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Your ref: Docket No. 52-006
Our ref: DCP_NRC_003024

August 25, 2010

Subject: AP1000 Response to Request for Additional Information (SRP 3)

Westinghouse is submitting a response to the NRC request for additional information (RAI) on SRP Section 3. This RAI response is submitted in support of the AP1000 Design Certification Amendment Application (Docket No. 52-006). The information included in this response is generic and is expected to apply to all COL applications referencing the AP1000 Design Certification and the AP1000 Design Certification Amendment Application.

Enclosure 1 provides the response for the following RAI(s):

RAI-SRP3.8.2-SEB1-02 R3
RAI-SRP3.8.2-SEB1-03 R3
RAI-SRP3.8.3-SEB1-04 R3
RAI-SRP3.8.3-SEB1-05 R3
RAI-TR85-SEB1-10 R6

Questions or requests for additional information related to the content and preparation of this response should be directed to Westinghouse. Please send copies of such questions or requests to the prospective applicants for combined licenses referencing the AP1000 Design Certification. A representative for each applicant is included on the cc: list of this letter.

Very truly yours,

Robert Sisk, Manager
Licensing and Customer Interface
Regulatory Affairs and Strategy

/Enclosure

1. Response to Request for Additional Information on SRP Section 3

D063
NRO

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ENCLOSURE 1

Response to Request for Additional Information on SRP Section 3

AP1000 TECHNICAL REPORT REVIEW

RAI Response Number: RAI-SRP-3.8.2-SEB1-02
Revision: 3

Question: (Revision 0)

Section 3.8.2.2, as well as other sections of the DCD related to structures, refers to DCD Section 1.9 for discussion of compliance with regulatory guides. The staff notes that for Regulatory Guides 1.7 and 1.57 the DCD complies with earlier revisions of the regulatory guides. For Regulatory Guide 1.160, the DCD indicates that it is not applicable to the AP1000 design certification and that Section 17.5 defines the responsibility for a plant maintenance program. Regulatory Guide 1.199 is not described at all in Section 1.9 of the DCD.

In view of the extension of the AP1000 design to soil sites, reanalysis for updated seismic spectra, design changes made to structures, and to ensure that the AP1000 meets the safety requirements in current staff positions, the staff requests Westinghouse to indicate whether the design, construction, and inspection of the AP1000 plant comply with the current regulatory guides stated above or explain how following the existing versions of the regulatory guides or Section 17.5 (for the plant maintenance program), referred to in the DCD, provides an equivalent level of safety to the guidance in the current versions of the regulatory guides. Describe the basis for the use of each regulatory guide, or alternative, separately.

In the case of Regulatory Guide 1.199, "Anchoring Components and Structural Supports in Concrete," what are the alternative requirements or criteria Westinghouse are using to meet the NRC's regulations in the design, evaluation, and quality assurance of anchors (steel embedments) used for component and structural supports on concrete structures as required by GDC 1, "Quality Standards and Records," GDC 2, "Design Bases for Protection Against Natural Phenomena," and GDC 4, "Environmental and Dynamic Effects Design Bases."

If your response to this request for additional information will reference Revision 17 to the AP1000 DCD, please provide an exact reference.

Additional Question: (Revision 1)

Design criteria/approach used by W vs the guidance contained in several key NRC regulatory guides not referenced in DCD. Westinghouse is requested to compare their design criteria/approach to the guidance contained in several key NRC regulatory guides. Westinghouse will provide a revised RAI response to address this item.

Additional Question: (Revision 2)

The staff reviewed the Westinghouse response to RAI-SRP3.8.2-SEB1-02, Rev.1 and determined that the response did not fully address all of the concerns related to NRC regulatory guides. Therefore, the following information is needed:

1. The RAI response did not identify whether the regulatory positions in RG 1.7 and RG 1.57 were applicable for the containment design for hydrogen generated pressure loads. Therefore, explain whether the regulatory positions in RG 1.7, Rev. 3 and RG 1.57, Rev. 1, related to containment structural integrity under the hydrogen generated pressure loads, were applicable or not. If not applicable, provide the methods used to address the containment design for hydrogen generated pressure loads.

2. The RAI response did not identify whether the regulatory positions in RG 1.57 were applicable for the containment design with respect to the design limits and load combinations in the RG. Therefore, explain whether the regulatory positions in RG 1.57, Rev. 1, related to the design limits and load combinations, were applicable or not. If not applicable, provide justifications for not using the design limits and load combinations in the RG and for the adequacy of the applicant design limits and loading combinations.

3. In the response, the applicant stated that DCD Appendix 1A indicates that RG 1.160 is, "Not applicable to AP1000 design certification. (DCD) Section 17.5 defines the responsibility for a Plant Maintenance Program," and the RAI response stated, "In (DCD) Subsection 17.5.6 the Combined License Information required to address the maintenance rule is provided." The staff believes that the DCD should document the basis for the design, construction, testing and inservice surveillance programs for plant structures. These review areas are clearly identified in SRP 3.8.1 through 3.8.5. In the case of the maintenance requirements, each SRP subsection identifies that RG 1.160 is applicable. Therefore, confirm that RG 1.160 is applicable for the maintenance of structures at the plant and confirm that it will be followed when implementing 10 CFR 50.65. Also, revise the DCD to reflect the applicability of RG 1.160, Rev. 2.

4. The response provided an assessment of the applicability of all regulatory positions in RG 1.199 (2003) to the AP1000 plant. In several cases, the response indicated that the regulatory positions do not have a design requirement or are not applicable to the AP1000 plant. As indicated in Item 3 above, the DCD should document the basis for the design, construction, testing and inservice surveillance programs for plant structures. Therefore, the DCD should be revised to indicate that the regulatory position in RG 1.199 (2003) is applicable for anchoring components and structural supports in concrete for the AP1000 plant or provide alternate methods for anchoring components and structural supports in concrete, or provide the basis for not following the position and any alternative taken.

Additional Question: (Revision 3)

The staff reviewed the Westinghouse response to RAI-SRP3.8.2-SEB1-02 Rev. 2, transmitted in their letter dated July 2, 2010. The information provided in the response addressed the concerns, related to the AP1000 design conformance with key NRC Regulatory Guides, except for two items.

(1) On Page 7 of 14 of the response, the entry for the DCD section number, in the fourth row and the third column (i.e., 3.8.4.6.3) of "Table 1.9-1 (Sheet 15 of 15)," needs to be revised to be consistent with the proposed DCD mark-ups shown on the other pages (i.e., include the 3 subsection nos. identified on pages 8 and 9 of 14).

(2) On page 8 of 14, the proposed mark-up for Subsection 3.8.4.5.1 indicates that the "[Design of fastening to concrete is in accordance with ACI 349-01, Appendix B]* and are ..." The text should be revised to be consistent with the corresponding proposed mark-ups to Subsections 3.8.3.5 and 3.8.5.5, where it states that "Design and construction of anchors and embedments conform to the procedures and standards of Appendix B to ACI 349-01 ...]." Also, for all of the three subsections, the Tier 2* information should be consistent (i.e., the portion of the statement that refers to ACI 349-01 Appendix B should be identified as Tier 2* as was done in Subsection 3.8.4.5.1.

Westinghouse Response: (Revision 0, 1)

Addition information is provided for Regulatory Guide 1.160 and Regulatory Guide 1.199 in Revision 1 of the response.

Regulatory Guide 1.7:

The current AP1000 certified design is consistent with Revision 3 of Regulatory Guide 1.7 (issued in March 2007). The AP1000 containment design is a passive system, using convective mixing. Design features promote free circulation of the containment atmosphere. NUREG-1793, "Final Safety Evaluation Report Related to the Certification of the AP1000 Standard Design, Docket 52-006," USNRC, Washington, DC, September 2004 (NUREG-1793), documents an analysis of the effectiveness of the passive mixing. Section 6.2.5.5 of NUREG-1793 concluded:

> "The (U.S. Nuclear Regulatory Commission) staff has determined that the Containment hydrogen control system meets the requirements of GDC 41 and 10 CFR 50.44, as well as the guidelines of draft RG 1.7, Revision 3."

Therefore, it can be concluded that there is no technical or safety issue impact due to this Regulatory Guide revision on the AP1000 design, design processes, or licensing documentation.

Regulatory Guide 1.57:

RG 1.57 Revision 1 (issued in March 2007) endorses ASME Boiler & Pressure Vessel Code (B&PV), Section III, "Rules for Construction of Nuclear Facility Components," Division 1, Subsection NE, "Class MC Components," 2001 Edition with 2003 Addenda and Section XI, "Rules for Inservice Inspection of Nuclear Power Plant Components," 2001 Edition with 2003 Addenda.

The containment vessel is designed to meet the requirements of ASME B&PV Code, Section III, "Rules for Construction of Nuclear Facility Components," Division 1, Subsection NE, "Class MC Components," 2001 Edition including 2002 Addenda. The 2003 Addenda did not include any requirements to impact the design of the containment vessel described in the DCD. There are only two changes (which are in Subsection NE-5000 "Examination") and are related to the examination of the welds and do not impact the design.

- NE-5222 socket welds: Socket welds shall be examined by the magnetic particle or liquid penetrant method.
- NE-5261 butt welded joints: All butt welded joints in pressure retaining parts not included in Categories A, B, C, and D – such as doors, opening frames, permanent attachments, and similar constructions – shall be radiographed.

Therefore, the containment vessel design is in conformance with this Regulatory Guide.

Regulatory Guide 1.160

Regulatory Guide 1.160 provides guidance on the effectiveness of maintenance at nuclear power plants. This guidance is provided for the establishment of maintenance programs consistent with the maintenance rule (10 CFR 50.65). This regulatory guide has no impact on the design, analysis, fabrication, or construction of the AP1000 containment. DCD Appendix 1A addresses the conformance of the AP1000 with Regulatory Guide 1.160. As noted in DCD Appendix 1A, Regulatory Guide 1.160 is "Not applicable to AP1000 design certification. Section 17.5 defines the responsibility for a Plant Maintenance Program." In Subsection 17.5.6 the Combined License information required to address the maintenance rule is provided. Section 17.5 has no impact on the design, analysis, fabrication, or construction of the AP1000 containment design. There are no special maintenance requirements for the containment identified in Subsection 3.8.2.

Regulatory Guide 1.199:

This new Regulatory Guide (Revision 0) was issued in November 2003, to provide guidance to licensees and applicants on methods acceptable to the U.S. NRC staff for complying with the U.S. NRC regulations in the design, evaluation, and quality assurance of anchors (steel embedments) used for component and structural supports on concrete structures. As a result of studies and tests performed, questions were raised regarding the design methodology used in Appendix B to American Concrete Institute (ACI) 349-80. After an extensive review of available test data, the ACI 349 code committee issued a revision to ACI 349 Appendix B in February 2001.

This Regulatory Guide 1.199 generally endorses Appendix B to ACI 349-01, with exceptions in the area of load combinations.

- The AP1000 nuclear island concrete structures are designed to meet the requirements of ACI 349-01 code, including Appendix B on the design of anchors in concrete.
- The load combinations used in the design of nuclear island concrete structures were reviewed and approved by the U.S. NRC, after the release of this Regulatory Guide, in the AP1000 design certification for the hard rock sites.

Itemized conformance with regulatory positions (discussed in Section C of this Regulatory Guide) is given in the attached table.

Westinghouse Additional Response: (Revision 2)

1. As noted previously, the AP1000 Containment Vessel design is consistent with the guidance of Regulatory Guides 1.7 Rev. 3 and 1.57 Rev. 1. These Regulatory Guides were published in March 2007 and are therefore not applicable to the AP1000 design certification. The method used to address hydrogen generated loads is included in the response to RAI-SRP3.8.2-SEB1-03, Rev. 1 Question #3.

2. The regulatory positions in RG 1.57, Rev. 1 are not applicable to the design of the AP1000 Containment Vessel. As noted previously, the containment vessel is consistent with the guidance provided in Regulatory Guide 1.57, Rev. 1. This Regulatory Guide was published in March 2007 and is therefore not applicable to the AP1000 design certification. Meeting the requirements provided in this RG and the SRP 3.8.2 are provided in the response to RAI-SRP3.8.2-SEB1-03, Rev. 2.

3. Although design information is provided in the DCD that supports the implementation of the maintenance program, conformance to Regulatory Guide 1.160 is the responsibility of the

COL applicant. Westinghouse cannot commit in the DCD to conformance to Regulatory Guide 1.160 for the COL applicant.

The design, construction information, and material requirements for the structures are captured in the DCD Section 3.8. Special construction techniques for the containment are outlined in DCD Subsection 3.8.2.6. Seals provided at the top of the concrete on the inside and outside of the vessel to prevent moisture between the vessel and concrete are identified in DCD Subsection 3.8.2.1.2. Provisions for corrosion protection for the containment are discussed in DCD Subsection 3.8.2.6. There are no inservice inspection requirements required to assure containment corrosion protection. Construction of the containment internal structures, including structural module is discussed in DCD Subsection 3.8.3.6. Special construction techniques for the structural module in the shield building and auxiliary building are the same as described in DCD Subsection 3.8.3.8. There are no special construction techniques used in the construction of the nuclear island structures foundation. The design and requirements for protective coatings are described in DCD Subsection 6.1.2.1. The COL Information Item for a coating program is described in DCD Subsection 6.1.3.2.

The inservice inspection and testing requirements for the containment are consistent with ASME Boiler and Pressure Vessel Code and Regulatory requirements. See Subsection 3.8.2.7 of the DCD. Construction inspection for the containment internal structures is addressed in DCD Subsection 3.8.3.8. DCD Subsection 3.8.4.7 addresses the examination requirements of structures supporting the passive containment cooling water storage tank on the shield building before and after first filling of the tank. Construction inspection for the nuclear island structures foundation is addressed in DCD Subsection 3.8.5.8.

The statements about inservice inspection in Subsection 3.8.3.7, 3.8.4.7, and 3.8.5.7 will be deleted and a statement about the establishment of a structures inspection program will be added.

The AP1000 structures, with the exception of the containment, do not include design features that require continuing monitoring or inservice inspection or testing. The AP1000 does not include post-tension tendons, seismic isolators, flood gates or other features that would require inspection and maintenance.

In Regulatory Guide 1.206 "Combined License Applications for Nuclear Power Plants" Criterion C.I.1.9.1 states that"...COL applicants should provide an evaluation of conformance with the guidance in NRC regulatory guides...". In practice the COL application incorporates by reference DCD Table 1.8-2 and supplements this with a table that completes the evaluation of regulatory guides for which the DCD does not provide a conformance evaluation. Westinghouse works closely with the AP1000 Design Centered Working Group and it would not be appropriate for Westinghouse to make commitments for the COL applicants for operational programs such as monitoring conditions of structures. A

COL information item on a structures inspection program will be added as Subsection 3.8.6.5.

4. Conformance to Regulatory Guide is defined in the appendix to this response. Conformance to Regulatory Guide 1.199 will be added to DCD Appendix 1A as shown below.

   Specific requirements for the design and construction of anchors and embedments to conform to the procedures and standards of Appendix B to ACI 349-01 conform with the regulatory positions of USNRC Regulatory Guide 1.199 will be added to Subsection 3.8.3.5, 3.8.4.5.1, and 3.8.5.5 as shown below.

   A specific commitment to conform to ASME NQA-2, 1983, for load-bearing steel embedments is not required or appropriate. As documented in DCD Section 17.3, NQA-1-1994 is the applicable revision of NQA-1 for work performed for the AP1000 project. The 1994 Edition of NQA-1 incorporates the quality assurance requirements of the previous NQA-2 as Part II of NQA-1. Subsection 3.8.4.6.2 applies the quality assurance program described in Chapter 17. This subsection is applicable to the structures discussed in Subsection 3.8.3, 3.8.4, and 3.8.5.

Westinghouse Additional Response: (Revision 3)

(1) The entry for Regulatory Guide 1.199 in Table 1.9-1 is corrected as shown below to include reference to Subsections 3.8.3.5, 3.8.4.5.1, and 3.8.5.5.

(2) The text in 3.8.4.5.1 is revised to include construction as well as design of fastening.

The text added to subsections 3.8.3.5 and 3.8.5.5 is marked as Tier 2* to be consistent with the marking of the information in Subsection 3.8.4.5.1.

The entry for ACI-349-01 in Table 1-1 of the DCD Introduction is revised to add reference to Subsection 3.8.3.5.

Design Control Document (DCD) Revision: (Revision 0, 1, 2, and 3)

Revise the entry for ACI-349-01 in Table 1-1 of the DCD Introduction as shown below

Table 1-1

Index of AP1000 Tier 2 Information Requiring NRC Approval for Change

Item	Expiration at First Full Power	Tier 2 Reference
Use of ACI-349-01	Yes	3.8.3.2 3.8.3.5 3.8.4.2 3.8.4.4.1 3.8.4.5 3.8.4.5.1 3.8.5.5 Table 3.8.4-2

Add an entry for Subsection 3.8.6.5 in Table 1.8-2 as shown at the end of this response.

Add the entries for Regulatory Guides 1.160 and 1.199 to Table 1.9-1 as follow:

Table 1.9-1 (Sheet 13 of 15)

REGULATORY GUIDE/DCD SECTION CROSS-REFERENCES

Division 1 Regulatory Guide		DCD Chapter, Section or Subsection
1.160	Monitoring the Effectiveness of Maintenance at Nuclear Power Plants (Rev. 2, March 1997)	3.8.6.5 17.5.6 The COL applicant is responsible for assessing conformance to Regulatory Guide 1.160 of monitoring the effectiveness of maintenance ~~This regulatory guide is not applicable to AP1000 design certification~~

Table 1.9-1 (Sheet 15 of 15) **REGULATORY GUIDE/DCD SECTION CROSS-REFERENCES**		
Division 1 Regulatory Guide		**DCD Chapter, Section or Subsection**
1.197	Demonstrating Control Room Envelope Integrity at Nuclear Power Reactors (Rev. 0, May 2003)	9.4.1 6.4.5
1.199	Anchoring Components and Structural Supports in Concrete (Rev. 0, November 2003	3.8.3.5 3.8.4.5.1 3.8.5.5

Revise the conformance of Reg. Guides 1.160 in Appendix 1A as follows:

Reg. Guide 1.160, Rev. 2, 3/97 – Monitoring the Effectiveness of Maintenance at Nuclear Power Plants

General	N/A	Not applicable to AP1000 design certification. Section 17.5 defines the responsibility for a Plant Maintenance Program. The COL applicant is responsible for assessing conformance to Regulatory Guide 1.160 of monitoring the effectiveness of maintenance

Add the following to Appendix 1A following the conformance of Reg. Guides 1. 197

Reg. Guide 1. 199, Rev. 0, 11/03 – Anchoring Components and Structural Supports in Concrete

C.1 –C.7	Conforms

Make this the third paragraph of Subsection 3.8.3.5-:

> [*The design and construction of anchors and embedments conform to the procedures and standards of Appendix B to ACI 349-01*]*and are in conformance with the regulatory positions of USNRC Regulatory Guide 1.199, Revision 0.

Revise DCD Subsection 3.8.3.7 as follows:

3.8.3.7 In-Service Testing and Inspection Requirements

~~There are no containment internal structures design features that require special in-service testing or inspection or special maintenance requirements.~~ The Combined License applicant is responsible for the establishment of an inspection program for structures. See Subsection 3.8.6.5.

Revise the third paragraph of Subsection 3.8.4.5.1 as follows:

[*Design and construction of fastening to concrete is in accordance with ACI 349-01, Appendix B*~~.~~]* and are in conformance with the regulatory positions of USNRC Regulatory Guide 1.199, Revision 0.

Revise Subsection 3.8.4.7 as follows:

3.8.4.7 Testing and In-Service Inspection Requirements

Structures supporting the passive containment cooling water storage tank on the shield building roof will be examined before and after first filling of the tank.

- The boundaries of the passive containment cooling water storage tank and the tension ring of the shield building roof will be inspected visually for excessive concrete cracking before and after first filling of the tank. Any significant concrete cracking will be documented and evaluated in accordance with ACI 349.3R-96 (reference 50).
- The vertical elevation of the passive containment cooling water storage tank relative to the top of the shield building cylindrical wall at the tension ring will be measured before and after first filling. The change in relative elevation will be compared against the predicted deflection.
- A report will be prepared summarizing the test and evaluating the results.

~~There are no other in-service testing or inspection requirements for the seismic Category I shield building and auxiliary building. However, during~~ During the operation of the plant the condition of these structures should be monitored by the Combined License holder to provide reasonable confidence that the structures are capable of fulfilling their intended functions. The Combined License applicant is responsible for the establishment of an inspection program for structures. See Subsection 3.8.6.5.

Add a new paragraph following the existing first paragraph of Subsection 3.8.5.5 as follows:

3.8.5.5 Structural Criteria

The analysis and design of the foundation for the nuclear island structures are according to ACI-349 with margins of structural safety as specified within it. The limiting conditions for the foundation medium, together with a comparison of actual capacity and estimated structure loads, are described in Section 2.5. The minimum required factors of safety against sliding, overturning, and flotation for the nuclear island structures are given in Table 3.8.5-1.

[*The design and construction of anchors and embedments conform to the procedures and standards of Appendix B to ACI 349-01*]* and are in conformance with the regulatory positions of USNRC Regulatory Guide 1.199, Revision 0.

Revise DCD Subsection 3.8.5.7 as follows:

3.8.5.7 In-Service Testing and Inspection Requirements

~~There are no nuclear island structures foundation design features that require special in-service testing, inspection, or special maintenance requirements.~~ The Combined License applicant is responsible for the establishment of an inspection program for structures. See Subsection 3.8.6.5.

The need for foundation settlement monitoring is site-specific as discussed in subsection 2.5.4.5.10.

Add a new Subsection 3.8.6.5 as follows:

3.8.6.5 Structures Inspection Program

Consistent with Subsection 17.5.6, the Combined License applicant is responsible for the establishment of a structures inspection program consistent with the maintenance rule (10 CFR 50.65) and guidance in Regulatory Guide 1.160 to address maintenance requirements for the seismic Category I and seismic Category II structures.

PRA Revision:
None

Technical Report (TR) Revision:
None

Add an entry for Subsection 3.8.6.5 in Table 1.8-2 as follows:

Table 1.8-2 (Sheet 4 of 13)

SUMMARY OF AP1000 STANDARD PLANT COMBINED LICENSE INFORMATION ITEMS

Item No.	Subject	Subsection	Addressed by Westinghouse Document	Action Required by COL Applicant	Action Required by COL Holder
3.8-4	Deleted In-Service Inspection of Containment Vessel	Deleted	APP-GW-GLR-021	N/A	N/A
3.8-5	Structures Inspection Program	3.8.6.5	N/A	Yes	–
3.9-1	Reactor Internal Vibration Response	3.9.8.1	WCAP-16687-P	No	No

Attachment 1 - Itemized Conformance with Regulatory Positions (from Section C of Regulatory Guide 1.199). Positions were provided for C1 through C1.4 in Revision 0. Positions for C1.5 through C7. are provided in Revision 1

REGULATORY POSITION	AP1000 POSITION
C1. The procedures and standards of Appendix B to ACI 349-01 are acceptable to the NRC staff as described and supplemented below. The recommendations are applicable to the types of anchors discussed in Section B.1, "Definitions," and B.2, "Scope," of Appendix B to ACI 349-01.	Conforms
C1.1 The notations and definitions given in Sections B.0 and B.1 of Appendix B toACI349-01 are acceptable to the NRC staff.	Conforms
C1.2 The position on load combinations is given in Regulatory Position 1.3. In addition to the guidance of Section B.3.3 of Appendix B, the testing recommendations defined in ASTME488-96, "Standard Test Methods for Strength of Anchors in Concrete and Masonry Elements," are acceptable to the NRC staff as a guide for establishing a testing program. Test methods not covered by ASTM E488-96 (e.g., combined tension and shear, cracked concrete) should be established and executed using good engineering judgment. ACI 355.2-01, "Evaluating the Performance of Post-Installed Mechanical Anchors in Concrete," provides guidance acceptable to the NRC staff for determining whether post-installed mechanical anchors are acceptable for use in uncracked as well as cracked concrete. For materials consideration, the NRC staff recommends that anchors be fabricated using a material that is compatible with the environment in which they will be installed.	Conforms

C1.3 The load factors used in Section 9.2.1 of ACI 349-01 are acceptable to the NRC staff except for the following: 1.3.1. In load combinations 9, 10, and 11, 1.2To should be used in place of I.05To. 1.3.2. In load combination 6, 1.4Pa should be used in place of 1.25Pa. 1.3.3. In load combination 7, 1.25Pa should be used in place of 1.15Pa. 1.3.4. The NRC staff endorses Section B.4, "General Requirements for Strength of Structural Anchors," of ACI 349-01. The NRC staff endorses the strength reduction factors given in Section B.4.4; however, load factors consistent with SRP Section 3.8.4, "Other Seismic Category I Structures," should be applied to the load combinations given in Section 9.2 of ACI 349-01.	Conforms
C1.4 The design standards given in Sections B.5, "Design Requirements for Tensile Loading," and B.6, "Design Requirements for Shear Forces," are acceptable to the staff.	Conforms
C1.5 The design standards given in Sections B.7, "Interaction of Tensile and Shear Forces," and B. 8, "Required Edge Distances, Spacing, and Thickness To Preclude Splitting Failure," are acceptable to the NRC staff.	Conforms
C1.6 Section B.9, "Installation of Anchors," is acceptable to the NRC staff.	Conforms
C1.7 The design standards given in Sections B.10, "Structural Plates, Shapes, and Specialty Inserts," and B.11, "Shear Capacity of Embedded Plates and Shear Lugs," are acceptable to the NRC staff. When grouting is the only option, it is recommended that tests be performed in accordance with Sections B.12.3 and B.12.4 of Appendix B.	Conforms

C2. All anchors should be inspected to verify that they are of the specified size and type. Installation standards should be consistent with accepted industry-specified tolerances. Anchor systems that are external (that part or portion of the anchor that is not embedded in concrete-visible part) to the concrete surface should be inspected to assure adequate performance during the life of the structure. In addition to the provisions in Section B.9.2 of Appendix B, the NRC staff recommends the following post-installed 6-step inspection program to verify the proper installation of post-installed anchors.	Conforms ~~This paragraph does not have a design requirement.~~
C3. All quality assurance standards of ASME NQA-2, 1983, "Quality Assurance Program Requirements for Nuclear Facilities," are applicable to load-bearing steel embedments and other load-bearing components of component and structural supports.	Conforms The AP1000 is in conformance with NQA-1-1994 which incorporated the assurance standards of ASME NQA-2, 1983 ~~This paragraph does not have a design requirement..~~
C4. The concrete constituents and embedded materials should be compatible with the anticipated environmental conditions to which they will be subjected during the life of the plant.	Conforms
C5. Loads and forces on embedments should be properly evaluated to account for baseplate flexibility and eccentricity of connections and the dynamic (strain rate and low-cycle fatigue) effects of loads and forces.	Conforms
C6. The hardness, materials, and heat treatment of high-strength anchor bolts and studs (Fy > 110 ksi) should be carefully controlled to prevent environmental and stress-corrosion cracking.	Conforms
C7. Because anchors are not generally specified for masonry, the NRC staff does not recommend the use of any type of anchor discussed in this guide to attach Seismic Category I components or systems to concrete block walls that are seismically qualified, except for extremely low load applications. In locations where it is impossible to avoid the use of anchors, users should verify through appropriate means (e.g., pull test) that the supports are structurally acceptable.	Conforms In AP1000 plant design, Seismic Category I components or systems are not attached to concrete block walls.

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-SRP3.8.2-SEB1-03
Revision: 3

Question: (Revision 0)

Table 3.8.2-1 of DCD Rev. 16, which provides the load combinations and service limits for the steel containment vessel, has been revised. Westinghouse is requested to explain the following items:

1. Why were other load combinations identified in NUREG-0800, SRP 3.8.2, Acceptance Criteria and Regulatory Guide 1.57, Rev. 1, omitted? (e.g., SRP 3.8.2 II.3.B.iii.(1)(a); II.3.B.iii.(3)(b), (d), and (e); and II.3.B.iii.(5) for post flooding condition). Please provide the bases for omitting the load combinations and reference any necessary documents to support this action.

2. A new load combination has been added in the DCD for Service Levels A and D, which includes the external pressure of 0.9 psid. Westinghouse is requested to provide the technical basis for this pressure load and provide the corresponding temperature value and the basis for this temperature.

   Clarify in the DCD what is meant by "loss of all AC in cold weather" used in Footnotes 3 and 5.

3. Although load combinations with OBE are not required because the OBE is defined as less than or equal to 1/3 of the SSE, there is no indication that the OBE loading is considered in the appropriate load combinations for fatigue as described in SRP 3.8.2 acceptance criterion - II.3.B.iii.(2).

If your response to this request for additional information will reference Revision 17 to the AP1000 DCD, please provide an exact reference.

Additional Question (Revision 1)

Confirm that several additional load combinations identified in the RAI were considered in the design of the containment. During a conversation with the NRC load combinations of interest were identified.

Additional Question (Revision 2)

10 CFR 50, Appendix A, General Design Criterion (GDC) 50, requires that nuclear power plant containment structures be designed with sufficient margin of safety to accommodate appropriate

design loads. In addition, 10 CFR 50.44 requires the capability of containments be demonstrated to resist loads associated with combustible gas generation from a metal-water reaction of the fuel cladding. As a result of the staff review of the Revision 1 response to RAI SRP-3.8.2-SEB1-03 and the related Revision 4 response to RAI TR09-08, the staff requests that the following items, related to the load combinations for the steel containment, be addressed:

1. Clarify the following items related to the revision of DCD Table 3.8.2-1 described in the Revision 4 response to RAI TR09-08;

   a. A load combination which combines W (wind) plus Pe (external pressure) is included in the proposed revision to Table 3.8.2-1. Explain why a load combination that combines W plus Pd (design pressure representing LOCA) is not also included. SRP 3.8.2 Section II.3.D indicates that for external environmental loads, a concrete shield building typically protects steel containments from the environment. If environmental loads external to the steel containment (e.g., wind, tornado, external flooding) either directly or indirectly impose loads on the steel containment, the design of the steel containment needs to consider these loads, such as W plus Pd. WEC should use load combinations and acceptance criteria that are consistent with those specified in SRP Section 3.8.1, or provide sufficient justification for an equivalent alternative.

   b. A load combination that combines Wt (tornado) plus Po (operating pressure) is included in the proposed revision to Table 3.8.2-1. Provide justification why a load combination that combines Wt plus Pe (external pressure) is not also included. Such a load combination is typically required for concrete containments.

   c. The proposed markup to the load combinations and the footnotes in DCD Table 3.8.2-1 identify four different pressures and associated temperatures of the containment. Provide, in the footnotes to the table, the values for the different pressures and the corresponding temperatures inside and outside containment that are used in each of these load combinations.

2. Clarify the following items related to the hydrogen generation load combinations described in the Revision 1 response to RAI SRP-3.8.2-SEB1-03:

   a. The RAI response states, "According to 10 CFR 50.34(f), the peak LOCA pressure plus the peak pressure from a hydrogen burn must be less than ASME Service Level C (not including buckling)." Provide a justification for the following: (1) 10 CFR 50.34(f) is identified rather than 10 CFR 50.44, since 10 CFR 50.34(f) is only applicable to a limited set of older plants, and (2) the term "peak LOCA pressure" is used rather than the term "hydrogen generated pressure loads from the 100% fuel clad metal-water reaction."

b. The RAI response states, "The peak pressure from the hydrogen burn (Pg1 +Pg2) is 90.3 psig as reported in Section 41.11 and Table 41-4 of the PRA report." Explain why the phrase "hydrogen burn" in this statement applies to both Pg1 and Pg2. The staff notes that SRP 3.8.2 defines Pg1 as the pressure load generated from the 100% fuel clad metal-water reaction and not the pressure due to hydrogen burn. Also, explain whether the 90.3 psig represents the maximum hydrogen generated pressure load from the fuel clad metal-water reaction plus the hydrogen burn load for the AP1000 plant.

Additional Question (Revision 3)

The staff reviewed the Westinghouse response to RAI-SRP3.8.2-SEB1-03 Rev. 2, transmitted in their letter dated July 2, 2010. The response addressed the concerns, related to load combinations considered in the design of the steel containment, except for one item. Since the proposed changes to the DCD were not presented, include in the RAI response the proposed DCD mark-ups showing revisions in the DCD associated with the change from 10 CFR 50.34(f) to 10 CFR 50.44.

Westinghouse Response: (Revision 0)

1. The containment vessel (CV) design has gone through a detailed review by the NRC staff and consultants. This review included the load combinations and service limits for the containment vessel.

   In the most recent Technical Audit meeting in Pittsburgh (held the week ending 5/23/2008), the CV design as described in the DCD Revision 16, and the CV design report and calculations related to Technical Report 9 (TR-09) (Reference 1) were reviewed in great detail. The CV design calculations include the various design load combinations. The governing combinations are present in DCD Rev. 16 Table 3.8.2-1, "Load Combinations and Service Limits for Containment Vessel." This table was revised in TR-09 and included in this review.

   The post flooding condition load combination was also discussed in the May 2008 NRC audit. In response to the revised request, RAI-TR09-005 Rev 2 was sent to the NRC in September 2008 (Reference 2). The following response is provided again:

   > *The post accident flooding load combination is not applicable in the design of the containment vessel. Containment flooding events are described in DCD subsection 3.4.1.2.2.1. Curbs are provided around openings through the maintenance floor at elevation 107'-2" to control flooding into the lower compartments. The maximum curb elevation of 110'-2" establishes the maximum flooding on the containment vessel boundary. There are seals at elevation 107'-2" between the containment vessel and maintenance floor as shown in sheet 2 of DCD Figure 3.8.2-8. In the event of seal leakage hydrostatic pressure could be imposed on the vessel behind the concrete.*

Pressure loads below elevation 100' are resisted by the mass concrete of the nuclear island basemat. Pressure loads above elevation 100' would be carried by the steel vessel. Hence, there could be a maximum hydrostatic head of 10' corresponding to a hydrostatic pressure of about 5 psi.

The containment vessel is designed for a design pressure of 59 psi. This pressure exceeds the maximum calculated pressure in design basis accidents.

Maximum flooding occurs late during the accident transient. The combination of hydrostatic pressure at elevation 100' and containment pressure is less than the design pressure of 59 psi. Hence, the post-LOCA flooding event is enveloped by the other design cases.

2. This load combination corresponds to an external pressure based on an evaluation of a credible initiating event in cold weather

   Several possible credible initiating events were evaluated in order to verify this external pressure. See the response to RAI –TR09-008, Rev. 4 for more information on these scenarios.

3. ASME Section III, Division 1, Subsection NE, Paragraph NE-3221.5 provides the requirements for analysis for cyclic operation. Paragraph NE-3221.5(d) 'Vessels Not Requiring Analysis for Cyclic Service' provides a list of six conditions that if the specified Service Loadings of the vessel or portion thereof meet all six conditions, an analysis for cyclic service is not required.

   Westinghouse has a calculation, available for audit, to show how these six conditions are met.

Westinghouse Additional Response: (Revision 1)

The containment vessel is protected from the direct effects of wind//tornado loads (and associated potential missiles) by virtue of its location inside the shield building. The differential pressure effects of a tornado are also reduced because of the location; and are bounded by other pressure loadings for which the containment vessel is designed.

Westinghouse confirms that, as shown in DCD Table 3.8.2-1, the Containment Vessel shell is designed for the Tornado (W_t) and Wind (W) loads.

In the following specific load combinations for which the NRC reviewer requested information are addressed. The load combinations identified in the Design Control Document (DCD) in support of the Design Certification Amendment are not changed from the Certified Design.

1. SRP 3.8.2 II.3.B.iii.(1)(a)

Normal operating plant condition

$D + L + T_o + R_o + P_o$

Response: This load combination calls for P_o, which is "External pressure loads resulting from pressure variation either inside or outside containment." For the AP1000 CV this results in an external pressure, "based on evaluation of credible initiating event in cold weather." Please note that the terms use in the DCD used for the load combinations are slightly different that the NRC guidance. Westinghouse uses P_e for external pressure.

2. SRP 3.8.2 II.3.B.iii.(3)(b)
   Operating plant condition in combination with SSE

   $D + L + T_o + R_o + P_o + E'$

   Response: This load combination is included in the AP1000 Design Control Document (DCD) and the containment design specification. It is captured as a Service Level D Service Limit load combination in DCD Table 3.8.2-1. The application of Service Level D limits to this load combination was included in the DCD Revision 15 that is referenced by the AP1000 Design Certification. Westinghouse has not changed this load combination or how it is applied to the containment vessel in the DCD that supports the design certification amendment. The NRC approval of the application of Service Level D limits to this load combination it documented in the AP1000 FSER (NUREG-1793) as follows:

   In addition to the four issues discussed above, the staff requested the applicant to provide the technical basis for using Service Level D allowable stress, instead of Service Level C allowable stress, for the load combination of seismic loads plus design external pressure when the evaluation of the containment vessel adequacy was performed. During the audit conducted on October 6–9, 2003, the applicant presented an evaluation based on the load combination, assuming that these two events occur simultaneously. In its submittal dated December 12, 2003 (Revision 3 of the response to Open Item 3.8.2.1-1), the applicant provided a final calculation that justifies the change of design basis from Service Level C to Service Level D. Based on its review of these documents and the discussion with the applicant, the staff found that the change from Service Level C to Service Level D for the load combination of seismic plus design external pressure is technically justified because of the extremely low sequence frequency (less than 1E-10 per year) leading to containment failure.

3. SRP 3.8.2 II.3.B.iii.(3)(d)

Deal Dead load plus pressure resulting from an accident that releases hydrogen generated from 100-percent fuel clad metal-water reaction accompanied by hydrogen burning

$D + P_{g1} + P_{g2}$

Response: The AP1000 addresses the production of large quantities of hydrogen from the oxidation of zirconium and other metals as a result of a postulated severe accident. The AP1000 includes hydrogen igniters inside containment to assure that hydrogen generated in a severe accident is burned prior to reaching an explosive mixture. The discussion of the generation and burning of hydrogen as a result of a severe accident is included in DCD Subsection 19.41.

The containment is also evaluated for the deterministic severe accident pressure capacity. This evaluation is discussed in DCD Subsection 3.8.2.4.2 "Evaluation of Ultimate Capacity". According to 10 CFR 50.34(f), the peak LOCA pressure plus the peak pressure from a hydrogen burn must be less than ASME Service Level C (not including buckling). The Service Level C maximum capacity is 117 psig at 300°F as presented in DCD section 3.8.2.4.2.8. The peak pressure from the hydrogen burn (Pg1 + Pg2) is 90.3 psig) as reported in Section 41.11 and Table 41-4 of the PRA report. The severe accident conditions are beyond design basis accidents and the load combinations for these severe accident evaluations are not included in the load combinations and service limits for the containment vessel provided in the DCD.

The containment ultimate capacity and the treatment of severe accidents that result in the generation of hydrogen is not altered from what was included in the AP1000 certified design. In the Final Safety Evaluation Report for AP1000 (NUREG-1793) the NRC states the following.

> "The staff considers the analysis procedures used in evaluating the ultimate capacity of the AP1000 containment to be consistent with sound engineering practice for such evaluations. On this basis, the staff concludes that the results of the AP1000 ultimate capacity evaluation constitute acceptable input for probabilistic risk assessment analyses and severe accident evaluations."

4. SRP 3.8.2 II.3.B.iii.(3)(e)

   $D + P_{g1} + P_{g3}$

   Response: The AP1000 does not have a post accident inerting system. Therefore, this load combination is not applicable to the AP1000.

5. SRP 3.8.2 II.3.B.iii.(5)

Post Flooding Condition

Response: This condition was previously addressed in the Response to Rev. 1.

Westinghouse Additional Response: (Revision 2)

1.

a. Design Wind load should only be considered in operating conditions. This is a mistake in the load combinations table as it is a construction wind load combined with a Service Level A load combination. During the service of the vessel, it will not experience a construction wind load. A load combination that combines design wind with operating pressure will be added to the Service Level A load combinations. The worst case of a tornado wind load, which results in the largest reduction in pressure, is included in load combination C2. The proposed load combination will be included in the markup of DCD Table 3.8.2-1 in RAI-TR09-008, Rev. 6. Justification for not combining W plus Pd is added to DCD Section 3.8.2.4.1.1 and is provided in the response to RAI-TR09-008, Rev. 6.

b. The AP1000 is designed to meet the ASME Boiler & Pressure Vessel Code, Section III, Subsection NE 2001 edition with the 2002 Addenda. The containment vessel is not designed to meet the requirements of a concrete containment and therefore does not need to be analyzed for tornado wind loads combined with Pe (external pressure). Environment loads such as tornados cannot impose a direct load on the containment vessel. The tornado wind load has been defined as a reduction in external pressure. If this is then combined with the external pressure, the tornado wind load would simply reduce the effect of the external pressure on the containment vessel. Therefore, this load combination was not considered to be evaluated in the design of the containment vessel.

c. Pressures and temperature values have been added to DCD Section 3.8.2.4.1.1. Please see the DCD Revision Section of the response to RAI-TR09-008, Rev. 6.

2.a. (1) 10 CFR 50.44 will be referenced in the DCD instead of 10 CFR 50.34(f).

(2) The term "peak LOCA pressure" should be "hydrogen generated pressure loads from 100% fuel clad metal-water reaction" in the RAI Response.

CHANGES TO RESPONSE TO REV. 1 (Revision 2)

Response: The AP1000 addresses the production of large quantities of hydrogen from the oxidation of zirconium and other metals as a result of a postulated severe accident. The AP1000 includes hydrogen igniters inside containment to assure that hydrogen generated in a severe accident is burned prior to reaching an explosive mixture. The discussion of the generation and burning of hydrogen as a result of a severe accident is included in DCD Subsection 19.41.

The containment is also evaluated for the deterministic severe accident pressure capacity. This evaluation is discussed in DCD Subsection 3.8.2.4.2 "Evaluation of Ultimate Capacity". According to 10 CFR 50.34(f), the hydrogen generated pressure loads from 100% fuel clad metal-water reaction ~~peak LOCA pressure~~ plus the peak pressure from a hydrogen burn must be less than ASME Service Level C (not including buckling). The Service Level C maximum capacity is 117 psig at 300°F as presented in DCD section 3.8.2.4.2.8. The peak pressure from the hydrogen generated pressure loads from 100% fuel clad metal-water reaction plus the hydrogen burn (Pg1 + Pg2) is 90.3 psig) as reported in Section 41.11 and Table 41-4 of the PRA report. The severe accident conditions are beyond design basis accidents and the load combinations for these severe accident evaluations are not included in the load combinations and service limits for the containment vessel provided in the DCD.

The containment ultimate capacity and the treatment of severe accidents that result in the generation of hydrogen is not altered from what was included in the AP1000 certified design. In the Final Safety Evaluation Report for AP1000 (NUREG-1793) the NRC states the following.

> "The staff considers the analysis procedures used in evaluating the ultimate capacity of the AP1000 containment to be consistent with sound engineering practice for such evaluations. On this basis, the staff concludes that the results of the AP1000 ultimate capacity evaluation constitute acceptable input for probabilistic risk assessment analyses and severe accident evaluations."

2.b. This was a mistake in the RAI response. Pg1 refers to the "hydrogen generated pressure loads from the 100% fuel clad metal-water reaction," and Pg2 refers to the pressure resulting from the hydrogen burn. Pg1 + Pg2 is 90.3 psig as reported in Section 41.11 and Table 41-4 of the PRA Report. See above changes to Response to Rev. 1 for corrections of this mistake.

References:

1. APP-GW-GLR-005, Revision 1, "Containment Vessel Design Adjacent to Large Penetrations," Technical Report Number 9, submitted with DCP/NRC1988, September 5, 2007.

2. Letter, Sisk (Westinghouse) to NRC, "AP1000 Response to Request for Additional Information (TR09)", DCP/NRC2261, September 15, 2008.

Westinghouse Additional Response: (Revision 3)

Revision 3 of this response is provided to deliver the DCD mark-ups inadvertently omitted from the Revision 2 response. These markups replace references to 10 CFR 50.34(f) -with 10 CFR 50.44 in the DCD for containment hydrogen issues.

Design Control Document (DCD) Revision: (Revision 3)
~~None~~

Revise the third paragraph of Subsection .3.8.2.4.2.8 as follows:

The maximum pressure that can be accommodated according to the ASME Service Level C stress intensity limits, excluding evaluation of instability, is determined by yield of the cylinder and is 135 psig at an ambient temperature of 100°F and 117 psig at 300°F. This limit is used in the evaluations required by 10 CFR ~~50.34(f)~~50.44.

Revise Item B. of Subsection 6.2.4.1 as follows:

B. The hydrogen control system is designed in accordance with the requirements of 10 CFR 50.44 ~~and 10 CFR 50.34(f)~~ and meets the NRC staff's position related to hydrogen control of SECY-93-087.

Revise the seventh bullet of Subsection 19.1.2 as follows:

- Demonstrate compliance with the hydrogen control criteria set forth in 10 CFR ~~50.34(f)(2)(ix)~~50.44.

Revise the seventh paragraph of Subsection 19.1.5 as follows:

The AP1000 containment is capable of providing an effective barrier to the release of fission-products to the environment and includes effective hydrogen control measures. The AP1000 design meets the criteria in 10 CFR ~~50.34(f)(2)(ix)~~50.44.

Revise the first paragraph of 19.34.2.7 as follows:

Reference 19.34-7 states that equipment identified as being useful to mitigate the consequences of severe accidents must be designed to provide reasonable assurance that it will continue to operate in a severe accident environment for the length of time needed to accomplish its function. Also, 10 CFR ~~50.34(f)~~50.44 requires safety equipment to continue performing its function after being exposed to a containment environment created as a consequence of generating a quantity of hydrogen equivalent to that from 100-percent cladding oxidation. As the AP1000 design uses thermal igniters to burn hydrogen in a controlled manner, it is necessary to demonstrate that the safety equipment can continue to perform its function in the high-temperature environment created by the hydrogen burning.

Revise the second paragraph of Subsection 19.41.3.4 as follows

Since the lowest hydrogen concentration for which deflagration-to-detonation transition has been observed in the intermediate-scale FLAME facility at Sandia is 15 percent (Reference 19.41-7), and 10 CFR ~~50.34(f)~~50.44 limits hydrogen concentration to less than 10 percent, the likelihood of deflagration-to-detonation transition is assumed to be zero if the hydrogen concentration is less than 10 percent.

Revise the second paragraph of Subsection 19.41.12 as follows:

Analyses are performed to meet the requirements of 10 CFR ~~50.34(f)~~50.44. Igniter burning analyses with rapid hydrogen generation and 100-percent cladding reaction conclude that the igniter system maintains the global uniform hydrogen concentration in the containment at or below lower flammability limits. If the stage 4 automatic depressurization system is available, the hydrogen is well mixed in the containment and no excessive concentrations are predicted in the in-containment refueling water storage tank or PXS valve/accumulator rooms. If the stage 4 automatic depressurization system is failed, hydrogen in the in-containment refueling water storage tank and PXS valve/accumulator rooms can reach high concentrations. However, the mixtures are oxygen starved and are not flammable or detonable. The safety margin basis containment performance requirement is met as the loss-of-coolant accident plus 100-percent active cladding reaction hydrogen burn peak pressure provides margin to the ASME Service Level C stress limits.

Revise item 32 in Table 19.59-18 as follows:

Table 19.59-18 (Sheet 16 of 25)

AP1000 PRA-BASED INSIGHTS

Insight	Disposition

Table 19.59-18 (Sheet 16 of 25) **AP1000 PRA-BASED INSIGHTS**	
Insight	**Disposition**
32. The containment structure can withstand the pressurization from a LOCA and the global combustion of hydrogen released in-vessel (10 CFR ~~50.34(f)~~50.44).	19.41

PRA Revision:
None

Technical Report (TR) Revision:
None

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-SRP3.8.3-SEB1-04
Revision: 3

Question:

Due to design changes, extension of the AP1000 design to soil sites, reanalysis for updated seismic spectra, and updates made to some critical sections, Westinghouse is requested to address a concern with the design details of the structural module connections to the reinforced concrete basemat. Section 3.8.3.5.3 of the DCD indicates that the steel plate modules are anchored to the reinforced concrete basemat by mechanical connections welded to the steel plate or by lap splices where the reinforcement overlays shear studs on the steel plate. Typical details of these two options are shown on DCD Figure 3.8.3-8, sheets 1 and 2. Westinghouse is requested to address the following two items:

1. The left side of Figure 3.8.3-8, sheet 2, shows that the mechanical connectors that are welded to a ¾ inch plate at the base of the module is identified as "CONT" (presumably meaning continuous) on one side of the module and on the other side the term "CONT" is struck out. Explain which detail is correct and revise the figure accordingly. Were the design detail calculations completed for this connection? Explain how the large loads coming from the CIS wall modules can be properly transferred from the module wall plate at a localized point to the embedded connectors.

2. The right side of Figure 3.8.3-8, sheet 2 shows #11 at 10 inch spacing span from the embedded basemat region into the wall module with about 3 inches of concrete cover. Since this type of connection is not addressed in ACI 349, describe how the loads from the module can be properly transferred from the module to the embedded bars in the basemat and how the design will be performed. When this detail was discussed with Westinghouse at an earlier audit this year, Westinghouse indicated that they would consider removing this second option.

If your response to this request for additional information will reference Revision 17 to the AP1000 DCD, please provide an exact reference.

Revision 1

Provide information on the connection of structural modules to base concrete in the RAI response.

Revision 2

The staff reviewed the response provided in Westinghouse letter dated March 12, 2010. The response provided information to address Item 1 of the original RAI question; however, the response contained in Item 2 did not provide all the needed information. Since the type of connection shown in the right side of DCD Figure 3.8.3-8, sheet 2, is not addressed in ACI 349, describe how the loads from the module can be properly transferred from the module to the embedded bars in the base concrete and how the design is performed. As background information, this detail was discussed with the applicant at an earlier audit, the applicant indicated that they would consider removing this connection as an option, and instead rely on the other existing option of transferring loads directly from the faceplates to the base concrete using vertical bars and mechanical connectors. In addition, address the items listed below related to the revised typical detail shown in Figure RAI-SRP-3.8.3-SEB1-04-01A (Figure 01A).

1. Explain why the horizontal #11 @ 18" rebars are identified with mechanical connectors, while no mechanical connectors are shown in the sketch.

2. Explain why the enlarged detail in the upper right corner does not show the continuity of the vertical L4 x 3 angle, which is identified in the overall connection of Figure RAI-SRP-3.8.3-SEB1-04-01A.

3. Welds for the connection on the left side of the module should be shown in the figure to demonstrate the load path from the module to the base concrete.

4. Explain why the title of the figure is labeled "...single layer of dowel bars" because the left side of the figure shows two layers of dowel bars.

For the updated DCD Figure 3.8.3-8, Sheet 2, provided in the RAI response, explain why the left hand side connection detail was replaced with the new connection detail shown in Figure RAI-SRP-3.8.3-SEB1-04-01A, rather than replacing the right hand side connection detail. This is not consistent with the text information provided in the RAI response.

Additional Question (Revision 3):

The staff reviewed the response provided in Westinghouse letter dated July 30, 2010 and concluded that the response addressed most of the concerns identified in this RAI; however, the following additional information is needed to resolve this RAI:

(1) Because both the text of the response and Figure RAI-SRP-3.8.3-SEB1-04-01 indicate that the mechanical connection detail illustrated in the figure is a representative connection detail that will be used, a note should be added to the figure that any deviation to the detail shown shall maintain a direct load path to transfer loads from both sides of the module surface plates to the vertical dowel bars in the base concrete through the use of intervening plates, mechanical connectors, and welds.

(2) Correct the typo on Page 4 of 7 of the response and under "Design Control Document (DCD) Revision." In the phrase, "The updated DCD Figure 3.8.3-8, Sheet 2 is provided on Page 6 of this response," "Page 6" should be "Page 5."

Westinghouse Response:

1. The plate at the base of the module does not need to be continuous. The revised typical detail is shown in Figure RAI-SRP-3.8.3-SEB1-04-01 ~~and will be included in the DCD~~.

~~An alternate version of this detail is used in cases where the trusses are extended into the basemat as shown in RAI-SRP-3.8.3-SEB1-04-02.~~ The vertical dowel bars are placed in two layers. The base plate is stiffened to transfer the loads from the module wall plate to the embedded connectors. The design of the surface plate, base plate, and vertical stiffeners is checked by finite element analysis using the model shown in Figure RAI-SRP-3.8.3-SEB1-04-03. Tension corresponding to yield is applied to each dowel bar. These design calculations have been completed.

2. This connection has been removed as an option, and loads are transferred directly from the faceplates to the base concrete using vertical bars and mechanical connectors. ~~The right side of Figure 3.8.3-8, sheet 2 shows a dowel bar adjacent to the surface plate. The design of this type of connection is based on recommendations and test data given in Reference 1. The reference provides a design equation to calculate the strength of the connection based on key parameters such as concrete strength, dowel bar length and spacing, and concrete cover. This detail may be used when loading on the surface plates is within the range of the test data. It is used at the base of the CA05 module inside containment and the CA20 module outside containment where design loads are smaller.~~

Westinghouse Response to Revision 1:

The revised typical detail is shown in Figure RAI-SRP-3.8.3-SEB1-04-01 and provides information on the connection of structural modules to base concrete. The revised DCD figure identifying the key features of this connection design is provided.

Westinghouse Response to Revision 2:

Westinghouse will remove the "alternate" detail with the lapped splice, and provide mechanical connection of the structural modules to the base concrete using reinforcing bars and mechanical connectors. The mechanical connection detail illustrated in Figure RAI-SRP-3.8.3-SEB1-04-01 is representative of the connection that will be used. This mechanical connection consists of a base plate welded to the CA module liner plate, with gusset plates and vertical reinforcing bars attached to the underside of the base plate. DCD figure 3.8.3-8 is modified to reflect changes to the connection, including deletion of lap splice details. The details presented as typical are intended to provide clarity of the design intent, but not to supersede any changes necessary to accommodate features needed to ensure functionality of the design.

- Based on NRC response to RAI-SRP3.8.3-SEB1-04, Westinghouse has deleted the connection detail that does not have a direct load transfer path from the modules to the base concrete.
- Westinghouse will provide a connection similar to the mechanical connection illustrated in Figure RAI-SRP-3.8.3-SEB1-04-01 (module wall plate welded to base plate with mechanical connectors/reinforcing bar)

- The changes to the design calculations and drawings include consideration of the details required to make the mechanical connection work in all areas and to address fabrication and construction issues

Item 1: Base connections with mechanical connectors

Figure RAI-SRP-3.8.3-SEB1-04-01 in the previous response is replaced by a revised Figure RAI-SRP-3.8.3-SEB1-04-01 which includes revisions to respond to the previous comments. The items listed in the question related to the revised typical detail shown in Figure RAI-SRP-3.8.3-SEB1-04-01 (Figure 01A in Rev 1 response) are addressed below.

1. The horizontal #11 @ 18" rebars are incorrectly identified with mechanical connectors. These bars terminate with hooks as shown in the sketch. This figure has been revised.

2. The enlarged detail in the upper right corner shows a section midway between the vertical L4 x 3 angles. As shown in Figure 3.8.3-SEB1-04-02, the base plate is not continuous across the vertical angles. This design will be clarified in implementation of the changes described in this response.

3. Welds for the mechanical connection have been added to the figure to demonstrate the load path from the module to the base concrete.

4. The title of the figure has been revised to show "...double layer".

Reference:

Westinghouse Response to Revision 3:

(1) A statement is added to the DCD text to state that loads in module surface plates are transferred directly from the faceplate to the base concrete through mechanical connectors and reinforcement bars.

(2) With the addition of the Revision 3 response and additional DCD text revision the revised DCD figure is now on Page 6 of the response.

Design Control Document (DCD) Revision:

Revise the third paragraph of 3.8.3.5.3 as shown below

Figure 3.8.3-8 shows the typical design details of the structural modules, typical configuration of the wall modules, typical anchorages of the wall modules to the reinforced base concrete, and connections between adjacent modules. Concrete-filled structural wall modules are designed as reinforced concrete structures in accordance with the requirements of ACI-349, as supplemented in the following paragraphs. The faceplates are considered as the reinforcing steel, bonded to the concrete by headed studs. The application of ACI-349 and the supplemental requirements are supported by the behavior studies described in subsection 3.8.3.4.1. The steel plate modules are anchored to the reinforced concrete basemat by mechanical connections welded to the steel plate ~~or by lap splices where the reinforcement overlaps shear studs on the steel plate~~. Loads are transferred directly from the faceplates to the base concrete using reinforcing bars, mechanical connectors, and welds. The design of the surface plate, base plate, and vertical stiffeners is checked by finite element analysis. The design of critical sections is described in subsection 3.8.3.5.8.

Revise DCD Rev 17 Figure 3.8.3-8, sheet 2 of 3. The updated DCD Figure 3.8.3-8, Sheet 2 is provided on Page 6 of this response.

The following note will be added to Figure 3.8.3-8,

> Changes to the mechanical connector detail shown will maintain a direct load path to transfer loads from the module face plates to the reinforcement bars in the base concrete through the use of intervening plates, mechanical connectors, and welds.

PRA Revision:

None

Technical Report (TR) Revision:

None

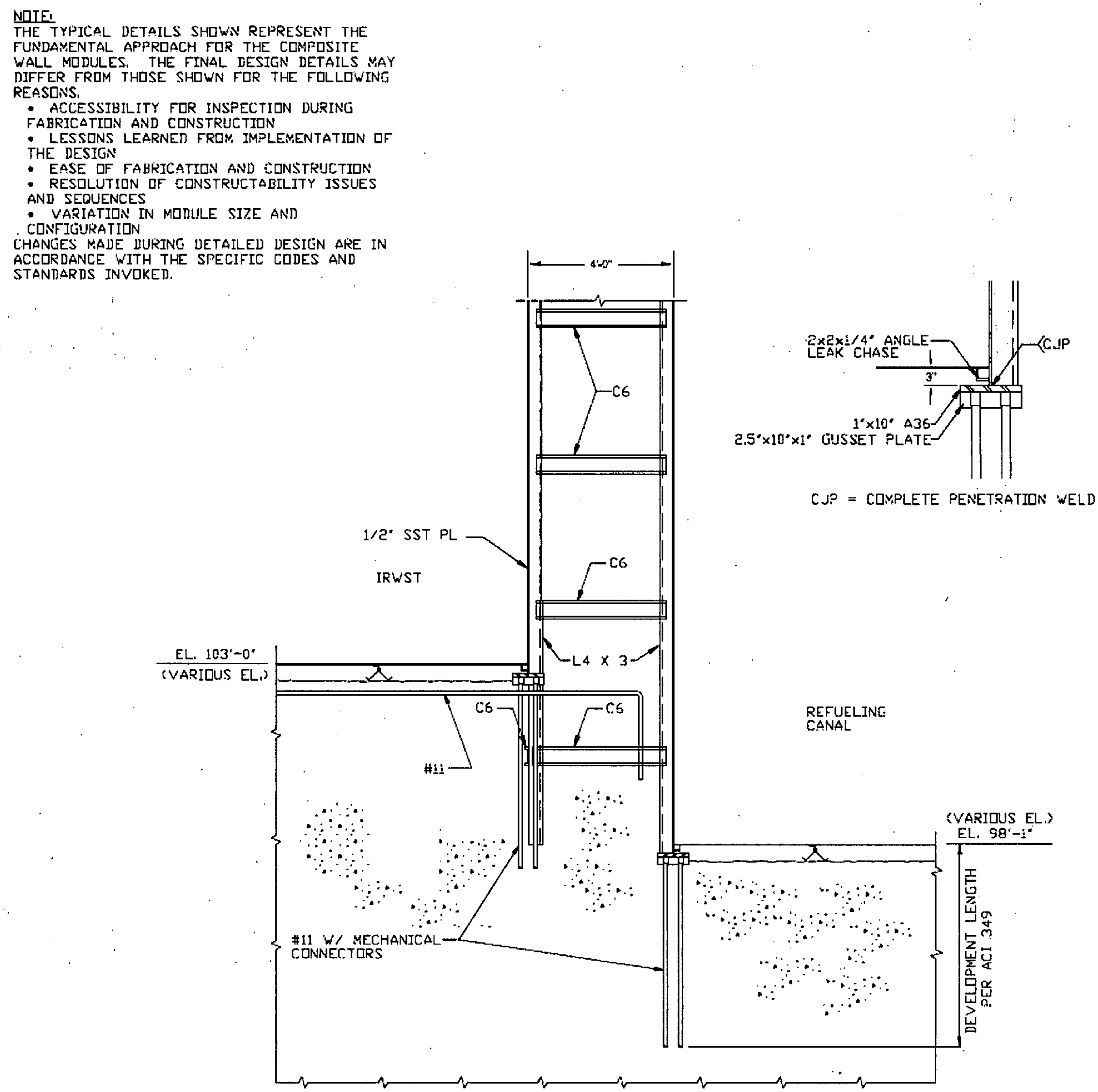


Figure RAI-SRP-3.8.3-SEB1-04-01
Representative details at base of CA01 Module Wall with a Double Layer of Dowel Bars

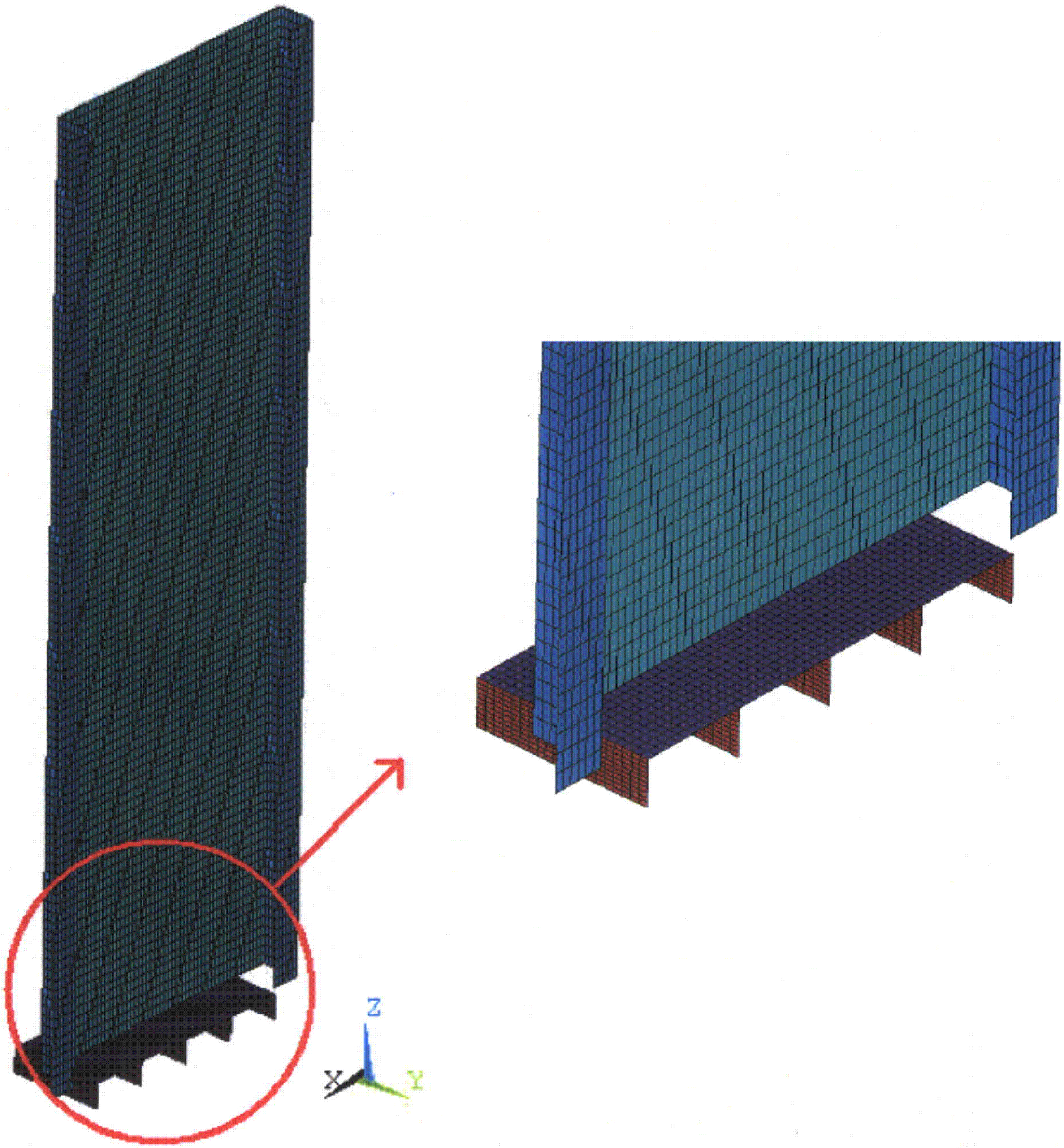
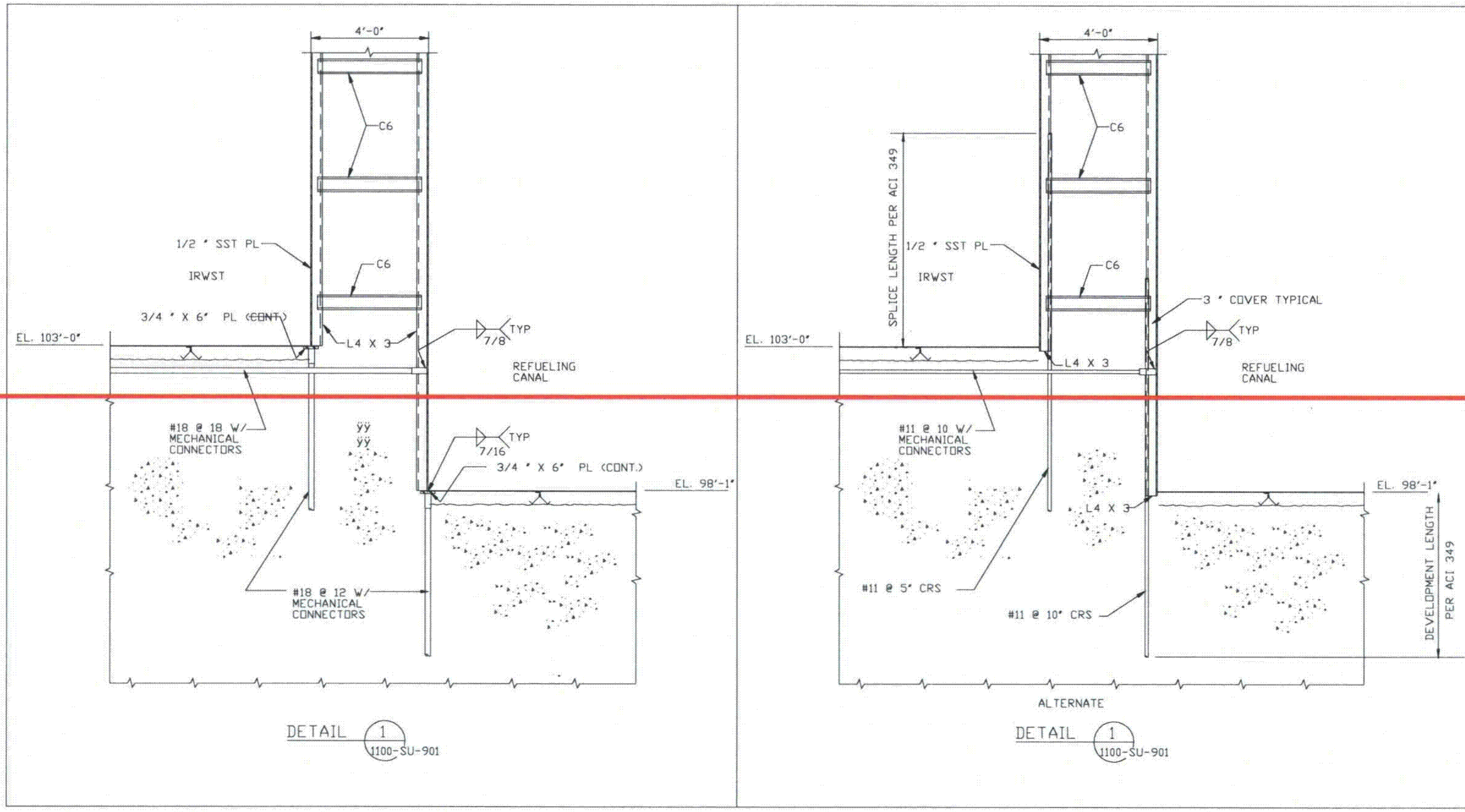


Figure RAI-SRP-3.8.3-SEB1-04-02



Replace Figure 3.8.3-8 (Sh2 of 3) with New Figure RAI-SRP-3.8.3-SEB1-04-01

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-SRP3.8.3-SEB1-05
Revision: 3

Question: (Revision 0)

DCD Section 3.8.3.5.8 describes the design summary of critical sections for the CIS. Westinghouse is requested to address the following items related to this revised section:

For DCD Section 3.8.3.5.8.1 – Structural Wall Modules

1. The last paragraph, was revised to eliminate some Tier 2* information and criteria (denoted by italicized text, square bracket, and a superscript *). Westinghouse is requested to provide the basis for removing this information. The information removed relates to DCD Rev. 16 Tables 3.8.3-3 through 3.8.3-6. These tables have been substantially revised from the prior DCD tables to remove significant design information. Westinghouse is requested to provide the same or comparable information that was provided in prior revisions of the DCD.

2. The last two sentences in the referenced paragraph are italicized but are outside the square bracket with a star. These sentences should be placed inside the square brackets.

3. The last sentence states "See Appendix 3H for more detailed discussion." Westinghouse should explain why a reference for more detailed information of structural wall modules inside containment is made to Appendix 3H which addresses auxiliary and shield building critical sections.

For DCD Section 3.8.3.5.8.2 – IRWST Steel Wall

4. Same issue discussed in item 3 above is also applicable to DCD Section 3.8.3.5.8.2.

For DCD Section 3.8.3.5.8.3 – Column Supporting Operating Floor

5. Same issues as items 1 and 3 above are also applicable to DCD 3.8.3.5.8.3

Updating of all analyses due to changes in seismic and other loads

6. Westinghouse is requested to explain whether the information presented for all structures in DCD Rev. 16, Sections 3.8.1 through 3.8.5, and associated appendices reflect the latest set of updated analyses for the revised seismic loads (e.g., extension of design to soil sites and resolution of RAIs related to seismic) and revision of other loads which might have been updated from the prior version of the DCD.

If your response to this request for additional information will reference Revision 17 to the AP1000 DCD, please provide an exact reference.

Additional Question: (Revision 2)

The staff reviewed the response provided in Westinghouse letter dated March 15, 2010 and concluded that the response addressed most of the concerns identified in this RAI; however, more information is needed to resolve the remaining items. In the response, most of the Tier 2* information, including descriptions, criteria, member forces, required plate thicknesses, and stress results, that were removed from the Section 3.8.3.5.8 of DCDs Rev. 16 and Rev. 17, will be placed back in DCD Sections 3.8.3.5.8.1 to 3.8.3.5.8.3 and Tables 3.8.3-4 through 3.8.3-6. However, in DCD Table 3.8.3-3, the applicant did not provide the required plate thicknesses which were provided in the same table in DCD Rev.15. In addition, there appears to be a Tier 2* "square bracket" missing in the last paragraph of the proposed mark-up to DCD Section 3.8.3.5.8.1, which, if in error, should be corrected. Therefore, provide the required plate thicknesses and correct DCD Section 3.8.3.5.8.1 for the missing square bracket.

Additional Question: (Revision 3)

The staff reviewed the Westinghouse response to RAI-SRP3.8.3-SEB1-05 Rev. 2, transmitted in their letter dated July 2, 2010. The response addressed the concerns, related to the removal of Tier 2* information for the DCD and definition of new Tier 2* criteria for critical sections, except for one item. On Page 5 of 37 of the response, in the last sentence of the second bullet of the proposed new criteria in Subsection 3.8.3.5.8, the reference to "Table 3H-3" should be revised to "Table 3H.5-3." Since this new Tier 2* criteria are utilized in several locations in the DCD, this correction should be made at all of the other locations as well.

Westinghouse Response: (Revision 0)

1. The removal of the subject information was identified and explained in APP-GW-GLR-045 (Reference 1). This report supports the removal of the design load summary tables in Design Control Document (DCD) Subsection 3.8.3 and 3.8.4 and the tables of member forces and moments in Appendix 3H. The last paragraph of DCD Section 3.8.3.5.8.1 in DCD Revision 15 referenced member forces tables in DCD Revision 15. The information removed from tables in the DCD represents the results of detailed calculations and analyses. These results change slightly during the design finalization due to changes related to constructability and construction sequence. Finalization of the design spectra can also result in minor changes in the as-designed results. The DCD changes between Revision 15 and Revision 16 also supported the change of the design spectra from a hard rock only case to design spectra acceptable for multiple rock and soil cases. Small changes in modeling and updates to software may also have a minor effect on the results. For these reasons, it is not practical to lock in these design and analysis results in the DCD.

Subsection 3.8.3, 3.8.4, and Appendix 3H as shown in Revision 17 provide information on the requirements and criteria for design configuration, and concrete reinforcement. These requirements and criteria lock-in the design for NRC review and demonstrate that the requirements and criteria for the design conforms with review guidance or otherwise uses appropriate design and analysis methods. The level of detail represented by the design summary tables of forces and moments does not appear to be consistent with the guidance of Regulatory Guide 1.70 and Standard Review Plan Section 3.8.4. SRP Section 3.8.3 and 3.8.4 do not suggest that this detailed information should be included in the DCD. Attempting to lock in the design loads results over specifies the design. The design loads and related information removed in DCD Revision 16 included the amount of reinforcement provided and identified the fraction of the limit calculated. This overly restricted the changes to the design during design finalization.

Based on the above information, Westinghouse does not believe it is necessary to return the information on member forces and moments and the specific amount of reinforcement provided removed in DCD Revision 16 to the DCD. Detailed results of the analyses of the critical structures and other structures are available for NRC audit and have been reviewed by NRC review staff. These detailed design calculations include the design summary Tables of Forces and Moments. One of the reasons that the specific results for the critical structures were included in the DCD through Revision 15 was because of the relatively limited amount of design information available for the NRC review staff to look at to make a judgment about the implementation of the design methods, requirements, and criteria in the structural design. The information now available for NRC review is much more complete and comprehensive. Finally, the sufficiency of the as-built structural design is subject to verification with reports required by the inspections, tests, analyses, and acceptance criteria (ITAAC) in Tier 1 of the DCD. Tier 1 of the DCD includes dimensional requirements for structures in the AP1000 design including critical structures.

Based on the above information Westinghouse does not believe it is appropriate to return the information on member forces and moments, and the specific amount of reinforcement provided, to the DCD.

2. In DCD Revision 17, the last two sentences of the last paragraph of DCD Section 3.8.3.5.8.1 were corrected to be standard, non-italic text because the text only provides cross-references, not design information critical to the NRC approval.

3. The last sentence of the last paragraph of DCD Section 3.8.3.5.8.1 should have been "See Technical Report APP-GW-GLR-045 for more details."

   This correction will be incorporated in next DCD revision as shown below.

4. The last sentence of the last paragraph of DCD Section 3.8.3.5.8.2 should have been "See Technical Report APP GW GLR 045 for more details."

The latest set of updated analyses for the revised seismic loads including the extension of design to soil sites (six soils cases) was included in DCD Revision 17. These analyses also reflected changes to methods and criteria that resulted from resolution of RAIs related to seismic design and analysis.

5. For the same reasons outlined in item 1 above Westinghouse does not believe it is necessary to return the information on member forces and moments and the specific amount of reinforcement provided removed in DCD Revision 16 to the DCD Section 3.8.3.5.8.3

   The last sentence of the last paragraph of DCD Section 3.8.3.5.8.2 should have been "See Technical Report APP-GW-GLR-045 for more details."

   This correction will be incorporated in next DCD revision as shown below.

6. The last sentence of the last paragraph of DCD Section 3.8.3.5.8.3 should have been "See Technical Report APP-GW-GLR-045 for more details."

   This correction will be incorporated in next DCD revision as shown below.

References:

1. APP-GW-GLR-045, "AP1000 Standard Combined License Technical Report, Nuclear Island, Evaluation of Critical Sections" Westinghouse Electric Company LLC.

Additional Westinghouse Response: (Revision 1)

This response addresses the tables that are contained in DCD Tier 2 Section 3.8. Comparable information removed from DCD Revision 16 is replaced in the DCD.

Also provided in this response are changes to Tier 2, Table 3.8.4-6, "Materials Used in Structural and Miscellaneous Steel." These changes resolve a Westinghouse corrective action issue report and an extent of condition review. It provides for new steel structural materials needed to support design changes in the AP1000 mechanical/structural modules and the enhanced shield building.

The revised Table 3.8.4-6 includes the major structural and miscellaneous steel shapes needed. The materials included in the table are consistent with the SRP guidance to include structural shapes and reinforcement. The changes are based on review of steel materials from the following sources: structural design changes (i.e. modules, enhanced shield building), materials listed in previous RAIs (RAI-SRP-3.8.3-SEB1-06; RAI-SRP-3.8.4-SEB1-02), design finalization, and conforming ASTM standards already listed in the DCD text or references. This is not an all-inclusive list and specifically excludes, for example, most pressure-retaining materials and fasteners (i.e. bolts, nuts, studs, and bolting materials).

Additional Westinghouse Response: (Revision 2)

Westinghouse has further updated DCD Tier 2 Table 3.8.3-3, "Definition Of Critical Locations And Thicknesses For Containment Internal Structures" below to include a column of the required plate thicknesses, as were provided in the same table in DCD Rev.15

Also, the unintentionally omitted square bracket has been restored in Section 3.8.3.5.8.1 at the end of the third sentence in the modified paragraph: *The other walls have stainless steel on one face and carbon steel on the other.*]*

The criterion in Subsection 3.8.3.5.8 on reporting requirements for Tier 2* information in Critical section tables is revised.

Additional Westinghouse Response: (Revision 3)

The reference to Table 3H.5-3 in 3.8.3.5.8 is corrected. In the previous revision the reference was to Table 3H-3.

A similar correction to Subsection 3H.1 is included in the response to RAI-SRP3.8.4-SEB1-03

Design Control Document (DCD) Revision: (Revision 0, 1, 2)

Revise DCD Tier 2, Section 3.8.3.5.8, "Design Summary of Critical Sections," as follows: (Revision 2)

3.8.3.5.8 Design Summary of Critical Sections

[Changes in the values in the critical section tables that are designated as Tier 2 must be reported to the NRC if*

- *A change to design parameters is required. These design parameters include reinforcement provided, concrete strength, and steel section size. Both design parameter increases and decreases must be reported.*
- *Changes in the values of loads, moments, and forces in the critical section tables that are designated as Tier 2* must be reported to the NRC if the change results in a required reinforcement (or plate thickness for CA modules) increase greater than 10% of the*

*provided reinforcement (or plate thickness for CA modules).]** For example the change must be reported if a change in moments or forces in Table 3H.5-2 results in a calculated required reinforcements value in Table 3H.5-3 more than 10% of the corresponding provided reinforcement value.

Revise DCD Tier 2, Section 3.8.3.5.8.1, "Structural Wall Modules," as follows:

3.8.3.5.8.1 Structural Wall Modules

(Previous paragraphs unchanged)

[*The three walls extend from the floor of the in-containment refueling water storage tank at elevation 103′ 0″ to the operating floor at elevation 135′ 3″. The south west wall is also a boundary of the refueling cavity and has stainless steel plate on both faces. The other walls have stainless steel on one face and carbon steel on the other.]** ~~Design summaries are given in Table 3.8.3-4, 3.8.3-5, and 3.8.3-6. See Appendix 3H for more detailed discussion.~~ For each wall design information is summarized in Tables 3.8.3-4, 3.8.3-5 and 3.8.3-6 at three locations. *[Results are shown at the middle of the wall (mid span at mid height), at the base of the wall at its mid point (mid span at base) and at the base of the wall at the end experiencing greater demand (corner at base). The first part of each table shows the member forces due to individual loading. The lower part of the table shows governing load combinations. The steel plate thickness required to resist mechanical loads is shown at the bottom of the table as well as the thickness provided. The maximum principal stress for the load combination including thermal is also tabulated. If this value exceeds the yield stress at temperature, a supplemental evaluation is performed*]* as described in subsection 3.8.3.5.3.4; [*for these cases the maximum stress intensity range is shown together with the allowable stress intensity range which is twice the yield stress at temperature.*]* See Technical Report APP-GW-GLR-045 (Reference 56) for more details.

Revise DCD Tier 2 Section 3.8.3.5.8.2, "In-Containment Refueling Water Storage Tank Steel Wall," as follows:

3.8.3.5.8.2 In-Containment Refueling Water Storage Tank Steel Wall

(first paragraph unchanged)

The wall is evaluated as vertical and horizontal beams. The vertical beams comprise the T-section columns plus the effective width of the plate. The horizontal beams comprise the L-section angles plus the effective width of the plate. Table 3.8.3-7 shows the ratio of the design stresses to the allowable stresses. When thermal effects result in stresses above yield, the evaluation is in accordance with the supplemental criteria]* as described in subsection 3.8.3.5.3.4. ~~See Appendix 3H for more detailed discussions.~~ See Technical Report APP-GW-GLR-045 (Reference 56) for more details.

Revise DCD Tier 2 Section 3.8.3.5.8.3, "Column Supporting Operating Floor," as follows:

3.8.3.5.8.3 Column Supporting Operating Floor

(first paragraph unchanged)

The load combinations in Table 3.8.4-1 were used to assess the adequacy of the column. ~~*See Appendix 3H for more detailed discussion.*]*~~ *. For mechanical load combinations, the maximum interaction factor due to biaxial bending and axial load is 0.59. For load combinations with thermal loads, the maximum interaction factor is 0.94. Since the interaction factors are less than 1, the column is adequate for all the applied loads.*]* See Technical Report APP-GW-GLR-045 (Reference 56) for more details.

Revise DCD Tier 2 Section 3.8.7, "References," as follows:

56. APP-GW-GLR-045, "AP1000 Standard Combined License Technical Report, Nuclear Island, Evaluation of Critical Sections" Westinghouse Electric Company LLC.

Revise DCD Tier 2 Table 3.8.3-3, "Definition Of Critical Locations And Thicknesses For Containment Internal Structures" as follows: (Revision 2)

Table 3.8.3-3

[*DEFINITION OF CRITICAL LOCATIONS AND THICKNESSES FOR CONTAINMENT INTERNAL STRUCTURES*[(1)]]*(4)

Wall Description	***Applicable Column Lines***	***Applicable Elevation Range***	***Concrete Thickness***[(2)]	***Required Thickness of Surface Plates (inches)***[(3)]	***Thickness of Surface Plates Provided (inches)***
Containment Structures					
Module Wall 1	*West wall of refueling cavity*	*Wall separating IRWST and refueling cavity from elevation 103' to 135'-3"*	*4'-0" concrete-filled structural wall module with 0.5-in.-thick steel plate on inside and outside of wall*	*0.25*	*0.5*
Module Wall 2	*South wall of west steam generator cavity*	*Wall separating IRWST and west steam generator cavity from elevation 103' to 135'-3"*	*2'-6" concrete-filled structural wall module with 0.5-in.-thick steel plate on inside and outside of wall*	*0.44*	*0.5*
CA02 Module Wall	*North east boundary wall of IRWST*	*Wall separating IRWST and maintenance floor from elevation 103' to 135'-3"*	*2'-6" concrete-filled structural wall module with 0.5-in.-thick steel plate on inside and outside of wall*	*0.37*	*0.5*

Notes:

1. *The applicable column lines and elevation levels are identified and included in Figures 1.2-9, 3.7.2-12 (sheets 1 through 12), 3.7.2-19 (sheets 1 through 3) and on Table 1.2-1.*
2. *The concrete thickness includes the steel face plates. Thickness greater than 3'-0" have a construction tolerance of +1", -3/4". Thickness less than or equal to 3'-0" have a construction tolerance of +1/2", -3/8".*
3. *These plate thicknesses represent the minimum thickness required for operating and design basis loads except for designed openings or penetrations. These values apply for each face of the applicable wall unless specifically indicated on the table. For load combinations with thermal loads, the evaluation is performed as described in DCD subsection 3.8.3.5.3.4.*
4. *See Subsection 3.8.3.5.8 for reporting requirements for changes to Tier 2* information in this section.* ~~**NRC Staff approval is required prior to implementing a change in this information; see DCD Introduction Section 3.5.*~~

Revise DCD Tier 2 Table 3.8.3-4, Design Summary of West Wall of Refueling Canal" as follows:

Table 3.8.3-4 (Sheet 1 of 3) [DESIGN SUMMARY OF WEST WALL OF REFUELING CANAL]*	
Element Number 101870	
Plate thickness provided	*= 0.50 inches(1)*
Thermal Load Combinations	
Yield stress at design temperature	*= 55.0 ksi*
Allowable stress intensity range for load combinations (including thermal)	*= 110.0 ksi(2)*

Notes:

1. *This is a lot more than the plate thickness required for load combinations excluding thermal.*
2. *The maximum principal stress and the maximum stress intensity range for these load combinations are much lower than the allowable.*

Table 3.8.3-4 (Sheet 1 of 3)

[DESIGN SUMMARY OF WEST WALL OF REFUELING CANAL
DESIGN LOADS, LOAD COMBINATIONS, AND COMPARISON TO ACCEPTANCE CRITERIA
*MID-SPAN AT MID-HEIGHT]** (3)

Load/Comb.	TX	TY	TXY	MX	MY	MXY	NX	NY	Comments
	k/ft	k/ft	k/ft	kft/ft	kft/ft	kft/ft	k/ft	k/ft	
Dead (D)	0	-18	0	2	1	0	0	1	–
Hydro (F)	3	4	1	22	28	0	0	1	–
Live (L)	1	-9	0	4	2	0	0	1	During refueling
Live (L_o)	0	-2	0	1	0	0	0	0	During operation
ADS	0	6	4	19	21	-3	0	1	–
E_s	14	31	75	29	33	9	2	4	–
Thermal (T_o)	-193	-165	-21	435	404	-15	8	-16	–
LC (1)	4	-13	8	68	76	-5	0	5	$1.4D+1.4F+1.7L_o+1.7ADS$
LC (2)	6	-35	1	40	44	0	0	5	$1.4D+1.4F+1.7L_r$
LC (3)	4	-9	8	66	76	-5	0	5	$1.4D+1.4F+1.7ADS$
LC (4)	17	21	80	73	83	12	2	7	$D+F+L_o+\|ADS\|+E_s$
LC (5)	-11	-53	-78	-23	-25	-12	-2	-3	$D+F+L_o-\|ADS\|-E_s$
LC (6)	-176	-144	59	508	487	-3	10	-9	$D+F+L_o+\|ADS\|+T_0+E_s$
LC (7)	-204	-218	-99	412	379	-27	6	-19	$D+F+L_o-\|ADS\|+T_0-E_s$
LC (8)	17	25	80	72	83	6	2	7	$0.9D+1.0F+1.0\|ADS\|+1.0E_s$

Notes:
x-direction is horizontal, y-direction is vertical.
element number101870

Plate thickness required for load combinations excluding thermal:	*0.042 inches*
Plate thickness provided:	*0.50 inches*
Maximum principal stress for load combinations including thermal:	*23.37 ksi*
Yield stress at temperature:	*55.0 ksi*
Maximum stress intensity range for load combinations including thermal:	*23.37 ksi*
Allowable stress intensity range for load combinations including thermal:	110.0 ksi

(3) See Subsection 3.8.3.5.8 for reporting requirements for changes to Tier 2* information in this section.
~~*NRC Staff approval is required prior to implementing a change in this information; see DCD Introduction Section 3.5.~~

Table 3.8.3-4 (Sheet 2 of 3) [DESIGN SUMMARY OF WEST WALL OF REFUELING CANAL]*	
Element Number 101788	
Plate thickness provided	*= 0.50 inches(1)*
Thermal Load Combinations	
Yield stress at design temperature	*= 55.0 ksi*
Allowable stress intensity range for load combinations (including thermal)	*= 110.0 ksi(2)*

Notes:
1. *This is a lot more than the plate thickness required for load combinations excluding thermal.*
2. *The maximum principal stress and the maximum stress intensity range for these load combinations are much lower than the allowable.*

Table 3.8.3-4 (Sheet 2 of 3)

[*DESIGN SUMMARY OF WEST WALL OF REFUELING CANAL DESIGN LOADS, LOAD COMBINATIONS, AND COMPARISON TO ACCEPTANCE CRITERIA MID-SPAN AT BASE*]* (3)

Load/Comb.	TX k/ft	TY k/ft	TXY k/ft	MX kft/ft	MY kft/ft	MXY kft/ft	NX k/ft	NY k/ft	Comments
Dead (D)	-1	-27	0	-1	-3	0	0	1	–
Hydro (F)	6	7	1	-5	-50	0	0	17	–
Live (L)	0	-8	0	0	-5	0	0	1	During refueling
Live (L_o)	0	-2	0	0	-1	0	0	0	During operation
ADS	6	15	4	-5	-41	-1	-1	10	–
E_s	14	44	85	14	96	3	3	11	–
Thermal (T_0)	-417	-157	-98	522	619	-14	-13	-24	–
LC (1)	17	-6	8	-17	-146	-2	-2	42	$1.4D+1.4F+1.7L_o+1.7ADS$
LC (2)	7	-42	1	-8	-83	0	0	27	$1.4D+1.4F+1.7L_r$
LC (3)	17	-3	8	-17	-144	-2	-2	42	$1.4D+1.4F+1.7ADS$
LC (4)	25	37	90	13	83	4	4	39	$D+F+L_o+\|ADS\|+E_s$
LC (5)	-15	-81	-88	-25	-191	-4	-4	-3	$D+F+L_o-\|ADS\|-E_s$
LC (6)	-392	-120	-8	535	702	-10	-9	15	$D+F+L_o+\|ADS\|+T_0+E_s$
LC (7)	-432	-238	-186	497	428	-18	-17	-27	$D+F+L_o-\|ADS\|+T_0-E_s$
LC (8)	25	42	90	3	2	2	2	39	$0.9D+1.0F+1.0\|ADS\|+1.0E_s$

Notes:	
x-direction is horizontal, y-direction is vertical.	
element number 101788	
Plate thickness required for load combinations excluding thermal:	*0.02inches*
Plate thickness provided:	*0.50 inches*
Maximum principal stress for load combinations including thermal:	*28.0 ksi*
Yield stress at temperature:	*55.0 ksi*
Maximum stress intensity range for load combinations including thermal:	*28.0 ksi*
Allowable stress intensity range for load combinations including thermal:	110.0 ksi

(3) See Subsection 3.8.3.5.8 for reporting requirements for changes to Tier 2* information in this section. ~~*NRC Staff approval is required prior to implementing a change in this information; see DCD Introduction Section 3.5.~~

Table 3.8.3-4 (Sheet 3 of 3) [DESIGN SUMMARY OF WEST WALL OF REFUELING CANAL]*	
Element Number 101794	
Plate thickness provided	*= 0.50 inches*(1)
Thermal Load Combinations	
Yield stress at design temperature	*= 55.0 ksi*
Allowable stress intensity range for load combinations (including thermal)	*= 110.0 ksi*(2)

Notes:
1. *This is a lot more than the plate thickness required for load combinations excluding thermal.*
2. *The maximum principal stress and the maximum stress intensity range for these load combinations are much lower than the allowable.*

Table 3.8.3-4 (Sheet 3 of 3)

*[DESIGN SUMMARY OF WEST WALL OF REFUELING CANAL DESIGN LOADS, LOAD COMBINATIONS, AND COMPARISON TO ACCEPTANCE CRITERIA NORTH END BOTTOM CORNER]** (3)

Load/Comb.	TX k/ft	TY k/ft	TXY k/ft	MX kft/ft	MY kft/ft	MXY kft/ft	NX k/ft	NY k/ft	Comments
Dead (D)	-2	-24	-6	0	-2	0	0	0	–
Hydro (F)	4	0	5	-8	-16	3	2	3	During operation
Live (L)	0	-13	-3	0	-1	0	0	0	During refueling
Live (L_o)	0	-2	0	0	0	0	0	0	During operation
ADS	7	-4	7	-5	-19	1	2	2	–
E_s	24	43	92	13	61	6	5	3	–
Thermal (T_0)	-294	-311	104	423	360	-24	-32	47	–
LC (1)	15	-44	11	-20	-58	6	6	8	$1.4D+1.4F+1.7L_o+1.7ADS$
LC (2)	3	-56	-7	-11	-27	4	3	4	$1.4D+1.4F+1.7L_r$
LC (3)	15	-40	11	-20	-58	6	6	8	$1.4D+1.4F+1.7ADS$
LC (4)	33	21	98	10	62	10	9	8	$D+F+L_o+\|ADS\|+E_s$
LC (5)	-29	-73	-100	-26	-98	-4	-5	-2	$D+F+L_o-\|ADS\|-E_s$
LC (6)	-261	-290	202	433	422	-14	-23	55	$D+F+L_o+\|ADS\|+T_0+E_s$
LC (7)	-323	-384	4	397	262	-28	-37	45	$D+F+L_o-\|ADS\|+T_0-E_s$
LC (8)	33	17	99	0	24	10	9	8	$0.9D+1.0F+1.0\|ADS\|+1.0E_s$

Notes:

x-direction is horizontal, y-direction is vertical.
element number 101794

Plate thickness required for load combinations excluding thermal:	*0.27 inches*
Plate thickness provided:	*0.50 inches*
Maximum principal stress for load combinations including thermal:	*28.1 ksi*
Yield stress at temperature:	*55.0 ksi*
Maximum stress intensity range for load combinations including thermal:	*35.26 ksi*
Allowable stress intensity range for load combinations including thermal:	*110.0 ksi*

(3) See Subsection 3.8.3.5.8 for reporting requirements for changes to Tier 2* information in this section. ~~*NRC Staff approval is required prior to implementing a change in this information; see DCD Introduction Section 3.5.~~

Revise DCD Tier 2 Table 3.8.3-5, "Design Summary of South Wall of Steam Generator Compartment," as follows:

~~Table 3.8.3-5 (Sheet 1 of 3)~~ ~~***[DESIGN SUMMARY OF SOUTH WALL OF STEAM GENERATOR COMPARTMENT]****~~	
~~***Element Number 104228***~~	
~~*Plate thickness provided*~~	~~*= 0.50 inches*[1]~~
~~***Thermal Load Combinations***~~	
~~*Yield stress at design temperature*~~	~~*= 36.0 ksi*~~
~~*Allowable stress intensity range for load combinations (including thermal)*~~	~~*= 72.0 ksi*[2]~~

~~***Notes:***~~
1. ~~*This is a lot more than the plate thickness required for load combinations excluding thermal.*~~
2. ~~*The maximum principal stress and the maximum stress intensity range for these load combinations are much lower than the allowable.*~~

Table 3.8.3-5 (Sheet 1 of 3)

[*DESIGN SUMMARY OF SOUTH WALL OF STEAM GENERATOR COMPARTMENT DESIGN LOADS, LOAD COMBINATIONS, AND COMPARISON TO ACCEPTANCE CRITERIA MID-SPAN AT MID-HEIGHT*]* (3)

Load/Comb.	TX	TY	TXY	MX	MY	MXY	NX	NY	Comments
	k/ft	k/ft	k/ft	kft/ft	kft/ft	kft/ft	k/ft	k/ft	
Dead (D)	-1	-20	0	1	0	0	0	0	–
Hydro (F_o)	-2	3	-7	19	22	0	0	-1	–
Live (L)	0	-10	0	2	0	0	0	0	During refueling
Live (L_o)	0	-3	0	0	0	0	0	0	During operation
ADS	-1	12	-16	15	16	0	0	1	–
E_s	11	42	78	28	31	3	3	3	–
Thermal (T_0)	-136	-139	-13	221	217	6	-3	-5	–
LC (1)	-6	-9	-37	54	58	0	0	0	$1.4D+1.4F+1.7L_o+1.7ADS$
LC (2)	-4	-41	-10	31	31	0	0	-1	$1.4D+1.4F+1.7L_r$
LC (3)	-6	-3	-37	54	58	0	0	0	$1.4D+1.4F+1.7ADS$
LC (4)	9	34	87	63	69	3	3	3	$D+F+L_o+\|ADS\|+E_s$
LC (5)	-15	-74	-101	-23	-25	-3	-3	-5	$D+F+L_o-\|ADS\|-E_s$
LC (6)	-127	-105	74	284	286	9	0	-2	$D+F+L_o+\|ADS\|+T_0+E_s$
LC (7)	-151	-213	-114	198	192	3	-6	-10	$D+F+L_o-\|ADS\|+T_0-E_s$
LC (8)	7	39	55	63	69	3	3	3	$0.9D+1.0F+1.0\|ADS\|+1.0E_s$

Notes:
x-direction is horizontal, y-direction is vertical.
element number 104228

Plate thickness required for load combinations excluding thermal:	*0.04 inches*
Plate thickness provided:	*0.50 inches*
Maximum principal stress for load combinations including thermal:	*23.0 ksi*
Yield stress at temperature:	*36.0 ksi*
Maximum stress intensity range for load combinations including thermal:	*23.0 ksi*
Allowable stress intensity range for load combinations including thermal:	*72.0 ksi*

(3) See Subsection 3.8.3.5.8 for reporting requirements for changes to Tier 2* information in this section.
~~*NRC Staff approval is required prior to implementing a change in this information; see DCD Introduction Section 3.5.~~

~~Table 3.8.3-5 (Sheet 2 of 3)~~ ~~[*DESIGN SUMMARY OF SOUTH WALL OF STEAM GENERATOR COMPARTMENT*]*~~	
~~***Element Number 101943***~~	
~~*Plate thickness provided*~~	~~*= 0.50 inches*(1)~~
~~***Thermal Load Combinations***~~	
~~*Yield stress at design temperature*~~	~~*= 36.0 ksi*~~
~~*Allowable stress intensity range for load combinations (including thermal)*~~	~~*= 72.0 ksi*(2)~~

~~***Notes:***~~
1. ~~*This is a lot more than the plate thickness required for load combinations excluding thermal.*~~
2. ~~*The maximum principal stress and the maximum stress intensity range for these load combinations are much lower than the allowable.*~~

Table 3.8.3-5 (Sheet 2 of 3)

[DESIGN SUMMARY OF SOUTH WALL OF STEAM GENERATOR COMPARTMENT DESIGN LOADS, LOAD COMBINATIONS, AND COMPARISON TO ACCEPTANCE CRITERIA MID-SPAN AT BASE]* (3)

Load/Comb.	TX	TY	TXY	MX	MY	MXY	NX	NY	Comments
	k/ft	k/ft	k/ft	kft/ft	kft/ft	kft/ft	k/ft	k/ft	
Dead (D)	-3	-24	0	0	0	0	0	0	–
Hydro (F)	3	4	-12	-5	-41	0	0	15	–
Live (L)	-1	-9	-1	0	-2	0	0	0	During refueling
Live (L_o)	0	-3	0	0	0	0	0	0	During operation
ADS	2	14	-15	-4	-30	0	0	9	–
E_s	18	50	71	4	32	2	1	13	–
Thermal (T_0)	-300	-40	33	240	266	7	8	-6	–
LC (1)	3	-9	-42	-14	-108	0	0	36	$1.4D+1.4F+1.7L_o+1.7ADS$
LC (2)	-2	-43	-19	-7	-61	0	0	21	$1.4D+1.4F+1.7L_r$
LC (3)	3	-4	-42	-14	-108	0	0	36	$1.4D+1.4F+1.7ADS$
LC (4)	20	41	74	3	21	2	1	37	$D+F+L_o+\|ADS\|+E_s$
LC (5)	-20	-87	-98	-13	-103	-2	-1	-7	$D+F+L_o-\|ADS\|-E_s$
LC (6)	-280	1	107	243	287	9	9	31	$D+F+L_o+\|ADS\|+T_0+E_s$
LC (7)	-320	-127	-65	227	163	5	7	-13	$D+F+L_o-\|ADS\|+T_0-E_s$
LC (8)	20	46	44	-5	-39	2	1	37	$0.9D+1.0F+1.0\|ADS\|+1.0E_s$

Notes:

x-direction is horizontal, y-direction is vertical.
element number 101943

Plate thickness required for load combinations excluding thermal: 0.04 inches
Plate thickness provided: 0.50 inches

Maximum principal stress for load combinations including thermal: 25.7 ksi
Yield stress at temperature: 36.0 ksi

Maximum stress intensity range for load combinations including thermal: 25.7 ksi
Allowable stress intensity range for load combinations including thermal: 72.0 ksi

(3) See Subsection 3.8.3.5.8 for reporting requirements for changes to Tier 2* information in this section.
~~*NRC Staff approval is required prior to implementing a change in this information; see DCD Introduction Section 3.5.~~

~~Table 3.8.3-5 (Sheet 3 of 3)~~ ~~[DESIGN SUMMARY OF SOUTH WALL OF STEAM GENERATOR COMPARTMENT]*~~	
~~*Element Number 101933*~~	
~~*Plate thickness provided*~~	~~*= 0.50 inches*[(1)]~~
~~*Thermal Load Combinations*~~	
~~*Yield stress at design temperature*~~	~~*= 36.0 ksi*~~
~~*Allowable stress intensity range for load combinations (including thermal)*~~	~~*= 72.0 ksi*[(2)]~~

~~*Notes:*~~
1. ~~*This is a lot more than the plate thickness required for load combinations excluding thermal.*~~
2. ~~*The maximum principal stress and the maximum stress intensity range for these load combinations are much lower than the allowable.*~~

Table 3.8.3-5 (Sheet 3 of 3)

*[DESIGN SUMMARY OF SOUTH WALL OF STEAM GENERATOR COMPARTMENT DESIGN LOADS, LOAD COMBINATIONS, AND COMPARISON TO ACCEPTANCE CRITERIA WEST END BOTTOM CORNER]** (3)

Load/Comb.	TX k/ft	TY k/ft	TXY k/ft	MX kft/ft	MY kft/ft	MXY kft/ft	NX k/ft	NY k/ft	Comments
Dead (D)	-6	-34	3	-1	3	0	-1	-3	–
Hydro (F)	6	16	-12	-5	-11	3	2	3	–
Live (L)	-3	-15	2	0	1	0	0	-1	During refueling
Live (L_o)	-1	-5	0	0	1	0	0	0	During operation
ADS	13	55	-16	-2	-13	2	3	5	–
E_s	44	193	78	6	26	4	9	26	–
Thermal (T_0)	-314	-139	179	170	341	12	-47	-123	–
LC (1)	20	60	-40	-12	-32	8	7	9	$1.4D+1.4F+1.7L_o+1.7ADS$
LC (2)	-5	-51	-9	-8	-10	4	1	-2	$1.4D+1.4F+1.7L_r$
LC (3)	22	68	-40	-12	-33	8	7	9	$1.4D+1.4F+1.7ADS$
LC (4)	56	225	85	2	32	9	13	31	$D+F+L_o+\|ADS\|+E_s$
LC (5)	-58	-271	-103	-14	-46	-3	-11	-31	$D+F+L_o-\|ADS\|-E_s$
LC (6)	-258	86	264	172	373	21	-34	-92	$D+F+L_o+\|ADS\|+T_0+E_s$
LC (7)	-372	-410	76	156	295	9	-58	-154	$D+F+L_o-\|ADS\|+T_0-E_s$
LC (8)	58	233	53	-2	5	9	13	31	$0.9D+1.0F+1.0\|ADS\|+1.0E_s$

Notes:	
x-direction is horizontal, y-direction is vertical.	
element number 101933	
Plate thickness required for load combinations excluding thermal:	*0.04 inches*
Plate thickness provided:	*0.50 inches*
Maximum principal stress for load combinations including thermal:	*43.1 ksi*
Yield stress at temperature:	*36.0 ksi*
Maximum stress intensity range for load combinations including thermal:	*52.6 ksi*
Allowable stress intensity range for load combinations including thermal:	*72.0 ksi*

(3) See Subsection 3.8.3.5.8 for reporting requirements for changes to Tier 2* information in this section. ~~*NRC Staff approval is required prior to implementing a change in this information; see DCD Introduction Section 3.5.~~

Revise DCD Tier 2 Table 3.8.3-6, "Design Summary of North-East Wall of IRWST," as follows:

~~Table 3.8.3-6 (Sheet 1 of 3)~~ ~~[*DESIGN SUMMARY OF NORTH-EAST WALL OF IRWST*]*~~	
~~*Element Number 140027*~~	
~~*Plate thickness provided*~~	~~*= 0.50 inches(1)*~~
~~*Thermal Load Combinations*~~	
~~*Yield stress at design temperature*~~	~~*= 36.0 ksi*~~
~~*Allowable stress intensity range for load combinations (including thermal)*~~	~~*= 72.0 ksi(2)*~~

~~*Notes:*~~

~~*1. This is a lot more than the plate thickness required for load combinations excluding thermal.*~~

~~*2. The maximum principal stress and the maximum stress intensity range for these load combinations are much lower than the allowable.*~~

Table 3.8.3-6 (Sheet 1 of 3)

*[DESIGN SUMMARY OF NORTH-EAST WALL OF IRWST DESIGN LOADS, LOAD COMBINATIONS, AND COMPARISON TO ACCEPTANCE CRITERIA MID-SPAN AT MID-HEIGHT]** (3)

Load/Comb.	TX	TY	TXY	MX	MY	MXY	NX	NY	Comments
	k/ft	k/ft	k/ft	kft/ft	kft/ft	kft/ft	k/ft	k/ft	
Dead (D)	-1	-13	3	0	3	1	-1	-2	–
Hydro (F)	-5	1	0	8	5	1	2	2	–
Live (L)	0	-12	3	1	8	4	-2	-3	During refueling
Live (L_o)	0	-2	2	2	9	4	-2	-3	During operation
ADS	-7	4	3	8	3	2	2	3	–
E_s	14	27	38	19	32	15	6	14	–
Thermal (T_0)	-84	-65	43	208	218	8	-10	-12	–
LC (1)	-20	-13	13	28	32	13	1	0	$1.4D+1.4F+1.7L_o+1.7ADS$
LC (2)	-8	-37	9	13	25	10	-2	-5	$1.4D+1.4F+1.7L_r$
LC (3)	-20	-10	9	25	16	6	5	5	$1.4D+1.4F+1.7ADS$
LC (4)	15	17	46	37	52	23	7	14	$D+F+L_o+\|ADS\|+E_s$
LC (5)	-27	-45	-36	-17	-18	-11	-9	-20	$D+F+L_o-\|ADS\|-E_s$
LC (6)	-69	-48	89	245	270	31	-3	2	$D+F+L_o+\|ADS\|+T_0+E_s$
LC (7)	-111	-110	7	191	200	-3	-19	-32	$D+F+L_o-\|ADS\|+T_0-E_s$
LC (8)	1	20	44	35	43	19	9	17	$0.9D+1.0F+1.0\|ADS\|+1.0E_s$

Notes:	
x-direction is horizontal, y-direction is vertical.	
element number 140027	
Plate thickness required for load combinations excluding thermal:	*0.04 inches*
Plate thickness provided:	*0.50 inches*
Maximum principal stress for load combinations including thermal:	*23.4 ksi*
Yield stress at temperature:	*36.0 ksi*
Maximum stress intensity range for load combinations including thermal:	*23.4 ksi*
Allowable stress intensity range for load combinations including thermal:	*72.0 ksi*

)

(3) See Subsection 3.8.3.5.8 for reporting requirements for changes to Tier 2* information in this section.
~~*NRC Staff approval is required prior to implementing a change in this information; see DCD Introduction Section 3.5.~~

~~Table 3.8.3-6 (Sheet 2 of 3)~~ ~~[*DESIGN SUMMARY OF NORTH-EAST WALL OF IRWST*]*~~	
~~***Element Number 140005***~~	
~~*Plate thickness provided*~~	~~*= 0.50 inches(1)*~~
~~***Thermal Load Combinations***~~	
~~*Yield stress at design temperature*~~	~~*= 36.0 ksi*~~
~~*Allowable stress intensity range for load combinations (including thermal)*~~	~~*= 72.0 ksi(2)*~~

~~***Notes:***~~

~~*1. This is a lot more than the plate thickness required for load combinations excluding thermal.*~~

~~*2. The maximum principal stress and the maximum stress intensity range for these load combinations are much lower than the allowable.*~~

Table 3.8.3-6 (Sheet 2 of 3)

*[DESIGN SUMMARY OF NORTH-EAST WALL OF IRWST
DESIGN LOADS, LOAD COMBINATIONS, AND COMPARISON TO ACCEPTANCE CRITERIA
MID-SPAN AT BOTTOM – ELEVATION 107'-2"]*(3)*

Load/Comb.	TX k/ft	TY k/ft	TXY k/ft	MX kft/ft	MY kft/ft	MXY kft/ft	NX k/ft	NY k/ft	Comments
Dead (D)	-1	-16	3	0	2	0	0	-1	–
Hydro (F)	-1	2	-1	0	-8	1	0	9	-
Live (L)	0	-11	1	0	2	0	0	-1	During refueling
Live (L_o)	0	-4	1	0	1	0	0	-1	During operation
ADS	-2	4	3	0	-6	2	0	6	–
E_s	18	31	40	16	58	9	6	11	–
Thermal (T_0)	-220	-163	80	212	213	1	4	6	–
LC (1)	-6	-20	10	0	-17	5	0	20	$1.4D+1.4F+1.7L_o+1.7ADS$
LC (2)	-3	-38	5	0	-5	1	0	10	$1.4D+1.4F+1.7L_r$
LC (3)	-6	-13	8	0	-19	5	0	21	$1.4D+1.4F+1.7ADS$
LC (4)	18	17	46	16	59	12	6	24	$D+F+L_o+\|ADS\|+E_s$
LC (5)	-22	-53	-40	-16	-69	-10	-6	-10	$D+F+L_o-\|ADS\|-E_s$
LC (6)	-202	-146	126	228	272	13	10	30	$D+F+L_o+\|ADS\|+T_0+E_s$
LC (7)	-242	-216	40	196	144	-9	-2	-4	$D+F+L_o-\|ADS\|+T_0-E_s$
LC (8)	14	23	45	16	46	12	6	25	$0.9D+1.0F+1.0\|ADS\|+1.0E_s$

Notes:
x-direction is horizontal, y-direction is vertical.
element number 140005

Plate thickness required for load combinations excluding thermal: 0.04 inches
Plate thickness provided: 0.50 inches

Maximum principal stress for load combinations including thermal: 22.8 ksi
Yield stress at temperature: 36.0 ksi

Maximum stress intensity range for load combinations including thermal: 22.8 ksi

Allowable stress intensity range for load combinations including thermal: 72.0 ksi

(3) See Subsection 3.8.3.5.8 for reporting requirements for changes to Tier 2* information in this section.
~~*NRC Staff approval is required prior to implementing a change in this information; see DCD Introduction Section 3.5.~~

~~Table 3.8.3-6 (Sheet 3 of 3)~~ ~~[*DESIGN SUMMARY OF NORTH-EAST WALL OF IRWST*]*~~	
~~***Element Number 140001***~~	
~~*Plate thickness provided*~~	~~*= 0.50 inches*~~[(1)]
~~***Thermal Load Combinations***~~	
~~*Yield stress at design temperature*~~	~~*= 36.0 ksi*~~
~~*Allowable stress intensity range for load combinations (including thermal)*~~	~~*= 72.0 ksi*~~[(2)]

~~***Notes:***~~
1. ~~*This is a lot more than the plate thickness required for load combinations excluding thermal.*~~
2. ~~*The maximum principal stress and the maximum stress intensity range for these load combinations are much lower than the allowable.*~~

Table 3.8.3-6 (Sheet 3 of 3)

[DESIGN SUMMARY OF NORTH-EAST WALL OF IRWST DESIGN LOADS, LOAD COMBINATIONS, AND COMPARISON TO ACCEPTANCE CRITERIA NORTH END BOTTOM CORNER – ELEVATION 107'-2"](3)*

Load/Comb.	TX	TY	TXY	MX	MY	MXY	NX	NY	Comments
	k/ft	k/ft	k/ft	kft/ft	kft/ft	kft/ft	k/ft	k/ft	
Dead (D)	-1	-21	3	0	0	0	0	0	–
Hydro (F)	-3	17	9	10	13	11	-6	-16	–
Live (L)	0	-15	2	0	0	0	0	0	During refueling
Live (L_o)	0	-6	1	0	0	0	0	0	During operation
ADS	-3	27	11	9	17	10	-5	-16	–
E_s	6	98	37	34	139	31	14	52	–
Thermal (T_0)	-49	-42	72	32	173	-40	-19	49	–
LC (1)	-11	30	37	29	47	32	-17	-50	$1.4D+1.4F+1.7L_o+1.7ADS$
LC (2)	-6	-31	20	14	18	15	-8	-22	$1.4D+1.4F+1.7L_r$
LC (3)	-11	40	36	29	47	32	-17	-50	$1.4D+1.4F+1.7ADS$
LC (4)	5	115	61	53	169	52	13	52	$D+F+L_o+\|ADS\|+E_s$
LC (5)	-13	-135	-35	-33	-143	-30	-25	-84	$D+F+L_o-\|ADS\|-E_s$
LC (6)	-44	73	133	85	342	12	-6	101	$D+F+L_o+\|ADS\|+T_0+E_s$
LC (7)	-62	-177	37	-1	30	-70	-44	-35	$D+F+L_o-\|ADS\|+T_0-E_s$
LC (8)	-1	123	60	53	169	52	3	20	$0.9D+1.0F+1.0\|ADS\|+1.0E_s$

Notes:	
x-direction is horizontal, y-direction is vertical.	
element number 140001	
Plate thickness required for load combinations excluding thermal:	*0.04 inches*
Plate thickness provided:	*0.50 inches*
Maximum principal stress for load combinations including thermal:	*32.3 ksi*
Yield stress at temperature:	*36.0 ksi*
Maximum stress intensity range for load combinations including thermal:	*32.4 ksi*
Allowable stress intensity range for load combinations including thermal:	*72.0 ksi*

(3) See Subsection 3.8.3.5.8 for reporting requirements for changes to Tier 2* information in this section.
~~*NRC Staff approval is required prior to implementing a change in this information; see DCD Introduction Section 3.5.~~

Westinghouse

Revise DCD Tier 2, Table 3.8.4-6, "Materials Used in Structural and Miscellaneous Steel," as follows:

Table 3.8.4-6

MATERIALS USED IN STRUCTURAL AND MISCELLANEOUS STEEL

Standard	Construction Material
ASTM A1	Carbon steel rails
ASTM A36/A36M	Rolled shapes, plates, and bars
ASTM A53	Welded and Seamless Steel Pipe, Grade B
ASTM A106	Seamless Carbon Steel Pipe for High Temperature Service
ASTM A108	Weld studs
ASTM A123	Zinc coatings (hot galvanized)
ASTM A167	Stainless and Heat-Resisting Chromium Nickel Steel Plate, Sheet and Strip.
ASTM A193	Alloy Steel and Stainless Steel Bolting Materials for High-Temperature Service
ASTM A194	Carbon and Alloy Steel Nuts and Bolts for High-Pressure and High-Temperature Service
ASTM A240	Duplex 2101 stainless steel (designation S32101)
ASTM A242	High-strength low alloy structural steel
ASTM A276	Stainless and Heat-Resisting Steel Bars and Shapes
ASTM A307	Low carbon steel bolts
ASTM A312	Seamless and Welded Austenitic Stainless Steel Pipe
ASTM A325	High strength bolts
ASTM A354	Quenched and tempered alloy steel bolts (Grade BC)

ASTM A441	High-strength low alloy structural manganese vanadium steel
ASTM A496	ASTM A496 - Standard Specification for Steel Wire, Deformed, for Concrete Reinforcement
ASTM A500	Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes
ASTM A501	Hot-Formed Welded and Seamless Carbon Steel Structural Tubing
ASTM A505	Standard Specification for Steel, Sheet and Strip, Alloy, Hot-Rolled and Cold-Rolled
ASTM A514	High-Yield Strength Quenched and Tempered Alloy Steel Plate, Suitable for Welding
ASTM A517	Standard Specification for Pressure Vessel Plates, Alloy Steel, High-Strength, Quenched and Tempered
ASTM A564	Hot-Rolled and Cold-Finished, Age Hardening Stainless and Heat-Resisting Steel Bars and Shapes
ASTM A570	Hot-Rolled Carbon Steel Sheets and Strip, Structural Quality, Grades C, D and E
ASTM A572	High-strength low alloy structural steel
ASTM A588	High-strength low alloy structural steel
ASTM A607	Steel Sheet and Strip, Hot-Rolled and Cold-Rolled, High-Strength, Low-Alloy, Columbium and/or Vanadium
ASTM A615	Deformed and Plain Billet Steel Bars for Concrete Reinforcement
ASTM A618	Hot-Formed Welded and Seamless High-Strength Low-Alloy Structural Tubing
ASTM A706	Low Alloy Steel Deformed Bars for Concrete Reinforcement
ASTM A970	Specification for Welded Headed Bars for Concrete Reinforcement
ASTM A992	Structural steel shapes
ASTM F1554	Steel anchor bolts, 36, 55, and 105-ksi Yield Strength

PRA Revision: None

Technical Report (TR) Revision: None

RAI Response Number: RAI-TR85-SEB1-10
Revision: 6

Question:

Section 2.4.1 indicates that "Table 2.4-2 shows the reactions at the underside of the basemat for each soil case. These are conservative estimates using the results of the 2D SASSI horizontal analyses..." The following items need to be addressed:

a. What is the technical basis that these results are considered to be conservative?

b. What is the technical basis for combining the M_{xx} EW seismic load with the vertical load by SRSS and similarly for the M_{yy} NS excitation load and the vertical load? (Normally SRSS is applicable to the use of three directional load combination. Since these loads are being used for the NI stability evaluation, normal practice is to utilize the summation of one horizontal load and vertical load, both acting in the worst direction. This would be repeated for the other horizontal load and vertical load.)

c. Footnote 2 of Table 2.4-2 (Page 13 of 83) states that reactions for horizontal input are calculated from the 2D SASSI analyses. Reactions due to vertical input are calculated from the maximum accelerations in 3D ANSYS or SASSI analyses for hard rock (HR), firm rock (FR), upper bound of soft medium soil (UBSM), and soft to medium soil (SM), and from 2D ANSYS analyses for soft rock (SR) and soft soil (SS). Was the 2D ANSYS analyses, referred to here, based on the linear or nonlinear ANSYS analyses? Also, why wasn't one consistent set of analyses (say 2D SASSI) used for both horizontal and vertical input in this evaluation?

Additional Request (Revision 1):

The staff reviewed the RAI response provided in Westinghouse letter dated 10/19/07. Based on the information provided, Westinghouse is requested to address the items listed below.

a. With the changes made to a number of seismic analyses, explain whether the maximum seismic reactions in Table 2.4-2, developed from the 2D SASSI analyses, are still relied upon for any purpose. If so, then explain where they are utilized and why combining the member forces above grade with the inertia forces below grade, using absolute sum, is considered to be conservative.

b. The use of the SRSS or the 100/40/40 combination method is only acceptable for combining the co-directional responses such as Mxx due to NS, EW, and vertical, in order to obtain a combined Mxx. However, it is not clear from TR 85, DCD Section 3.8.5, nor from the RAI response, how the stability calculations are performed once the individual three loads Mxx, Myy, and vertical (each of these already combined by SRSS or 100/40/40 due to the three earthquake inputs) are determined. DCD 3.8.5.5.4, for example, discusses the overturning evaluation and presents the equation for the factor of safety as the resisting moment divided by the overturning moment. However, this does not explain how the vertical seismic force is considered. The traditional method for evaluating stability (sliding and overturning) of nuclear plant structures in accordance with SRP 3.8.5 is to perform two separate 2-D evaluations, one for the N-S and vertical directions and one for the E-W and vertical directions. Thus, for overturning evaluation as an example, the minimum vertical downward load (deadweight minus upward buoyancy force minus upward vertical seismic force) is considered in calculating the resisting moment and this is then compared to the overturning moment about one horizontal direction (i.e., EW axis); then a similar comparison is made for the same minimum downward vertical load with the overturning moment about the other perpendicular horizontal direction (i.e., NS axis). Westinghouse is requested to clarify if they follow this analytical method for the stability evaluations (sliding and overturning) and document the approach in TR85 and the DCD. If not, then Westinghouse is requested to justify any other alternative method used. Note, with the changes recently made in the various seismic analyses, explain whether the maximum seismic reactions in Table 2.4-2, developed from the 2D SASSI analyses, are still relied upon for use in the stability evaluations performed in Section 2.9 of TR85.

<u>Note</u>: that the issues described above are applicable to all stability evaluations including the new 3D NI20 model using response spectrum analysis with ANSYS, which is used for stability evaluation.

c. With the changes made to a number of seismic analyses, explain whether the results from Table 2.4-2 and footnote 2, developed from the 2D SASSI analyses, are still relied upon for any purpose. If so, then Westinghouse is requested to provide the technical basis for the statement "...different models give consistent results and use of results from different analyses is acceptable."

Additional Request (Revision 2):

In the response for item b of the RAI, Westinghouse indicated that the analysis for stability has been revised to utilize the 3D ANSYS finite element NI20 model using a mode superposition time history analysis (linear with no lift-off). A separate 2D ANSYS lift-off analysis demonstrated that the minor lift-off is negligible. Since the 3D ANSYS NI20 model analysis using three input motions applied simultaneously is utilized for the stability evaluation, the concern raised by the directional combination methods no longer applies. Therefore, this concern has been adequately addressed. However, the RAI response discussed the need to utilize some passive pressure resistance capability of the soil when performing the sliding stability analyses. The passive pressure resistance curve as a function of displacement is based on Reference 1 (Hsai-Yang Fang, "Foundation Engineering Handbook," 1991) given in the RAI response. Westinghouse is requested to provide the complete text in the applicable section or chapter of the referenced book which describes the approach for determining the passive pressure resistance function.

Additional Request (Revision 3):

1. Remove the F_P term in the equations and explain the removal in DCD sections 3.8.5.5.3 and 3.8.5.5.4.
2. Check the reference to Table 3.8.5-2 and clarify the reference in the second paragraph of the revised DCD markup in the RAI response.
3. Check DCD Table 3.8.5-2 for use of zero passive pressure and explain or justify use.

Additional Request (Revision 4):

a) The staff reviewed the response provided in Westinghouse letter dated September 22, 2009 and found that insufficient information was provided. In the response, the passive earth pressure was removed from the seismic stability analysis, an explanation was provided why reliance on soil passive pressure is not required for stability evaluation, and related tables were revised in the corresponding subsections of the DCD and TR-85. This information is subject to an audit for its adequacy.

b) As a result of the staff's structural audit conducted during the week of August 10, 2009, the NRC staff requested the justification as to why TR-85 is not identified as Tier 2* since it is referenced in DCD Section 3.8.5 and it contains key details of the design of the foundation. Similarly, justification as to why TR-9 (Containment Vessel Design Adjacent to Large Penetrations) and TR-57 (Nuclear Island: Evaluation of Critical Sections) are not identified as Tier 2* information because they contain key analysis and design information for the containment, and the auxiliary and shield buildings, which are not sufficiently described in the DCD, was not provided. Therefore, the staff requests either that TR-9, TR-57, and TR-85 be identified as Tier 2* information in the DCD, or a justification provided.

Additional Request (Revision 5):

During the NRC seismic audit conducted during the week of June 14 to 18, 2010, the NRC staff discussed with Westinghouse the response to Revision 4 of this RAI. The staff clarified that the request to identify selected technical reports as Tier 2* information was to assure that important information on the design and analysis of structures is captured as information that requires review before it is revised or changed. Key details of the analysis and design of the containment, auxiliary building, shield building, and foundation need to be included in the licensing basis. Westinghouse should provide a means to accomplish this. The technical reports that should be considered when identifying the key details of the analysis and design include TR-03, TR-9, TR-57, TR-85, TR-115, and the shield building report.

It is requested that changes in the nuclear island seismic sliding displacements due to modifications of the non-linear seismic sliding model be reflected in this RAI.

Additional Request (Revision 6):

The staff reviewed the Westinghouse response to RAI-TR85-SEB1-10, Rev. 5, transmitted in their letter dated July 30, 2010. The proposed approach, to add the specific Tier 2* information from the applicable technical reports (TR) and shield building (SB) report(s) to the AP1000 DCD, is acceptable, because mark-ups of 3.7 and 3.8, and related appendices, will be provided and give the staff an opportunity to confirm that the required information from all of the applicable technical reports and RAI responses will be identified as Tier 2* in the DCD. The response regarding revised nuclear island seismic sliding displacements is also acceptable, because it provides the mark-ups for the changes to the DCD and TR-85 to reflect the changes in the sliding evaluation and the increases in seismic displacements due to sliding. However, this RAI still remains unresolved, primarily because the review of the specific Tier 2* information will be completed after the responses for the remaining unresolved RAIs are finalized, as indicated in the response. In addition, the following information needs to be provided:

(1) In the sixth paragraph of page 19 of 29 of the RAI response, include Appendix 3H along with Appendices 3G and 3I for review of Tier 2* information.

(2) Last paragraph in the mark-up for DCD Section 3.8.5.5.3 – Sliding, on page 21 of 29, and TR-85 Section 2.9, page 25 of 29 state:

"The governing friction value in the interface zone is a thin soil layer below the mudmat with an angle of internal friction of 35° giving a static coefficient of friction of 0.55."

This is not necessarily true; there are a number of sliding friction interfaces and relying on only the "thin soil layer below the mudmat" for the "governing" coefficient of friction of 0.55 could cause another sliding interface beneath this thin layer to govern or above the thin soil layer (concrete to soil interface) to govern. Therefore, the sentence should be revised to indicate that the friction value used in the evaluation is based on a governing angle of internal friction of 0.55 for the soil beneath the foundation.

(3) Second paragraph in the mark-up for DCD Section 3.8.5.5.5 – Seismic Stability Analysis, on page 22 of 29, and TR-85 Section 2.9, page 26 of 29 state:

"Therefore, it can be concluded that the Nuclear Island is stable against sliding, and there is no quality requirement for the backfill material adjacent to the NI (side soil) to maintain stability against sliding."

This statement is misleading and can be misinterpreted that there is no quality requirement for backfill material of the NI. Therefore, the last part of this statement should be deleted because it does not add any useful information; and more importantly, there are requirements for backfill material for other considerations.

Westinghouse Response:

a. The results in Table 2.4-2 are conservative because of the method of combination of member forces and inertia forces below grade. The maximum member forces at grade are translated down to the underside of the basemat with an absolute combination of the effects of the horizontal shear forces and the moments. The horizontal loads on the portion below grade are added absolutely to the sum of the member forces above grade.

b. As described in DCD subsection 3.7.2.6,

> In analyses with the earthquake components applied separately and in the response spectrum and equivalent static analyses, the effect of the three components of earthquake motion are combined using one of the following methods:
>
> - The peak responses due to the three earthquake components from the response spectrum and equivalent static analyses are combined using the square root of the sum of squares (SRSS) method.

- The peak responses due to the three earthquake components are combined directly, using the assumption that when the peak response from one component occurs, the responses from the other two components are 40 percent of the peak (100 percent-40 percent-40 percent method). Combinations of seismic responses from the three earthquake components, together with variations in sign (plus or minus), are considered. This method is used in the nuclear island basemat analyses, the containment vessel analyses and the shield building roof analyses.

In the combination shown in Table 2.4-2, the moment M_{xx} due to input in the NS direction is zero. Thus the SRSS combination combines two components (EW seismic load and vertical load).

c. The 2D ANSYS analyses referred to in Footnote 2 of Table 2.4-2 were based on linear ANSYS analyses. As described in TR85 many analyses have been performed using a variety of models. At the time of the stability evaluation there was not a consistent set available. However, the different models give consistent results and use of results from different analyses is acceptable.

Westinghouse Response (Revision 1):

a. As discussed in RAI-TR85-SEB1-04, part (2), Revision 1, the 2D SASSI reactions (Fx, Fy, and Fz) are used to obtain seismic response factors between the hard rock case to the upper-bound-soft-to-medium (UBSM) soil case, and the soft-to-medium (SM) soil case. These factors were used to adjust the hard rock time history to reflect the seismic response for the other two potential governing soil cases UBSM and SM. The firm rock, soft rock, and soft soil have lower seismic response. Combining the member forces above grade with the inertia forces below grade using absolute sum is conservative since it assumes the structures above grade, and those below grade are in phase (modes closely spaced). Otherwise, one could have used the SRSS method.

b. Westinghouse agrees that the SRSS and 100/40/40 combination method is only acceptable for combining the co-directional responses. When Westinghouse has used this combination method it has been applied only to co-directional responses. The NRC has previously reviewed the acceptable use of the 100/40/40 method as part of the AP600 and the hard rock certification. The NRC in their FSER (NUREG-1793) related to AP1000 hard rock licensing states:

> "As for the suitability of using the 100 percent, 40 percent, 40 percent combination method, the applicant, during audits performed by the staff, provided calculations to demonstrate that the combination method always gives reasonable results by comparing the results with those from the SRSS combination method. From its review of the design calculations, the staff also finds that the difference between results obtained using the two methods was less than 5 percent which is considered insignificant and, therefore, is acceptable."

The NRC review and audit considered stability, and it is further stated in FSER Section 3.7.2.17:

"... When the equivalent acceleration static analysis method is used, the SRSS method or 100 percent, 40 percent, 40 percent method was used to combine spatial response in conformance with RG 1.92 and consistent with accepted common industry practice. ... Torsional effects and stability against overturning, sliding, and flotation are considered.

When it is necessary to combine co-directional responses, Westinghouse is not using any different methodology that wasn't reviewed and accepted by the NRC previously.

For the seismic stability analysis Westinghouse is using the 3D NI20 model. Time history analyses using ANSYS has been used. This is discussed in RAI-TR85-SEB1-004, part (2). It was not necessary to use the 100 percent, 40 percent, 40 percent method. However, if this method was used the following method would have been used to calculate the co-directional responses:

- The seismic maximum moment about an edge (e.g. column line I) is calculated considering the maximum moment due to the horizontal excitation combined with 40 percent of the moment due to the maximum vertical seismic excitation. (Note that using 100 percent of maximum vertical seismic excitation, and 40 percent of the maximum moment due to horizontal excitation will not control.) This moment is used as the maximum SSE overturning moment in the stability evaluation.
- For sliding 40 percent of the maximum vertical seismic component is considered in the reduction of the normal force in the calculation of the friction force.

Using the maximum time history results a comparison of the stability factors of safety obtained to the 100 percent, 40 percent, 40 percent method to the stability factors of safety obtained from the time history analysis is made. The time history analysis calculates the stability factors of safety at each time step, and the minimum factor of safety used. The coefficient of friction considered is 0.55. This comparison is given in Table RAI-TR85-SEB1-10-01a for sliding in the NS and EW direction, and overturning about the West side of the Shield Building and about column line 11. Also, the comparison is given for the hard rock (HR), upper-bound-soft-to-medium (UBSM) case, and the soft-to medium (SM) case. As seen from this comparison, the 100, 40 percent, 40 percent method is more conservative compared to the time history method for the overturning factors of safety. For sliding partial passive pressure is required to meet the 1.1 limit. To compare the two methods the amount of deflections required to obtain the required passive resistance are compared. This comparison is given in Table RAI-TR85-SEB1-10-01b. As seen from this comparison the NS deflections are essentially the same, and for the EW deflections the 1 x 0.4 x 0.4 method is conservative (larger deflections).

It is noted that Westinghouse has not used response spectrum analysis to perform the stability evaluation.

Table RAI-TR85-SEB1-10-01a: Factor of Safety Comparisons for 1 x 0.4 x 0.4 and TH Methods

Stability Factors of Safety	1 x 0.4 x 0.4 Method			T.H. Method		
	HR	UBSM	SM	HR	UBSM	SM
Sliding N-S SSE $\mu = 0.55$	1.1	1.1	1.1	1.1	1.1	1.1
Sliding E-W SSE $\mu = 0.55$	1.1	1.1	1.1	1.1	1.1	1.1
Overturning WSB SSE	1.31	1.17	1.17	1.62	1.44	1.46
Overturning Col. 11 SSE	1.78	1.77	1.79	2.06	2.00	1.92

Table RAI-TR85-SEB1-10-01b: Displacement Comparisons for 1 x 0.4 x 0.4 and TH Methods

Units: inches

Stability Factors of Safety	1 x 0.4 x 0.4 Method			T.H. Method		
	HR	UBSM	SM	HR	UBSM	SM
Sliding N-S SSE $\mu = 0.55$	0.11	0.10	0.07	0.12	0.12	0.08
Sliding E-W SSE $\mu = 0.55$	0.10	0.79	0.65	0.09	0.50	0.49

Provided below is a summary of the stability evaluation performed using the 3D NI20 model and ANSYS time history seismic analyses. Three cases are considered: HR, UBSM, and SM. The other three cases firm rock, soft rock, and soft soil do not control the stability evaluation.

Seismic Overturning Stability Evaluation

It is not necessary to consider passive pressure in the overturning evaluation. Therefore, in the calculation of the factor of safety for overturning the resistance moment associated with passive pressure is zero (Mp = 0). In Table RAI-TR85-SEB1-10-02 is given the factors of safety associated with overturning about column lines 11, 1, I and west side of shield building. All of the factors of safety are above the established limit of 1.1.

Table RAI-TR85-SEB1-10-02: Overturning Factors of Safety

Column Line / Wall	HR F.S.	UBSM F.S.	SM F.S.
Column Line 11 (North)	2.06	2.00	1.92
Column Line 1 (South)	1.83	1.79	1.77
Column Line I (East)	1.31	1.18	1.17
West side of Shield Building (West)	1.62	1.44	1.46

Seismic Sliding Evaluation

In the evaluation of sliding different coefficients of friction are considered. They are 0.7, 0.6, and 0.55. Also, it is necessary to rely on passive pressure. Using Case 15 (RAI-TR85-SEB1-35, R1, Table RAI-TR85-SEB1-35-1), and the methodology given in Reference 1 using a soil friction angle of 35°, a relationship between passive pressure and displacement at grade elevation can be defined. This relationship is shown in Figures RAI-TR85-SEB1-10-1 and RAI-TR85-SEB1-10-2 for the first 5 inches of deflection. Curves are given for the North-South and East-West directions. The passive pressure at zero deflection is equal to the at rest pressure. The total passive soil pressure resistance force is 43,500 kips for the North-South direction, and 69,100 kips for the East-West direction. It is noted that to achieve the full passive pressure displacements in excess of 10 inches are required.

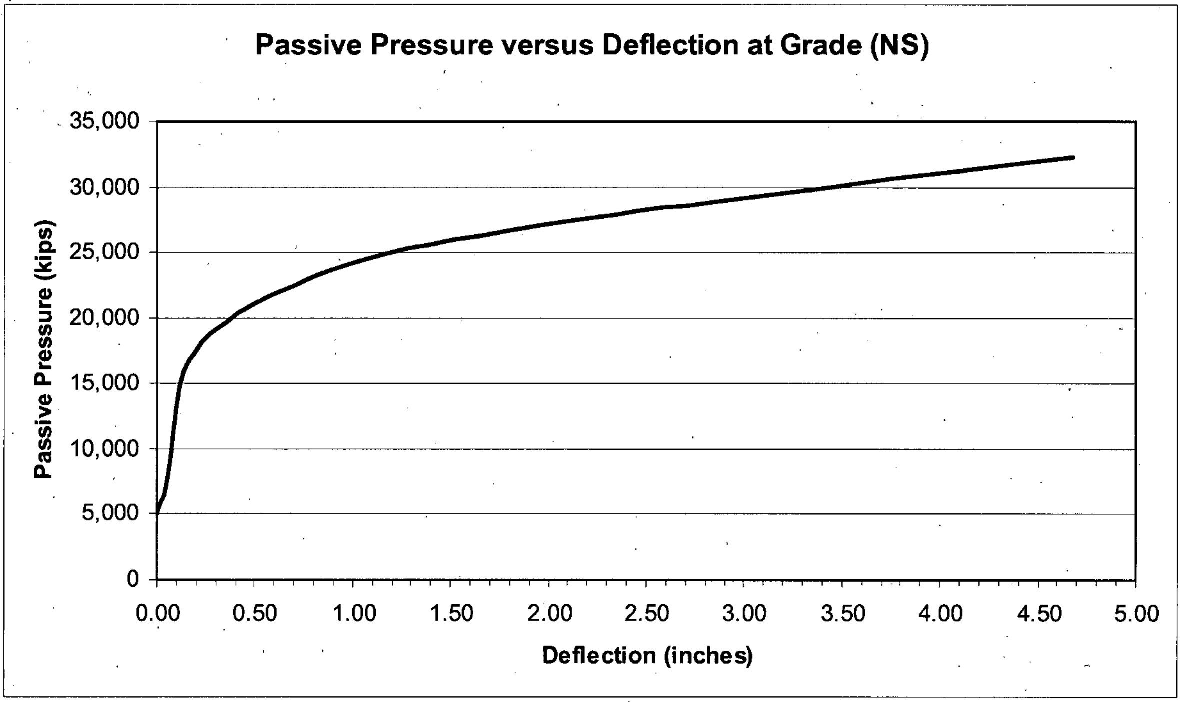


Figure RAI-TR85-SEB1-10-1 – Passive Pressure versus Deflection at Grade (North-South Excitation)

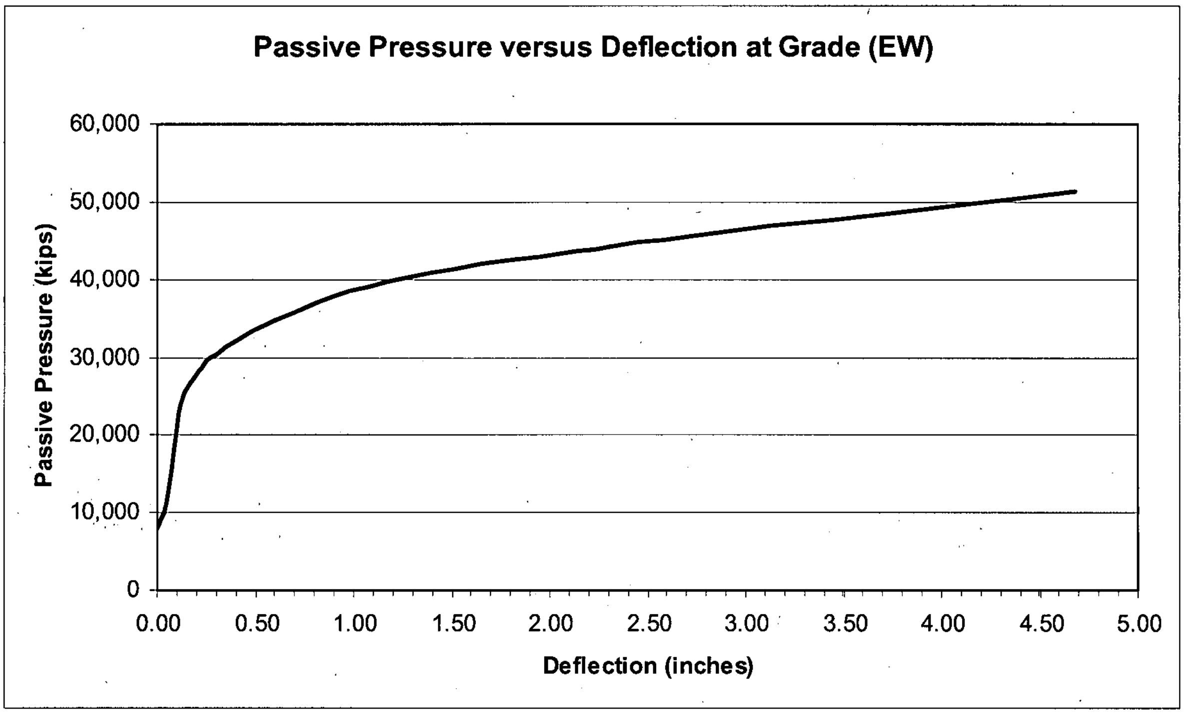


Figure RAI-TR85-SEB1-10-2 – Passive Pressure versus Deflection at Grade (East-West Excitation)

During the sliding stability calculation it was determined that the factor of safety for sliding drops below the limit of 1.1 for a very short time if passive pressure is not considered. Plots of the factor of safety (FS) versus time for the hard rock case and the North-South and East-West directions are given in Figures RAI-TR85-SEB1-10-3 and RAI-TR85-SEB1-10-4 using a coefficient of friction of 0.55. As seen from these figures the time at which the factor of safety drops below 1.1 is very short. This is the only time during the seismic event that this occurs. When the passive pressure is considered, the factor of safety remains above the limit of 1.1.

In Tables RAI-TR85-SEB1-10-3 to RAI-TR85-SEB1-10-5 are given a summary of the results for the three coefficient values. Provided is the required passive pressure to maintain the factor of safety equal to or above 1.1. As seen from this summary using a coefficient of friction of 0.55 or higher, deflections less than 0.15 inch for hard rock, less than or equal to 0.5 inch for upper bound soft to medium and soft to medium soil conditions are needed to develop the required amount of passive pressure.

The coefficient of friction is changed from 0.7 to 0.55 for the soils. The coefficient of friction for the waterproofing membrane is also changed from 0.7 to 0.55.

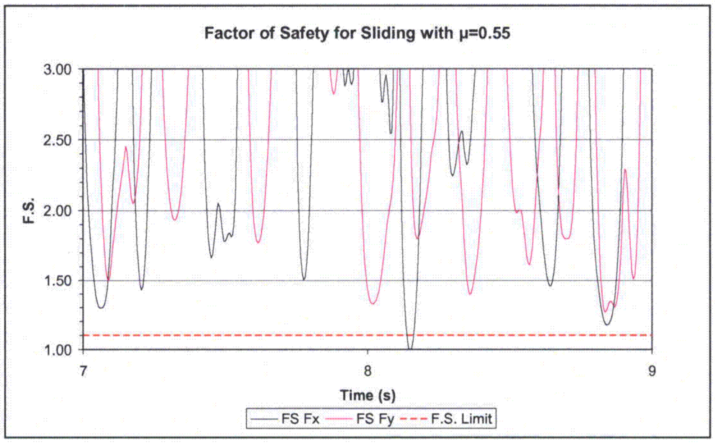


Figure RAI-TR85-SEB1-10-3 - North-South FS without Passive Pressure

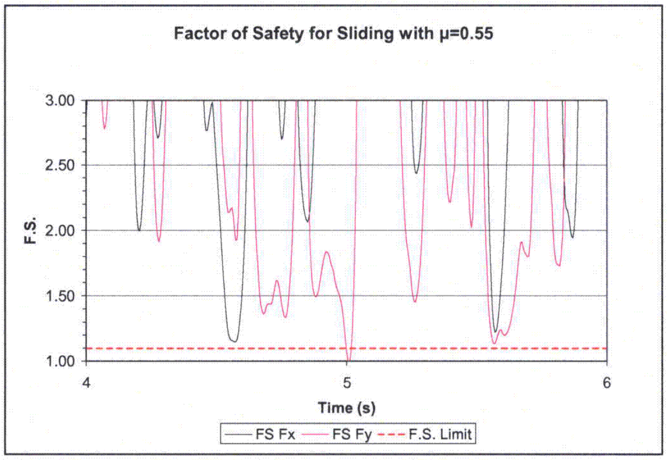


Figure RAI-TR85-SEB1-10-4 – East-West FS without Passive Pressure

Table RAI-TR85-SEB1-10-3 - Factors of Safety against Sliding for Hard Rock

Direction	Coefficient of Static Friction	Passive Pressure Resistance Force Required	Displacement at Grade	F.S.
North – South (Xg)	0.70	(1)	0.00 in	1.24
East – West (Yg)	0.70	(1)	0.00 in	1.23
North – South (Xg)	0.60	7,166 kip	0.05 in	1.10
East – West (Yg)	0.60	10,802 kip	0.04 in	1.10
North – South (Xg)	0.55	15,142 kip	0.12 in	1.10
East – West (Yg)	0.55	18,402 kip	0.09 in	1.10

Note (1) - At rest pressure

Table RAI-TR85-SEB1-10-4 - Factors of Safety against Sliding for Upper Bound Soft to Medium

Direction	Coefficient of Static Friction	Passive Pressure Resistance Force Required	Displacement at Grade	F.S.
North – South (Xg)	0.70	(1)	0.00 in	1.28
East – West (Yg)	0.70	11,127 kip	0.05 in	1.10
North – South (Xg)	0.60	6,992 kip	0.05 in	1.10
East – West (Yg)	0.60	25,927 kip	0.16 in	1.10
North – South (Xg)	0.55	14,817 kip	0.12 in	1.10
East – West (Yg)	0.55	33,352 kip	0.50 in	1.10

Note (1) - At rest pressure

Table RAI-TR85-SEB1-10-5 - Factors of Safety against Sliding for Soft to Medium

Direction	Coefficient of Static Friction	Passive Pressure Resistance Force Required	Displacement at Grade	F.S.
North – South (Xg)	0.70	(1)	0.00 in	1.29
East – West (Yg)	0.70	11,627 kip	0.05 in	1.10
North – South (Xg)	0.60	(1)	0.00 in	1.11
East – West (Yg)	0.60	25,977 kip	0.16 in	1.10
North – South (Xg)	0.55	11,092 kip	0.08 in	1.10
East – West (Yg)	0.55	33,202 kip	0.49 in	1.10

Note (1) - At rest pressure

c. The justification of the statement made that "...different models give consistent results and use of results from different analyses is acceptable." Is given in RAI-TR85-SEB1-04, part (2), Revision 1, where it is shown that the reactions obtained using the 2D SASSI seismic response factor applied to the time history response result in conservative reactions when compared to the 3D SASSI analysis results. Therefore, the acceptability of the seismic response factors developed from the 2D SASSI models for use in the seismic stability evaluations is acceptable.

Westinghouse Response (Revision 2):

In the May 4 to 8, 2009 audit, the NRC reviewed the displacements based on the displacement curves given in Reference 1. The displacements given in the Revision 1 response to this RAI is based on the passive pressures defined using the Case 15 soil parameters as defined in RAI-TR85-SEB1-35. As part of the review of RAI-TR85-SEB1-35, the NRC requested Westinghouse to explain why, for sliding stability evaluation, a high passive pressure was used for resistance of the backfill adjacent to the Nuclear Island (NI) rather than a lower bound value based on soil parameters such as those defined by Case 21 (soil parameters defined in RAI-TR85-SEB1-35). Westinghouse stated that a lower bound was used in for the soil properties similar to Case 21. A comparison of geotechnical parameters and lateral earth pressures was given during the audit and is presented in Table RAI-TR85-SEB1-10-6. Presented in Tables RAI-TR85-SEB1-10-7 to RAI-TR85-SEB1-10-12 are the stability results for Case 15 and the lower bound soil case evaluated. It is noted that the displacements given for Case 15 are slightly different from those given in Revision 1 of this response because the active and dynamic surcharge pressures were slightly modified to be more representative (e.g. dynamic surcharge acting only on one side; active pressure acts below adjacent building foundations). The deflections obtained were discussed. It was stated by Westinghouse that the analysis methodology used was the conservative equivalent static. This will result in large deflections since the seismic loads are considered to be constant and do not reflect the short time duration as shown in Figures RAI-TR85-10-3 and RAI-TR85-10-4. It was requested that Westinghouse perform a more realistic non-linear analysis with sliding friction elements using a 2D ANSYS model.

Westinghouse modified the 2D ANSYS model that was used to study the basemat uplift. This model is described in Subsection 2.4.2 of TR85. This 2D non-linear model is for the East-West direction. There is no need to modify this model for the North-South direction since the NI deflections calculated to maintain a factor of safety of 1.1 is largest in the East-West direction. This model was modified introducing friction elements along the bottom of the basemat that is at the interface of the basemat and soil media.

Direct time integration analysis was performed that is also described in Subsection 2.4.2 of TR85. The three cases that have the lowest factor of safety related to sliding were evaluated. These three cases are HR, UBSM, and SM. The seismic input was increased by 10% so as to maintain the factor of safety against sliding of 1.1. No passive soil resistance is considered in the analyses. The resulting deflections at the base using a coefficient of friction of 0.55 are given in Table RAI-TR85-10-13 for the three cases. As noted above this model did consider vertical uplift in addition to sliding. As seen from this table the Nuclear Island experiences negligible sliding during the seismic event, and no passive soil resistance is necessary from the backfill. This is consistent with the observation made in Revision 1 of this response that:

> *"During the sliding stability calculation it was determined that the factor of safety for sliding drops below the limit of 1.1 for a very short time if passive pressure is not considered. Plots of the factor of safety (FS) versus time for the hard rock case and the North-South and East-West directions are given in Figures RAI-TR85-SEB1-10-3 and RAI-TR85-SEB1-10-4 using a coefficient of friction of 0.55. As seen from these figures the time at which the factor of safety drops below 1.1 is very short."*

Therefore, it can be concluded that the Nuclear Island is stable against sliding, and there is no quality requirement for the backfill material adjacent to the NI (side soil) to remain stable against sliding. Also, as noted in Revision 1 of this response, there is no passive pressure required to maintain stability against overturning.

The factors of safety related to wind and tornado loads have also been revised to remove passive pressure from the calculation of the factor of safety. All of the factors of safety are above the limits established for stability. Changes to the DCD and Technical Report are reflected below under Design Control Document (DCD) Revision and Technical Report (TR) Revision.

During the review of the response given for RAI-TR85-SEB1-04, it was requested that Westinghouse include in the DCD a description of the evaluations performed for the foundation stability which consists of a summary of the analyses presented in TR85, Rev. 1. This request is reflected in this RAI under the DCD revision section below.

Table RAI-TR85-SEB1-10-6 – Comparison of Geotechnical Parameters and Lateral Earth Pressures

Soil Properties/ Parameters	Case 15 Soil	Case 21 Soil	Lower Bound Soil Evaluated
Total Unit Weight (pcf)	150.0	95.0	122.4
Effective Unit Weight (pcf)	87.6	60.0	60.0
Friction Angle (degrees)	35.0	32.0	35.0
At-Rest Earth Pressure Coefficient (Ko)	0.426	0.470	0.426
Lateral Ko Earth Pressure at Elev. 60.5 (psf)	1,529	1,147	1,064
Full At-Rest Resistance Force (kips)	7,985 (E-W) 5,022 (N-S)	5,957 (E-W) 3,746 (N-S)	5,635 (E-W) 3,544 (N-S)
Passive Earth Pressure Coefficient (Kp)	3.690	3.255	3.690
Lateral Kp Earth Pressure at Elev. 60.5 (psf)	13,229	7,941	9,206
Full Passive Resistance Force (kips)	69,098 (E-W) 43,456 (N-S)	42,244 (E-W) 25,939 (N-S)	48,758 (E-W) 30,664 (N-S)

Table RAI-TR85-SEB1-10-7 – Sliding Factors of Safety with Hard Rock Case 15 Soil Passive Resistance

Direction	Coefficient of Friction	At-Rest Force Applied (kips)	Passive Force Applied (kips)	% of Full Passive Force	Displacement at Grade (inch)	Factor of Safety
North – South	0.70	5,017	N/A	N/A	0.000	1.22
East – West	0.70	7,977	N/A	N/A	0.000	1.24
North – South	0.60	N/A	9,166	21.1	0.065	1.10
East – West	0.60	N/A	10,076	14.6	0.030	1.10
North – South	0.55	N/A	17,116	39.4	0.188	1.10
East – West	0.55	N/A	17,676	25.6	0.082	1.10

Table RAI-TR85-SEB1-10-8 – Sliding Factors of Safety with Upper Bound Soft to Medium Case 15 Soil Passive Resistance

Direction	Coefficient of Friction	At-Rest Force Applied (kips)	Passive Force Applied (kips)	% of Full Passive Force	Displacement at Grade (inch)	Factor of Safety
North – South	0.70	5,017	N/A	N/A	0.000	1.22
East – West	0.70	N/A	10,390	15.0	0.035	1.10
North – South	0.60	N/A	8,910	20.5	0.063	1.10
East – West	0.60	N/A	25,250	36.6	0.145	1.10
North – South	0.55	N/A	16,750	38.5	0.132	1.10
East – West	0.55	N/A	32,610	47.2	0.453	1.10

Table RAI-TR85-SEB1-10-9 – Sliding Factors of Safety with Soft to Medium Case 15 Soil Passive Resistance

Direction	Coefficient of Friction	At-Rest Force Applied (kips)	Passive Force Applied (kips)	% of Full Passive Force	Displacement at Grade (inch)	Factor of Safety
North – South	0.70	5,017	N/A	N/A	0.000	1.27
East – West	0.70	N/A	10,900	15.8	0.042	1.10
North – South	0.60	N/A	5,350	12.3	0.008	1.10
East – West	0.60	N/A	25,300	36.6	0.146	1.10
North – South	0.55	N/A	12,980	29.9	0.099	1.10
East – West	0.55	N/A	32,400	46.9	0.439	1.10

Table RAI-TR85-SEB1-10-10 – Sliding Factors of Safety with Hard Rock Lower Bound Evaluated Soil Passive Resistance

Direction	Coefficient of Friction	At-Rest Force Applied (kips)	Passive Force Applied (kips)	% of Full Passive Force	Displacement at Grade (inch)	Factor of Safety
North – South	0.70	3,544	N/A	N/A	0.000	1.18
East – West	0.70	5,635	N/A	N/A	0.000	1.17
North – South	0.60	N/A	8,200	26.7	0.087	1.10
East – West	0.60	N/A	8,650	17.7	0.052	1.10
North – South	0.55	N/A	16,170	52.7	0.796	1.10
East – West	0.55	N/A	16,250	33.3	0.112	1.10

Table RAI-TR85-SEB1-10-11 – Sliding Factors of Safety with Upper Bound Soft to Medium Lower Bound Evaluated Soil Passive Resistance

Direction	Coefficient of Friction	At-Rest Force Applied (kips)	Passive Force Applied (kips)	% of Full Passive Force	Displacement at Grade (inch)	Factor of Safety
North – South	0.70	3,544	N/A	N/A	0.000	1.18
East – West	0.70	N/A	9,000	18.5	0.055	1.10
North – South	0.60	N/A	8,100	26.4	0.085	1.10
East – West	0.60	N/A	23,900	49.0	0.535	1.10
North – South	0.55	N/A	15,850	51.7	0.711	1.10
East – West	0.55	N/A	31,250	64.1	2.33	1.10

Table RAI-TR85-SEB1-10-12 – Sliding Factors of Safety with Soft to Medium Lower Bound Evaluated Soil Passive Resistance

Direction	Coefficient of Friction	At-Rest Force Applied (kips)	Passive Force Applied (kips)	% of Full Passive Force	Displacement at Grade (inch)	Factor of Safety
North – South	0.70	3,544	N/A	N/A	0.000	1.22
East – West	0.70	N/A	9,500	19.5	0.059	1.10
North – South	0.60	N/A	4,500	14.7	0.031	1.10
East – West	0.60	N/A	23,900	49.0	0.535	1.10
North – South	0.55	N/A	12,100	39.5	0.189	1.10
East – West	0.55	N/A	31,000	63.6	2.24	1.10

Table RAI-TR85-SEB1-10-13 – Seismic Deflections at Bottom of Nuclear Island Basemat due to Sliding (Coefficient of Friction equal to 0.55)

Case	Deflection @ 60.5' El Without buoyant force inches	Deflection @ 60.5' El With buoyant Force Inches
HR	0.02	0.06
UBSM	0.08	0.15
SM	0.12	0.19

Reference:

1. Hsai-Yang Fang, "Foundation Engineering Handbook," Second Edition, 1991, Van Nostrand Reinhold.

Westinghouse Response (Revision 3):

Clarified the safe shut down earthquake sliding equation in DCD section 3.8.5.5.3 removing F_P, F_H, and clarifying the definition for F_D. The M_P term in the equation in DCD section 3.8.5.5.4 is removed. A sentence is added to both sections to explain why those terms are not included. In the second paragraph of DCD section 3.8.5.5.4 the phrase "the static moment balance approach" is removed and replaced by "time history analysis."

The second paragraph of the revised DCD markup for Subsection 3.8.5.5.5 is modified to remove confusion related to passive pressure. The reference to Table 3.8.5-2 in Subsection 3.8.5.5.5 is removed.

Seismic deflections at the bottom of the Nuclear Island Basemat due to sliding (coefficient of friction equal to 0.55) are given for both cases with/without buoyant force in the DCD (buoyant force deflections are added to Table RAI-TR85-SEB1-10-13).

Footnote (3) in DCD Table 3.8.5-2 will be removed to clarify that the values in the table use zero passive pressure. .

Westinghouse Response (Revision 4):

a) The response to Revision 3 of this RAI was discussed with the NRC staff prior to submittal. The response to Revision 3 and the associated DCD revisions provided the changes requested by the staff during these conversations. The responses are supported by the analyses that support the stability evaluation. It is not practicable to include the information in the supporting analyses in the RAI response or DCD mark-up. Westinghouse will schedule a time for the staff to audit theses analyses.

b) APP-GW-GLR-044 (TR-85), APP-GW-GLR-005 (TR-09), and APP-GW-GLR-045 (TR-57) were prepared to support and inform the NRC of changes to structural design of the AP1000 in the design certification amendment. These changes included completion and design finalization of portions of the structural design. The design completion addressed COL information items by completion of design activities that were not complete at the time of design certification. Design finalization changes to the DCD included changes due to the expansion of the seismic response spectra in the Design Certification amendment and were incorporated in the critical section information in the DCD. These reports were formally transmitted by letter to the NRC and are included on the AP1000 Design Certification docket.

The design basis for the structural design of the AP1000 is documented in calculations and design reports that are available for review and audit. The information included in the DCD should be sufficient to satisfy the guidance of the Standard Review Plan (SRP). The SRP includes guidance that design criteria and design and analysis methods be included in the DCD. The SRP guidance does not include the inclusion of design details in the DCD.

Documents in the DCD currently identified as Tier 2* are codes and standards or are topical reports that provide methods and criteria. Defining the subject documents as Tier 2* would include excessive detail as Tier 2* information. The subject reports include the results of calculations. Because reanalysis may result in small changes in the results these results should not be identified as Tier 2*. Incorporating the subject reports as Tier 2* would require NRC approval for changes at a level of detail that is not appropriate.

If there is specific information in the subject documents that the staff considers to be in the nature of criteria and methods and should be included in the DCD that information should be identified for inclusion in the DCD. Given the justification in the preceding paragraphs Westinghouse will not be incorporating APP-GW-GLR-044 (TR-85), APP-GW-GLR-005 (TR-09), and APP-GW-GLR-045 (TR-57) as Tier 2* information.

Westinghouse Response (Revision 5):

Westinghouse will review the information in RAI responses and the structural technical reports for the key analysis and design information that should be included in the AP1000 Design Control Document (DCD). Consistent with the Standard Review Plan, this information is expected to include design criteria, design methods, modeling requirements, analysis methods, and limits. This information will be included in the DCD so that the staff will not rely on information in the technical reports to support the SER conclusions. Commitments to the NRC included in the technical reports will be included. Modifications to standard industry methods and computer codes will be identified.

The review will be completed after the responses for the remaining unresolved RAIs are finalized. Westinghouse will provide DCD mark-ups for the complete Sections 3.7 and 3.8 identifying the information to be added or revised. Appendices 3G and 3I will also be reviewed for revisions.

Subsequent to agreement on the content of the DCD, portions of the DCD that should be identified as Tier 2* will be identified.

The revised nuclear island seismic sliding displacements due to modifications of the non-linear seismic sliding model are reflected in this RAI. The revised nuclear island seismic sliding displacement values are reflected in Table RAI-TR85-SEB1-10-13, DCD subsection 3.8.5.5.5, footnote 2 of DCD Table 3.8.5-2, TR85 section 2.9, and footnote (2) of table 2.9-1.

Westinghouse Response (Revision 6):

(1) DCD Appendix 3H will also be reviewed for revision needed to incorporate information from the technical reports.

(2) The second sentence in Paragraph 3 of 3.8.5.5.3 will be revised to remove the reference to thin soil layer and will read as follows:

> The governing friction value in the soil below the mudmat has an angle of internal friction of 35° giving a static coefficient of friction of 0.55.

(3) The reference to the backfill requirement in the last sentence of paragraph 2 of DCD Subsection 3.8.5.5.5 will be deleted as show below.

Design Control Document (DCD) Revision:

Modify the first sentence in the last paragraph of DCD subsection 3.4.1.1.1.1, Revision 17, to read as follows:

> The waterproof function of the membrane is not safety-related; however, the membrane between the mudmats must transfer horizontal shear forces due to seismic (SSE) loading. This function is seismic Category I. The specific ~~static~~ coefficient of friction between horizontal membrane and concrete is ≥~~0.7~~ 0.55.

Modify the following DCD Revision 17 subsections related to seismic stability.

3.8.5.5.3 Sliding

The factor of safety against sliding of the nuclear island (NI) during a tornado or a design wind is shown in Table 3.8.5-2 and is calculated as follows:

$$F.S. = \frac{F_S}{F_H}$$

where:

F.S. = factor of safety against sliding from tornado or design wind

F_S = shearing or sliding resistance at bottom of basemat

~~F_P = maximum soil passive pressure resistance, neglecting surcharge effect~~

F_H = maximum lateral force due to active soil pressure, including surcharge, and tornado or design wind load

The factor of safety against sliding of the nuclear island during a safe shutdown earthquake is shown in Table 3.8.5-2 and is calculated as follows:

$$F.S. = \frac{F_s}{F_D}$$

where:

F.S. = factor of safety against sliding from a safe shutdown earthquake
F_S = shearing or sliding resistance at bottom of basemat
F_P = maximum soil passive pressure resistance, neglecting surcharge effect
F_D = maximum dynamic lateral force, including dynamic active earth pressures seismic force from safe shutdown earthquake
F_H = maximum lateral force due to all loads except seismic loads

The sliding resistance is based on the friction force developed between the basemat and the foundation. The governing friction value in the interface zone is a thin soil layer below the mudmat with hasan angle of internal friction of 35° giving a static coefficient of friction of 0.700.55. The effect of buoyancy due to the water table is included in calculating the sliding resistance. F_P Passive soil pressure resistance is not included in the equations above because passive pressure is not considered for sliding stability. Since there is no passive pressure considered, active and overburden soil pressures are also not considered.

3.8.5.5.4 Overturning

The factor of safety against overturning of the nuclear island during a tornado or a design wind is shown in Table 3.8.5-2 and is calculated as follows:

$$F.S. = \frac{M_R}{M_O}$$

where:

F.S. = factor of safety against overturning from tornado or design wind
M_R = resisting moment
M_O = overturning moment of tornado or design wind

The factor of safety against overturning of the nuclear island during a safe shutdown earthquake is shown in Table 3.8.5-2 and is evaluated using the static moment balance approachtime history analysis assuming overturning about the edge of the nuclear island at the bottom of the basemat. The factor of safety is defined as follows:

$$F.S. = (M_R + M_P)/(M_O + M_{AO})$$

where:

F.S. = factor of safety against overturning from a safe shutdown earthquake
M_R = nuclear island's resisting moment against overturning
M_O = maximum safe shutdown earthquake induced overturning moment acting on the nuclear island, applied as a static moment
M_P = Resistance moment associated with passive pressure
M_{AO} = Moment due to lateral forces caused by active and overburden pressures

The resisting moment is equal to the nuclear island dead weight, minus buoyant force from ground water table, multiplied by the distance from the edge of the nuclear island to its center of gravity. The overturning moment is the maximum moment about the same edge from the time history analyses of the nuclear island lumped mass stickNI20 model described in subsection 3.7.2 and 3G.2. M_PResistance moment due to passive pressure is not included in the equation above because passive pressure is not considered for overturning stability.

3.8.5.5.5 Seismic Stability Analysis

The factors of safety for sliding and overturning for the SSE are calculated for each soil case for the base reactions in terms of shear and bending moments about column lines 1, 11, I and the west side of the shield building at each time step of the seismic time history. The 2D SASSI reactions (Fx, Fy, and Fz) are used to obtain seismic response factors between the hard rock case to the upper-bound-soft-to-medium (UBSM) soil case, and the soft-to-medium (SM) soil case. These factors were used to adjust the hard rock time history to reflect the seismic response for the other two potential governing soil cases UBSM and SM. The firm rock, soft rock, and soft soil cases have higher factors of safety against sliding and therefore not considered.

A non-linear analysis with sliding friction elements using a 2D ANSYS model was performed. The 2D ANSYS model that was used to study the basemat uplift (see Subsection 3.8.5.5.6 and Appendix 3G). This 2D non-linear model is for the East-West direction. There is no need to consider the North-South direction since the NI deflections calculated to maintain a factor of safety of 1.1 is largest in the East-West direction. This model was modified introducing friction elements along the bottom of the basemat and soil media interface. Direct time integration analysis was performed with vertical uplift and sliding allowed. The three cases that have the lowest factor of safety related to sliding were evaluated. These three cases are HR, UBSM, and SM. The seismic input was increased by 10% to maintain the factor of safety against sliding of 1.1. No passive soil resistance is considered. The resulting maximum displacement at the base of the NI basemat (EL 60.5') using a coefficient of friction of 0.55 is 0.12" without buoyant force consideration, and 0.19" with buoyant force considered. This is negligible sliding during the seismic event, and no passive soil resistance is necessary from the backfill (side soil). Therefore, it can be concluded that the Nuclear Island is stable against sliding. , and there is no quality requirement for the backfill material adjacent to the NI (side soil) to maintain stability against sliding.

The minimum seismic stability factors of safety values are reported in Table 3.8.5-2.

3.8.5.5.~~5~~6 Effect of Nuclear Island Basemat Uplift on Seismic Response

The effects of basemat uplift were evaluated using an east-west lumped-mass stick model of the nuclear island structures supported on a rigid basemat with nonlinear springs. Floor response spectra from safe shutdown earthquake time history analyses, which included basemat uplift, were compared to those from analyses that did not include uplift. The comparisons showed that the effect of basemat uplift on the floor response spectra is not significant.

3.8.7 References

56. Hsai-Yang Fang, "Foundation Engineering Handbook," Second Edition, 1991, Van Nostrand Reinhold.

Table 3.8.5-2

FACTORS OF SAFETY FOR FLOTATION, OVERTURNING AND SLIDING OF NUCLEAR ISLAND STRUCTURES

Environmental Effect	Factor of Safety[1]
Flotation	
High Ground Water Table	3.7
Design Basis Flood	3.5
Sliding	
Design Wind, North-South	~~23.2~~ 14.0
Design Wind, East-West	~~17.4~~ 10.1
Design Basis Tornado, North-South	~~12.8~~ 7.7
Design Basis Tornado, East-West	~~10.6~~5.9
Safe Shutdown Earthquake, North-South	~~1.28~~1.1[2]
Safe Shutdown Earthquake, East-West	~~1.33~~1.1[2]
Overturning	
Design Wind, North-South	51.5
Design Wind, East-West	27.9
Design Basis Tornado, North-South	17.7
Design Basis Tornado, East-West	9.6
Safe Shutdown Earthquake, North-South	~~1.35~~1.77
Safe Shutdown Earthquake, East-West	~~1.12~~1.17 ~~[3]~~

Note:

1. Factor of safety is calculated for the envelope of the soil and rock sites described in subsection 3.7.1.4.
2. From non-linear sliding analysis using friction elements the horizontal movement is negligible (0.12"without buoyant force consideration, and 0.19" with buoyant force considered). ~~Factor of safety is shown for soils below and adjacent to nuclear island having angle of internal friction of 35 degrees.~~
3. ~~ASCE/SEI 43-05, Reference 42, recognizes that there is considerable margin beyond that given by the moment balance formula and permits a nonlinear rocking analysis. The nonlinear (liftoff allowed) time history analysis described in Appendix 3G.10 showed that the nuclear island basemat uplift effect is insignificant. Further, these analyses were performed for free field seismic ZPA input as high as 0.5g without significant uplift. Therefore the factor of safety against overturning is greater than 1.67 (0.5g/0.3g).~~

APPENDIX 3G NUCLEAR ISLAND SEISMIC ANALYSES

Modify the second paragraph in Section 3.G.1 changing Reference number.

Analyses were performed in accordance with the criteria and methods described in Section 3.7. Section 3G.2 describes the development of the finite element models. Section 3G.3 describes the soil structure interaction analyses of a range of site parameters and the selection of the parameters used in the design analyses. Section 3G.4 describes the fixed base and soil structure interaction dynamic analyses and provides typical results from these dynamic analyses. In Reference ~~3~~6 are provided a summary of dynamic and seismic analysis results (i.e., modal model properties, accelerations, displacements response spectra) and the nuclear island liftoff analyses. The seismic analyses of the nuclear island are summarized in a seismic analysis summary report. Deviations from the design due to as-procured or as-built conditions are acceptable based on an evaluation consistent with the methods and procedures of Sections 3.7 and 3.8 provided the following acceptance criteria are met:

3G.5 References

6. APP-GW-GLR-044, "Nuclear Island Basemat and Foundation," Revision 1, Westinghouse Electric Company LLC

PRA Revision:

None

Technical Report (TR) Revision:

~~None~~

The following modifications are Post Revision 1.

Modify the last paragraph of Section 2.4.1, 2D SASSI Analyses to the following:

Table 2.4-2 shows the reactions at the underside of the basemat for each soil case. These are conservative estimates using the results of the 2D SASSI horizontal analyses also used for the member forces in Table 2.4-1. Horizontal loads on the portion below grade are added absolutely to the sum of the member forces above grade. The 2D SASSI reactions (Fx, Fy, and Fz) are used to obtain seismic response factors between the hard rock case to the upper-bound-soft-to-medium (UBSM) soil case, and the soft-to-medium (SM) soil case. These factors were used to adjust the hard rock time history to reflect the seismic response for the other two potential governing soil cases UBSM and SM.

Modify Section 2.9 as follows:

2.9 Nuclear island stability

The factors of safety associated with stability of the nuclear island (NI) are shown in Table 2.9-1 for the following cases:

- Flotation Evaluation for ground water effect and maximum flood effect
- The Nuclear Island to resist overturning during a Safe Shutdown Earthquake (SSE)
- The Nuclear Island to resist sliding during the SSE
- The Nuclear Island to resist overturning during a tornado/wind/hurricane condition
- The Nuclear Island to resist sliding during a tornado/wind/hurricane condition.

~~The factors of safety for sliding and overturning for the SSE are calculated for each soil case for the base reactions in terms of shear and bending moments about column lines 1, 11, I and the west side of the shield building at each time step of the seismic time history. The seismic time history analysis used the ANSYS computer code and the NI20 model.~~ The minimum stability factors of safety values are reported in Table 2.9-1. The method of analysis is as described in subsection 3.8.5.5 of the DCD and the coefficient of friction of 0.55 is used. The governing friction value at the interface zone is a thin soil layer (soil on soil) under the mud mat assumed to have a friction angle of 35 degrees. The Combined License applicant will provide the site specific angle of internal friction for the soil below the foundation. ~~For seismic overturning no passive pressure was considered. For sliding partial passive pressure is considered (less than 35% NS and 48% EW). The relationship between passive pressure and displacement at grade is shown in Figures 2.9-1 and 2.9-2. These curves are based on the methodology given in Reference 10.~~

The factors of safety for sliding and overturning for the SSE are calculated for each soil case for the base reactions in terms of shear and bending moments about column lines 1, 11, I and the west side of the shield building at each time step of the seismic time history. The 2D SASSI reactions (Fx, Fy, and Fz) are used to obtain seismic response factors between the hard rock case to the upper-bound-soft-to-medium (UBSM) soil case, and the soft-to-medium (SM) soil case. These factors were used to adjust the hard rock time history to reflect the seismic response for the other two potential governing soil cases UBSM and SM. The firm rock, soft rock, and soft soil cases have higher factors of safety against sliding and therefore not considered.

The seismic time history analysis used the ANSYS computer code and the NI20 model. The minimum stability factors of safety values are reported in Table 2.9-1. For seismic overturning no passive pressure was considered. For sliding partial passive pressure is considered. Two soil cases are considered for sliding, the soil parameters used for design (friction angle of 35°, and submerged weight of 87.6 pcf), and a lower bound soil density (friction angle of 35°, and submerged weight of 60 pcf). For the design case the amount of passive pressure required to meet the 1.1 factor of safety is 40% for the North-South seismic event, and 47% of the East-West excitation of full passive pressure. For the lower bound case the amount of passive pressure required to meet the 1.1 factor of safety is less than 53% for the North-South seismic event, and 64% of the East-West excitation of full passive pressure. The relationship between passive pressure and displacement at grade is obtained based on the methodology given in Reference 10. The relationship between passive pressure and displacement at grade is shown in Figures 2.9-1 and 2.9-2. The maximum Nuclear Island displacement of the Nuclear Island at grade to develop the required passive resistance is 0.5" for the design case, and 2.3" for the lower bound case. These deflections are based on conservative equivalent static analysis. This will result in large deflections since the seismic loads are considered to be constant and do not reflect the short time duration that they exist during the seismic event. A more realistic non-linear analysis with sliding friction elements using a 2D ANSYS model was performed. The 2D ANSYS model that was used to study the basemat uplift (see Subsection 2.4.2). This 2D non-linear model is for the East-West direction. There is no need to consider the North-South direction since the NI deflections calculated to maintain a factor of safety of 1.1 is largest in the East West direction. This model was modified introducing friction elements along the bottom of the basemat and soil media interface. Direct time integration analysis was performed with vertical uplift and sliding allowed. The three cases that have the lowest factor of safety related to sliding were evaluated. These three cases are HR, UBSM, and SM. The seismic input was increased by 10% to maintain the factor of safety against sliding of 1.1. No passive soil resistance is considered. The resulting maximum displacement at the base of the NI basemat (EL 60.5') using a coefficient of friction of 0.55 is 0.12" without buoyant force consideration, and 0.19" with buoyant force considered. This is negligible sliding during the seismic event, and no passive soil resistance is necessary from the backfill (side soil). Therefore, it can be concluded that the Nuclear Island is stable against sliding, and there is no quality requirement for the backfill material adjacent to the NI (side soil) to maintain stability against sliding.

Table 2.9-1 – Factors of Safety Related to Stability of AP1000 NI

Load Combination	Sliding		Overturning		Flotation	
	Factor of Safety	Limit	Factor of Safety	Limit	Factor of Safety	Limit
D + H + B + W	Design Wind					
North-South	~~23.2~~14.0	1.5	51.5	1.5	–	–
East –West	~~17.4~~10.1	1.5	27.9	1.5	–	–
D + H + B + W_t	Tornado Condition					
North-South	~~12.8~~7.7	1.1	17.7	1.1	–	–
East –West	~~10.6~~5.9	1.1	9.6	1.1	–	–
D + H + B + W_h	Hurricane Condition					
North-South	~~18.1~~10.3	1.1	31.0	1.1	–	–
East –West	~~14.2~~8.1	1.1	16.7	1.1	–	–
D + H + B + E_S	SSE Event					
North-South	1.1(2)	1.1	–	–	–	–
East-West	1.1(2)	1.1	–	–	–	–
Line 1	–	–	1.77(1)	1.1	–	–
Line 11	–	–	1.~~92~~93(1)	1.1	–	–
Line I	–	–	1.17(1)	1.1	–	–
West Side Shield Bldg	–	–	1.44(1)	1.1	–	–
	Flotation					
D + F	–	–	–	–	3.51	1.1
D + B	–	–	–	–	3.70	1.5

Notes:

(1) No passive pressure is considered.

(2) No passive pressure is considered. From non-linear sliding analysis using friction elements the horizontal movement is negligible (0.12"without buoyant force consideration, and 0.19" with buoyant force considered). ~~Factor of safety for sliding considers that the soils below and adjacent to the nuclear island have an angle of internal friction of 35 degrees. Also, the coefficient of friction for soils below the nuclear island is equal to 0.55. The maximum deflection of the nuclear island needed to develop the required passive pressures are less than 0.15 inch for hard rock, less than or equal to 0.5 inch for upper bound soft to medium (UBSM) and soft to medium (SM) soil conditions. The other soil conditions have smaller deflection requirements than the UBSM and SM cases.~~

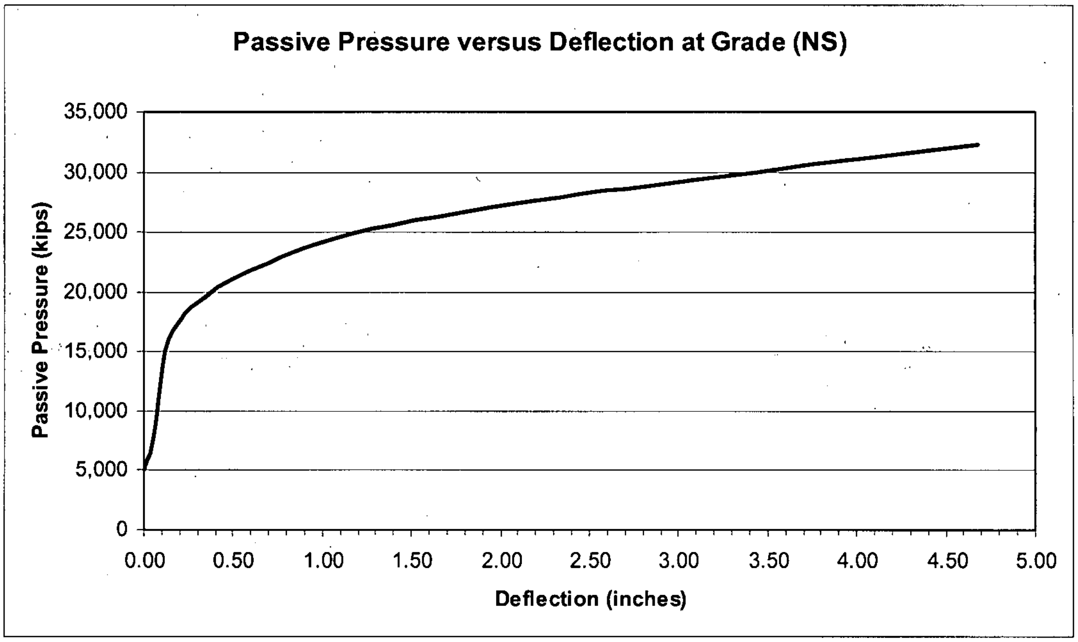


Figure 2.9-1 – Passive Pressure versus Deflection at Grade (North-South Excitation)

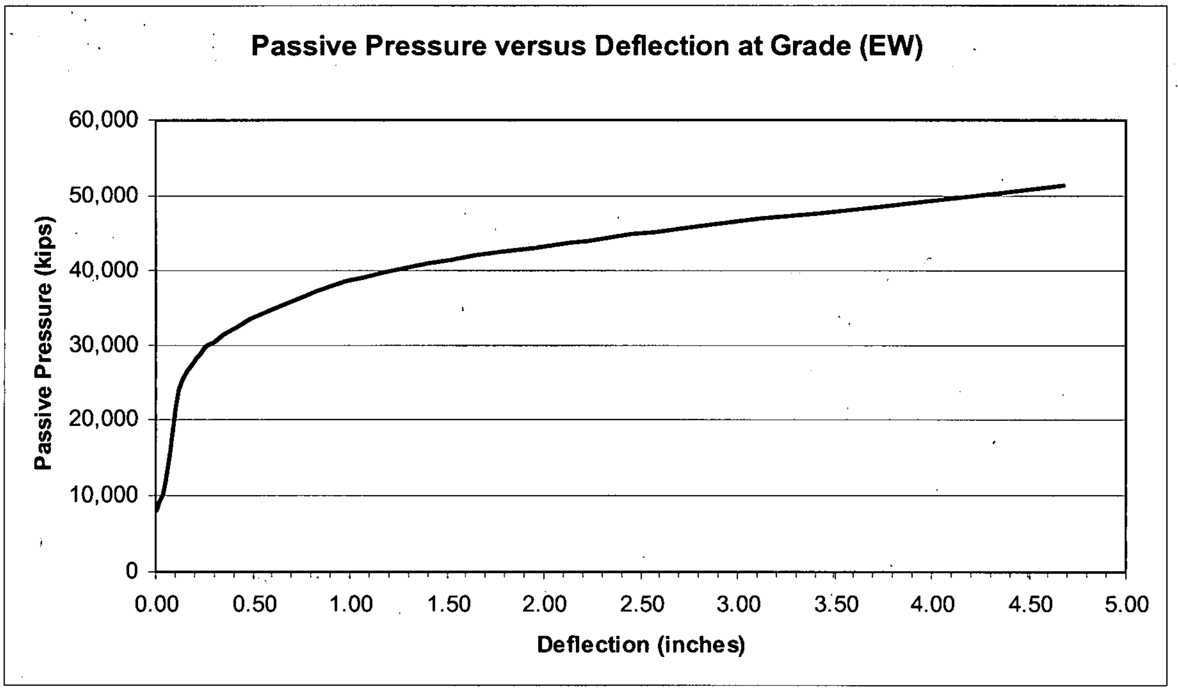


Figure 2.9-2 – Passive Pressure versus Deflection at Grade (East-West Excitation)

4. REFERENCES

10. HSAI-Yang Fang, "Foundation Engineering Handbook," Second Edition, 1991, Van Nostrand Reinhold.