REQUEST FOR ADDITIONAL INFORMATION 905-6311 REVISION 3

2/27/2012

US-APWR Design Certification

Mitsubishi Heavy Industries

Docket No. 52-021

SRP Section: 03.08.03 - Concrete and Steel Internal Structures of Steel or Concrete Containments
Application Section: 3.8.3

QUESTIONS for Structural Engineering Branch 1 (AP1000/EPR Projects) (SEB1)

03.08.03-67

1. As indicated in MHI Technical Report (TR) MUAP-11019-P (R0), the key design philosophy for steel-concrete (SC) walls includes the prevention of SC specific failure modes and limit states by designing and detailing the section adequately. One of the potential SC specific failure modes is local buckling of the steel faceplates. The TR concludes that the SC specific limit state of steel faceplate local buckling is prevented by designing or detailing plate slenderness (defined by s/t_p where s= shear connector spacing and t_p = plate thickness) to be less than or equal to 20 everywhere in the containment internal structure (CIS). As discussed in Section 2.2 of the TR, this conclusion is based on a test performed on several specimens that were subjected to axial compression. It appears that the test subjected the specimens to compressive loads but did not consider the additional expansion effects of the steel plates due to operating and accident temperature loading. MHI Technical Report (TR) MUAP-11005-P (R0) does describe some tests performed on SC members considering temperature effects to determine whether buckling occurs; however, these tests do not appear to include compressive loads. Therefore, the staff requests that the applicant discuss the combined effects of temperature and axial loading on steel faceplate local buckling and whether additional tests are needed to consider the combined effects.

2. As discussed above, Section 2.2 of the subject TR made reference to MHI Technical Report (TR) MUAP-11005 (R0) for the compressive test of several specimens to examine buckling. However, this specific test could not be located in TR MUAP-11005 which contains over 16 different test papers. Therefore, in this case and in other cases, whenever a reference is made to a test report or paper, the applicant is requested to clearly identify where the report/paper is located.

3. Due to (a) the limited number of test specimens (five) shown on page 2-2 in TR MUAP-11019-P, (b) the question of similarity of the test specimens to the US-APWR SC members, and (c) the potential test scale effects, the applicant is requested to also perform a calculation for the US-APWR specific design configuration of the SC members to support the test results and to demonstrate the conservatism of the slenderness ratio design criteria to preclude local buckling.

4. The results of the 1/10th scale test, such as those presented on page 2-11 of MHI TR MUAP-10002-P (R0), indicate that buckling failures occurred in various SC walls of the test model during the test. Considering that the 1/10th scale test model has a steel faceplate slenderness ratio of 18, explain whether or not the buckling failures occurred in

the 1/10th scale test model are local buckling failures in the steel faceplates of the SC walls. If so, this would not be consistent with the design criteria for slenderness ratio of 20 to preclude local buckling; and therefore, the applicant is requested to justify the conservatism of the selected design criteria.

03.08.03-68

As indicated in the MHI Technical Report (TR) MUAP-11019-P (R0), another potential SC specific failure mode is interfacial shear failure of the connectors used to anchor the steel faceplates to the concrete infill. Section 2.3 of the subject TR states that "… the limit state associated with concrete pryout in shear for the groups of shear studs with spacing of 8 inches will be evaluated using ACI 349-06 Section D.6.3." The excerpt of the code is shown in Figure 2.3-2 of the TR. However, the calculation which followed in the same section of the TR for the design shear strength of the shear studs only considers the shear strength of the shear stud steel material, not the effect of the concrete pryout in shear as stated in the TR. The calculated design shear strength of the shear studs was then used to determine the steel faceplate development length in Section 2.4 of the TR and the interfacial shear strength of SC walls in Section 2.5. Explain why the effect of the concrete pryout was not included in the calculations of the design shear strength of the shear studs. Also explain how the effects due to spacing and embedment depth of the shear studs, and the effects of concrete cracking are considered in the design using ACI 349-06 Appendix D. If tensile forces perpendicular to the faceplate occur (e.g., equipment attachment loads, local (in-plane shear) induced faceplate buckling, concrete delamination/splitting forces which may occur between the ties), then explain how these tensile loads acting on the studs are considered, including interaction effects between the shear and tensile loads acting on the stud.

03.08.03-69

MHI Technical Report (TR) MUAP-11019-P (R0) indicates that splitting or delamination failure of the composite section through the concrete infill is another potential SC specific failure mode. The Design Philosophy and Executive Summary of the TR, page viii, states that "... steel tie bars provide structural integrity to the SC section and prevent SC specific delamination or splitting failure mode from occurring .... The area and spacing of the tie bars are designed to meet the minimum shear reinforcement requirements." The staff notes that there is no specific design criterion discussed in this report which demonstrates that these tie bar sizes and spacing will prevent delamination or a splitting failure mode; therefore, explain how the delamination or splitting failure mode can be prevented by providing adequate out-of-plane shear strength. Also, 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.

03.08.03-70

The Design Philosophy and Executive Summary of the MHI Technical Report (TR) MUAP-11019-P (R0), page viii, states that "... the stud spacing and strength are further designed to prevent interfacial shear failure as an SC specific limit state. This is done by

designing the interfacial shear strength (of the shear studs) to be always greater than the corresponding out-of-plane shear strength of the SC section." To support the design philosophy, Section 2.5 compares the interfacial shear strength with the out-of-plane shear strength of the SC section using the strengths per unit concrete area. To ensure that the design approach for the SC members is acceptable, the following information is requested:

1. According to the TR, the interfacial shear strength of the SC section is governed by the design shear strength of the shear studs, and the out-of-plane shear strength of the SC section is governed by tie bar and concrete strengths. These two strength calculations are based on different loads acting on different sections of the SC members (one being the shear load acting on the studs and the other is the out-of-plane shear acting on the tie bars and concrete). Therefore, explain how the comparison of these two different strengths can prevent interfacial shear failure as an SC specific limit state.
2. As discussed in the second question of this RAI, the stud shear strength used in the comparison did not consider the effect of the concrete pryout; therefore, the comparison calculation may need to be revised based on the answer to question Q-2.
3. At the end of Section 2.5, the report indicates that the compliance of the design philosophy (which is, preventing interfacial shear failure from occurring by designing the interfacial shear strength of the shear studs to be always greater than the corresponding out-of-plane shear strength of the SC section) must be checked for all SC walls and locations. Therefore, the applicant is requested to provide the check for all of the CIS critical sections, e.g., the connection regions where additional tie bars are used, as discussed at the end of Section 2.6 on page 2-9.
4. Explain whether the application of this design philosophy was verified by prior testing and provide this information. If existing test data are not available, then explain how the proposed additional confirmatory tests identified in Table A3-1 can demonstrate the design philosophy to preclude interfacial shear strength failure.

03.08.03-71

The Design Philosophy and Executive Summary of the MHI Technical Report (TR) MUAP-11019-P (R0), page viii, indicates that tie bars consist of two half-length pieces that are stud welded to the opposite steel faceplates, and the two half-length pieces are spliced in the center using a mechanical coupler. If the two half-length pieces of a tie bar will be welded to the opposite steel plates first, then spliced using a mechanical coupler, identify what tolerances need to be specified for the offset of the two half-length pieces of a tie bar. Explain how stresses in a tie bar due to the offset are evaluated and included in the design calculations. Since the tie bars will hold the two faceplates during the concrete pour, explain how the stresses in the tie bars and the faceplates due to the wet concrete are considered in the design of these structural elements. Explain how the assumption of the maximum wet concrete height that would be used to calculate additional stresses in the structural elements is ensured during the actual construction.

03.08.03-72

As discussed in DCD Sections 3.8.3 and 6.2, stainless steel material will be used on the wall of the refueling water storage pit (RWSP) to prevent corrosion from occurring. Describe the details of the stainless steel material (e.g., will stainless steel (SS) faceplates be used instead of carbon steel faceplates or will SS cladding on the carbon steel faceplate be used? Will studs/tie bars be SS? What are the thicknesses and material types etc.?). If stainless steel is combined with carbon steel, explain how the different material properties are considered in the analysis models and design.

03.08.03-73

Section 2.6 of the MHI Technical Report MUAP-11019-P (R0) discusses tie bar spacing and size. On the top of page 2-9, it states that "the maximum spacing requirement for tie bars is based on ACI 349-06 Section 11.5.5," which requires that spacing of shear reinforcement placed perpendicular to axis of member shall not exceed d/2 in nonprestressed members (d is the effective depth of a flexural member). The section on page 2-9 also states that "Typical SC modules include … A496 deformed wire with … orthogonal spacing grid of 24 in x 24 in." Explain what orthogonal spacing grid will be used for tie bars in the various SC wall thicknesses, including those less than 48 inches in thickness where the spacing would need to be smaller.

03.08.03-74

Section 2.8 of the MHI Technical Report MUAP-11019-P (R0) discusses SC faceplate penetration detailing. It indicated that, similar to the reinforced concrete (RC) application, an additional cover plate with thickness and width equal to that of the plate interrupted by the penetration is provided on all sides of the penetration. However, unlike the RC walls where the integrity of the structure is achieved through bond between concrete and steel reinforcement, the integrity of the SC walls relies on anchor studs, tie bars and welds. Therefore, explain whether additional studs and tie bars are added to the sides of the penetration and how are the size and spacing for these studs and tie bars determined. Also, provide representative penetration details including, thickened faceplate dimensions, penetration sleeves and associate anchors, faceplate studs, tie bars, welds (locations, types and sizes), how these elements are sized, and how local thermal effects due to hot piping are considered.

03.08.03-75

Section 4.3 of the MHI Technical Report MUAP-11019-P (R0) discusses additional considerations for SC compressive strength. It states that, "For the end restraint conditions encountered by most of the SC walls in the CIS, an effective length factor of k = 0.7 (fixed-pinned) may be used" (to consider slenderness effects). It also states that "This factor is reasonable and conservative for all of the walls below the operating deck, as they are bounded by the basemat and the major elevated floor slabs." Since ACI 349-06 indicates that k = 1 should be used unless analysis shows that a lower value is justified, the applicant is requested to (1) revise the wording "may be used" and provide a definitive value and (2) perform an analysis in accordance with ACI 349 Section 10.12.1 which demonstrates that k = 0.7 is appropriate for all walls. Note that for linear

elastic members such as steel columns (i.e., not concrete walls that may have reduced moment of inertia due to cracking) and for no sidesway motion, k = 0.7 is considered a theoretical value for ideal conditions; it is not necessarily a recommended design value.

03.08.03-76

Section 5.3.2 of the MHI Technical Report MUAP-11019-P (R0) discusses uniaxial moment capacity for SC sections considering compression reinforcement. As indicated, the moment capacity considering compression reinforcement is determined based upon yielding of the tension faceplate and elastic property of concrete (i.e., a linear relationship between stress and strain for concrete). In Section 5.3.3, for various SC walls, the moment capacities considering compression reinforcement are compared with the moment capacities without considering compression reinforcement. As shown in Section 5.3.1, the moment capacity without considering compression reinforcement is determined based upon plastic property of concrete. Explain why the moment capacity considering compression reinforcement in Section 5.3.2 is determined based upon elastic property of concrete, not plastic property of concrete as used for the moment capacity without considering compression reinforcement in Section 5.3.1. Explain why the moment capacities for these two cases can be based on different stress strain relationships of the concrete.

03.08.03-77

Section 6.0 of the MHI Technical Report (TR) MUAP-11019-P (R0) discusses the development of design equations for the out-of-plane shear strength of SC walls. The concrete contribution (Vc) to the shear strength of a structural member depends on the direction (tension or compression) of the axial load applied to the member, as shown in the ACI code for RC structures and by the recommended corresponding equations for SC walls. To ensure the design for out-of-plane shear strength for SC members is acceptable, the following information is needed:

1. For equation 6.2-3 presented in Section 6.2 for SC members subjected to axial tension load, a note similar to the one given in ACI 349-06 Section 11.3.2.3 should be added, i.e., “but not less than zero, where Nu is negative for tension.”
2. Section 6.4 of the report states that “as mentioned in ACI 349-06 Section 11.3.1.3, in the presence of significant axial tension, the out of plane shear strength can be calculated by considering the contribution of the shear reinforcement (Vs) alone and neglecting the contribution (Vc) of the concrete." Clarify what is meant by the phrase “can be calculated,” i.e., explain what approach is used in the design of the out of plane shear strength for all of the APWR SC walls when tension loads exist. Explain whether Vc is always conservatively neglected for all out-of-plane shear strength calculations, or is equation 6.2-3 sometimes used to consider the effect of the magnitude of the axial tension force. If it is the latter, then confirm that both plus and minus member forces due to seismic loadings are considered which would eliminate or diminish the benefit of using Equation 6.2-3.
3. If any of the three equations 6.2-1, 6.2-2, and/or 6.2-3 will be used to include the shear strength from the concrete infill, then explain why reducing the factor from 2.0 to 1.5 for Vc is adequate. Based on Figure 6.2-1 of the TR, the use of the 1.5

factor does not result in a lower bound to the data points for shear strength provided by concrete for sections corresponding to 36 inches or more (which match the SC section thicknesses).

4. In addition to Section 6.4 discussed above, there are several other locations in the TR where the phrase “can be estimated,” “can be used,” “can be calculated,” or “may be used” …. are utilized. As example, p 2-4 of the TR states that “The maximum axial compressive strength of the US-APWR SC modules can be estimated according to ACI Equation 10-2 shown in Figure 2.2-2. To avoid confusion, the wording should be revised to state definitively whether this approach or equation(s) are used or only sometimes used because the phrases indicate “can be…” or “may be…”

03.08.03-78

Section 8.2 of the MHI Technical Report MUAP-11019-P (R0) discusses design equations for combined axial tension, flexure, and in-plane shear. Section 8.3 of the report discusses design equations for combined axial compression, flexure, and in-plane shear. To ensure that the equations are acceptable for use in the design of the SC members, provide the information requested below.

1. For seismic force demand calculations, explain whether the SRSS directional combination method is used (not the 100-40-40 combination method) and whether the combination method used is consistent with RG 1.92, Rev. 2.
2. Considering that the sign of all seismic forces is eliminated when using the SRSS combination method, explain whether both sets of equations, for combined axial tension and axial compression with flexure and in-plane shear, will be used when evaluating the design for seismic force demands calculated using the SRSS combination method.
3. The equations in Section 8.3, which consider both axial compression and flexure, are developed based upon yielding of faceplates in tension. Considering that the faceplates in tension may not yield when the compression force applied to the member is large enough, explain how the cases with non-yielding faceplates in tension which consider axial force and bending moment interaction (comparable to column P-M interaction diagram) are not evaluated.
4. Unlike reinforced concrete sections with distinct areas of steel in the x and y directions, the SC members have a single faceplate on each face. Therefore, explain why equations 8.2-4 and 8.3-4 check whether $A_x^{available}$ is greater than $A_x^{required}$ separately from the $A_y^{available}$ versus $A_y^{required}$. This approach is in effect double counting the provided steel in both directions. The faceplates are actually subject to plane stresses in two directions simultaneously, which the present design equations do not consider.

03.08.03-79

Explain/clarify the inconsistencies or typos in the MHI Technical Report MUAP-11019-P (R0) listed below.

1. Tests summarized in Table 2.2-1, on page 2-2, shows that steel plates are 6 mm with yield strength of 403 MPa, while page 2-4 shows values of 4.5 mm and 240 MPa.
2. Page 2-9, equation 2.6-1, shows “0.39 …>= 0.41” should be revised to “0.39 … and >= 0.41”
3. Page 6-4, third paragraph, the phrase “According to Section 11.5.6.2 of ACI 349 …” should read “According to Section 11.5.7.2 of ACI 349 …”
4. Page 7-2, third paragraph, fourth line down, the phrase “… steel plate uniaxial tension strength …” should read as “… steel plate uniaxial shear strength …”
5. Page 8-5, “Nx = … =0”, “=0” should be deleted, since this is a case with compression force, Nx.

03.08.03-80

Section 2.3.2 of MHI Technical Report MUAP-10002-P (R0), subsection (4) on page 2-19, compares the theoretical ultimate strengths (bending and shear) with the experimental test results for the 1/10th scale test model. To understand the comparisons being made and the results obtained, the staff requests that the applicant provide the following additional information:

1. Explain how the calculation methods, used for the theoretical ultimate strengths presented in this subsection of the report, compare with the methods used for the U.S. APWR CIS SC structure design.

2. The test results of the 1/10th scale test, such as those presented on page 2-11 of the report, indicated that buckling failures occurred in various SC walls of the test model during the test. Explain how the buckling failures affect the ultimate strengths of the test model. Also, explain whether the effects of the buckling failures were taken into account in the calculations for SC wall ultimate strengths for the test model.

3. Page 2-19 of the report indicates that the horizontal force corresponding to the theoretical ultimate bending strength of the test model overestimates the value from the test by a factor of 1.77. Explain why the calculated ultimate bending strength of the test model overestimates the value from the test by this margin.

4. Page 2-20 of the report states that "The pressurizer chamber wall experienced considerable damage. Its (calculated strength ÷ measured maximum horizontal force) ratio was between 1.41 and 1.45, indicating poor correspondence. The likely reason is as follows..." However, the sentences/paragraphs in the report following the above quoted sentences are not clear to the staff on what the possible reason is. Therefore, explain why the calculated strength of the pressurizer chamber wall is more than 40% higher than the measured maximum horizontal force. Explain whether the effect of buckling plays a role in the poor correspondence, since page 2-11 of the report indicates that buckling occurred

in the pressurizer chamber wall and was at an advanced stage before the maximum load was reached.

5. Page 2-20 of the report states that "These findings indicate that the test model's strength was determined by its shear strength..." Clarify whether the "shear strength" means in-plane shear strength, out-of-plane shear strength, or both. If the shear strength contribution included the out-of-plane shear strength, then describe how this was calculated and provide the calculated contributions of the out-of-plane and the in-plane shear strengths.