RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

12/27/2013

US-APWR Design Certification

Mitsubishi Heavy Industries

Docket No. 52-021

RAI NO.:	**NO. 1051-7176 REVISION 4**
SRP SECTION:	**03.08.03 – Concrete and Steel Internal Structures of Steel or Concrete Containments**
APPLICATION SECTION:	**3.8.3**
DATE OF RAI ISSUE:	**08/30/2013**

QUESTION NO. 03.08.03-115:

The staff reviewed MUAP-11013-P Revision 2, dated February 28, 2013 (referred to as the report in the following discussion). The staff found that the updated report incorporated some SC design changes, information resulting from the RAI questions issued before this updated report, and added technical information in new Appendices A and B, which have not been reviewed before. Based on the review of the revised and new information, some additional items need to be addressed, as given below, to ensure that the design and validation methodologies of SC structures, described in MUAP-11013 (R2), are acceptable.

1. As indicated in the report, in Task 3, detailed nonlinear inelastic finite element (NIFE) models of SC walls were developed and benchmarked using test results; and in Task 4, NIFE models of SC walls were used to confirm the stiffness and dynamic characteristics of the linear elastic finite element (LEFE) models utilized in the seismic SSI analysis (Task 1-A) and in the response spectrum analysis for the CIS (Task 1-B), as well as to demonstrate the performance for beyond SSE loads. From the information provided in the report, it is not clear which specific NIFE model(s) were used for the development of the LEFE model utilized for the seismic analysis of the APWR CIS and how the benchmarking was performed for validating these NIFE and LEFE models. Therefore, the applicant is requested to describe (1) which NIFE model was used for determining the adequacy of the LEFE model utilized in the seismic analysis of the APWR CIS, (2) what specific benchmarking is relied upon for this NIFE model and discuss the adequacy of the comparison of the analysis versus test results, and (3) how the LEFE model was benchmarked/calibrated using this specific NIFE model. The above information should be included in the applicable portions of MUAP-11013-P.

   For the benchmarking analysis of the 1/10th scale model test, Section A-7.1 indicates that the full 3-D FE model was first analyzed for monotonic loading condition to refine and verify the model. The staff requests that the applicant describe how the refinement was done.

2. No detailed descriptions for Task 4 were provided in the report. Since the purpose of this report is to provide the overall methodology for the analysis and design of the CIS, the

staff requests that the applicant provide a description of the evaluations performed for Task 4 comparable to those provided for Task 3 in Appendix A.

3. Regarding the stud to concrete shear force-slip relationship model for connector elements, (1) on page A-4-1, the report states that “the behavior of some of these specimens was influenced significantly by the shear studs, and the accuracy of shear stud force vs. slip relationship had a significant contribution;” and (2) on pages A-7-5 and A-8-10, the report indicates that the analysis comparison results show that there is functionally no difference between FE models with the shear force-slip relationship model and with the fully tied interaction. Therefore, the staff requests that the applicant explain these differing statements, and if the shear force-slip relationship model is important, describe how it is used in Task 4 for confirming the stiffness and dynamic characteristics of the LEFE models of the CIS structure utilized in Tasks 1-A and 1-B.

4. In the push-out tests summarized in Section B-7 of the report, tie bars were included in the test specimens. This is consistent with the actual configuration of the SC walls. The staff requests that the applicant describe how the stress in tie-bars due to interfacial shear is also included in the design stress analysis of the tie bars; or provide the technical justification for not doing so.

5. The staff requests the applicant to correct the following inconsistencies, missing information, and typos in MUAP-11013 (R2):

   Page A-3-1, the last paragraph, the reference to Section 2.6 and Section 2.7 probably should refer to Section 2.5 and Section 2.6, respectively.

   Page A-5-5, Figures A-5-4 and A-5-5, refer to results obtained from “MBM;” provide a definition of this term and explain how this was done.

   Page A-7-2, regarding the modeling of the RC loading slab to CIS connection in the full 3-D FE model for the 1/10th scale test, the third paragraph states that the bottom RC loading slab is connected to the concrete infill, but the last paragraph states that the top and bottom RC loading slabs are only connected to the steel specimen in the FE model. Clarify or correct this inconsistency.

   Page A-7-6, the last paragraph, reference for results in Section 8.2 should be corrected to identify Section 7.2.

   Page B-1-2, TS 3.0, Item (ii) indicates that Specimen 3.2 had tie bars perpendicular to specimen length; however, Figure B-10.6-1 on page B-10-32 shows that the tie bars are parallel to the specimen length. This inconsistency should be corrected.

   Page B-1-5, Table B-1-1, for Specimen SP2.2.1, third column, 2nd row, it indicates that Specimen SP2.2.1 had tie bars parallel to specimen length; however, Figure B-9.2-3 on page B-9-5 shows that the tie bars are perpendicular to the specimen length. This inconsistency should be corrected.

   Page B-7-2, the second paragraph from the bottom of the page, it states that **[** **]**. However, in the 2nd paragraph on this page, it states **[** **]**. This inconsistency should be corrected.

Page B-9-11, the third paragraph, a statement is made that “The deflection measurements taken at the mid-span region were used to generate an average moment-curvature response using the second order central difference method as shown in.” The missing information at the end of this sentence should be provided.

ANSWER:

1. (1) A nonlinear inelastic finite element (NIFE) model consisting of layered composite shell (LCS) elements was used to confirm the adequacy of the linear elastic finite element (LEFE) model utilized in the seismic analysis of the US-APWR containment internal structure (CIS). As presented in detail in Technical Report MUAP-11013, Rev. 2, Appendix A, the LCS NIFE model models the steel concrete (SC) walls using shell elements with three plies; two for the steel faceplates and one for the concrete infill. The steel plies consist of three layers and the concrete ply consists of nine layers. The steel plies are associated with the elasto-plastic steel material model with multiaxial plasticity and strain hardening. The concrete ply is associated with the inelastic concrete material model with multiaxial damaged plasticity in compression, and cracked oriented damaged elasticity with smeared cracking in tension. The three plies of the SC composite section are fully bonded to each other, i.e., there is no slip between the steel and concrete plies. Additional details of the LCS NIFE model are provided in Technical Report MUAP-11013, Rev. 2, Appendix A.

   (2) The NIFE model described above is benchmarked against the 1/10th scale physical test described in Technical Report MUAP-11005, Rev. 1, Appendix E, Reference E-4. The specific benchmarking that is relied upon for this NIFE model considers the following:

   - The web plates in the SC walls in the bottom portion of the 1/10th scale CIS test structure could not be modeled explicitly by the LCS elements. These web plates were smeared into the plies modeling the steel faceplates. This specific benchmarking assumption has no influence on the modeling of the US-APWR CIS because it does not include any continuous web plates in the SC walls. The SC walls of the US-APWR CIS have discontinuous discrete tie bars, which did not need to be smeared into the steel faceplate plies.

   - The SC walls have full strength connections anchoring them to the concrete basemat. This specific benchmarking assumption has no influence on the modeling of the US-APWR CIS because all the SC walls have been designed with full strength connections, and their full strength behavior has been confirmed experimentally as described in Technical Report MUAP-11013, Rev. 2, Appendix B.

   - The nonlinear inelastic material model for the steel and concrete materials of the LCS elements. These include the multiaxial plasticity models, smeared cracking models, uniaxial inelastic stress-strain behavior, hysteretic behavior, and cyclic damage rules for the steel and concrete material models. The details of the steel and concrete material models are provided in Technical Report MUAP-11013, Rev. 2, Appendix A. These benchmarked material models were used with identical parameters to model the steel and concrete materials of the SC walls of the US-APWR CIS.

The comparisons of the analysis results using the LCS NIFE model are presented in detail in Technical Report MUAP-11013, Rev. 2, Appendix C. The monotonic analysis of the NIFE model was compared with the envelope of the cyclic responses from the 1/10th scale test, and the cyclic analysis of the NIFE model was compared with the hysteretic responses from the 1/10th scale test. As shown, the NIFE model predicts the experimental lateral load-displacement responses reasonably and accurately.

(3) The linear elastic FE model was not calibrated to the nonlinear inelastic FE model in any way. The LEFE model modeled the SC walls of the US-APWR CIS using shell elements with only one linear elastic material model. The parameters of the LEFE model shell elements including the section thickness, material elastic modulus, and poisson ratio were calibrated to the appropriate flexural stiffness and in-plane shear stiffness of SC walls using procedures described in detail in Technical Report MUAP-11018, Rev. 1. The LEFE model was benchmarked to the NIFE model by comparing the overall structure stiffness of both models in the elastic range of the response corresponding to safe-shutdown earthquake (SSE) level loads. The comparison of the elastic range stiffness of the LEFE and NIFE models of the US-APWR CIS are provided in Technical Report MUAP-11013, Rev. 2, Appendix C.

Technical Report MUAP-11013 will be revised as indicated on the attached markups to include a summary in Section A-7.1 of the analysis performed to refine and verify the 1/10th scale model.

2. The evaluations performed for Task 4 are now included in Technical Report MUAP-11013, Rev. 2, Appendix C with the same level of details provided for Task 3 in Appendix A.

3. The discussion on Technical Report MUAP-11013, Rev. 2, page A-4-1 focuses on the out-of-plane behavior of SC specimens that were included in the comprehensive database of SC tests reported in Appendix A. These tests are not specific or representative to the US-APWR SC design. The statement in the report regarding the accuracy of the shear stud force-vs. slip relationship, and its contribution to the behavior of the specimen does not apply to the US-APWR SC design, which uses significant section detailing requirements outlined in Technical Report MUAP-11019, Rev. 1, to prevent SC specific failure modes including excessive slip between the steel faceplates and concrete infill.

   On Pages A-7-5 and A-8-10, the report discusses the behavior of the 1/10th scale test performed on the CIS in Japan. This test structure had good SC section detailing as evident from: (i) the lack of SC specific failure modes in the test, and (ii) the use of extensive shear studs and web plates in the SC walls. Additional geometric and material details of the 1/10th scale structure are provided in Technical Report MUAP-11005, Rev. 1, Appendix E, Reference E-4. For this 1/10th scale structure, the analysis results showed that there was no functional difference between the behavior predicted by FE models with shear force-slip relationship models for the studs and FE models with fully tied interaction.

   The SC walls in the US-APWR CIS use even better section detailing than the SC walls in the 1/10th scale structure. These included closely spaced shear studs, lower steel faceplate slenderness ratio (spacing-to-plate thickness, s/t_p ratio), and closely spaced large area tie bars designed in accordance with Technical Report MUAP-11019, Rev. 1, to prevent SC specific failure modes. Therefore, modeling the shear force-slip relationship behavior of the shear studs is not necessary, and fully tied models are used for the NIFE analysis.

4. As discussed in Technical Report MUAP-11019, Rev. 1, Section 2.5, the interfacial shear strength of the US-APWR SC walls is conservatively calculated using the strength of the shear studs only. Nevertheless, it is recognized that the tie bars also contribute directly to the total interfacial shear strength, as demonstrated in the Series 1.0 pushout tests. Technical Report MUAP-11013, Rev. 2, Appendix B-7 discusses how the tie bar contribution was shown to be a function of the tie bar orientation to the applied loading. Equation B-7.4-2 was postulated for the combined capacity of the studs and tie bars when the tie bars are oriented in the longitudinal position (or parallel to the applied load), and Equation B-7.5-2 was postulated for the combined capacity when the tie bars are oriented transverse to the applied load. Although these equations are not used for design, they were demonstrated to provide reasonable representations of the interfacial shear strengths developed by the Series 1.1 and 1.2 specimens, respectively. The tests demonstrated that the largest combined interfacial shear capacity was achieved for the transverse tie bar orientation (Series 1.2), with the increase vs. the longitudinal orientation attributed to the increased capacity of the tie bar when its gusset plate area is engaged in interfacial shear resistance.

   Although the Series 1.0 tests illustrated the significant involvement of the tie bars in interfacial shear resistance, the Series 2.1 tests confirmed that this involvement does not significantly reduce the out-of-plane shear strength of the tie bars vs. their capacities calculated in accordance with the American Concrete Institute (ACI) 349-06 code, regardless of tie bar orientation (longitudinal or transverse). Test Series 2.1.1 and 2.1.2 involved specimens with a/d ratios of 2.0 and faceplates that were artificially thickened vs. the actual US-APWR design, in order to force out-of-plane shear failure to be the governing failure mode of the tests. Specimen 2.1.1 used the transverse tie bar orientation, and Specimen 2.1.2 used the longitudinal tie bar orientation. As shown in Technical Report MUAP-11013, Rev. 2, Figure B-8.6-4 (repeated below), the total out-of-plane shear strength including the tie bar and concrete contributions was substantially greater than the nominal capacity ($V_n = V_c + V_s$) calculated in accordance with Technical Report MUAP-11019, Rev. 1, Equations 6.2-1 and 6.2-4, despite the fact that the tie bars were also loaded in interfacial shear in these same tests. The figure illustrates that the transverse orientation of the tie bars in Specimen 2.1.1 actually resulted in a larger ultimate out-of-plane shear capacity vs. Specimen 2.1.2, despite the larger interfacial shear forces carried by the tie bars in this orientation.

Figure 4-1 (MUAP-11013 Rev. 2 Fig. B-8.6-4): Load vs. Deflection Comparison of SP2.1.1 and SP2.1.2.

Specimens 2.1.3 and 2.1.4 were also shear-controlled specimens as a result of artificially thickened faceplates, but they evaluated a more typical shear span (a/d) ratio of 3.0. As shown in Technical Report MUAP-11013, Rev. 2, Figure B-8.8-4 (repeated below), these tests demonstrated consistent out-of-plane shear performance with Series 2.1.1 and 2.1.2. The tests reaffirmed the conservatism of the Technical Report MUAP-11019, Rev. 1 out-of-plane shear strength equations vs. the actual strength of the US-APWR SC wall designs, and the transverse tie bar orientation was again shown to result in slightly larger out-of-plane shear strength. Additionally, these tests more clearly illustrated the relative contributions of the concrete and the tie bars to out-of-plane shear strength. After significant vertical deflection (approximately 1.7 in. for SP2.1.3) and development of wide shear cracks, the concrete contribution to out-of-plane shear strength dropped precipitously due to loss of aggregate interlock. After this reduction in the concrete shear strength contribution, it was shown that the specimens were able to sustain out-of-plane loading at a reduced level that was still slightly larger than the combined nominal out-of-plane shear strengths of the concrete and steel calculated with the Technical Report MUAP-11019, Rev. 1 equations.

Figure 4-2 (Technical Report MUAP-11013, Rev. 2, Figure B-8.8-4): Load vs. Deflection Comparison of SP2.1.3 and SP2.1.4.

In summary, the Series 2.1 tests confirmed that the ACI 349-06 out-of-plane shear strength equations as modified by Technical Report MUAP-11019, Rev. 1 are conservative vs. the actual strength of the US-APWR SC wall designs. The tests also confirmed that exposure of the tie bars to the interfacial shear demands that are attendant with out-of-plane shear loading does not reduce the capacity of the tie bars below their calculated strengths. Therefore, the design calculations for the actual US-APWR SC wall tie bars need not include an evaluation of the interaction of stresses due to combined interfacial and out-of-plane shear demands.

5. Technical Report MUAP-11013, Rev. 2 will be revised as follows:

   a) References to Sections 2.6 and 2.7 of the third paragraph of page A-3-1 will be revised to refer to Sections 2.5 and 2.6, respectively.

   b) "MBM" refers to the mechanics based model described in Technical Report MUAP-11018, Appendix A. A sentence will be added to the first paragraph of page A-5-4 to explain this.

   c) The third paragraph of page A-7-2 refers to the concrete basemat, not the bottom reinforced concrete loading slab. Therefore there is no inconsistency with the last paragraph of page A-7-2. The second sentence of the third paragraph of page A-7-2 will be modified as follows for clarity. "The steel plates partially extend into the concrete basemat shown in Figure A-7-1 and are anchored to the basemat at the location where they are terminated."

   d) Reference to Section 8.2 of the last paragraph of page A-7-6 will be revised to refer to Section 7.2.

   e) The description of Specimen 3.2 on page B-1-2, TS 3.0, Item (ii) will be revised to identify tie bars as parallel to specimen length.

   f) The description of Specimen 2.2.1 on page B-1-5, Table B-1-1 will be revised to identify tie bars as perpendicular to specimen length.

g) The inconsistency will be corrected by deleting the first sentence of the second paragraph from the bottom of page B-7-2.

h) The first sentence of the third paragraph of page B-9-11 will be expanded to reference Equation B-9.5-1.

Impact on DCD

DCD Table 3.8.3-7 will be revised as indicated on the attached markup.

Impact on R-COLA

There is no impact on the R-COLA.

Impact on PRA

There is no impact on the PRA.

Impact on Technical/Topical Report

Technical Report MUAP-11013 will be revised as indicated on the attached markups.

This completes MHI's response to the NRC's question.

Table 3.8.3-7 Summary of Confirmatory Physical Test Results (Sheet 1 of 3)

Test Series	Acceptance Criteria	Summary of Test Results
1.0 - Pushout Test	The push-out tests are to experimentally confirm that the shear strength of steel headed shear studs used in US-APWR SC walls can be calculated conservatively using MUAP-11019, Equation 2.3-1. As explained in MUAP-11019, this equation is based on ACI 349-06 Appendix D.6.1 Equation D-18 and the applicable resistance factor of 0.75 from ACI 349-06 Appendix D.4.5.	ACI 349-06 previsions are conservative when compared to test results; SP1.1 : Tie bar oriented parallel to the force direction, Acceptance Ratio = 1.34 SP1.2 : Tie bar oriented perpendicular to the force direction, Acceptance Ratio = 1.84
2.1 - Scaled Out-of-Plane Shear Tests	Out of Plane (OOP) Shear Scaled Tests is to experimentally confirm that the out-of-plane shear strength of USAPWR SC walls with their specific rectangular tie bar details can be predicted conservatively using ACI 349-06 code equations, modified by technical report MUAP-11019, Section 6.2.	ACI 349-06 provisions modified by MUAP-11019 are conservative when compared to test results; SP2.1.1 : Tie bars oriented perpendicular to the specimen length, a/d = 2.0: Acceptance Ratio = 1.52 SP2.1.2 : Tie bars oriented parallel to the specimen length, a/d = 2.0: Acceptance Ratio = 1.42 SP2.1.3 : Tie bars oriented perpendicular to the specimen length, a/d = 3.0: Acceptance Ratio = 1.26 SP2.1.4 : Tie bars oriented parallel to the specimen length, a/d = 3.0: Acceptance Ratio = 1.23
2.2 - Full Scale Out-of-Plane Shear Tests	Full Scale Out-of-Plane (Monotonic Loading) Shear Tests is to experimentally confirm that the out-of-plane strength of US-APWR SC walls with their specific rectangular tie bar detail designs is governed by flexural yielding rather than brittle shear behavior for shear span ratios greater than or equal to 2. Flexural strength of SC walls is provided by technical report MUAP-11019 Section 5.3.	The Test Series 2.2 specimens have failed in flexure, confirming the objective of the test series. Additionally MUAP-11019 provisions are conservative when compared to test results; SP2.2.1 : Tie bars oriented ~~parallel~~ perpendicular to the specimen length, a/d = 2.0: Acceptance Ratio = 1.18 SP2.2.2 : Tie bars oriented parallel to the specimen length, a/d = 2.0: Acceptance Ratio = 1.24

APPENDIX C

TABLE OF CONTENTS

LIST OF FIGURES .... **C-ii**
LIST OF TABLES .... **C-iv**

INTRODUCTION **C-1**

C1 ANALYSIS OF FULL-SCALE PRIMARY SHIELD STRUCTURE **C-2**
C1.1 Overall Approach C-2
C1.2 Geometric Modeling of Primary Shield Model C-2
C1.3 Material Model Parameters C-4
C1.3.1 Summary of Concrete Material Model C-4
C1.3.2 Summary of Steel Material Model C-4
C1.4 Interaction and Boundary Conditions C-4
C1.5 Mesh and Element Types C-5
C1.5.1 Concrete Elements C-5
C1.5.2 Steel Elements C-7
C1.7 Analysis Results C-9
C1.7.1 FEM Results C-9
C1.8 Design Criteria for Primary Shield Structure C-13
C1.8.1 Design Shear Strength Criteria C-13
C1.8.2 Comparison of Design Shear Strength to Analysis C-15
C1.8.3 Design Flexural Strength and Flexure-Axial Interaction C-15
C1.8.4 Comparison of Structure Stiffness from NIFE and LEFE models C-17
C1.8.5 Reconciliation of NIFE and LEFE model stiffness variations C-18
C1.9 SUMMARY AND CONCLUSIONS C-20

C2 PUSHOVER ANALYSIS OF US-APWR CIS **C-22**
C2.1 US-APWR – CIS MODEL ANALYSIS C-22
C2.2 Pushover Analysis Results: Condition A C-23
C2.3 Pushover Analysis Results: Condition B C-29
C2.3.1 Condition B: Short-Term Heating C-31
C2.3.2 Condition B: Long-Term Heating C-35
C2.4 Summary and Conclusions C-39

LIST OF FIGURES

Figure 1 Elevation Dimensions used in Model C-3
Figure 2 Section View of Middle Portion on the Left and Upper Portion on the Right........ C-3
Figure 3 Steel Shell Thicknesses........ C-3
Figure 4 Steel Shell Structure (left) and Concrete Solid (right) C-5
Figure 5 Front, Top, and Isometric Views of Coincident Shear Stud Connector Locations........ C-5
Figure 6 Steel and Concrete Element Meshes........ C-7
Figure 7 Lateral loading applied at: (i) 35'-11" top of middle portion, (ii) 46'-11" top of upper portion C-6
Figure 8 Temperature History used in Accident Thermal Analysis C-7
Figure 9 Concrete Temperature Gradients at 0, 12, 24, 36, 48, and 60 Seconds C-8
Figure 10 Operation Thermal Load Displacement Curves C-9
Figure 11 Comparison of Operation Thermal and Accident Thermal with Lateral Load Applied at Elevation 46'-11" C-10
Figure 12 Displacement Gradient for the Operation Thermal Load Condition and a Force Level of 37,000 kips (Load applied at 46'-11")........ C-10
Figure 13 Steel Von Mises Stresses for the Operation Thermal Load Case (Load applied at 46'-11") C-11
Figure 14 Steel Von Mises Stresses for the Accident Thermal Load Case (Load applied at 46'-11") C-11
Figure 15 Concrete Maximum Tension Strains for the Operation Thermal Load Case (Load applied at 46'-11") C-12
Figure 16 Concrete Maximum Compressive Stresses for the Operation Thermal Load Case (Load applied at 46'-11")........ C-12
Figure 17 Nominal Design Shear Strength as a Function of Angle........ C-14
Figure 18 Design Shear Strength of Middle Portion........ C-14
Figure 19 Design Shear Strength of Upper Portion........ C-14
Figure 20 Comparison of Analytical Strength to Design Strength C-15
Figure 21 P-M Interaction Curves for the Three Middle Wall Segments C-16
Figure 22 Comparison of P-M Interaction Curves to Design Shear Strengths........ C-16
Figure 23 Comparison of Non-Linear and Linear Stiffnesses C-18
Figure 24 Finite Element Analysis Mesh C-22
Figure 25 Basemat and PSW Finite Element Representation C-23
Figure 26 Base Shear vs. Top Slab Displacement Response Comparison C-24
Figure 27 Base Shear vs. Top Slab Displacement Response up to SSE Load C-24
Figure 28 Concrete Cracking at 25k kips Base Shear C-25
Figure 29 Concrete Cracking at 32k (left) and 39k (right) kips Base Shear C-26
Figure 30 Concrete Cracking at 85k kips Base Shear C-26
Figure 31 Steel Plate Yielding at 300k kips Base Shear C-27
Figure 32 Steel Plate Yielding at 425k kips Base Shear C-28
Figure 33 Lateral Load Resistance Distribution in CIS for Condition A........ C-29
Figure 34 Initial Stiffness Comparison up to SSE Level........ C-29
Figure 35 Thru-Thickness Temperature Distribution of a typical 4' SC Wall C-31
Figure 36 Temperatures of CIS at One Hour of Heating........ C-31
Figure 37 Concrete Maximum Tensile Strain at the End of Heat Analysis........ C-32
Figure 38 Temperature Distribution in Center Mass Concrete Region C-32
Figure 39 Concrete Cracking in Center Mass Concrete Region C-33
Figure 40 Base Shear vs. Top Slab Displacement Response Comparison C-33

Figure 41 Plastic Strain Distribution in SC Walls at 135k kips base shear........................... C-34
Figure 42 Base Shear vs. Top Slab Displacement Response Comparison C-34
Figure 43 Condition B Lateral Load Resistance Distribution in CIS C-35
Figure 44 Thru-Thickness Temperature Distribution of a typical 4' SC Wall C-36
Figure 45 Temperatures of CIS after 20 Hours of Heating.. C-36
Figure 46 Base Shear vs. Top Slab Displacement Response Comparison C-37
Figure 47 Plastic Strain Distribution in SC Walls at 200k kips C-37
Figure 48 Base Shear vs. Top Slab Displacement Response Comparison C-38
Figure 49 Condition B Lateral Load Resistance Distribution in CIS C-38

LIST OF TABLES

Table 1 Summary of Analysis Runs C-8
Table 2 Secant and LEFE Stiffness Values for Operation and Accident Thermal Conditions C-18
Table 3 Impact of PSW Stiffness Reduction on CIS Modal Frequencies and Mass Participation... C-19
Table 4 Summary of Concrete Cracking Formation Load Levels C-27
Table 5 ummary of Steel Plate Yielding Formation Load Levels for Condition A.............. C-28
Table 6 Heat Transfer Analysis Material Properties... C-30