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December 28, 2010

Document Control Desk
U.S. Nuclear Regulatory Commission
Washington, DC 20555-0001

Attention: Mr. Jeffery A. Ciocco

Docket No. 52-021
MHI Ref: UAP-HF-10351

Subject: MHI's Responses to US-APWR DCD RAI No. 657-5135 Revision 2 (SRP 03.08.05)

Reference: 1) "Request for Additional Information No. 657-5135 Revision 2, SRP Section: 03.08.05 - Foundations," dated 11/15/2010.

With this letter, Mitsubishi Heavy Industries, Ltd. ("MHI") transmits to the U.S. Nuclear Regulatory Commission ("NRC") a document entitled "Responses to Request for Additional Information No. 657-5135, Revision 2."

Enclosed are the responses to 6 RAIs contained within Reference 1. This transmittal completes the response to this RAI.

Please contact Dr. C. Keith Paulson, Senior Technical Manager, Mitsubishi Nuclear Energy Systems, Inc. if the NRC has questions concerning any aspect of this submittal. His contact information is provided below.

Sincerely,

Atsushi Kumata for

Yoshiki Ogata,
General Manager- APWR Promoting Department
Mitsubishi Heavy Industries, LTD.

Enclosure:

1. Response to Request for Additional Information No. 657-5135, Revision 2

CC: J. A. Ciocco
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NRO

Enclosure 1

UAP-HF-10351
Docket No. 52-021

Response to Request for Additional Information No. 657-5135,
Revision 2

December, 2010

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

12/28/2010

US-APWR Design Certification

Mitsubishi Heavy Industries

Docket No. 52-021

RAI NO.:	**NO. 657-5135 REVISION 2**
SRP SECTION:	**03.08.05 – Foundations**
APPLICATION SECTION:	**3.8.5**
DATE OF RAI ISSUE:	**11/15/10**

QUESTION NO. RAI 03.08.05-36:

In the response to Question 03.08.05-23, MHI states that "Figure 3.8.5-5 is a plan view and not a cross section." This is somewhat confusing because in MHI's previous response to the initial Question 03.08.05-23, it is stated that Figure 3.8.5-5 is a cross section. Figure 3.8.5-5 cannot be both a cross section and not a cross section. MHI is requested to clarify this inconsistency.

It is further noted that results of the seismic response analyses of the revised model (made necessary by changes in the R/B geometry) will be provided in the revised technical report MUAP-08005. The staff will review this revised report when it is received. In addition, the staff will review the changes in the R/B geometry (including the coordinates of the mass centers of basemat and the structures supported on it) and the changes in the seismic analysis and structural design will be incorporated into a future revision of the DCD.

ANSWER:

MHI's initial response to RAI 340-2004 Rev. 0, Question 03.08.05-01 stated in the second sentence that "Figure 3.8.5-5 is a cross section" is incorrect. This error was corrected in MHI's response to RAI 496-3735 Rev. 0, Question 03.08.05-23 which regarded the same issue.

Technical Report (TR) MUAP-08005 has generally been superseded by TR MUAP-10001. See MHI's responses to Questions 03.07.02-65 and 03.07.02-68 of RAI 660-5134 for additional discussions on TRs MUAP-10001 and MUAP-08005. Please note that TRs MUAP-10001 and MUAP-10006 and revision to MUAP-08005 as discussed in Questions 03.07.02-65 and 03.07.02-68 of RAI 660-5134 fulfill the technical report commitment in RAI 496-3735 Question 03.08.05-23.

Impact on DCD

There is no impact on the DCD in the response to this question. For impact to the DCD on the concrete filling of the "dent", see response to Question 03.08.05-38 of this RAI.

Impact on COLA

There is no impact on the COLA.

Impact on PRA

There is no impact on the PRA.

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DATE OF RAI ISSUE:	**11/15/10**

QUESTION NO. RAI 03.08.05-37:

In the response to Question 03.08.05-27, MHI states that the rigid elements used in the model couple the motion of the nodes that they connect with in all six degree of freedom. The shell elements modeling the walls of the building are extended into the layer of solid elements to transmit nodal rotations to the solid elements. The staff finds that additional information is needed in order to complete its evaluation of the response. For example, is the use of the rigid elements equivalent to providing fixed boundary condition to the rotational degree of freedom of the shell element? Further, how do shell elements transmit nodal rotations to the solid elements by extending into the layer of solid elements? MHI is requested to provide answers to these questions.

ANSWER:

The response to Question 03.08.05-27 referred to "rigid" elements that connect wall shell elements to the mat solid elements. These elements are not used in the updated analyses of the Reactor Building structures including the PCCV. Shell elements that extend into the layer of solid elements of the mat have been used to represent the bending stiffness of the wall. Wall to mat coupling of the three translations and coupling of the out-of-plane rotation about a horizontal axis are necessary to reflect the fixed axial, shear, and moment boundary condition of walls connected to the mat.

To adequately represent the fixed boundary condition between the shell elements of the wall and the solid elements of the mat to transmit out-of-plane nodal rotation requires more complex modeling than was previously explained. The modeling technique can be described as follows:

1. ANSYS SOLID65 elements are used to represent the mat.

2. ANSYS SHELL181 elements are used to represent the walls above the solid elements of the mat.

3. ANSYS SHELL63 elements are used to represent only the bending stiffness of the elements that extend the wall into the first layer of solid elements of the mat via use of the KEYOPT(1) = 2 option. These elements have no mass or weight.

The boundary condition achieved with the above technique results in translational displacement of nodes at the top and bottom of the top layer of solid elements matching the translational displacement of the nodes at the top and bottom of the embedded SHELL63 elements. Because rotational degrees of freedom are compatible between SHELL63 and SHELL181 elements, the rotations of the top of the SHELL63 element embedded in mat are the same as the rotations of the bottom of the SHELL181 wall elements. Without membrane stiffness present in the embedded SHELL63 elements, no artificial stiffening of the mat in out-of-plane bending due to membrane behavior of the wall affects the mat along the length of the wall in the plane of the wall. Some artificial bending stiffness of the mat about a vertical axis along the length of the wall occurs but is not significant since the mat is essentially rigid for mat in-plane bending. This technique is used for all Reactor Building wall connections with the mat including the PCCV and provides a fixed boundary condition for the rotational degree of freedom of the wall elements connecting to the mat solid elements. Figure 1 is a plot of a cut away view of the ANSYS model at the interaction of the wall and mat.

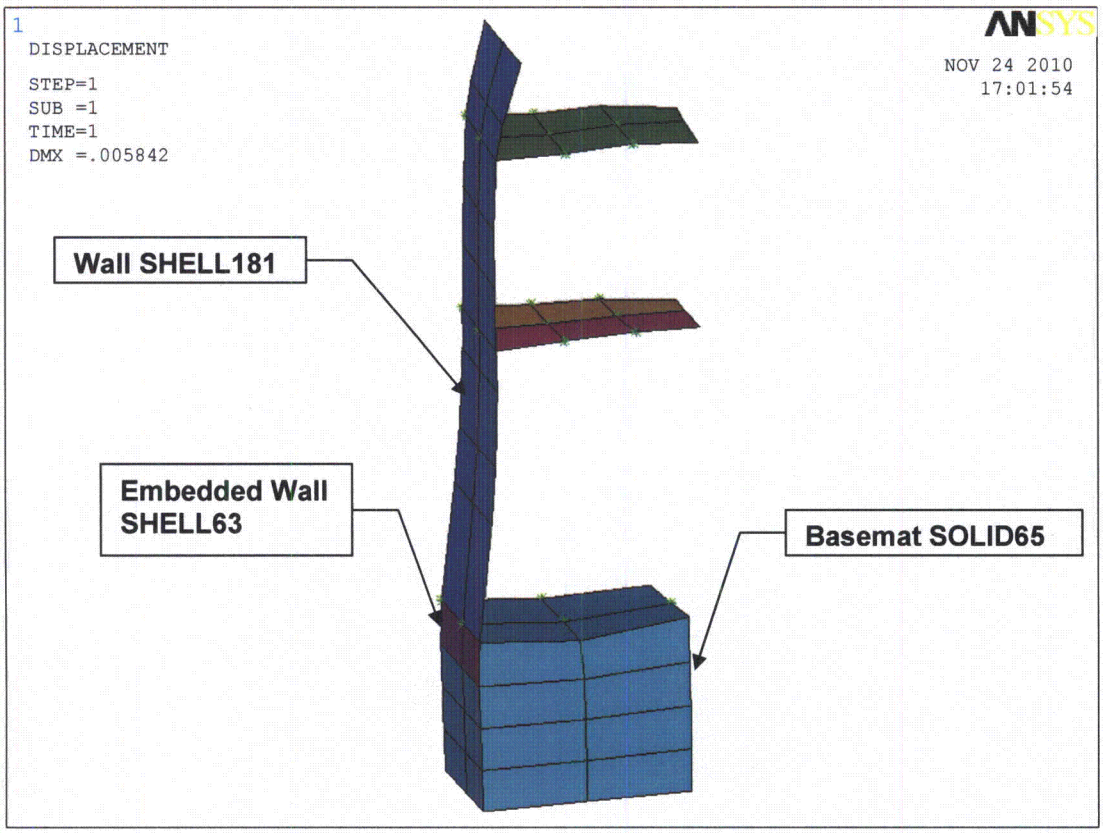

Figure 1 - Cut-Away View of ANSYS Model's Wall to Basemat Configuration
(Exaggerated Deflected Shape Due to East-West Earthquake)

Impact on DCD

There is no impact on the DCD.

Impact on COLA

There is no impact on the COLA.

Impact on PRA

There is no impact on the PRA.

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APPLICATION SECTION:	**3.8.5**
DATE OF RAI ISSUE:	**11/15/10**

QUESTION NO. RAI 03.08.05-38:

In its response to Part 5 of Question 03.08.05-28, MHI states that the high water table will not affect the soil spring stiffness for the R/B-PCCV analyses. However, the Applicant does not present any data to substantiate this claim. The staff disagrees with this position because the equations of motion for elastic waves in fluid-saturated porous media are different from those used in the derivation of the soil springs given in ASCE 4-98. MHI is requested to provide numerical data to support their argument that the high water table has no effect on the soil spring constants.

In the response to Part (c), MHI states that Figures 3.8.5-11 and 3.8.5-12 of US-APWR DCD Revision 2 indicate that the dent in the R/B basemat is filled with concrete and becomes part of the structural basemat. However, the staff notices that the description of the basemat presented in the first paragraph of the US-APWR DCD Revision 2 Subsection 3.8.5.1.1 has not been updated. MHI is requested to update this description.

ANSWER:

A) As explained in detail in the *Report on Subgrade Modeling* (2010b), the derivations for soil spring stiffness recommended by ASCE 4-98 are no longer used in the finite element analyses for the DCD. It is also explained in the URS (2010b) that soil spring representation of the subgrade in the finite element analyses was replaced by full continuous finite element modeling of the soil-foundation system. Therefore, the effects of groundwater table on soil spring stiffness, as addressed by the response to RAI Question 3.8.5-38, can be referred to as the effects of groundwater table on the subgrade properties used in the finite element analyses.

The DCD establishes the maximum groundwater level (GWL) as 1ft below grade. This means that, during the life of the structure, GWL can be anywhere at or below that elevation. The calculations account for this uncertain value through conservative considerations and soil property assessments leading to results that envelope all possible situations regarding the GWL, as demonstrated hereafter.

The effects of GWL on soil stiffness can be briefly described as follows:

1. Soil stiffness is a function of the soil deformation moduli (e.g., shear modulus, G, and bulk modulus, K)

2. The soil deformation moduli depend on the effective confining stress and shear strain level.

3. Effective confining stress in soil can be changed due to presence of groundwater by two mechanisms: (A) buoyancy, and (B) shear-induced plastic dilation combined with undrained or partly drained loading.

With reference to **Statement 2**, through the soil deformation moduli (for example G), soil stiffness is a function of the mean effective confining stress σ'_m via (e.g., Hardin and Drnevich, 1972):

$$G = G_0 \left(\frac{\sigma'_m}{\sigma'_{m0}} \right)^n \tag{1}$$

and similar relations are valid for the bulk and Young's moduli. In eq. (1), G_0 is the low strain shear modulus of soil at a reference effective confining stress, σ'_{m0}, while G is the low strain shear modulus corresponding to a different effective confining stress, σ'_m. Usual values for the power exponent "n" range between 0.4 and 0.6 for cohesionless soils and between 0.5 and 1 for cohesive soils (e.g. Richart et al. 1970). The dependence on shear strain level is not affected by presence of groundwater and is addressed in Technical Report MUAP-10001.

With reference to **Statement 3(A)**, one effect of groundwater is reducing the effective confining stress through buoyancy. That means that the overburden effective stress at a given point in a soil deposit is lower as the GWL is higher and, by virtue of eq. (1), the shear modulus is lower as GWL is higher. As stated in Technical Report MUAP-10001 and confirmed by Silva (2010), the shear wave velocity values provided in each soil layer of the recommended soil profiles come from records in soil deposits with high GWL. That means that the values of shear wave velocity, Vs, used in the finite element analyses to infer the shear modulus in soil via $G = Vs^2 \rho$ (where ρ is the mass density of soil) have been recorded in saturated soils. It can be therefore concluded that Vs values in MUAP 10001 provide values corresponding to the highest GWL and therefore to the lowest G. Consequently, the lower bound for G values is used in the finite element analyses for the soil in Profile 270, 500 of MUAP 10001. On the other hand, rock deformation moduli are not affected by GWL, so the upper bound for G values considered in the finite element analyses comes from Profile 2032, 100 in MUAP 10001, irrespective of the level of groundwater. Any GWL in soil lower than the maximum level would induce deformation moduli above the lower bound and will, therefore, be contained in the range of soil moduli already considered in the analysis. It is mentioned that Profile 270, 500 represents the softest soil conditions from all generic layered soil profiles recommended in MUAP-10001, while Profile 2032, 100 represents the stiffest rock conditions.

A numerical example is presented to illustrate the effect of GWL on shear modulus due to buoyancy (Statement 3A). Considering a location "A", at a depth z = 50ft below grade (about 10ft below the basemat elevation), the shear modulus computed based on the shear wave velocity and mass density at that depth from Profile 270, 500 in MUAP-10001 is G_{sat} = 6.38×10^3 ksf (see Table AP. 3-1 in URS 2010a). This value corresponds to fully saturated soil, and therefore to an effective overburden stress:

$$\sigma'_{sat} = z \cdot \gamma' = 4.17 ksf \tag{2}$$

where γ'=83.3pcf was calculated based on the soil density in Table Ap. 3-1 of URS (2010a) and assuming a void ratio e=0.5 for very dense soil. If the GWL goes below 50ft depth, the effective overburden stress at location "A" becomes:

$$\sigma'_{dry} = z \cdot \gamma_{dry} = 6.25ksf \quad (3)$$

with γ_{dry} = 125pcf, from Table AP. 3-1. The new value of shear modulus at location "A" becomes (via eqn. 1 and using the effective stresses in eqns. 2 and 3): G_{dry} = 7.81x10^3 ksf, well below the value of 9.81x10^3 ksf already considered in the calculations for soil Profile 560, 200 in MUAP-10001 (see Table Ap. 3-2 in URS 2010a).

With reference to **Statement 3(B)**, shear induced dilation is the development of plastic volumetric strains in soils subjected to shear (such as in seismic loading). In particular, loose and medium dense granular soils and normally consolidated clays experience compactive volumetric strains under shear, while dense granular soils and over-consolidated clays experience mainly dilative plastic volumetric strains. It has been verified experimentally that, under drained loading (static loads), the results in terms of effective stresses and the mechanical behavior of cohesionless soils was not affected by presence of water in soil (e.g., Finn et al 1978).

In case of relatively fast cyclic loading (e.g., seismic loads) of saturated soils, the pore water does not have time to move out from (or into) the pores and the volume of soil remains practically constant for a short time (undrained loading). In this situation, for loose to medium dense granular sand and normally consolidated clay the tendency of volumetric compaction translates into an increase in the pore water pressure, with corresponding reduction of the effective stress and hence of both deformation moduli and shear strength of the soil. Conversely, for dense sand and over-consolidated clay the tendency of dilating under shear translates into a reduction of the pore water pressure (suction), with corresponding increase in effective stress and therefore in deformation moduli and shear strength. The soils considered in the DCD are very dense granular soils (Vs > 1000ft/s even for the softest soil – Profile 270, 500 in MUAP 10001, corresponding to Standard Penetration Test blowcounts N_{60} > 50...100blows/ft – see e.g. Bowles 1996, eq. 20-17, corresponding in turn to very dense soil – e.g. Bowles 1996, Table 3-4). For this type of soils, presence of groundwater leads to increase in deformation moduli and shear strength during cyclic loading. Increase in stiffness and shear strength under shear due to plastic dilation is not warranted for non-saturated soils (i.e. above GWL) and therefore the conservative approach accounting for any level of the groundwater table is to ignore these effects. The finite element analyses for the DCD assume elastic behavior for soil, therefore ignoring any plastic dilation effects.

References:

Bowles, J.E. (1996) *Foundation Analysis and Design*, Fifth Edition, McGraw-Hill.

Finn, W.D.L., Vaid, Y.P. and Bhatia, S.K. (1978) Constant volume cyclic simple shear testing, *Proceedings 2nd International Conference on Microzonation*, San Francisco, CA, pp.839-851.

Hardin, B.O. and Drnevich, V.P. (1972). Shear modulus and damping in soils:design equations and curves. J. Soil Mech. Div., ASCE, 98(SM7):667-692.

MUAP 1001: Mitsubishi Heavy Industries (2010). *Seismic Design Bases for the US-APWR Standard Plant*. Report No. MUAP-10001 (R0), Rev. 0. February, 2010.

Richart, F.E., Hall, R.J. and Woods, R.D. (1970). *Vibrations of Soils and Foundations*. Prentice Hall.

Silva, W. (2010). Personal Communication.

URS (2010a). *US-APWR – Basic Analysis and Design of R/B*. Calculation REB-13-05-230-001 for Mitsubishi Heavy Industries.

URS (2010b). *Subgrade Modeling in Finite Element Analyses Nuclear Island Structures*. Report No. REF-13-05-160-005 for Mitsubishi Heavy Industries

B) The description of the basemat presented in the first paragraph of the US-APWR DCD Revision 2 Subsection 3.8.5.1.1 has been updated as shown in the attached markup.

Impact on DCD

See Attachment 1 for the markup of DCD Tier 2, Section 3.8 changes to be incorporated.

- Revise the fourth and fifth sentences in the first paragraph of Subsection 3.8.5.1.1 to read as follows: "The length of the basemat in the north-south direction is 309 ft, 0 in., and in the east-west direction is 210 ft, 0 in, as shown in Figure 3J-1. The central region, generally circular with a diameter of approximately 187 ft, supports the PCCV and containment internal structure with a thickness of approximately 38 ft, 2 in."

Impact on COLA

There is no impact on the COLA.

Impact on PRA

There is no impact on the PRA.

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

12/28/2010

US-APWR Design Certification

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Docket No. 52-021

RAI NO.:	**NO. 657-5135 REVISION 2**
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APPLICATION SECTION:	**3.8.5**
DATE OF RAI ISSUE:	**11/15/10**

QUESTION NO. RAI 03.08.05-39:

In the response to Part 1 of Question 03.08.05-30, MHI states that the total dynamic lateral pressure is the sum of the Wood's pressure and the Westergaard's hydrodynamic pressure. The staff does not accept this answer unless additional data are provided to support this response by MHI. The Wood's solution is based on the classical elastic wave theory which is not applicable for the fluid-saturated porous media. MHI is requested to provide numerical data to support the statement that the lateral pressure based on the elastic wave theory in the porous media is enveloped by the Wood's pressure.

In the response to Part 2, MHI states that the lateral earth pressure induced by the vertical earthquake is given by $K_0\ (a_v/g)\gamma_e Z_w$ in which K_0 is at-rest coefficient of soil. The staff is not aware of this equation. MHI is requested to provide the technical basis for this equation.

In the response to Part 3, MHI provides a detailed answer that includes the explanation of the active and passive pressure. The staff finds the response somewhat hard to follow. Perhaps the question asked by the staff was not clearly stated. Reiterating its concern as expressed in the initial RAI question: the staff noticed that Wood's solution does not consider the earth pressure due to the rotation of the wall at its base and requested that the Applicant provide information addressing this earth pressure. Notice that the rotation of the wall at its base is a result of the SSI analysis. So, the earth pressure should be calculated within the frame of theory of elasticity, because the SSI analysis performed is within the frame of linear elasticity. MHI is requested to address this concern.

ANSWER:

As described in Subsection 3.8.4.4.3 of DCD Tier 2, the basemats and below grade exterior walls of the US APWR seismic category I buildings are designed using load combinations accounting for sub-grade loads including static and dynamic lateral earth pressure, soil surcharges, and effects of maximum water table. The earth pressure loads used for the standard design envelope the static and dynamic earth pressure demands of wide range of candidate sites within continental US. The magnitudes of the earth pressure design loads are developed following the requirements of SRP 3.8.4 Acceptance Criterion 4.H that states:

"Consideration of dynamic lateral soil pressures on embedded walls is acceptable if the lateral earth pressure loads are evaluated for two cases. These are (1) lateral earth pressure equal to the sum of the static earth pressure plus the dynamic earth pressure calculated in accordance with ASCE 4-98, Section 3.5.3.2, and (2) lateral earth pressure equal to the passive earth pressure. If these methods are shown to be overly conservative for the cases considered, then the staff reviews alternative methods on a case-by-case basis."

The first case required by Acceptance Criterion 4.H is addressed as described in Section 3.6.3 of MUAP-10006(R0) by considering lateral earth pressure equal to the sum of the static earth pressure at rest (including additional pressure due to vertical inertial forces in backfill – see Part 2 of this Answer), plus the dynamic earth pressure calculated in accordance with ASCE 4-98, Section 3.5.3.2. The calculations of the lateral seismic earth pressure use a total unit weight of 130 pcf for the embedment soil and assume that the water table is at plant grade elevation and that the response of the two phase system consisting of submerged soil and the water are completely coupled and in-phase. As described in the response to RAI 660-5134 question 3.7.2-62, Section 3.6.3 of MUAP 10006 will be revised to document the calculations of the dynamic earth pressures based on seismic coefficient of 0.6 g that is twice the free field peak ground acceleration at the bottom of the foundations to account for dynamic response amplification in the embedment soil as calculated using elastic wave theory (e.g. Veletsos and Younan 1994, Wu and Finn 1999) and for effects of wall rotation and deformations.

Regarding the second case of Acceptance Criterion 4.H, MHI assumes the position that based on the seismic deformations of the basements of Category I buildings, the consideration of lateral pressure equal to the passive earth pressure is overly conservative. The results of the site-independent soil-structure interaction analyses for maximum lateral displacements relative to free field ground motion that are presented in Tables 4-1 to 4-4 in MUAP 10006(R0) show that the maximum displacements at nominal plant grade elevation is less than 0.75 in. The necessary lateral displacements to mobilize passive pressure in a 40ft deep granular fill are between 0.5% and 1% of fill depth (e.g. Das 2006), i.e. about 2.5in to 5in, much larger than the actual maximum lateral displacements obtained from site-independent SSI analyses of non-embedded structures. As lateral displacements toward fill of embedded structures are expected to be even lower than those calculated from the SSI analyses of non-embedded structures, it can be concluded that the state of strains in the embedment soil will not result in mobilization of earth pressures that are equal to the passive resistance of the soil.

Part 1.

It is deemed that the use of saturated unit weight for the soil provides the most conservative case for including the effects of groundwater in the calculations of the dynamic earth pressures because it considers that the response of the two phases of the system, the ground water and the soil to be completely in-phase and does not consider the dissipation of energy due to the viscous flow of the ground water. In the response to Part 1 of Question 3.8.5-30, MHI provided alternative computations of the total dynamic lateral pressure assuming no interaction between the embedment soil and the ground water. This approach also assumed the responses of the two phases of the dynamic system are in-phase by adding together the maximum responses from soil and water dynamic pressure. Moreover, the Westergaard's formula provides hydrodynamic pressure representing the increase in pore pressure under seismic loads on a vertical wall bordering a free body of water (e.g. reservoir). Therefore, this approach provides an upper bound estimate of the ground water pressures since it does not consider the dissipation of energy due to the viscous flow of the pore water that reduces significantly the peak response of the embedment soil as shown in the study presented in (Theodorakopoulos, at. al., 2001). The calculations are presented in Part 1 of Question 3.8.5-30.

Part 2.

A simple quasi-static approach is employed to account for the effect of the vertical component of the ground motion on the horizontal seismic earth pressures. The pressure $\Delta p = Ko(a_h/g)\gamma_{eff}Z$ represents the additional horizontal earth pressure at rest produced by an increase in vertical effective stress $\Delta\sigma_{eff} = (a_h/g)\gamma_{eff}Z$ induced by vertical inertial forces at depth Z. The calculations use vertical seismic coefficient of 0.6 g which takes in account the amplifications due to vertical response of the embedment soil. The seismic lateral earth pressures due to the vertical and horizontal component of the earthquake are added algebraically by conservatively assuming that the peak vertical and horizontal response accelerations in the embedment soil occur simultaneously.

Part 3.

In the response to RAI 212-1950 question 3.7.2-14, MHI described that DCD COL item 3.7(23) requires the applicability of the standard plant design to be verified by site-specific SSI analyses using SASSI methodology. The site-specific SSI analyses of embedded foundations are required to demonstrate that the standard design which is based on the seismic responses obtained from site-independent SSI analyses of surface foundations, envelopes the effects of embedment soil including the site-specific dynamic earth pressure demands.

The SASSI program provides frequency domain elastic solution of the response of the structure interacting with the soil by employing finite element models representing the stiffness, damping and mass inertia properties of the foundation and the surrounding soil. The site-dependent SASSI analyses of embedded foundations provide elastic solutions for the dynamic earth pressures that include the effects of the actual response of the integral dynamic system consisting of the building structures and the surrounding soil. Therefore, the dynamic earth pressures obtained from the site-specific SSI analyses include effects that are due to the rotation of the base, the elastic deformations of the wall, the variation of the embedment soil properties, etc. The site-specific SSI analyses use total unit weight for the soil located below the site-specific elevations of the water table and provide an upper bound values for the dynamic earth pressures from the saturated soil. The comparison of the SASSI calculated site-specific dynamic earth pressures with the dynamic earth pressures used in the standard design ensure that the site specific earth pressure demands are enveloped by the standard design.

References:

(1) Das, B.M. (2006). *Principles of Geotechnical Engineering*, 6th edition, Thomson.

(2) Veletsos, A.S. and Younan, A.H. (1994). Dynamic soil pressures on rigid vertical walls. *Earthq. Eng. Struct. Dynamics*, 23:275-301.

(3) Wu, G. and Finn, W.D.L. (1999). Seismic lateral pressures for design of rigid walls. *Can. Geotech. J.*, 36:509-522.

(4) D.D. Theodorakopoulos, A.P. Chassiakos, and D.E. Beskos (2001): "Dynamic pressures on rigid cantilever walls retaining poroelastic soil media" Soil Dynamics and Earthquake Engineering 21 January 2001, pg. 315:338. (www.elsevier.com/locate/soildyn)

Impact on DCD

There is no impact on the DCD.

Impact on COLA

There is no impact on the COLA.

Impact on PRA

There is no impact on the PRA.

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

12/28/2010

US-APWR Design Certification

Mitsubishi Heavy Industries

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QUESTION NO. RAI 03.08.05-40:

In the response to Part 1 of Question 03.08.05-31, MHI provides a technical rationale for choosing 1/3 of the estimated maximum settlement for the differential settlement. The staff reviewed this response and considers the answer to be acceptable. However, the staff notices that in US-APWR DCD Revision 2 Table 2.0-1, Key Site Parameters, the last row specifies that the maximum tilt of R/B complex foundation generated during operational life of the plant is limited to1/2000. Given the size of the R/B foundation, B_{equiv}, as 240ft, 1/2000 of 240ft is 1.44 in. However, 1/3 of the maximum settlement specified is 2 in. which is larger than 1.44 in. MHI is requested to clarify this discrepancy. In the response to Part c of the question, MHI states that the stresses generated by the 2 in. differential settlement are not critical for the design of the mat. The staff accepts this answer; however, in the response, MHI did not address the effects of the 2 in. on the super structure and supported equipment. For example, the p-Δ effect on the structural members and the possibility of pounding between structures and supported equipment should be considered. MHI is requested to provide information that indicates these effects have been included in the study.

ANSWER:

1) Tilt, as defined in US-APWR DCD Tier 2, Table 2.0-1, is the rigid body rotation of the structure. Mat differential settlements depend on both tilt and mat flexibility. Therefore, the tilt and settlement parameters given in US-APWR DCD Tier 2 Table 2.0-1 are intended to represent two conditions that need to be checked independently of each other, to ensure that neither is exceeded. Moreover, the specified 2 in differential settlement is related with the overall structural integrity of the building and includes the differential settlement of the building generated during construction and operation of the plant. The 1/2000 tilt limitation is defined based on the demands for safe operation of NSSS components and includes the post construction differential settlements generated during the operation of the plant.

2) With respect to the effects of tilting of the structure to a maximum of 1/2000, it is to be noted, for example, that the height of the PCCV from the highest point to the bottom of the mat is 268' and the maximum height of R/B structure is 190' at the south side and 167' at the north side. On the south side, the adjacent portion of the Turbine Building extends approximately 90' above the base of its mat foundation. The lateral

displacement due to the maximum allowed rotation would be 1.6" at the top of the containment and 1.14" at the top of the R/B". The maximum lateral displacement due to tilt at the roof level of the Turbine building would be approximately 0.54", and the maximum lateral displacement due to tilt in the R/B at the elevation corresponding to the adjacent T/B roof would be approximately 0.58". There is a gap of 4" between adjacent buildings so there is ample space to assure that the buildings do not pound each other due to tilting. Moreover, the adequacy of 4" gap is required to be confirmed as part of the overall design for lateral displacements due to applicable loads including seismic loading, including effects due to tilt, as discussed above and in the response to RAI 497-3734 Question 03.08.04-33. With regard to supported equipment, story-to-story differential displacement is insignificant with respect to typical equipment installation clearances, and therefore typical clearances are sufficient to prevent pounding between equipment and walls.

3) In addition the P-Δ effect of the lateral deflection due to tilting of the structure (Max 1.14") for the columns in the R/B was considered in the verification of the columns and found to be of no consequence. The 1/2000 out of plumbness of the columns is within the acceptable limits. The P-Δ effect on the walls is even less severe and does not control nor has any effect on the design of the walls.

Impact on DCD

There is no impact on the DCD.

Impact on COLA

There is no impact on the COLA.

Impact on PRA

There is no impact on the PRA.

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

12/28/2010

US-APWR Design Certification

Mitsubishi Heavy Industries

Docket No. 52-021

RAI NO.:	**NO. 657-5135 REVISION 2**
SRP SECTION:	**03.08.05 – Foundations**
APPLICATION SECTION:	**3.8.5**
DATE OF RAI ISSUE:	**11/15/10**

QUESTION NO. RAI 03.08.05-41:

In the response to Question 03.08.05-33, MHI states that the value of the coefficient of friction, μ, for concrete-to-concrete friction ranges from 0.6 to 1.4 per ACI 349 Section 11.7. The value of μ=0.6 is for concrete placed against hardened concrete not intentionally roughened, while the value of 1.4 is for concrete placed monolithically. MHI further states that the construction sequence of the foundation for US-APWR will allow the use of the value of 0.7 and quotes the publication "State of the Art Report on Finite Element Analysis of Reinforced Concrete: ASCE," as the supporting document. The staff disagrees with this position since US-APWR is stated to be designed in accordance with ACI 349, and not a report of ASCE. MHI also states that at certain sites minor roughening of the fill concrete surface *may* be required. The staff finds that unless the requirement for a "roughened surface" is specified in DCD, a conservative value should be used in the analyses, i.e., μ=0.6. MHI is requested to specify "roughened surface" for the fill concrete to justify use of μ=0.7, or to use μ=0.6 in the analysis.

ANSWER:

MHI agrees with the NRC Staff's position to provide a "roughened surface" for fill concrete when using a coefficient of friction, μ, equal to 0.7 for calculating the shear (or sliding) resistance value, F_s. If a coefficient of friction of less than 0.7 is justified by the COL Applicant, "roughening" of the concrete is not required. The DCD will be modified as described below on "Impact on DCD" to incorporate these points.

Impact on DCD

See Attachment 1 for a mark-up of DCD Tier 2, Subsection 3.8.5 for changes to be incorporated.

- Add at the end of Subsection 3.8.5.5.2 the following paragraph:

  "When a coefficient of friction of 0.7 is used in calculating sliding resistance F_s, roughening of fill concrete is required per criteria given in Section 11.7.9 of ACI 349 (Reference 3.8-8). If a coefficient of friction of less than 0.7 is used by the COL Applicant, roughening of fill concrete is not required."

- Add COL 3.8(30) at the end of Subsection 3.8.6 to read as follows:

"COL 3.8(30) When a coefficient of friction of 0.7 is used in calculating sliding resistance F_s, roughening of fill concrete is required per criteria given in Section 11.7.9 of ACI 349 (Reference 3.8-8). If a coefficient of friction of less than 0.7 is used by the COL Applicant, roughening of fill concrete is not required."

See Attachment 2 for the mark-up of the DCD Tier 2, Section 1.8, changes to be incorporated.

- Add COL 3.8(30) above to Table 1.8-2 (sheet 12 of 44)

Impact on COLA

- Section 3.8.5.5 and 1.8 of the R-COLA will be revised to incorporate impacts on the FSAR text due to the DCD revisions and the new COL Item given above. A cross-reference will be added to the coefficient of friction of 0.6; roughening of fill concrete is not required for the R-COLA.

- Section 3.8.5.5 and 1.8 of the S-COLA will be revised to incorporate impacts on the FSAR text due to the DCD revisions and the new COL Item given above. A cross-reference will be added to the coefficient of friction of 0.7 for fill concrete cited in Table 2.5-212 of the S-COLA. Based on the coefficient of friction of 0.7, roughening of fill concrete is required for the S-COLA.

- Additional Sections will be revised in the R-COLA and S-COLA if required to align with the revised DCD text.

Impact on PRA

There is no impact on the PRA.

ATTACHMENT 1
to RAI 657-5135

3.8.4.7.1 Construction Inspection

Inspections relating to the construction of seismic category I and II SSCs are conducted in accordance with the codes applicable to the construction activities and/or materials. In addition, weld acceptance is performed in accordance with the NCIG, Visual Weld Acceptance Criteria for Structural Welding at Nuclear Power Plants, NCIG-01, Revision 2 (Reference 3.8-31).

3.8.5 Foundations

3.8.5.1 Description of the Foundations

Each building is isolated on a separate concrete basemat as identified in Subsection 3.8.4. The PCCV and the containment internal structure are integral with the R/B on a common basemat. Adjoining building basemats, such as the east and west PS/Bs, A/B, and T/B, are structurally separated by a 4 in. gap at and below the grade. This requirement does not apply to engineered mat fill concrete that is designed to be part of the foundation subgrade.

Basemats are located at a depth below the zone of maximum frost penetration, taken as 4 ft below grade. The COL Applicant is to determine if the site-specific zone of maximum frost penetration extends below the depth of the basemats for the standard plant, and to pour fill concrete under any basemat above the frost line so that the bottom of fill concrete is below the maximum frost penetration level.

3.8.5.1.1 Reactor Building and Enveloped Structures

The R/B, with the PCCV and containment internal structure at its center, is built on a common basemat and isolated from the adjacent A/B, east and west PS/Bs, and T/B. The basemat of the R/B is essentially a rectangular-shaped reinforced concrete mat and is composed of two parts. One part of the basemat is for the PCCV and containment internal structure, and the other part is for the remaining seismic category I basemat for the R/B. The length of the basemat in the north-south direction is 309 ft, 0 in., and in the east-west direction is 210 ft, 0 in, as shown in Figure 3J-1. The central region, generally circular with a diameter of approximately 187 ft~~188 ft, 0 in.~~, supports the PCCV and containment internal structure with a thickness of approximately ~~varying from 11 ft, 7 in. to~~ 38 ft, 2 in. The peripheral portion which supports the R/B is 9 ft, 11 in. thick.

The basemat includes hollow portions such as the tendon gallery, tendon gallery access tunnel, and other portions such as in-core chase and CV recirculation sump. Since the vertical tendons are anchored at the roof of the tendon gallery, the upper part of the tendon gallery is important from the structural point of view.

The basemat reinforcement consists of a top horizontal layer of reinforcement, a bottom horizontal layer of reinforcement, and vertical shear reinforcement. The bottom layer of reinforcement is arranged in a rectangular grid. The top layer of reinforcement is arranged in a rectangular grid at the center of the mat and radiates outward in a polar pattern in order to avoid interference with PCCV reinforcement. The top and bottom reinforcement at the upper portion of the tendon gallery is in a polar pattern.

ATTACHMENT 1
to RAI 657-5135

F_h = Lateral force due to active soil pressure, including surcharge, and tornado or wind load, as applicable

The factor of safety against sliding caused by earthquake is identified by the ratio:

$FS_{se} = [F_s + F_p] / [F_d + F_h]$, not less than FS_{sl} as determined from Table 3.8.5-1

where

FS_{se} = Structure factor of safety against sliding caused by earthquake

F_s = Shear (or sliding) resistance along bottom of structure basemat

F_p = Resistance due to maximum passive soil pressure, neglecting any contribution of surcharge. No credit is taken for passive soil pressure in calculating the factor of safety against sliding in standard plant building structures.

F_d = Dynamic lateral force, including dynamic active earth pressures caused by seismic loads

F_h = Lateral force due to all loads except seismic loads

When a coefficient of friction of 0.7 is used in calculating sliding resistance F_s, roughening of fill concrete is required per criteria given in Section 11.7.9 of ACI 349 (Reference 3.8-8). If a coefficient of friction of less than 0.7 is used by the COL Applicant, roughening of fill concrete is not required.

3.8.5.5.3 Flotation Acceptance Criteria

The factor of safety against flotation is identified as the ratio of the total dead load of the structure including foundation (D_r) divided by the buoyant force (F_b). Therefore,

$FS_f = D_r / F_b$, not less than FS_{fl} as determined from Table 3.8.5-1.

where

FS_f = Structure factor of safety against flotation by the maximum design basis flood or ground water table.

D_r = Total dead load of the structure including foundation.

F_b = Buoyant force caused by the design basis flood or high ground water table, whichever is greater.

3.8.5.6 Materials, Quality Control, and Special Construction Techniques

Subsection 3.8.4.6 describes the materials, quality control, and special construction techniques applicable to seismic category I foundations, including water control structures and below-grade concrete walls and foundations. Subsection 3.8.1.7 provides testing and surveillance requirements relating to the PCCV basemat.

COL 3.8(25) *The site-specific COL are to assure the design criteria listed in Chapter 2, Table 2.0-1, is met or exceeded.*

COL 3.8(26) *Subsidence and differential displacement may therefore be reduced to less than 2 in. if justified by the COL Applicant based on site specific soil properties.*

COL 3.8(27) *The COL Applicant is to specify normal operating thermal loads for site-specific structures, as applicable.*

COL 3.8(28) *The COL Applicant is to specify concrete strength utilized in non-standard plant seismic category I structures.*

COL 3.8(29) *The COL Applicant is to provide design and analysis procedures for the ESWPT, UHSRS, and PSFSVs.*

COL 3.8(30) *When a coefficient of friction of 0.7 is used in calculating sliding resistance F_s, roughening of fill concrete is required per criteria given in Section 11.7.9 of ACI 349 (Reference 3.8-8). If a coefficient of friction of less than 0.7 is used by the COL Applicant, roughening of fill concrete is not required.*

3.8.7 References

3.8-1 Combined License Applications for Nuclear Power Plants, RG 1.206, Rev. 0, U.S. Nuclear Regulatory Commission, Washington, DC, June 2007.

3.8-2 Rules for Construction of Nuclear Facility Components, Division 2, Concrete Containments. Section III, American Society of Mechanical Engineers, 2001 Edition through the 2003 Addenda (hereafter referred to as ASME Code).

3.8-3 Design Limits, Loading Combinations, Materials, Construction, and Testing of Concrete Containments. RG 1.136, U.S. Nuclear Regulatory Commission, Washington, DC, Revision 3, March 2007.

3.8-4 Rules for Inservice Inspection of Nuclear Power Plant Components. Section XI, American Society of Mechanical Engineers, 2001 Edition through the 2003 Addenda.

3.8-5 Inservice Inspection of Ungrouted Tendons in Pre-stressed Concrete Containments. RG 1.35, U.S. Nuclear Regulatory Commission, Washington, DC, Revision 3, July 1990.

3.8-6 Determining Pre-stressing Forces for Inspection of Pre-stressed Concrete Containments. RG 1.35.1 U.S. Nuclear Regulatory Commission, Washington, DC, July 1990.

3.8-7 Concrete Containment, Design of Structures, Components, Equipment, and Systems, Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants. NUREG-0800 SRP 3.8.1, U.S. Nuclear Regulatory Commission, Washington, DC, March 2007.

Table 1.8-2 Compilation of All Combined License Applicant Items for Chapters 1-19 (sheet 12 of 44)

COL ITEM NO.	COL ITEM
COL 3.8(22)	*The COL Applicant is to establish a site-specific program for monitoring and maintenance of seismic category I structures in accordance with the requirements of NUMARC 93-01 (Reference 3.8-28) and 10 CFR 50.65 (Reference 3.8-29) as detailed in RG 1.160 (Reference 3.8-30). For seismic category I structures, monitoring is to include base settlements and differential displacements.*
COL 3.8(23)	*The COL Applicant is to determine if the site-specific zone of maximum frost penetration extends below the depth of the basemats for the standard plant, and to pour fill concrete under any basemat above the frost line so that the bottom of fill concrete is below the maximum frost penetration level.*
COL 3.8(24)	*Other non-standard seismic category I buildings and structures of the US-APWR are designed by the COL Applicant based on site-specific subgrade conditions.*
COL 3.8(25)	*The site-specific COL are to assure the design criteria listed in Chapter 2, Table 2.0-1, is met or exceeded.*
COL 3.8(26)	*Subsidence and differential displacement may therefore be reduced to less than 2 in. if justified by the COL Applicant based on site specific soil properties.*
COL 3.8(27)	*The COL Applicant is to specify normal operating thermal loads for site-specific structures, as applicable.*
COL 3.8(28)	*The COL Applicant is to specify concrete strength utilized in non-standard plant seismic category I structures.*
COL 3.8(29)	*The COL Applicant is to provide design and analysis procedures for the ESWPT, UHSRS, and PSFSVs.*
COL 3.8(30)	*When a coefficient of friction of 0.7 is used in calculating sliding resistance F_s, roughening of fill concrete is required per criteria given in Section 11.7.9 of ACI 349 (Reference 3.8-8). If a coefficient of friction of less than 0.7 is used by the COL Applicant, roughening of fill concrete is not required.*
COL 3.9(1)	*The COL Applicant is to assure snubber functionality in harsh service conditions, including snubber materials (e.g., lubricants,*