RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

05/31/2013

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. 03.08.03-102:

The staff reviewed the applicant's response to RAI 905-6311, Question 03.08.03-67 regarding the design criteria to preclude local buckling of the steel faceplates. Regarding the response to Item 4, the first sentence, "The selected steel face plate slenderness ratio (s/tp) prevents face plate yielding before local buckling," does not appear to be appropriate because the statement implies that "yielding" should occur after "local buckling", which is not consistent with the rest of the RAI response to Item 4.

In addition, if this statement is corrected, it appears that the design approach in the RAI response only requires that the slenderness ratio be selected such that buckling occurs after yielding. However, that does not appear to be sufficient since Section 2.2 of Technical Report (TR) MUAP-11019-P (R0) describes the design approach, which selects the slenderness ratio of 20 such that the failure mode of local buckling does not occur before the compressive strength of the section is reached. Thus, the staff requests that the applicant revise the RAI response to reflect the design approach presented in the TR, which was also verified by tests.

In addition, since this RAI addresses important analysis and design information for the CIS, details of the approach should be included in the TR and then summarized in the DCD.

ANSWER:

There was a typographical error in the original response to RAI 905-6311, Question 03.08.03-67, Item 4. This was corrected in the revised response transmitted by letter UAP-HF-13064 (ML13107B428). The corrected sentence reads, "The selected face plate slenderness ratio (s/t_p) prevents face plate local buckling before yielding."

The design criteria in Section 2.2 of Technical Report MUAP-11019, Rev.1 is sufficient because:

(i) The yield strain of the steel faceplates corresponds to the compressive strength of the concrete infill in compression as well. For example, Figure 03.08.03-102.1 shows the comparison of the normalized stress-strain curves for steel faceplates and concrete infill respectively. The normalized stress for steel is calculated as the stress divided by the yield stress (F_y), and the normalized stress for concrete is calculated as the stress divided by the compressive strength (f'_c). The concrete stress-strain curve in the figure is based on Popovic's empirical stress-strain model for concrete. Figure 03.08.03-102.1(a) compares the normalized stress-strain curve using nominal yield stress (50 ksi) for steel and nominal compressive strength (4000 psi) for concrete. Figure 03.08.03-102.1(b) compares the normalized stress-strain curve using expected yield stress (55 ksi) for steel and expected compressive strength (6000 psi) for concrete.

[

] Thus, the design approach in Technical Report MUAP-11019, Rev. 1 Section 2.2 is sufficient for both preventing yielding before local buckling, and allowing the axial load capacity to exceed the axial compressive strength (P_{no}) calculated using Equation 2.2-1.

Figure 03.08.03-102.1 Normalized stress-strain curves for steel and concrete of SC walls.

(ii) The occurrence of local buckling before yielding has not been shown to prevent the axial load capacity of the steel-concrete (SC) walls from exceeding the axial compressive strength (P_{no}) calculated using Equation 2.2-1 of Technical Report MUAP-11019, Rev. 1. This is confirmed in Table 03.08.03-102.1, which compares the axial load capacities of SC specimens from Table D-1 (compression load tests) of Technical Report MUAP-11005, Rev. 1 Appendix D with the axial compressive strength (P_{no}) calculated using Equation 2.2-1 of Technical Report MUAP-11019, Rev. 1.

As shown in Table 03.08.03-102.1, even SC specimens with slender steel faceplates (that is, with ratios substantially greater than the US-APWR design limit of 20) develop axial load capacities greater than or equal to P_{no} calculated using Equation 2.2-1. This is not unexpected because even for filled composite columns the occurrence of steel local buckling (before yielding) does not precipitate immediate failure because: (i) the concrete infill carries the loads shed by the steel, and (ii) the local buckling of the steel is constrained (slowed) by the concrete before compressive failure.

In summary, the design criteria in Section 2.2 of Technical Report MUAP-11019, Rev. 1 is more appropriate than the more limited criteria identified in the RAI (i.e., only ensuring yielding occurs before buckling). As confirmed by the consideration of faceplate and concrete material properties in (i) above, and the testing presented in Technical Report MUAP-11019, Rev. 1 Section 2.2, limiting s/t_p ratios to be less than or equal to 20 is sufficient for both preventing yielding before local buckling, and for allowing the axial load capacity to exceed the axial compressive strength (P_{no}) calculated using Equation 2.2-1. The criteria is based on the AISC N690 classification of sections for compression as slender or non-slender depending on the slenderness ratio of steel plates. Non-slender sections can develop the axial load capacity of the section because the steel plates undergo yielding in compression before local buckling.

Table 03.08.03-102.1. Comparison of specimen axial load capacity (P_{exp}) with axial compressive strength (P_{no}) calculated using Equation 2.2-1 for SC specimens with different s/t_p ratios

Impact on DCD

DCD Subsection 3.8.3.4.5.6 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

There is no impact on the Technical/Topical Report.

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

throughout the expanse of the SC walls for fabrication simplicity. Finally, the selected tie bar size and spacing is confirmed to maintain structural integrity of the SC walls by preventing section delamination or splitting failure, as discussed in Technical Report MUAP-11019, Section 2.7 (Reference 3.8-71). MIC-03-03-00057

3.8.3.4.5.6 Design of Shear Studs

The SC modules are designed as reinforced concrete elements, with the faceplates serving as reinforcing steel. Since the faceplates do not have deformation patterns typical of reinforcing steel, shear studs are provided to transfer the forces between the concrete and the steel faceplates. The shear studs are designed according to Appendix D of ACI 349-06 (Reference 3.8-8), as supplemented by Sections 2.1~~, 2.2, and 2.3~~ through 2.5 of Technical Report MUAP-11019 (Reference 3.8-71). ~~The shear studs make the concrete and steel faceplates interact compositely. In addition, the shear studs permit anchorage for piping and other items attached to the walls.~~As discussed in Technical Report MUAP-11019, Section 2.2 (Reference 3.8-71), the shear stud spacing is selected so that the shear stud spacing to faceplate thickness ratio, or faceplate slenderness ratio, is less than or equal to 20. This is to prevent faceplate local buckling under applied compression based on the behavior observed in experimental research. This research is summarized in Technical Report MUAP-11005, Appendix C (Reference 3.8-63). As discussed in Technical Report MUAP-11019, Section 2.3, the design shear strength of the studs is determined in accordance with ACI 349-06 Appendix D Section D.4.5 (Reference 3.8-8). Using these provisions, the shear studs are sized to prevent interfacial shear failure of the cross section under out-of-plane loading, as discussed in Technical Report MUAP-11019, Section 2.5. Finally, as discussed in Technical Report MUAP-11019, Section 2.4, the shear studs are confirmed to provide faceplate development lengths comparable to those of standard reinforcing bars typically used in reinforced concrete nuclear structures. MIC-03-03-00066 MIC-03-03-00057 DCD_03.08.03-102

3.8.3.4.6 Floor Slabs

The ~~floor slab of~~ reinforced concrete floor slabs ~~is~~are analyzed and designed according to ACI 349-06 (Reference 3.8-8) considering the same ~~loads~~design loading conditions as for the SC modules. The floor design does not rely on composite action with supporting structural steel beams. MIC-03-03-00066

3.8.3.4.7 Structural Steel Design and Analysis

Structural steel framing within the interior of the PCCV is primarily for support of floor slabs, equipment, distribution systems, and access platforms. Design and analysis procedures, including assumptions on boundary conditions and expected behavior under loads, are in accordance with the allowable stress design (ASD) method in AISC-N690 (Reference 3.8-9). Analysis methods are generally simple calculations using seismic ~~accelerations~~loads obtained from Section 3.7 methodologies in load combinations. Frame connections are detailed for simply-supported beams unless otherwise analyzed and detailed. MIC-03-03-00066

3.8.3.4.8 RCL Supports

The RCL piping and support system is analyzed for the dynamic effects of a SSE. A coupled model of the containment internal~~s~~ structure and the RCS is dynamically MIC-03-03-00066