SUPPLEMENTAL RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

12/27/2013

US-APWR Design Certification

Mitsubishi Heavy Industries

Docket No. 52-021

RAI NO.:	**NO. 985-6948 REVISION 3**
SRP SECTION:	**03.08.03 – Concrete and Steel Internal Structures of Steel or Concrete Containments**
APPLICATION SECTION:	**3.8.3**
DATE OF RAI ISSUE:	**01/08/2013**

QUESTION NO. 03.08.03-104:

The staff reviewed the applicant's response to the RAI 905-6311, Question 03.08.03-69 on how the delamination or splitting failure mode of the concrete in the SC sections can be prevented by providing adequate out-of-plane shear strength. Regarding the first example case, where a segment of the steel-concrete (SC) wall is subjected to an axial compression force on the concrete only at one end, a 3T (T =wall section thickness) transfer length is assumed in the calculation for the resisting moment. However, the staff noticed that Section 2.4 of technical report (TR) MUAP-11019 indicates a steel faceplate development length of 2T for a typical 48 inch thick SC wall. Therefore, the staff requests that the applicant reconcile the use of the 3T transfer length versus the 2T value, which was utilized in Section 2.4 of TR MUAP-11019.

Regarding the second example case, in which the splitting moment results from slightly different yield forces in the two steel faceplates, the staff found that insufficient information is provided for the derivation of the splitting moment. Therefore, the staff requests that the applicant provide additional information on how the splitting moment in the second example case is obtained, e.g., include a complete free body diagram showing all forces balancing each other.

In addition, as requested in the original RAI, identify what tests exist which provide additional justification to show that delamination or splitting would not occur anywhere for the configuration that is the same or similar to the US-APWR configuration.

Furthermore, the RAI response stated that the splitting or delamination failure is hypothesized, and its force demand is not real and will not be combined with other load demands. It is unclear to the staff why the failure mode is not a real case; therefore, the applicant is requested to provide additional explanation regarding this issue.

ANSWER:

This answer supplements the previous MHI answer that was transmitted by letter UAP-HF-13112 (ML13172A082). The supplemental response presented below was discussed with the Nuclear Regulatory Commission (NRC) staff during the Design Certification Document (DCD) Tier 2, Section 3.8 Audit conducted during the week of November 4, 2013.

The original response to this RAI remains correct with the following supplemental information regarding distribution of localized forces. For clarity, the original response is repeated below and the supplemental information follows.

The transfer length for a steel-concrete (SC) wall section is not the same as the development length for the steel faceplates. Transfer length (L_T) is defined as the length over which the shear studs develop composite action in terms of strain compatibility in the SC wall, as shown in Technical Report MUAP-11019, Rev. 1 Figure 2.7-1. Development length (L_d) is defined as the length over which the shear studs develop the yield strength of the steel faceplates, as shown in Technical Report MUAP-11019, Rev. 1 Figure 2.4-1.

The development length of the steel faceplates of SC wall is dependent upon the relative strengths of the steel faceplates and the shear studs. It was calculated in Technical Report MUAP-11019, Rev. 1 Section 2.4, and was shown to be less than two times the wall thickness (T) for typical US-APWR SC walls.

The transfer length depends on the relative stiffness of the shear studs and the steel faceplates. It was not calculated explicitly, but is expected to be greater than the development length (two times the wall thickness T). It was assumed to be equal to 3T in the sample calculation presented in Technical Report MUAP-11019, Rev. 1 Section 2.7. Figure 2.7-4 shows the free body diagram for the case with slightly different yield forces in the two steel faceplates, and the resulting splitting moment. This figure will be revised as indicated on the attached markup to provide additional information on how the splitting moment is obtained.

There are no tests that show that delamination or splitting will not occur for the configuration that is the same or similar to the US-APWR configuration. Delamination or splitting failure has been observed only once in the laboratory for tests conducted on SC walls without tie bars. Delamination or splitting failure has not been observed in any tests conducted on SC walls with tie bars exceeding the minimum requirements of American Concrete Institute (ACI) 349-06 Section 11.5.6.3, which are discussed in Section 2.6 of Technical Report MUAP-11019, Rev. 1.

Delamination or splitting failure is not a plausible failure mode for the US-APWR design because:

(i) All the SC wall connections (SC wall-to-concrete basemat, or SC wall-to-SC wall connections) involve force transfer to all elements of the cross-section, i.e., both the steel faceplates and the concrete infill. As shown in Technical Report MUAP-11019, Rev. 1 Section 2.7 and Figures 2.7-1, 2.7-2, and 2.7-3, delamination or splitting failure is possible when forces are transferred to only some elements of the cross-section, namely, either only the steel faceplates or only the concrete infill of the SC wall.

(ii) As discussed in Technical Report MUAP-11020, Rev. 1, Section 1, all US-APWR SC walls, including the connection regions, are specifically designed and detailed to undergo yielding and energy dissipation during overloads.

(iii) The design loads and load combinations for the SC walls produce eight different load effects, namely, three in-plane forces, three out-of-plane moments, and two out-of-plane shears. As described in MUAP-11019 Rev. 1, steel faceplate yielding is the governing failure mode for all these load effects (except for pure axial compression, which causes steel yielding accompanied with concrete crushing).

Impact on DCD

There is no impact on the DCD.

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-11019, Rev. 1 Figure 2.7-4 will be revised as indicated on the attached markup.

SUPPLEMENTAL INFORMATION

During the NRC Audit of the US-APWR Containment Internal Structure analysis and design, a question was raised that is related to the topics addressed in this RAI, including faceplate development length and ensuring ductility of the SC walls in the design process. The NRC requested MHI to provide a basis (e.g., design code, standard, or reference) to show why localized forces can be distributed over a distance of two times the wall thickness. The following discussion is provided in response to that request.

The standard design policy employed in the evaluation of SC walls for the US-APWR design is to calculate demand/capacity ratios on an element-by-element basis, using the procedures defined in Technical Report MUAP-11019, Rev. 1. Shell element force and moment demands are calculated at the centroid of each element, and the elements have typical widths and lengths of approximately T/2, where T is the SC wall thickness. In limited cases involving high demands in localized areas, aggregation of calculated demands over a distance no larger than 2T is permitted for comparison with capacities calculated over the same length, on the basis of the known ductility of SC walls (as demonstrated by testing) and their capability to locally redistribute steel faceplate stresses. The localized yielding required for this redistribution is considered acceptable relative to the ACI 349-06 Chapter 21 performance objective of essentially elastic response under the design loading conditions.

In response to the stated NRC request, there is no specific basis in ACI 349-06 for distributing localized forces occurring in a wall over a particular distance such as 2T, nor is any specific basis provided in other ACI design standards to MHI's knowledge. Rather, the approach utilized for consideration of localized demands in SC walls reflects typical industry practice for reinforced concrete slab and wall design utilizing finite element analysis. In such practice, finite element results are often aggregated over selected distances representing effective member widths, and the aggregated demands are then compared with code-defined capacities calculated for the selected widths. This approach is similar to ACI code-endorsed practices for certain member designs in which effective member widths are more explicitly defined and demands and capacities are calculated in terms of the specified width. Examples include:

- T-beams: effective flange widths are defined in ACI 349-06 Section 8.10 as eight times the slab thickness on either side of the beam web.
- Two-way slabs: effective widths are defined for column strips and middle strips of two-way slab systems in accordance with ACI 349-06 Section 13.2, and are taken as 0.25 times the shortest span length for column strips and 0.5 times the shortest span length for middle strips. For typical two-way slab systems, these strips have effective widths of as much as eight to ten times the slab thickness.
- Moment transfer between slabs and columns: ACI 349-06 Section 13.5.3.2 states that unbalanced moments at slab-to-column interfaces shall be transferred by flexure within an effective width between lines that are 1-1/2 slab or drop panel thicknesses (1.5h)

outside opposite faces of the column or capital. This corresponds to a total effective width of greater than three times the slab thickness.

- Shear walls: ACI 349-06 Section 21.7.4 defines in-plane shear capacity for shear walls in terms of the full cross-section of the shear wall including the concrete area *(A_{cv})* and steel reinforcement distributed over that area *($A_{cv}\rho_t$)*. As such, the effective length over which in-plane shear demands are typically calculated is also that of the full shear wall cross-section. Similar approaches are applied to diaphragms, in accordance with ACI 349-06 Section 21.9.7.

Each of these approaches accounts for the ability of the given member to redistribute locally higher demands over the effective width considered. This redistribution is feasible because of the ductility of the member, or its ability to undergo localized plasticity without fracturing and/or loss of design basis load carrying function.

For example, it is understood that localized flexural stresses beyond yielding in slabs can be redistributed to adjacent regions as a result of the ductility of the slab flexural reinforcement. This capability is reflected in the definition of geometric strips for which flexural reinforcement is sized based on the total factored bending moments applied to the strip. It is noted that such considerations are permitted even for slabs that do not meet the specific requirements of the direct design method; ACI 349-06 Section 13.5.1 indicates that any procedure may be used that satisfies "conditions of equilibrium and geometric compatibility if shown that the design strength at every section is at least equal to the required strength" and the commentary (R13.5.1) mentions that "the design of the slab may be achieved through…numerical solutions based on discrete elements." The published case studies provided by ACI for flat slab design reflect such an approach, with total demands aggregated over strips of elements with width of approximately ten times the slab thickness (see http://www.concrete.org/Tools/318Information/CaseStudies.aspx, Case Study 4, Structural Analysis- Methods and Assumptions: Flat Slab).

The ductility required for localized stress redistribution in reinforced concrete members has been assured through code requirements for ductile section detailing, as well as extensive laboratory testing and field performance of reinforced concrete structures. Similarly, section detailing of the SC walls used in the US-APWR has been carefully selected to ensure a high degree of ductility, and physical testing has been performed to confirm ductile response of SC members with the selected detailing. The ductility of SC walls, and particularly the US-APWR SC wall design, is ensured by the following detailing approaches:

- Equal reinforcement ratios on each face of the cross section. Since the same steel faceplate cross-sectional area and nominal yield stress is used on both faces, SC walls are ensured to undergo ductile faceplate yielding in flexure prior to concrete crushing due to flexural compression. For the US-APWR SC walls, this has been demonstrated by the Series 2.2, 3.1, and 5.2 tests (see Technical Report MUAP-11013, Rev. 2, Appendix B, Sections 9, 10, and 13), wherein typical CIS SC sections were tested under out-of-plane loading and confirmed to undergo faceplate yielding in flexure prior to concrete crushing.
- Tie bars spaced at T/2 that provide out-of-plane shear strength greater than M_p/2T, where M_p is the plastic moment capacity of the section. This detailing ensures that the steel faceplates will undergo ductile yielding under out-of-plane loading prior to nonductile shear failure for shear spans greater than 2T. Typical shear span ratios in the CIS (evaluated as the ratio of out-of-plane moment to out-of-plane shear) are always greater than 2T, and are typically 3T to 5T. The tie bar spacing of T/2 also ensures development of a truss mechanism such that out-of-plane point loads can be distributed over a projected length of the SC wall, approximately equal to 2T. Ductile yielding prior to out-of-plane shear failure was also confirmed in the Series 2.2, 3.1, and 5.2 physical tests mentioned above.
- Shear studs spaced to develop the SC wall faceplate yield strength over a distance less than 2T. For the typical 48-in thick SC wall, the ½-in. thick faceplates are developed within a distance of 75 inches or 1.56T. This is comparable to typical development

lengths for the large reinforcing bars used in nuclear power plant structures, and ensures that the SC wall longitudinal reinforcement is fully developed within 2T of supports. It is further noted that the large, closely-spaced tie bars used in the US-APWR design also contribute significantly to interfacial shear strength and thus faceplate development, as shown in the Series 1.0 confirmatory tests (see MUAP-11013, Appendix B, Section 7). As a result, actual faceplate development lengths are less than 1.5T for the US-APWR design.

Together, these detailing approaches provide certainty that the US-APWR SC walls will respond in a ductile manner to out-of-plane loading and that localized faceplate tensile stresses in excess of yield can be redistributed to adjacent regions of the wall. The Series 2.2 and 3.1 tests confirmed ductile response of SC members detailed in this manner under both operating and accident thermal conditions. It is important to note that the specific ability of the SC members to redistribute forces after yielding was confirmed via faceplate strain gauge data collected in these tests. As shown in MUAP-11013 Appendix B Figures B-9.6-9 and B-10.5-17, after the applied loads resulted in first yielding of the bottom plate in the constant moment region between the load points, the test specimens continued to carry load by redistributing forces to un-yielded areas along the span. Faceplate strains up to 2.5 times yield were exhibited, along with yielding over a span in excess of 2T prior to termination of loading. Importantly, this represents more extensive yielding than accounted for in the applied design approach wherein demands were aggregated over a distance less than 2T, and the associated demand/capacity ratios were confirmed to be less than 0.9.

The aggregation of demands over a distance of less than or equal to 2T represents reasonable but not extensive yielding corresponding to the first onset of significant inelastic deformation, resulting typically from combined SSE and accident thermal loading. As such, this approach is considered to be in accordance with the performance objectives of ACI 349-06 Chapter 21 (Section R21.2.1), which include essentially elastic response (no significant damage) under SSE-level demands, and substantial assurance of structural integrity under beyond-design-basis earthquake loading. It is further noted that localized faceplate yielding induced by restraint of accident thermal growth will act to partially relieve the restraint causing the faceplate demands, and in turn reduce the thermal load demands. This self-relieving nature of accident thermal loading is discussed in Section RE.1.2 of ACI 349-06, where it is recognized that a portion of accident thermal stresses due to restraint may be relieved due to cracking, yielding, and other time-dependent deformations. Although cracking and localized yielding will tend to reduce accident thermal demands, the US-APWR design conservatively does not take any credit for such demand reduction.

Impact on DCD

There is no impact on the DCD.

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

There is no impact on a Technical/Topical Report.

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