with N' to get the final axial load.
8. The anchorage deformation is calculated based on either an elastic solution or plastic solution, whichever is appropriate based on the stress level in the anchor.
9. The energy required in obtaining equilibrium in the anchor is calculated as the area under the Load-Displacement curve of the anchor. This energy is compared against the total energy obtained from test results presented in Appendix B of BC-TOP-01. The required energy is less than the total energy obtained from test.

The inward curvature of the liner plate is evaluated as no more than 1/8 inch during fabrication and erection of the liner plate as given in Section 3.1 of BC-TOP-1.

Plate thickness variation is defined by the standard rolling tolerance. For a ¼ inch plate, a thickness variation within the limits of +16 percent and -4 percent is allowed. In keeping with development of BC-TOP-1, for a plate with -4 percent variations in thickness, a lower stiffness would result in an increase in anchor loading. This condition is highly improbably and therefore it is not necessary to consider the case of a plate which is -4 percent under the nominal thickness. A plate with +16 percent thickness variations is conservative as long as the excess thickness is constant throughout a large area; a thickness panel with inward curvature would be stiffer and therefore, the anchor load would decrease. In the analysis, a panel with outward curvature that is +16 percent over the nominal thickness is considered adjacent to a plate with inward curvature of nominal thickness.

The liner plate differs from a typical structural component such that, a lower value of yield will limit the stress and the forces on the liner plate and the anchors. For a liner plate with higher yield stress due to rolling processes and biaxial loading, the anchor will be subjected to a higher load. Consistent with ASME (CC3810(c),1), this is adequately satisfied by converting liner strain to stress and membrane forces assuming the plate remains elastic.

Weld offset is mitigated through quality control in accordance with ASME Section III Division 2 CC-4523.2. The structural discontinuities areas, such as pipe penetration and openings, are designed as special regions. The construction sequence used is such that effects of concrete voids behind the liner are mitigated as the liner is used as a form, in case of the cylinder and dome. The base plate is grouted to fill voids after installation.

Anchorage spacing will affect the stiffness of the system. If anchors are further apart than specified, panels with inward curvature will have less stiffness than a panel with the specified spacing which would result in higher anchor loads. Since it is relatively simple to fabricate anchors to the specified spacing, any small variation in spacing is not considered to have any appreciable effect on the anchorage system.

Variation of the concrete modulus affects the energy required for the anchorage system to reach equilibrium. A higher concrete modulus is advantageous as the energy required to reach equilibrium is lower. On the contrary, a lower concrete modulus requires higher energy absorption to reach equilibrium. In practice, a lower modulus is mitigated due to the code required overstrength and the extensive performance testing required of the concrete mix.

When an anchorage system has sufficient safety factors, the local yielding of concrete will be minor and it is not an area of concern since it is simply a mean of stress redistribution to obtain a maximum load capacity.

There is no stud in the anchorage system. Stud anchors are not applicable to the U.S.EPR design.

U.S. EPR FSAR Tier 2, Section 3.8.1.1.3 will be revised to include a summary of the U.S. EPR liner anchorage system. U.S. EPR FSAR Tier 2, Section 3.8.6 will be updated to add BC-TOP-01 Rev. 1 as a reference.

FSAR Impact:

The US EPR FSAR, Tier 2 Sections 3.8.1.1.3 and 3.8.6 will be revised as described in the response and indicated on the enclosed markup.

Question 03.08.01-55:

Follow-up to RAI 306, Question 3.8.1-42

The RAI response states that the design of structural steel members is based on the conservative use of the minimum allowable material stress values provided in FSAR Table 3.8-8. The design specifies a particular minimum value to be used in the fabrication of the component, and the stress values of the materials actually used in fabrication will be confirmed by certified material test reports and certificates.

To resolve this RAI: (a) add the above statements to the FSAR, and (b) explain why FSAR Table 3.8-8 lists A276 (martensitic) steel twice, with inconsistent stress ranges.

Item a:

The requested information has been added to the U.S. EPR FSAR Tier 2, Table 3.8-8.

Item b:

U.S. EPR FSAR Tier 2, Table 3.8-8 has been revised to remove the second reference to A276 (martensitic) steel.

FSAR Impact:

U.S. EPR FSAR Tier 2, Table 3.8-8 will be revised as described in the response and indicated on the enclosed markup.

U.S. EPR Final Safety Analysis Report Markups

Table 1.9-2—U.S. EPR Conformance with Regulatory Guides
Sheet 3 of 19

RG / Rev	Description	U.S. EPR Assessment	FSAR Section(s)
1.29, R4	Seismic Design Classification	Y	3.2.1
1.30, 08/1972	Quality Assurance Requirements for the Installation, Inspection, and Testing of Instrumentation and Electric Equipment	N/A-COL	N/A
1.31, R3	Control of Ferrite Content in Stainless Steel Weld Metal	Y	3.6.3
			5.2.3
			6.1.1
1.32,R3	Criteria for Power Systems for Nuclear Power Plants	Y	6.3
			Table 8.1-1
			8.2
			8.3.2.2.3
1.33,R2	Quality Assurance Program Requirements (Operation)	N/A-COL	N/A
1.34, 12/1972	Control of Electroslag Weld Properties	Y	5.2.3.4
1.35, R3	Inservice Inspection of Ungrouted Tendons in Prestressed Concrete Containments	N/A-~~OTHER (No ungrouted tendons)~~	N/A
1.35.1, 07/1990	Determining Prestre[03.08.01-50 and 03.08.01-53]ion of Prestressed Concrete	Y ~~N/A-OTHER (No ungrouted tendons)~~	3.8.1.2.5 ~~N/A~~
1.36, 02/1973	Nonmetallic Thermal Insulation for Austenitic Stainless Steel	Y	5.2.3.4.3
			6.1.1
1.37, R1	Quality Assurance Requirements for Cleaning of Fluid Systems and Associated Components of Water-Cooled Nuclear Power Plants	Y	3.13
			5.2.3
			5.3.1
			5.4.2
			6.1.1
			17.5
			10.3
1.38, R2	Quality Assurance Requirements for Packaging, Shipping, Receiving, Storage, and Handling of Items for Water-Cooled Nuclear Power Plants	N/A-COL	N/A
1.39, R2	Housekeeping Requirements for Water-Cooled Nuclear Power Plants	N/A-COL	N/A

Table 1.9-2—U.S. EPR Conformance with Regulatory Guides
Sheet 7 of 19

RG / Rev	Description	U.S. EPR Assessment	FSAR Section(s)
1.82, R3	Water Sources for Long-Term Recirculation Cooling Following a Loss-of-Coolant Accident	Y	6.3
1.84, R33	Design, Fabrication, and Materials Code Case Acceptability, ASME Section III	Y	3.8.1 3.8.2 4.5.2 5.2.1 5.4.2.4.1 10.3
1.86, 06/1974	Termination of Operating Licenses for Nuclear Reactors	N/A-COL	N/A
1.87, 06/1975	Guidance for Construction of Class 1 Components in Elevated-Temperature Reactors	N/A-OTHER	N/A
1.89, R1	Environmental Qualification of Certain Electric Equipment Important to Safety for Nuclear Power Plants	Y	3.11 Appendix 3D Attach C
1.90, R1	Inservice Inspection of Prestressed Concrete Containment Structures with Grouted Tendons 03.08.01-50 and 03.08.01-53	Y ~~EXCEPTION (No forced monitoring)~~	3.8.1.2.5 3.8.1.7.2 ~~5.3.1.6~~ ~~3.8.1.2.5 and 3.8.1.7.2~~
1.91, R1	Evaluations of Explosions Postulated To Occur on Transportation Routes Near Nuclear Power Plants	N/A-COL	N/A
1.92, R2	Combining Modal Responses and Spatial Components in Seismic Response Analysis	Y	3.7.2 3.7.3 Appendix 3D Attach E
1.93, 12/1974	Availability of Electric Power Sources	Y	16.B3.8
1.94, R1	Quality Assurance Requirements for Installation, Inspection, and Testing of Structural Concrete and Structural Steel During the Construction Phase of Nuclear Power Plants	Y	3.8.1.2.5
1.96, R1	Design of Main Steam Isolation Valve Leakage Control Systems for Boiling Water Reactor Nuclear Power Plants	N/A-BWR	N/A

3.8.1.1.2 Post-Tensioning System

Tendons are provided both horizontally and vertically in the cylindrical portion of the RCB. Tendons are provided in two orthogonal directions in the plan view of the containment dome. Layouts of the tendons vary to accommodate penetrations through the RCB wall.

03.08.01-50 and 03.08.01-53

The Freyssinet C-range post-tensioning system is the tendon system used for post-tensioning the concrete RCB. The Freyssinet 55C15 tendon system is made up of 55 seven-wire strands in each tendon. Section 3.8.1.6.3 describes the material properties of the tendon system. With the exception of the three greased test tendons of each type (vertical, gamma, and horizontal hoop) provided for force monitoring, the other tendons are grouted in place after tensioning. ~~The tendons are grouted in place after tensioning.~~

A total of 119 horizontal hoop tendons are provided around the cylindrical shell of the RCB. The tendons terminate at the three vertical buttresses provided around the outside of the containment wall. Terminations alternate so that each buttress has a horizontal tendon terminating every third hoop (i.e., each hoop tendon extends the full circumference of the building).

A total of 47 vertical tendons are provided around the cylindrical shell of the RCB. The vertical tendons terminate at the top of the ring girder that is provided at the transition of the wall to the spherical dome roof. A total of 104 gamma tendons are also provided vertically up through the containment wall where they then wrap over the dome and terminate at the ring girder on the opposite side of the wall. The gamma tendons are separated into two groups that are placed 90° apart in the RBC dome. The bottom of both the vertical tendons and the gamma tendons terminate at the tendon gallery.

03.08.01-50 and 03.08.01-53

The U.S. EPR design is based on the use of Alternative B of RG 1.90, Revision 1 for monitoring deformations under pressure. Membrane compression will be maintained and the maximum stress in the tensile reinforcing will be limited to one-half the yield strength of the reinforcing steel (0.5fy), under the peak expected pressure for inservice inspection (ISI) tests.

Additional information on layout and design of the tendons is provided in Appendix 3E for the RCB cylindrical wall, and buttress areas. The minimum required post tensioning force to offset the structural integrity test (SIT) pressure loading is 801k/ft hoop force, 401k/ft vertical force, and 548k/ft in both directions for the dome.

Figure 3.8-18—Finite Element Model of Reactor Containment Building Tendon Layout in Cylindrical Wall and Figure 3.8-19—Finite Element Model of Reactor Containment Building Tendon Layout in Dome show the finite element model of the tendon layout.

3.8.1.1.3 Liner Plate System

A carbon steel liner plate covers the entire inside surface of the RCB, excluding penetrations. The steel liner is 0.25 inch thick and is thickened locally around penetrations, large brackets, and at major attachments. Except for the bottom horizontal surface, angle and channel steel sections anchor the liner plate to the concrete containment structure. The in-containment refueling water storage tank (IRWST), including the containment sumps, are lined with 0.25 inch thick stainless steel liner plates that serve as additional corrosion protection for the underlying carbon steel liner. See Section 3.8.3 for a description of the IRWST.

Steel shapes reinforce the plate both longitudinally and laterally to provide rigidity during prefabrication, erection, and concrete placement. The steel shapes are welded to the liner plate and are fully embedded in the concrete to provide a rigid connection to the inside surface of the RCB concrete. The concrete foundation of the RB internal structures is poured on top of the liner plate at the basemat surface, embedding the lower region of the liner plate in the foundation. The liner plate is not used as a strength element to carry design basis loads; however, the liner supports the weight of wet concrete during the construction of the RCB.

03.08.01-54

Section CC3810 of ASME Section III, Division 2 prescribes the criteria for design of liner anchorage system. The U.S. EPR liner anchorage system is designed using an energy approach described in BC-TOP-01, Revision 1 (Reference 69), which addresses ASME criteria. The methodology considers the variation in liner yield strength analytically by converting liner strain to stress and membrane forces assuming the plate remains elastic. In addition, the variation of liner plate thickness is accounted for by considering a thicker panel (+16 percent) with outward curvature being adjacent to a nominal plate with inward curvature (refer to Figure 2 through 4 of Reference 69). The inward curvature is evaluated as no more than 1/8 inch during fabrication and erection of the liner plate as given in Reference 69. The weld offset is mitigated through quality control in accordance with ASME Section III Division 2 CC-4523.2. The effects of concrete voids behind the liner are mitigated by the construction method employed. Lower concrete modulus is mitigated due to the code required over strength and the extensive performance testing required of the concrete mix. The variation of anchorage spacing is mitigated by quality control during the fabrication process. The anchorage system is designed with a safety factor so that the local crushing of the concrete is limited and a means of stress redistribution to obtain a maximum load capacity. The structural discontinuities areas, such as pipe penetration and openings, are designed as special regions.

Section 3.8.2 contains a description of the penetrations through the containment liner, including the equipment hatch, airlocks, piping penetration sleeves, electrical penetration sleeves, and the fuel transfer tube penetration sleeve.

- Article CC-3000 of the ASME Code, 2004 Edition, Section III, Division 2 (GDC 1, GDC 2, and GDC 16).
- ASME Code 2004 Edition, Section XI, Subsection IWL, Requirements for Class CC Concrete Components of Light-Water Cooled Plants.
- ASME Code 2004 Edition, Section XI, Subsection IWE, Requirements for Class MC and Metallic Liners of Class CC Concrete Components of Light-Water Cooled Power Plants.

3.8.1.2.4 Regulations

- 10 CFR 50 – Licensing of Production and Utilization Facilities.
- 10 CFR 50, Appendix A – General Design Criteria for Nuclear Power Plants (GDC 1, 2, 4, 16, and 50).
- 10 CFR 50, Appendix J – Primary Reactor Containment Leakage Testing for Water Cooled Power Reactors.
- 10 CFR 100 – Reactor Site Criteria.

3.8.1.2.5 NRC Regulatory Guides

Regulatory Guides applicable to the design and construction of the RCB:

03.08.01-50 and 03.08.01-53

- RG 1.7, Revision 3.
- RG1.35.1, July 1990.
- RG 1.84, Revision 33.
- RG 1.90, Revision 1 ~~(exception described in 3.8.1.7)~~.
- RG 1.94, Revision 1.
- RG 1.107, Revision 1.
- RG 1.136, Revision 3 (exception described in 3.8.1.3).
- RG 1.199, November 2003 (exception described in 3.8.1.4).
- RG 1.216, August 2010.

3.8.1.3 Loads and Load Combinations

The U.S. EPR standard plant design loads envelope includes the expected loads over a broad range of site conditions. Loads and load combinations for the RCB are in accordance with the requirements of Article CC-3000 of the ASME Code, Section III, Division 2, Code for Concrete Containments and ACI Standard 359, and RG 1.136

3.8.1.6.3 Tendon System Materials

Tendons

The post-tensioning tendon system consists of load-carrying and non-load-carrying components. The load-carrying components include the post-tensioning wires that make up the tendons, and anchorage components composed of bearing plates, anchor heads, wedges, and shims. Non-load-carrying components include the tendon sheathing (including sheaths, conduits, trumpet assemblies, couplers, vent and drain nipples, and other appurtenances) and corrosion prevention materials.

Materials used for the RCB post-tensioning system (including post-tensioning steel, anchorage components, and non-load-carrying and accessory components) meet the requirements of Subarticle CC-2400 of the ASME Code, Section III, Division 2.

The Freyssinet C-range post-tensioning system has the following properties:

- ASTM A416 (Reference 36), Grade 270, low-relaxation tendon material.
- Tendon ultimate strength $F_{pu} = 270$ ksi
- Tendon minimum yield strength $F_{py} = (0.9)(270) = 243$ ksi
- Modulus of elasticity of tendon material $E_{ps} = 28{,}000$ ksi
- Number of strands per tendon $N_{strands} = 55$
- Total area of each tendon $A_p = 12.76\ in^2$

The materials used for the anchorage components are compatible with the tendon system. Tendon raceways consist of corrugated steel ducts and rigid metal conduit. These components are non-structural and are sealed to prevent the intrusion of concrete during construction.

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Grouting of Tendons

Cement grout for the grouted tendons in the prestressing system in the RCB is selected based on the testing and material requirements of the ASME Code, Section III, Division 2, as amended by RG 1.136, which endorses the Regulatory Positions of RG 1.107, Qualifications for Cement Grouting for Prestressing Tendons in Containment Structures.

Greasing of Tendons

Grease for the greased test tendons in the prestressing system in the RCB is selected based on the testing and material requirements of the ASME Code, Section III, Division 2.

3.8.1.7 Testing and Inservice Inspection Requirements

3.8.1.7.1 Structural Integrity Test

Following construction, the RCB is proof-tested at 115 percent of the design pressure. During this test, deflection measurements and concrete crack inspections are made to confirm that the actual structural response is within the limits predicted by the design analyses (GDC 1).

The SIT procedure complies with the requirements of Article CC-6000 of the ASME Code, 2004 Edition, Section III, Division 2, and with Subsections IWL and IWE of Section XI of the ASME Code.

3.8.1.7.2 Long-Term Surveillance

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The RCB is monitored periodically throughout its service life in accordance with 10 CFR 50.55a and 10 CFR 50, Appendix J, to evaluate the integrity of containment over time (GDC 1 and GDC 16). As part of this monitoring program, containment deformations and exterior surface conditions are determined while the building is pressurized. ~~at the maximum calculated DBA pressure (P_a). Initial conditions, baseline measurements taken at P_a, during depressurization following the SIT are established prior to initial operation.~~ Initial measurements and in-service inspection meet the requirements of the following:

- ASME Code, 2004 Edition, Section XI, Rules for Inservice Inspection of Nuclear Power Plant Components, Subsections IWE and IWL.
- Supplemental Inspection Requirements of 10 CFR 50.55a.
- ASME Code, 2004 Edition, Section XI, Rules for Inservice Inspection of Nuclear Power Plant Components, Subsection IWL, does not contain specifications for inservice inspection of grouted tendons. For inservice inspection of grouted tendons, the guidelines of RG 1.90, Revision 1 are followed. ~~with the following exceptions:~~
  - ~~Force monitoring of ungrouted test tendons is not provided:~~
    - ~~This exception to RG 1.90 is acceptable because all tendons used within the RCB are fully grouted.~~
  - ~~Pressurization at year one uses P_a instead of P_N:~~
    - ~~This exception is acceptable because the value of P_a is higher than that of P_N.~~
  - ~~Pressurization at years three and seven uses P_a instead of $1.15P_D$:~~

- ~~This exception is acceptable because the structural integrity is confirmed at year zero. Additional overpressurization to 1.15P_D unduly cycles the structure and interrupts the surveillance tracking of containment response to P_a.~~

~~The EPR containment uses fully grouted tendons in each location. This methodology has several advantages:~~

- ~~Tendons are surrounded with a cementitious grout injected into the tendon duct; the alkaline composition of the grout mixture, in accordance with RG 1.107, Revision 1 (February 1977), inhibits corrosion of the steel strands and prevents the ingress of corrosive fluids (e.g., water).~~
- ~~In the event of one or more strand failures during the life of the structure, the bond of the strand with grout and the grout to the concrete wall enables the remaining portion of the post-tensioning to be transmitted to the structure.~~
- ~~Grouted tendons and tendon anchorages are less vulnerable to local damage than ungrouted tendons. Therefore, if the end anchorages are damaged, for instance by fire or missile impact, the post-tensioning force will be maintained along the effective length of the tendon.~~
- ~~Grouted tendons increase the overall wall tightness by filling any voids from within the structure. This reduces the risk of water or other contaminates from entering through wall cracks or tendon end caps.~~
- ~~European experience has found that grouted tendons significantly improve concrete crack distribution when the containment is pressurized to a point where the tensile stress of the concrete is exceeded. Less local large tensile strains are likely to occur thus diminishing the risk of having large concrete cracks behind the containment liner. The absence of large cracks improved the safety margin of the liner with regard to air tightness.~~

~~The use of grouted tendons precludes the possibility of directly measuring the post-tension force over time by lifting off at the anchorages. The U.S. EPR mitigates this concern by extensively monitoring the movement of the RCB during 10 CFR 50, Appendix J, leak-rate testing at P_a. The pressure test schedule is a part of the inservice inspection program. Movements obtained from the initial test will be used to baseline a structural analysis that will be used to predict the capacity of the RCB over time. Thirty-six RCB locations will be monitored for radial displacement, 6 for vertical displacement and 13 on the dome for tri-directional displacement. Table 3.8-7—ISI Schedule for the U.S. EPR.~~

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~~The RCB is fully enclosed by the RSB; therefore, the potential for corrosion of the tendon system is significantly reduced.~~

The U.S. EPR containment differs in some aspects from the "reference containment" as defined in RG 1.90, Revision 1. The U.S. EPR containment ISI program will be

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developed using the concepts presented in RG 1.90, Revision 1. In accordance with RG 1.90, Revision 1, the tendons for the U.S. EPR will be included in an ISI program. The program will consist of three items;

- Force monitoring of ungrouted test tendons.
- Monitoring of deformations under pressure at prescribed locations (Alternative B of RG 1.90, Revision 1).
- Visual inspection of exposed structurally critical areas of the containment and containment prestressing system.

The ISI Program will also include a fourth component for the inspection of the test tendons filler grease. The inspection of filler grease will be performed consistent with the guidance in RG 1.35, R3, Regulatory Position 6.

Three greased tendons of each type (vertical, gamma and horizontal hoop) will be provided for force monitoring. The test tendons are included in the number of tendons required by design and will be subjected to force measurement by lift-off or load cells to assess the effects of concrete shrinkage and creep and relaxation of the tendon steel. The nine greased tendons form the sample size for load cell or lift-off testing,

In accordance with the Alternative B of RG 1.90, Revision 1, the points to be instrumented for measurement of radial displacements under pressure will be located in six horizontal planes in the cylindrical portion of the shell with a minimum of four points in each plane.

The points to be instrumented for measurement of vertical (or radial) displacements under pressure will be located at the top of the cylinder relative to the base, at a minimum of four approximately equally spaced azimuths. Locations will also be selected at the apex of the dome and one intermediate point between the apex and the springline on at least three equally spaced azimuths .

The visual inspections will be performed of representative areas at structural discontinuities, areas around large penetrations or a cluster of small penetrations, and other areas where heavy loads are transferred to the containment structure. Visual inspection of these selected areas will be completed during the pressure tests while the containment is at maximum test pressure. Also included will be samples of the exposed portions of the tendon anchorage assembly hardware. The tendon anchorage assemblies utilized for the greased tendons will be representative of the grouted tendons except that provisions will be provided to allow force measurement by lift-off of load cells. The sample size of tendon anchorage assemblies will comply with the requirements of RG 1.90, Revision 1.

The pressure test schedule is part of the ISI program and is provided in Table 3.8-7 - ISI Schedule for the U.S. EPR.

Section 6.2.6 contains a description of the associated leak-rate test procedure, Containment Integrated Leakage Rate Test (CILRT). Containment pressure testing will occur in conjunction with the CILRT.

Sufficient physical access is provided in the annulus between the RCB and the RSB to perform inservice inspections on the outside of the containment. Space is available inside of the RCB to perform inservice inspections of the liner plate. Gaps are provided between the liner and RB internal structures concrete structural elements, which provide space necessary to inspect the liner at wall and floor locations inside containment. Inservice inspection of the embedded portion of the containment liner and the surface of the concrete containment structure covered by the liner are exempted in accordance with Section III of the ASME Code for Class CC components.

3.8.2 Steel Containment

The steel containment section describes major RCB penetrations and portions of penetrations not backed by structural concrete that are intended to resist pressure. Section 3.8.1 describes the concrete RCB.

3.8.2.1 Description of the Containment

Steel items that are part of the RCB pressure boundary and are not backed by concrete include the equipment hatch, airlocks, construction opening, piping penetration sleeves, electrical penetration sleeves, and fuel transfer tube penetration sleeve. Section 3.8.1.1 describes RCB steel items that are backed by concrete, such as the liner plate.

3.8.2.1.1 Equipment Hatch, Dedicated Spare Penetration, Airlocks, and Construction Opening

The equipment hatch, illustrated in Figure 3.8-25 is a welded steel assembly with a double-sealed, flanged, and bolted cover. The cover for the equipment hatch attaches to the hatch sleeve from inside of the RCB. The cover seats against the sealing surface of the penetration sleeve mating flange when subjected to internal pressure inside the RCB. The RCB penetration sleeve and the RSB penetration sleeve are connected by an expansion joint to allow for differential movement between the two walls, as shown in Figure 3.8-25. The equipment hatch opens into the Seismic Category I FB, which provides protection of the hatch from external environmental hazards (e.g., high wind, tornado wind and missiles, and other site proximity hazards, including aircraft hazards and blasts). The equipment hatch sleeve has an inside diameter of approximately 27 feet, 3 inches.

54. ASTM 325-07, "Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength," American Society for Testing and Materials, 2007.

55. ASTM A490-06, "Standard Specification for Structural bolts, Alloy Steel, Heat Treated, 150 ksi Minimum Tensile Strength," American Society for Testing and Materials, 2006.

56. ASTM A307-07, "Standard specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength," American Society for Testing and Materials, 2007.

57. Gazetas, George, "Foundation Vibrations," Chapter 15 in Foundation Engineering Handbook, 2nd Edition, edited by Hsai-Yang Fang, CBS Publishers, New Delhi, India, 1997.

58. ACI 350-06, "Code Requirements for Environmental Engineering Concrete Structure," American Concrete Institute, 2006.

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

60. ASME B31.3, "Process Piping, American Society of Mechanical Engineers," American Society of Mechanical Engineers, 1996.

61. ASME B31.4, "Liquid Transportation System for Hydrocarbon, Liquid Petroleum Gas, Anhydrous Ammonia, and Alcohols," American Society of Mechanical Engineers, 1992.

62. ASME B31.8, "Gas Transportation and Distribution Piping Systems," American Society of Mechanical Engineers, 1995.

63. ACI 349-06/349R-06, "Code Requirements for Nuclear Safety-Related Concrete Structures" and Commentary, American Concrete Institute, 2006.

64. NUREG/CR-5096 - "Evaluation of Seals for Mechanical Penetrations of Containment Buildings," August 1998.

65. IBC-2009, International Code Council, International Building Code, 2009 edition.

66. ACI 229R-99, "Controlled Low-Strength Materials," American Concrete Institute, 1999.

67. ASTM D-1557-09, "Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort," American Society for Testing and Materials, 2009.

68. EM 1110-1-1904, "Settlement Analysis," U.S. Army Engineering Manual, 1990.

69. Bechtel Power Corporation Topical Report, BC-TOP-1, Containment Building Liner Plate Design Report, Revision 1, December 1972.

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Table 3.8-7—ISI Schedule for the U.S. EPR

	Test Pressure	
Year	**U.S. EPR ISI**	**RG 1.90**
0	$1.15{*}P_d$ ~~and P_a~~	$1.15{*}P_d$
1	$P_{\cancel{a}\underline{N}}$	P_N
3	$\underline{1.15}{*}P_{\cancel{a}\underline{d}}$	$1.15{*}P_d$
7	$\underline{1.15}{*}P_{\cancel{a}\underline{d}}$	$1.15{*}P_d$
Thereafter	P_a	P_a

Notes:

1. ~~At year 0, the baseline measurements will be taken following the SIT, at a test pressure of P_a.~~

   P_N – Normal operating pressure or zero.

   P_d – Containment design pressure, ~~.~~ $\underline{P_d = 62\ \text{psig}}$.

   Pa – Maximum calculated DBA pressure, ~~.~~ $\underline{P_a = 55\ \text{psig}}$.

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Table 3.8-8—Materials for Structural Steel Shapes and Plates
Sheet 1 of 2

ASTM Designation	F_y	F_u
A36	36 ksi	58 to 80 ksi
A53 (Type E or S) (Gr. B)	35 ksi	60 ksi
A106 Grade A Grade B Grade C	 30 ksi 35 ksi 40 ksi	 48 ksi 60 ksi 70 ksi
A167	27 to 39 ksi	73 to 94 ksi
A240 Austenitic Duplex Ferritic or Martensitic	 25 to 70 ksi 58 to 80 ksi 25 to 90 ksi	 70 to 125 ksi 87 to 116 ksi 55 to 115 ksi
A242	42 to 50 ksi	63 to 70 ksi
A276 Austenitic Austenitic-ferritic Ferritic Martensitic	 25 to 125 ksi 65 to 105 ksi 30 to 60 ksi 30 to 100 ksi	 70 to 145 ksi 90 to 125 ksi 60 to 75 ksi 60 to 125 ksi
A312	25 to 62 ksi	70 to 115 ksi
A441	40 to 50 ksi	60 to 70 ksi
A479 03.08.01-55 Austenitic Austenitic-ferritic Ferritic	 25 to 125 ksi 65 to 85 ksi 25 to 55 ksi	 70 to 145 ksi 90 to 118 ksi 60 to 70 ksi
Martensitic	40 to 100 ksi	70 to 130 ksi
~~A276 (Martensitic)~~	~~40 to 100 ksi~~	~~70 to 130 ksi~~
A500 (round) Grade A Grade B Grade C Grade D	 33 ksi 42 ksi 46 ksi 36 ksi	 45 ksi 58 ksi 62 ksi 58 ksi
A500 (square & rectangular) Grade A Grade B Grade C Grade D	 39 ksi 46 ksi 50 ksi 36 ksi	 45 ksi 58 ksi 62 ksi 58 ksi
A501	36 ksi	58 ksi
A514	90 to 100 ksi	100 to 130 ksi
A515	32 to 38 ksi	60 to 90 ksi
A516	30 to 38 ksi	55 to 90 ksi

Table 3.8-8—Materials for Structural Steel Shapes and Plates
Sheet 2 of 2

ASTM Designation	F_y	F_u
A570	30 to 55 ksi	49 to 70 ksi
A572	42 to 65 ksi	60 to 80 ksi
A588	42 to 50 ksi	63 to 70 ksi
A607		
Class I	45 to 70 ksi	60 to 85 ksi
Class II	45 to 70 ksi	55 to 80 ksi
A618		
Grade Ia, Ib & II	46 to 50 ksi	67 to 70 ksi
Grade III	50 ksi	65 ksi
A709	36 to 50 ksi	58 to 80 ksi
A913	50 to 70 ksi	65 to 90 ksi
A992	50 to 65 ksi	65 ksi

Note:

1. The design of structural steel members is based on the conservative use of the minimum allowable material stress values provided in Table 3.8-8. The design specifies a particular minimum value to be used for the fabrication of the component, and the stress values of the materials actually used in fabrication will be confirmed by certified material test reports and certificates of conformance.

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5.5 Programs and Manuals

5.5.4 Component Cyclic or Transient Limit

This program provides controls to track the FSAR Section 3.9.1.1, cyclic and transient occurrences to ensure that components are maintained within the design limits.

5.5.5 Pre-Stressed Concrete Containment Tendon Surveillance Program

This program provides for the monitoring of the containment post tensioning force over time. Tendons used in the containment structure are fully grouted and the structure itself is not exposed to the environment during its operational life. The program shall include baseline measurements prior to initial operation. The Tendon Surveillance Program, inspection frequencies, and acceptance criteria shall be in accordance with FSAR Section 3.8.1.7.

The provisions of SR 3.0.3 are applicable to the Tendon Surveillance Program Inspection frequencies.

~~Containment Post Tensioning Surveillance Program~~

~~This program provides for the monitoring of the containment post tensioning force over time. Tendons used in the containment structure are fully grouted and the structure itself is not exposed to the environment during its operational life. The program shall include initial base line measurements prior to initial operation. The Containment Post Tensioning Surveillance Program, inspection frequencies, and acceptance criteria shall be in accordance with Regulatory Guide 1.90, Rev. 1 with the following exceptions:~~

~~-Force monitoring of ungrouted test tendons is not provided~~
~~-Pressurization at year one uses~~ $\cancel{P_a}$ ~~instead of~~ $\cancel{P_N}$
~~-Pressurization at years three and seven use~~ $\cancel{P_a}$ ~~instead of~~ $\cancel{1.15P_D}$

~~The provisions of SR 3.0.3 are applicable to the Containment Post Tensioning Surveillance Program inspection frequencies.~~

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