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TOKYO, JAPAN

July 3, 2009

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- 09360

Subject: MHI's Responses to US-APWR DCD RAI No. 342-2000

Reference: 1) "Request for Additional Information No. 342-2000 Revision 0, SRP Section: 03.08.04 – Other Seismic Category I Structures, Application Section: 3.8.4," dated 4/21/2009.

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. 342-2000, Revision 0."

Enclosed are the responses to 31 RAIs contained within Reference 1.

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,

Y. Ogata

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

Enclosure:

1. Responses to Request for Additional Information No. 342-2000, Revision 0

CC: J. A. Ciocco
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NRO

Contact Information

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Enclosure 1

UAP-HF- 09360
Docket No. 52-021

Responses to Request for Additional Information No. 342-2000, Revision 0

July, 2009

RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

7/03/2009

US-APWR Design Certification

Mitsubishi Heavy Industries

Docket No. 52-021

RAI NO.:	**NO. 342-2000 REVISION 0**
SRP SECTION:	**3.8.4 – Other Seismic Category I Structures**
APPLICATION SECTION:	**03.08.04**
DATE OF RAI ISSUE:	**4/21/2009**

QUESTION NO.: 3.8.4-1

In DCD Subsection 3.8.4.1, the first paragraph (Page 3.8-45) states, "Adjoining building basemats 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 basemat subgrade for the interface between the R/B, and east and west PS/Bs. To be consistent with seismic modeling requirements of Section 3.7, no 4 in. gap is permitted in the fill concrete between these buildings."

The applicant is requested to provide the following information:

(a) Provide a description for the engineered mat fill concrete, including its dimensions and thickness and concrete strength.

(b) The last sentence of the above quote states that "To be consistent with ... no 4 in gap is permitted ..." What are the specific seismic modeling requirements of DCD Section 3.7 that make it necessary to eliminate the 4 in. gap between basemats of certain buildings?

ANSWER:

(a) A thin layer of fill concrete is placed before the start of construction of the basemat to provide a level surface that matches the bottom-of-foundation elevations. In cases where over-excavation is required in order to reach suitable materials and/or materials with uniform stiffness, the thickness of the layer of fill concrete is increased accordingly. The strength of the fill concrete is selected based on the site-specific properties of the subgrade. In order to perform the seismic response analysis required by Subsection 3.7.2.4.1, the COL Applicant is to address these site-specific aspects of the fill concrete design, including the dimensions, thickness, and strength of the concrete. The site-specific SASSI analysis discussed in DCD Subsection 3.7.2.4.1 will address the effect of the fill concrete on the seismic response, considering the horizontal extent of the fill concrete as discussed in (b) below.

(b) The standard seismic analyses presented in Subsection 3.7.2 of the DCD and the standard design of the foundations presented in Subsection 3.8.5 of the DCD assume that the foundations of the US-APWR Category I buildings rest directly on any of the surfaces of the four generic subgrades considered. The engineered mat fill concrete is considered as part of the basemat subgrade, thus implying that the layer of fill concrete is horizontally infinite and has the same material properties as the supporting subgrade. Therefore, in order to ensure the applicability of the simplifying modeling assumption of horizontally infinite fill concrete layer used in the standard plant design, no 4 in. gap is permitted.

In accordance with Subsection 3.7.2.4.1 of the DCD, the COL Applicant must verify the applicability of the standard design for the site specific conditions by performing site-specific soil-structure interaction analyses to demonstrate that the site-specific effects are enveloped by the standard design. The site-specific analyses have to consider the effects of fill concrete thickness and stiffness on the seismic response of the building. Based on the site-specific demands and design solutions, the fill concrete can be considered either as part of the subgrade (same approach as standard plant) or as part of the structural model for SSI analyses. If, in the site-specific design, the thickness and horizontal extent of the fill concrete away from the edge of the foundation is sufficient to fulfill the standard plant modeling approach, the fill concrete can be included in the site model as a horizontally infinite layer. In that case, FIRS are developed representing the site response conditions at top of the fill concrete layer. If the horizontal extent of the fill concrete does not allow the assumption of an infinite layer, the site-specific soil-structure interaction analyses have to model the fill concrete under the category I foundations as part of the structural model.

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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SRP SECTION:	**3.8.4 – Other Seismic Category I Structures**
APPLICATION SECTION:	**03.08.04**
DATE OF RAI ISSUE:	**4/21/2009**

QUESTION NO.: 3.8.4-2

DCD Section 3.8.4.1, the second paragraph (Page 3.8-45) states, "The minimum gaps between building superstructures is two times the absolute sum of the maximum displacement of each building under the most unfavorable load combination, or a minimum of 4 in."

The applicant is requested to provide the following information:

How was the maximum displacement of each building calculated? Was it based on the elastic analysis? Per ASCE/SEI 7-05 Section 12.8.6, this displacement needs to be amplified by the deflection amplification factor, C_d. Was the deflection amplification factor considered in your calculation? If yes, what is the value used. If not, provide the technical basis for not using it. Also, was the effect of the differential settlement at the basemat included in the maximum displacement calculation?

ANSWER:

For the US-APWR seismic category I and II building structures discussed in DCD Subsection 3.8.4, the minimum gaps between building superstructures are calculated as the absolute sum of the maximum seismic and static lateral displacements of the adjacent buildings. The time history seismic response analyses described in Section 3.7 of the DCD provide the maximum seismic displacement of the buildings at lumped mass nodes located at the major floor elevations of the building. The seismic response analyses use linear elastic stick elements to model the stiffness properties of the structures and lumped soil-structure interaction (SSI) parameters to model the interaction of the foundation with the subgrade. The time history analyses provide seismic displacements that include both the elastic deformation of the structure (inter-story drifts) and the foundation displacements and rotations. The envelope of the maximum displacement results obtained from the analyses of the four generic soil cases are used as input for the evaluation of the gaps between buildings. The static lateral displacements include the effects of the foundation differential settlements generated by the combined dead and live loads (D+L). In the response to Open Item RGS1 2.5.4, Table 2.1-1 of DCD Tier 1 and Table 2.0-1 of DCD Tier 2 are revised to provide a differential settlement of 2 in. across the length of the Reactor Building (R/B) complex foundation. This reference value is used to calculate the maximum static displacements for the R/B complex foundation.

The seismic displacements calculated from the lumped-mass stick soil-structure interaction (SSI) models used for the seismic response analyses of the US-APWR seismic category I and II building structures are not amplified by the deflection factor C_d specified in ASCE/SEI 7-05 Section 12.8.6. The methodology used for the seismic analyses of US-APWR seismic category I and II building is based on the requirements of SRP 3.7.1 and 3.7.2, which are different from and more stringent than those of ASCE/SEI 7-05 for seismic design of commercial and residential buildings. Further justification is provided as follows through general discussion of the seismic response analysis approaches of ASCE/SEI 7-05, SRP 3.7.1, and SRP 3.7.2:

1) The seismic input motion used for the seismic analysis and design of the US-APWR standard plant is based on generic ground motion response spectra that are required to be confirmed as valid for each particular plant site. This is accomplished by developing site-specific ground motion spectra as stated in DCD Subsection 3.7.1.1, and comparing the site-specific spectra to the standard plant spectra. The site specific spectra is developed by an elaborate probabilistic seismic hazard analysis (PSHA) using either the reference probability method of RG 1.165 or the performance-based method of RG 1.208 and considering mean annual probability of exceedance lower than or equal to 10^{-4}. Comprehensive geological, seismological, geophysical, and geotechnical investigations of the site and regions around the site are performed, and uncertainties related to identification and characterization of the seismic sources are evaluated. The site-specific ground motion spectra are developed at the ground surface considering site-specific soil amplifications. ASCE/SEI 7-05 code seismic design is based on design ground motion with mean annual probability of exceedance of approximately 10^{-2}. The design ground motion is defined by mapped spectral values without explicit consideration of site-specific geological, seismological, geophysical or geotechnical conditions. The ASCE/SEI 7-05 design approach relies on amplification factors (such as the deflection amplification factor, importance factor, etc.) and empirical relations to account for any uncertainties related to identification and characterization of seismic sources, site-specific soil amplifications, and the lower intensity of the considered design ground motion.

2) ASCE/SEI 7-05 provisions do not contain explicit requirements for SSI analysis. The seismic design is based on linear elastic analyses of fixed-base models as indicated in Figure 12.8-2 that do not include the displacements of the building foundation due to SSI translations and rotations. The deflection amplification factor C_d, and other factors specified in Chapter 12 of ASCE/SEI 7-05, serve to account for the SSI effects on the building displacements results obtained from the dynamic analyses of elastic fixed-base models. However, the US-APWR seismic design specifically considers SSI effects in accordance with the more rigorous requirements of SRP 3.7.2. The seismic analyses that form the basis for the US-APWR standard seismic design neglect the effect of the foundation embedment, and thus also provide building displacements that take into account the effects of the foundation translations and rotations in a conservative manner. The comparison of maximum displacement results of the seismic response analyses presented in Table 3H.3-11 and Table 3H.3-14 of DCD Appendix H indicate that the envelope maximum displacements of the R/B are mainly due to translation and rotation of the R/B complex common foundation. The contribution of the SSI effects on the overall maximum displacement of the building is at least 2.5 times the contribution of the elastic deformations of the structure. This value is based on comparison of the displacments for soft soil subgrade condition versus the displacements of the hard rock (fixed base) subgrade condition.

Considering the above differences in defining the design ground motion and methodologies used for seismic response analyses, it is deemed that the provisions of Section 12.8.6 of ASCE/SEI 7-05 for calculation of seismic displacements of commercial and residential buildings are not applicable for the standard seismic design of US APWR seismic category I and II buildings discussed in DCD Section 3.8.4.

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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SRP SECTION:	**3.8.4 – Other Seismic Category I Structures**
APPLICATION SECTION:	**03.08.04**
DATE OF RAI ISSUE:	**4/21/2009**

QUESTION NO.: 3.8.4-3

In DCD Subsection 3.8.4.1.1, the second paragraph (Page 3.8-45) states, "The R/B consists of the following five areas, defined by their functions.

- PCCV and containment internal structure
- Safety system pumps and heat exchangers area
- Fuel handling area
- Main steam and feed water area
- Safety-related electrical area"

The applicant is requested to provide the following information:

(a) PCCV and containment internal structures are not part of the R/B. Clarify the above quoted statement of the first bullet.
(b) Provide floor plans for each of the four areas in the above list (excluding the PCCV and containment internal structure).

ANSWER:

(a) The DCD will be revised to delete PCCV and containment internal structure from the bulleted list in DCD Subsection 3.8.4.1.1, and the first paragraph of DCD Subsection 3.8.4.1.1 will be clarified accordingly. Note that the upper roof elevation of the R/B is also corrected to elevation 154'-6". The shape description of the R/B is also changed from "nearly square" to "basically rectangular".

(b) The floor plans for the areas cited in the question are provided in Chapter 1 of Tier 2 of the DCD and in the general arrangements shown in MHI US-APWR Drawing No. 4CS-UAP-20070026, Revision 4.

Impact on DCD

See Attachment 1 for a mark-up of DCD Tier 2, Section 3.8 changes to be incorporated.

- Revise the first paragraph of Subsection 3.8.4.1.1 to the following:

"The R/B has five main floors. In plan, the R/B surrounds the PCCV and containment internal structure, and is founded with those structures on a single common basemat. The outer perimeter of the R/B is basically rectangular, and is constructed of reinforced concrete walls, floors, and roofs. In cross-section, the height of the R/B varies from roof elevation 101 ft, 0 in. to 154 ft, 6 in., and the PCCV extends above the R/B to elevation 232 ft, 0 in."

- Revise the second paragraph of Subsection 3.8.4.1.1 to the following:

  "The R/B consists of the following areas, defined by their functions.

  - Safety system pumps and heat exchangers area
  - Fuel handling area
  - Main steam and feed water area
  - Safety-related electrical area"

Impact on COLA

There is no impact on the COLA.

Impact on PRA

There is no impact on the PRA.

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QUESTION NO.: 3.8.4-4

In DCD Subsection 3.8.4.3.2, the first paragraph (Page 3.8-49) states, "Hydrodynamic loads due to seismic sloshing are calculated per ASCE Standard 4-98...."

The applicant is requested to provide the following information:

Were the hydrodynamic loads associated with the impulsive mode included? (Note that impulse mode is that mode in which a portion of the water moves in unison with the tank, wall, and is not due to sloshing.) If yes, provide information for how they were calculated. If not, explain why they were not considered.

ANSWER:

Hydrodynamic loads associated with the impulsive mode are included in the relevant analyses, as discussed in DCD Subsection 3.7.3.9. As clarified in the response to question 3.7.2-01 of RAI 212-1950, Revision 1, the methods used for the calculation of the hydrodynamic loads are consistent with the provisions of SRP 3.7.3, Subsection II.14 and the guidance of ASCE 4-98. Further explanation is provided as follows.

The Housner method contained in TID-7024 is used for computing the impulsive and convective pressure loads. The Housner method is considered appropriate because wall flexibility does not affect the validity of the loads determined when considering fluid-structure interaction of the impulsive mode. This is because the reinforced concrete walls of the cavities and pits identified in Subsection 3.7.3.9 of the DCD are rigid portions of the building structures. However, for the formulas shown in TID-7024, different coordinate systems are used for the impulsive pressure and convective pressure; in addition, TID-7024 does not specifically address how to define the input motion when the response analysis results of the building are used as input. Therefore, the formulas of TID-7024 are adjusted for performing the design as shown in Table 1 and Figure 1 below.

Table 1 lists the various formulas for calculating the pressure distributions $\left({}_{I}P_{W}, {}_{I}P_{S}, {}_{C}P_{W}, {}_{C}P_{S}\right)$ on the side wall (at $x = \pm L$) and bottom (at y = 0) of the cavity or pit.

Table 1 Hydrodynamic pressure equations for rectangular pools

	Side wall	Base plate
Impulsive Pressure	${}_I P_W = \rho L \ddot{X} \frac{\sqrt{3}}{2} \frac{H}{L}\left[1-\left(\frac{y}{H}\right)^2\right]\tanh\left(\sqrt{3}\frac{L}{H}\right)$	${}_I P_S = \rho L \ddot{X} \frac{\sqrt{3}}{2} \frac{H}{L} \frac{\sinh\left(\sqrt{3}\frac{x}{H}\right)}{\cosh\left(\sqrt{3}\frac{L}{H}\right)}$
Convective Pressure	${}_C P_W = \rho L S_A(\omega_1)\frac{10}{12}\frac{\cosh\left(\sqrt{\frac{5}{2}}\frac{y}{L}\right)}{\cosh\left(\sqrt{\frac{5}{2}}\frac{H}{L}\right)}$	${}_C P_S = \rho L S_A(\omega_1)\frac{5}{4}\left[\frac{x}{L}-\frac{1}{3}\left(\frac{x}{L}\right)^3\right]\frac{1}{\cosh\left(\sqrt{\frac{5}{2}}\frac{H}{L}\right)}$

The variables in Table 1 are defined as:

$\omega_1 = \sqrt{\sqrt{\frac{5}{2}}\frac{g}{L}\tanh\left(\sqrt{\frac{5}{2}}\frac{H}{L}\right)}$: fundamental angular frequency of free water

ρ : mass of liquid per unit width $(tf \cdot s^2 / m^2)$

h : depth of liquid

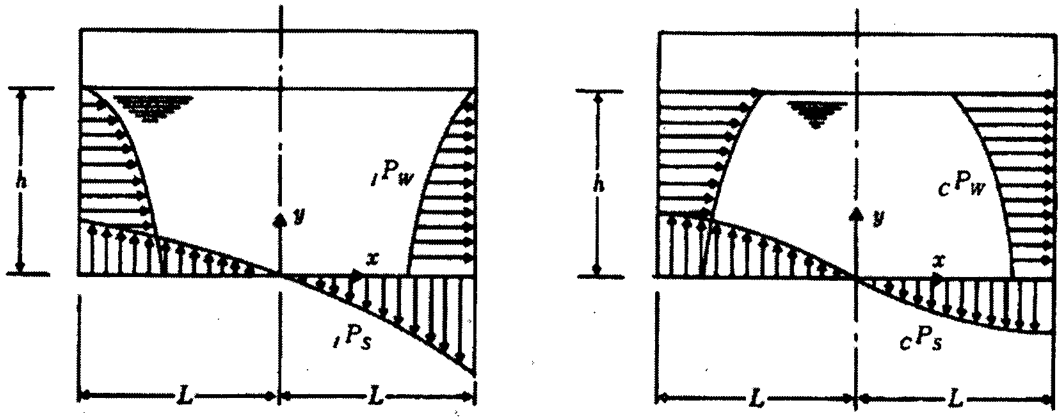
H : $H = h(h \le 1.5L), H = 1.5L(h > 1.5L)$

$2L$: width of the rectangular pool (m)

$\ddot{X}$: maximum response acceleration of the floor on which the pool is setting, or the average maximum response acceleration of the base floor and the upper-story floor (m/s^2)

$S_A(\omega_1)$: floor response spectrum for ω_1 for $\ddot{X}(t)$

g : gravitational acceleration (m/s^2)



(a) Impulsive pressure distribution profile **(b) Convective pressure distribution profile**

Figure 1 Dynamic hydraulic pressure distribution

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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QUESTION NO.: 3.8.4-5

In DCD Subsection 3.8.4.3.3, it (Page 3.8-50) states, "The dynamic soil pressure, induced during an SSE event, is considered as an earthquake load E_{ss}."

The applicant is requested to describe how the dynamic soil pressure, E_{ss}, was calculated. Was the soil considered as fully saturated to account for ground and flood water levels?

ANSWER:

A description of the calculation of dynamic soil pressure is provided in the response to Question 3.7.2-13, Item 4, of RAI 212-1950, Revision 1. As noted in that response, the soil was considered fully saturated.

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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QUESTION NO.: 3.8.4-6

The subject of DCD Subsection 3.8.4.3.4 is live loads and different live loads are listed from DCD Subsection 3.8.4.3.4.1 to DCD Subsection 3.8.4.3.4.9.

The applicant is requested to provide the following information:

There is no roof live load in the list. Per ASCE 7-05 (note that ASCE 7-05 is listed in subsection 3.8.4.2 as one of the Applicable Codes, Standards, and Specifications for US-APWR), roof live load is a load on roof produced during maintenance by workers, equipment and materials, and a minimum roof live load in addition to the snow load should be specified and included in the analysis. Provide a rationale for not including any roof live load, or to specify the magnitude of roof live load.

ANSWER:

MHI agrees to specify a roof live load in the DCD. Consistent with Chapter 2 of ASCE 7-05, the roof live load is not added to snow loads in evaluating design load combinations. Subsection 3.8.4.3.4.2 of the DCD will be revised as shown below in "Impact to DCD" to specify a roof live load of 40 psf. The revision shown below includes changes previously made to Subsection 3.8.4.3.4.2 in response to question 2.3.1-16 of RAI 59-1086, Revision 0.

Impact on DCD

See Attachment 1 for a mark-up of DCD Tier 2, Section 3.8 changes to be incorporated.

- Replace Subsection 3.8.4.3.4.2 with the following:

  "**3.8.4.3.4.2 Roof Snow Loads and Roof Live Loads**

  The roof is designed for uniform snow live load as specified in Chapter 2. Normal winter precipitation roof loads are added to all other live loads that may be expected to be present at the time to determine the design live load on the roof, and include appropriate load factors in applicable loading combinations. The extreme winter precipitation roof load is included as live load in extreme loading combinations using the applicable load factor. Other extreme environmental loads (e.g., seismic and tornado loads) are not considered as occurring simultaneously. Slope roof snow loads, partially loaded, unbalanced roof snow loads, and drifts (including sliding snow) on lower roofs, as applicable, are determined in accordance with ASCE 7-05 (Reference 3.8-35).

  The roof design accommodates a roof live load of 40 psf to account for loads produced by workers, equipment, and materials. Roof live load is not added to roof snow load when evaluating design load combinations."

Impact on COLA

There is no impact on the COLA.

Impact on PRA

There is no impact on the PRA.

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QUESTION NO.: 3.8.4-7

In DCD Subsection 3.8.4.3.4.7, the second paragraph (Page 3.8-52) states, "Impact allowance for traveling crane supports and runway horizontal forces are in accordance with AISC N690 (Reference 3.8-9) for seismic category I and II structures, unless the crane manufacturer's design specifies higher impact loads. The vertical live load is increased by 25% to account for vertical impact of cab-operated traveling cranes and 10% of pendant-operated traveling cranes. A lateral force, equal to 20% of the lifted load and crane trolley are applied at the top and perpendicular to the crane rails. A longitudinal force equal to 10% of the maximum wheel load is applied at the top of the rails."

The applicant is requested to provide the following information:

(a) Per AISC N690, the crane runway shall also be designed for crane stop forces. Explain why these impact forces were not included.
(b) provide information for the deflection criteria used for the crane runway.

ANSWER:

(a) MHI agrees that crane stop impact forces are to be considered in the runway design as required by AISC N690-1994. The discussion in DCD Subsection 3.8.4.3.4.7 will be revised accordingly as described below.

(b) Deflection criteria used for standard plant crane runways are in accordance with CMAA 70. Vertical deflection of crane runways is limited to (span length/600) based on maximum wheel loads without impact, and lateral deflection is limited to (span length/400) based on a lateral load of 10% of the maximum wheel loads without impact. In addition, building frame lateral deflection due to crane or other gravity loads is sufficiently limited to prevent undue distortion of the runway to ensure proper crane operation.

Impact on DCD

See Attachment 1 for a mark-up of DCD Tier 2, Section 3.8 changes to be incorporated.

- Add the following sentence as the last sentence in the second paragraph of Subsection 3.8.4.3.4.7: "Crane runways are also designed for crane stop forces."

Impact on COLA

There is no impact on the COLA.

Impact on PRA

There is no impact on the PRA.

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QUESTION NO.: 3.8.4-8

In DCD Subsection 3.8.4.3.6.2, the second paragraph states that, "In addition to the dead load, 25% of the floor live load during normal operation or 75% of the roof snow load, whichever is applicable, is also considered as accelerated mass in the seismic models."

The applicant is requested to provide the following information:

For the live load to be considered in the seismic model, ASCE 4-98 subsection C3.1.4.2 (Page 64) refers to the Earthquake Loads Section of ASCE Standard 7. Item 4 of subsection 12.7.2 (Page 128) of ASCE 7-05 states that 20% of the roof snow load shall be included if the roof snow load exceeds 30 psf. This snow load should be added to the 25% of the live load (note that this is "and" not "or"). Provide the technical basis for not following the ASCE recommendation.

ANSWER:

The technical basis for the second paragraph of DCD Subsection 3.8.4.3.6.2 is SRP 3.7.2 Acceptance Criterion II.3.D, which states that "mass equivalent to 25 percent of the floor design live load and 75 percent of the roof design snow load, as applicable, should be included." MHI's understanding of SRP 3.7.2 Acceptance Criterion II.3.D is that for purposes of dynamic modeling, 25% of the floor design live load is applicable to floors, and 75% of the roof design snow load is applicable to roofs. The US-APWR dynamic models therefore include mass equivalent to 25% of the floor live load on floors and mass equivalent to 75% of the roof snow load on roofs, applied concurrently. Mass equivalent to a floor load of 50 psf is also included as described in DCD Subsection 3.8.4.3.1.2. Neither Section 12.7.2 nor Chapter 2 of ASCE 7-05 requires roof live load to be evaluated concurrently with roof snow load in the design load combinations. Roof live load is discussed further in the response to question 3.8.4-6 of this RAI and roof snow load is discussed further in the response to question 2.3.1-16 of RAI 59-1086, Revision 0.

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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QUESTION NO.: 3.8.4-9

In DCD Subsection 3.8.4.3.4.3, the first paragraph (Page 3.8-51) states, "Roof rain load is accounted for in accordance with Chapter 8 of ASCE 7-05...Subsection 3.4.1.2 provides additional discussion of design features to limit ponding of rain on the roofs of plant buildings."

The applicant is requested to provide the following information:

In DCD Subsection 3.4.1.2, it is stated that sloped roofs are designed to preclude roof ponding. What is the value of the roof slope specified in the design?

ANSWER:

The roofs of the seismic category I R/B and PS/Bs are designed as flat roofs in accordance with ASCE 7 with a roof slope of not less than ¼" per foot. This is a general requirement which meets the provisions of ASCE 7 with regard to minimizing the potential for ponding, and exceptions to this minimum slope may occur in small roof areas or niches due to geometry or spacing limits. In all areas where drainage off the roof may be constricted due to antecedent snowpack, investigations are performed in accordance with ASCE 7 and ISG-07 to assure that instability due to progressive ponding will not occur. Snow and precipitation loads with respect to potential ponding are determined in accordance with ASCE 7 and ISG-07 guidance, as addressed in the response to RAI 59-1086, Revision 0.

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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7/03/2009

US-APWR Design Certification

Mitsubishi Heavy Industries

Docket No. 52-021

RAI NO.:	**NO. 342-2000 REVISION 0**
SRP SECTION:	**3.8.4 – Other Seismic Category I Structures**
APPLICATION SECTION:	**03.08.04**
DATE OF RAI ISSUE:	**4/21/2009**

QUESTION NO.: 3.8.4-10

In DCD Subsection 3.8.4.3.6.2, the third paragraph (Page 3.8-53) gives a load combination equation which is shown below.

$$1.0D+(1.0L \text{ or } 0.75\ S)+\ a_v\ (D+0.5\ (L \text{ or } S))$$

The applicant is requested to (1) explain the meaning of a_v (vertical seismic acceleration) and its value, and (2) provide the technical basis for this equation.

ANSWER:

(1) The magnitudes of the seismic design loads are developed using the coefficient a_v representing vertical seismic acceleration, for which the values are determined from the results of seismic response analyses. This process is discussed in Subsection 3.7.2 of the DCD and in the response to Question 3.8.4-11 of this RAI. In accordance with SRP 3.7.2, Section II.3.D and as stated in DCD Subsection 3.8.4.3.6.2, seismic dynamic analysis models for seismic category I buildings include 25% of the floor live load or 75% of the roof snow load, whichever is applicable, for determining the effective seismic weight. For the local design of members loaded individually, such as the floors and beams, seismic member forces include the vertical response due to masses equal to 50% of the specified floor live loads instead of 25% of floor live load used in the seismic dynamic model. For members with snow loads, there is no need to increase the weight considered since 75% of the snow load is already included in the dynamic model.

(2) The intent of the equation in the DCD is to provide conservatism in terms of strength/capacity in the structural design of certain slabs and beams subjected to seismic load due to the effective floor live loads. The additional load is applied on selected members and is considered conservative because it introduces margin in the structural design beyond that which is achieved by following the requirements of ASCE 7 Section 12.7.2 and SRP Section II.3.D. Further, consistent with recommendations in ASCE 4, Section C3.1.4.2, when full live loads are always expected to be present, the percentage of live load is increased up to 100% for the affected members.

Based on the explanations provided above, the DCD will be revised by deleting the first portion of the equation, since the combination of loads is addressed in Tables 3.8.4-3 and 3.8.4-4. Further, the term "a_v (D+0.5 (L or S))" was intended to define the seismic design load acting in the vertical direction. However, based on the explanations provided above, the equation will be shortened to show only the increase in member load. A statement will also be added explaining that the percentage of live load may be increased beyond 50% for members in locations where live loads are expected to be always present. The revised formula and modified portion of DCD Subsection 3.8.4.3.6.2 are given below in "Impact on DCD".

Impact on DCD

See Attachment 1 for a mark-up of DCD Tier 2, Section 3.8 changes to be incorporated.

- Change the third paragraph of Subsection 3.8.4.3.6.2 and associated formula to:

  "For the local design of members loaded individually, such as the floors and beams, seismic member forces include the vertical response due to masses equal to 50% of the specified floor live loads instead of 25% of floor live load, as follows:

  $$a_v(0.5L)$$

  where

  a_v = Vertical seismic acceleration obtained from the seismic dynamic analysis results

  L = Floor live load per Subsection 3.8.4.3.4

- Add the following statement after the above-shown text:

  "In locations where live loads are expected to always be present, the_percentage of live load acting as accelerated mass is increased up to 100% of the live load for the affected members."

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

7/03/2009

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Docket No. 52-021

RAI NO.:	**NO. 342-2000 REVISION 0**
SRP SECTION:	**3.8.4 – Other Seismic Category I Structures**
APPLICATION SECTION:	**03.08.04**
DATE OF RAI ISSUE:	**4/21/2009**

QUESTION NO.: 3.8.4-11

In DCD Subsection 3.8.4.4.1, the fourth paragraph (Page 3.8-56) states, "Seismic forces are obtained from the dynamic analysis of the three-dimensional lumped-mass stick model described in Subsection 3.7.2. These loads are applied to the linear elastic FE model fixed at elevation 3 ft. 7 in. as equivalent static forces."

The applicant is requested to provide the following information:

(a) Seismic forces obtained from the dynamic analysis of the 3-D lumped-mass stick model are in the time domain. Explain how forces in the time history response are converted to equivalent static forces.

(b) Where are the equivalent static forces applied at the FE model? Are these concentrated forces or distributed stresses? Explain how the equivalent static forces from the lumped-mass stick model are mapped into the 3-D Finite Element model.

(c) Provide the technical basis for using the fixed base boundary condition at elevation 3 ft, 7 in. for the FE model; whereas, the seismic forces are obtained from the dynamic analysis of the three-dimensional lumped-mass stick model that is elastically supported at the base.

ANSWER:

(a) Equivalent static forces are developed for each major floor elevation in two horizontal and vertical directions to serve as the input for safety shutdown earthquake (SSE) design loads for the design of the structural members. The seismic response analyses of the reactor building (R/B), prestressed concrete containment structure (PCCV) and the containment internal structure presented in Subsection 3.7.2 of the DCD are performed on lumped mass stick models where the stiffness of the structural members between two floor elevations is modeled by stick elements. The results of the seismic response analyses for the maximum member forces in the stick elements serve as a basis for the development of the equivalent static forces.

The maximum story shear force results from two sets of direct integration time history analyses are used to develop the SSE loads: (1) a set of 12 analyses (three runs for each generic soil case considered) where the three components of the input earthquake excitation are applied on the model separately; and (2) a set of 4 analyses (one run for each generic soil case considered) where the three components of the input earthquake excitation are applied on the model simultaneously. The development of design SSE loads uses the results for the response in the direction of the input ground motion from the first set of analyses with unidirectional earthquake. The results of eight time history analyses, two analysis (one with unidirectional and one with multidirectional design ground motion input) for each of the four generic soil case considered, are enveloped and then increased to account for the floor rotational response, accidental torsion and to introduce additional margin of safety in the seismic design of the structural members.

Story shear force diagrams presented in Table 1 and Table 2 are developed from the results of the direct integration time history analyses and used to calculate the maximum member shear forces in the north-south (NS) and east-west (EW) directions, respectively. The horizontal equivalent floor loads obtained from the different seismic response analyses are enveloped and adjusted to account for the torsional response of the floor based on the seismic response analyses results for the maximum torsional moment and to account for the effects of accidental torsion as required by Subsection 3.1.1(c) of ASCE 4-98 code. The seismic analyses results from story axial force diagrams, listed in Table 3, are used to develop the vertical member axial forces presented in Table 3. The magnitudes of the vertical SSE loads are calculated from the axial force diagrams. The floor vertical loads are adjusted to account for the effects of floor rocking based on the maximum bending moment results, as discussed in response to Item (b) below.

Table 1 Design SSE Loads in North-South Direction

Structure	Floor Location	Elev. (ft)	NS Shear Load (kip)									SSE Load (kip)
			Separate Directional Input				Combine Input				Envelope	
			Soft	Med 1	Med 2	Hard	Soft	Med 1	Med 2	Hard		
R/B	FH08	154.5	2,940	10,270	13,320	9,640	3,060	9,820	12,790	9,810	13,320	16,000
	FH07	125.7	4,860	14,810	18,670	13,570	5,010	14,250	17,440	13,580	18,670	22,500
	FH06	101.0	6,490	16,920	21,230	15,580	6,620	16,400	19,590	15,740	21,230	25,500
	RE41	101.0	5,040	8,330	7,190	8,130	5,280	11,150	13,160	11,090	13,160	15,400
	RE42	101.0	2,640	4,280	4,350	3,830	2,570	5,110	5,490	5,660	5,660	7,800
	RE05	115.5	6,050	10,610	9,980	12,370	6,170	10,510	11,080	13,450	13,450	17,600
	RE04	101.0	10,380	17,950	17,790	22,120	10,720	18,270	22,520	23,110	23,110	30,800
	RE03	76.4	47,300	77,610	66,720	63,750	47,490	79,900	70,870	64,530	79,900	95,800
	RE02	50.2	71,410	112,290	98,270	91,790	70,960	114,700	106,350	95,270	114,700	137,600
	RE01	25.3	91,560	138,620	123,070	111,610	90,890	140,260	130,630	116,130	140,260	168,200
PCCV	CV11	230.2	470	1,580	1,640	1,100	460	1,600	1,680	1,100	1,680	1,910
	CV10	225.0	2,650	9,010	9,370	6,250	2,630	9,110	9,570	6,250	9,570	10,900
	CV09	201.7	6,420	21,630	22,390	14,770	6,370	21,960	22,760	14,770	22,760	25,900
	CV08	173.1	10,280	33,900	34,880	22,650	10,300	34,630	35,220	22,650	35,220	40,100
	CV07	145.6	15,440	48,730	49,640	31,950	15,640	50,150	49,760	31,950	50,150	56,500
	CV06	115.5	19,110	57,690	58,290	38,240	19,350	59,640	58,280	38,240	59,640	66,200
	CV05	92.2	21,990	63,770	63,960	43,160	22,220	66,110	63,870	43,160	66,110	73,600
	CV04	76.4	23,680	66,940	66,840	45,990	23,910	69,510	66,700	45,990	69,510	78,200
	CV03	68.3	25,250	69,690	69,290	48,570	25,460	72,470	69,110	48,570	72,470	82,000
	CV02	50.2	27,670	73,250	72,260	52,260	27,870	76,300	72,030	52,260	76,300	87,100
	CV01	25.3	30,190	75,940	74,160	55,450	30,360	79,140	74,140	55,450	79,140	90,500
Containment Internal Structure	IC09	139.5	350	1,500	1,850	1,660	360	1,430	1,820	1,730	1,850	2,300
	IC08	112.3	1,200	3,940	4,680	4,250	1,210	3,850	4,710	4,310	4,710	5,800
	IC18	110.8	1,350	4,360	5,160	4,690	1,360	4,270	5,200	4,750	5,200	6,300
	IC61	96.6	1,290	2,390	3,680	3,580	1,320	2,390	4,270	3,580	4,270	5,300
	IC62	96.6	1,340	2,460	3,850	3,730	1,390	2,640	4,270	3,670	4,270	5,300
	IC05	76.4	10,410	18,150	25,140	20,660	10,550	17,360	27,820	21,060	27,820	33,800
	IC15	59.2	10,580	18,410	25,470	20,920	10,710	17,590	28,190	21,350	28,190	34,200
	IC04	50.2	15,620	26,220	34,680	27,860	15,580	24,880	37,930	28,990	37,930	45,900
	IC14	45.7	16,590	27,660	36,290	29,400	16,510	26,370	39,610	30,250	39,610	47,900
	IC03	35.6	20,510	33,120	41,730	34,830	20,330	32,300	45,000	34,320	45,000	54,300
	IC02	25.3	25,930	40,200	47,940	40,990	25,750	40,330	50,890	40,580	50,890	61,300
	IC01	16.0	31,620	47,420	53,950	46,970	31,450	48,690	56,540	46,660	56,540	68,000

Table 2 Design SSE Loads in East-West Direction

Structure	Floor Location	Elev. (ft)	EW Shear Load (kip)									SSE Load (kip)
			Separate Directional Input				Combine Input				**Envelope**	
			Soft	Med 1	Med 2	Hard	Soft	Med 1	Med 2	Hard		
R/B	FH08	154.5	3,020	5,930	7,000	6,820	3,070	5,890	6,850	7,190	7,190	8,500
	FH07	125.7	5,020	9,550	10,730	11,090	5,100	9,560	10,970	11,600	11,600	13,600
	FH06	101.0	6,670	12,400	13,600	14,300	6,780	12,520	13,950	14,900	14,900	17,300
	RE41	101.0	2,810	5,680	5,840	5,920	2,980	5,920	7,970	6,710	7,970	9,200
	RE42	101.0	1,760	3,280	3,610	3,680	1,800	3,340	4,190	3,720	4,190	4,400
	RE05	115.5	6,590	12,520	14,250	13,930	6,600	12,150	14,330	14,530	14,530	18,600
	RE04	101.0	14,320	27,270	30,380	30,020	14,760	27,390	29,380	31,730	31,730	40,200
	RE03	76.4	48,190	89,580	90,480	92,980	48,430	89,370	90,710	93,920	93,920	108,100
	RE02	50.2	71,620	126,630	126,980	125,650	72,220	128,200	127,290	127,390	128,200	152,100
	RE01	25.3	90,640	151,770	155,960	144,850	91,320	154,180	155,900	151,320	155,960	184,000
PCCV	CV11	230.2	630	1,460	1,730	1,060	630	1,470	1,740	1,060	1,740	1,910
	CV10	225.0	3,580	8,350	9,830	6,030	3,590	8,390	9,890	6,030	9,890	10,900
	CV09	201.7	8,520	20,180	23,410	14,470	8,570	20,240	23,540	14,470	23,540	25,900
	CV08	173.1	13,310	31,890	36,270	22,630	13,370	31,930	36,450	22,630	36,450	40,100
	CV07	145.6	19,120	46,340	51,260	32,350	19,180	46,310	51,530	32,350	51,530	56,500
	CV06	115.5	22,800	55,380	60,230	38,150	22,800	55,330	60,580	38,150	60,580	66,200
	CV05	92.2	25,910	61,690	67,700	41,990	25,920	61,640	67,500	41,990	67,700	73,600
	CV04	76.4	27,670	65,060	71,750	43,880	27,690	65,010	71,450	43,880	71,750	78,200
	CV03	68.3	29,270	67,990	75,270	45,440	29,300	67,950	74,870	45,440	75,270	82,000
	CV02	50.2	31,630	71,860	79,770	47,270	31,670	71,820	79,250	47,270	79,770	87,100
	CV01	25.3	33,920	74,830	82,800	50,440	33,960	74,790	82,230	50,440	82,800	90,500
Containment Internal Structure	IC09	139.5	400	1,390	1,950	1,850	400	1,310	1,920	1,850	1,950	2,500
	IC08	112.3	1,330	3,600	5,020	5,080	1,340	3,420	4,950	5,080	5,080	6,500
	IC18	110.8	1,500	4,000	5,550	5,650	1,500	3,800	5,470	5,640	5,650	7,200
	IC61	96.6	1,380	3,000	4,090	3,260	1,360	2,920	4,090	3,440	4,090	5,100
	IC62	96.6	1,440	3,140	4,270	3,500	1,440	3,090	4,330	3,330	4,330	5,300
	IC05	76.4	11,060	18,750	24,470	21,890	11,020	18,660	24,510	21,860	24,510	30,400
	IC15	59.2	11,220	18,990	24,760	22,150	11,180	18,900	24,800	22,110	24,800	30,800
	IC04	50.2	15,990	25,730	32,610	28,720	15,950	25,700	32,400	28,670	32,610	40,600
	IC14	45.7	16,900	27,110	33,990	29,860	16,850	27,100	33,730	29,800	33,990	42,300
	IC03	35.6	20,480	32,170	38,610	33,630	20,420	32,240	38,150	33,570	38,610	48,000
	IC02	25.3	25,380	38,650	44,100	38,340	25,290	38,830	43,390	38,310	44,100	54,700
	IC01	16.0	30,670	45,200	49,620	43,920	30,520	45,510	48,680	43,920	49,620	61,300

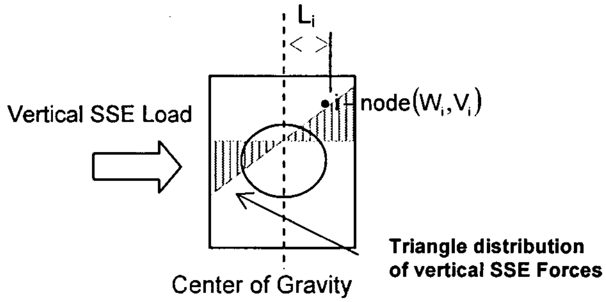
Table 3 Design SSE Loads in Vertical Direction

Structure	Floor Location	Elev. (ft)	Vertical Load (kip)									SSE Load (kip)
			Separate Directional Input				Combine Input				Envelope	
			Soft	Med 1	Med 2	Hard	Soft	Med 1	Med 2	Hard		
R/B	FH08	154.5	1,950	3,450	4,790	6,640	2,740	4,570	6,790	7,570	7,570	10,000
	FH07	125.7	3,350	5,820	8,110	10,710	4,690	7,530	11,190	12,580	12,580	16,400
	FH06	101.0	4,630	7,890	10,990	13,700	6,410	9,940	14,760	16,510	16,510	21,200
	RE41	101.0	2,660	4,380	6,070	5,910	3,170	5,760	7,630	7,670	7,670	9,200
	RE42	101.0	2,100	3,360	4,430	4,300	2,290	4,800	7,110	5,690	7,110	8,100
	RE05	115.5	4,730	7,280	9,640	7,890	4,480	11,620	15,150	13,440	15,150	19,700
	RE04	101.0	9,450	14,250	18,770	15,230	8,980	23,400	29,900	26,180	29,900	38,900
	RE03	76.4	37,160	54,030	67,720	61,120	35,810	55,170	83,120	72,360	83,120	100,300
	RE02	50.2	57,670	79,800	99,750	90,030	55,760	82,980	119,150	102,660	119,150	143,700
	RE01	25.3	75,430	100,390	124,310	111,770	73,070	106,030	145,740	123,900	145,740	175,700
PCCV	CV11	230.2	300	720	1,150	1,120	300	710	1,180	1,120	1,180	1,300
	CV10	225.0	1,670	3,720	6,080	5,750	1,700	3,650	6,170	5,750	6,170	6,800
	CV09	201.7	4,050	8,370	13,050	11,890	4,090	8,530	13,170	11,890	13,170	14,400
	CV08	173.1	6,600	13,310	19,710	18,210	6,640	13,530	19,830	18,210	19,830	21,800
	CV07	145.6	10,160	20,070	28,760	27,160	10,190	20,370	28,710	27,160	28,760	31,900
	CV06	115.5	12,750	24,770	35,010	33,270	12,770	25,110	34,710	33,270	35,010	38,900
	CV05	92.2	14,830	28,380	39,660	37,850	14,840	28,770	39,380	37,850	39,660	44,000
	CV04	76.4	16,090	30,470	42,270	40,400	16,080	30,890	42,010	40,400	42,270	46,900
	CV03	68.3	17,270	32,390	44,600	42,690	17,240	32,840	44,390	42,690	44,600	49,500
	CV02	50.2	19,140	35,270	47,920	45,920	19,090	35,760	47,810	45,920	47,920	53,200
	CV01	25.3	21,160	38,100	50,800	48,740	21,070	38,660	50,920	48,740	50,920	56,600
Containment Internal Structure	IC09	139.5	210	320	360	560	210	370	480	640	640	750
	IC08	112.3	800	1,180	1,310	2,030	790	1,380	1,790	2,310	2,310	2,700
	IC18	110.8	810	1,180	1,310	2,040	900	1,550	2,020	2,610	2,610	3,100
	IC61	96.6	650	910	1,030	1,240	1,100	1,570	2,070	2,180	2,180	2,900
	IC62	96.6	680	990	1,110	1,330	1,100	1,840	2,420	2,350	2,420	2,900
	IC05	76.4	6,650	9,130	10,150	13,030	7,900	10,760	14,270	15,990	15,990	20,600
	IC15	59.2	6,810	9,330	10,370	13,300	8,030	10,920	14,490	16,220	16,220	20,900
	IC04	50.2	10,870	14,730	16,080	20,040	12,060	16,160	20,790	23,130	23,130	29,300
	IC14	45.7	10,930	14,810	16,160	20,140	12,820	17,220	21,960	24,370	24,370	30,900
	IC03	35.6	14,210	19,120	20,490	24,920	15,970	21,500	26,520	28,750	28,750	36,000
	IC02	25.3	20,430	27,240	28,380	33,190	20,410	27,390	32,540	33,780	33,780	42,200
	IC01	16.0	25,160	33,370	34,230	39,150	25,130	33,630	38,790	38,810	39,150	48,100

(b) In the detailed finite element (FE) model, the design SSE loads presented in Table 1, Table 2, and Table 3 are applied as concentrated forces on all nodes of each nominal floor elevation. The magnitude of the SSE load is distributed across the floor proportional to the weight pertaining to each nodal point.

Additionally, a converted triangle distribution is used to apply an adjustment to the vertical seismic loads (ΔV_i) in order to account for the floor rocking moment at each nominal floor elevation as shown in the sketch below. The adjustments to the vertical nodal forces are calculated as follows:

$$\Delta V_i = \frac{W_i \cdot L_i}{\sum_{floor} \left(W_i \cdot L_i^2\right)} \cdot \Delta M$$



where

ΔV_i := Adjustment to vertical nodal force due to floor rocking

W_i := Weight pertaining to node "i"

L_i := Nodal distance from the floor center of gravity

ΔM := Moment for compensating for the shortfall of SSE floor rocking moment

The SSE floor rocking moments are calculated from the results of the time history sesimic response analyses. Bending moment diagrams are developed for each time history analyses performed using the stick member results for maximum bending moments. The floor rocking moments at each floor lumped mass location are calculated from these bending moment diagrams. As shown in Table 4 and Table 5 below, the SSE floor rocking moment is obtained by applying a load margin to the envelope the value of the results obtained for each soil case considered.

Table 4 Floor Rocking Moments in North-South Direction

Structure	Floor Location	Elev. (ft)	NS Rocking Moments ($x10^3$ kip-ft)									SSE Floor Rocking ($x10^3$kip-ft)
			Separate Directional Input				Combine Input				Envelope	
			Soft	Med 1	Med 2	Hard	Soft	Med 1	Med 2	Hard		
R/B	FH08	154.5	87	335	442	328	90	318	432	326	442	533
	FH07	125.7	208	726	935	670	216	693	897	687	935	1,130
	FH06	101.0	367	1155	1474	1063	378	1112	1399	1066	1474	1,780
	RE41	101.0	164	308	342	282	180	366	372	323	372	442
	RE42	101.0	86	168	179	132	133	226	281	301	301	333
	RE05	115.5	80	166	219	230	116	246	355	351	355	458
	RE04	101.0	326	570	665	783	370	578	787	898	898	1,170
	RE03	76.4	3492	7096	8458	6177	3592	8665	9997	6131	9997	12,100
	RE02	50.2	5703	10942	12047	9467	5944	12720	13613	8687	13613	16,300
	RE01	25.3	8148	14503	14793	12235	8419	16341	16134	11052	16341	19,700
PCCV	CV11	230.2	2.5	8.3	9.2	5.9	2.4	8.3	9.2	5.9	9.2	10.3
	CV10	225.0	70	236	247	166	68	238	253	166	253	288
	CV09	201.7	278	938	980	653	269	945	1005	653	1005	1,140
	CV08	173.1	599	2006	2088	1376	579	2028	2132	1376	2132	2,430
	CV07	145.6	1109	3668	3796	2464	1076	3728	3851	2464	3851	4,370
	CV06	115.5	1578	5153	5301	3414	1549	5254	5356	3414	5356	6,080
	CV05	92.2	1938	6258	6413	4115	1920	6396	6464	4115	6464	7,340
	CV04	76.4	2138	6862	7017	4496	2128	7018	7065	4496	7065	8,020
	CV03	68.3	2583	8172	8313	5337	2600	8376	8352	5337	8376	9,500
	CV02	50.2	3273	10063	10170	6579	3313	10341	10194	6579	10341	11,700
	CV01	25.3	3995	11888	11930	7814	4039	12234	11941	7814	12234	13,800
Contianment Internal Structure	IC09	139.5	10	42	52	46	10	40	51	48	52	63.3
	IC08	112.3	12	52	64	58	13	50	63	60	64	83.3
	IC18	110.8	58	202	242	219	59	197	242	223	242	300
	IC61	96.6	27	51	80	82	28	52	93	84	93	117
	IC62	96.6	28	52	83	85	29	57	93	86	93	1,170
	IC05	76.4	311	558	792	762	326	626	848	797	848	1,040
	IC15	59.2	406	713	1023	923	423	779	1103	975	1103	1,350
	IC04	50.2	486	853	1239	1112	500	918	1348	1167	1348	1,650
	IC14	45.7	653	1134	1615	1389	668	1178	1759	1428	1759	2,150
	IC03	35.6	870	1488	2075	1720	883	1500	2263	1743	2263	2,770
	IC02	25.3	1125	1888	2582	2073	1150	1909	2782	2114	2782	3,390
	IC01	16.0	1581	2580	3383	2609	1616	2588	3604	2634	3604	4,380

Table 5 Floor Rocking Moments in East-West Direction

Structure	Floor Location	Elev. (ft)	EW Rocking Moments ($x10^3$kip-ft)									SSE Floor Rocking ($x10^3$kip-ft)
			Separate Directional Input				Combine Input				Envelope	
			Soft	Med 1	Med 2	Hard	Soft	Med 1	Med 2	Hard		
R/B	FH08	154.5	124	262	371	305	123	272	362	333	371	425
	FH07	125.7	274	542	737	643	274	552	733	696	737	850
	FH06	101.0	456	868	1105	1018	454	880	1070	1070	1105	1,270
	RE41	101.0	58	135	185	178	75	177	232	222	232	275
	RE42	101.0	39	71	93	86	43	83	134	120	134	150
	RE05	115.5	242	428	680	548	251	443	732	628	732	942
	RE04	101.0	699	1246	1732	1531	728	1285	1882	1754	1882	2,420
	RE03	76.4	3153	5726	6503	6391	3106	5775	6963	6839	6963	8,160
	RE02	50.2	5308	9618	9926	9813	5235	9772	10001	10247	10247	12,200
	RE01	25.3	7492	13417	13369	12735	7424	13603	13438	12945	13603	16,100
PCCV	CV11	230.2	3.4	7.8	9.2	5.7	3.4	7.8	9.2	5.8	9.2	10.3
	CV10	225.0	95	217	259	160	96	218	261	160	261	288
	CV09	201.7	381	864	1027	628	383	868	1033	628	1033	1,140
	CV08	173.1	815	1855	2179	1338	819	1862	2193	1338	2193	2,430
	CV07	145.6	1493	3417	3938	2438	1499	3424	3960	2438	3960	4,370
	CV06	115.5	2097	4830	5482	3414	2104	4837	5511	3414	5511	6,080
	CV05	92.2	2551	5893	6622	4137	2559	5900	6656	4137	6656	7,340
	CV04	76.4	2802	6478	7239	4529	2810	6483	7278	4529	7278	8,020
	CV03	68.3	3328	7755	8581	5378	3338	7760	8626	5378	8626	9,500
	CV02	50.2	4096	9614	10519	6588	4105	9620	10573	6588	10573	11,700
	CV01	25.3	4843	11419	12386	7726	4852	11425	12450	7726	12450	13,800
Containment Internal Structure	IC09	139.5	11	39	55	53	11	38	54	53	55	71.7
	IC08	112.3	13	51	72	68	13	48	71	68	72	100
	IC18	110.8	65	187	263	261	65	176	259	260	263	342
	IC61	96.6	28	60	83	65	29	60	82	68	83	100
	IC62	96.6	29	63	86	70	28	63	88	68	88	108
	IC05	76.4	354	637	834	808	353	653	844	811	844	1,040
	IC15	59.2	455	793	1059	1002	454	812	1070	1003	1070	1,330
	IC04	50.2	545	931	1278	1163	541	950	1292	1164	1292	1,590
	IC14	45.7	717	1203	1631	1447	712	1201	1643	1447	1643	2,030
	IC03	35.6	936	1541	2064	1786	932	1535	2073	1786	2073	2,560
	IC02	25.3	1194	1909	2533	2139	1188	1902	2539	2140	2539	3,130
	IC01	16.0	1641	2546	3255	2758	1634	2535	3251	2758	3255	4,030

(c) The design of the R/B reinforced concrete structural members is based on a series of static analyses performed on the detailed finite element (FE) model. The demands under mechanical design loads on the building walls and slabs of the above grade portion are obtained from the static analysis of FE model fixed at a nominal grade elevation of 3'-7". The demands under thermal design loads are obtained from the thermal stress analysis of a FE model that also includes the R/B basement where the basemat is supported by a subgrade with stiffnesses represented by soil springs. The fixed base boundary conditions are established at nominal grade to simplify the analysis. For most of the load combinations considered, this fixed base modeling simplification produces conservative results for shear force and bending moment demands on the shear walls of the above grade portion of the building. As discussed in the response to Question 3.8.4-26 of this RAI, a confirmatory analysis will be performed to demonstrate that sufficient design margin exist to cover effects of the interaction between the subgrade, basemat and R/B superstructure.

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

7/03/2009

US-APWR Design Certification

Mitsubishi Heavy Industries

Docket No. 52-021

RAI NO.:	**NO. 342-2000 REVISION 0**
SRP SECTION:	**3.8.4 – Other Seismic Category I Structures**
APPLICATION SECTION:	**03.08.04**
DATE OF RAI ISSUE:	**4/21/2009**

QUESTION NO.: 3.8.4-12

In DCD Subsection 3.8.4.4.1, the sixth paragraph (Page 3.8-56) states, "The R/B is analyzed using a three-dimensional FE model with the NASTRAN computer codes (Reference 3.8-13)."

The applicant is requested to provide the following information:

(a) Were the upper bound and lower bound values of elastic modulus and shear modulus of concrete suggested by ASCE 4-98 (Subsection C3.1.3.1 in Page 63) used in the FE analyses? If not, provide the technical basis that shows your results for both the floor response spectra and the design of the R/B are conservative.
(b) Were the cracked sections of concrete considered in the analyses as suggested in Design and Analysis Procedure 4B of SRP 3.8.4 (page 3.8.4-10 of SRP 3.8.4 Revision 2, March 2007)? If not, provide the reason for not doing it.
(c) Provide information for the types of element used in the FE model.

ANSWER:

(a) The results of the seismic response analyses of lumped mass stick models presented in Subsection 3.7.2 of the DCD serve as basis for development of the in-structure response spectra (ISRS) that are used for design of Category I and II subsystems and components and seismic design loads used for design of the structural members of reactor building (R/B). The ISRS are developed from the R/B stick model time history analyses results according to the requirements of RG 1.122 to account for the frequency variation due to concrete stiffness changes. Per RG 1.122, smoothing and broadening the ISRS by +/-15% is sufficient to account for the uncertainties related to variation of material properties of the structure and soil, damping values and the approximations in the structural and soil-structure interaction modeling techniques. Per Subsection C3.1.3.1 of ASCE 4-98 code, the 25% variation of the concrete stiffness that was considered by the Standard 4 Working Group, can result in frequency variation of up to 12%. The Standard 4 Working Group concluded that the +/-15% peak broadening used for the ISRS, which is consistent with RG 1.122, is sufficient to account for the frequency variation due to concrete stiffness changes, as well as other uncertainties in the analysis.

The seismic design of the R/B reinforced concrete members is based on demands obtained from static analyses performed on the detailed finite element (FE) model shown in Figure 3.8.4-2 of the DCD. The static analyses are performed by applying safe shutdown earthquake (SSE) design loads, developed based on the results of the seismic response analyses as described in the response to question 11 of this RAI. The design margins introduced in the development of the equivalent seismic loads accommodate the effects of variation of the concrete stiffness.

Frequencies in the range of 4.5 HZ to 12 Hz characterize the dynamic properties of the lumped mass stick models used for these seismic response analyses as shown in Tables 3H.3-1, 3H.3-2 and 3H.3-3 in Appendix 3H of the DCD. Due to the broad band nature of the CSDRS in this frequency range, the variation of structural frequencies that are due to the changes in concrete stiffness have small effect on the calculated seismic loads. The 7% damping CSDRS curves show that in the dominant structural frequency range, the 12% variation in structural frequencies will result in not more than 2% change in the peak spectral acceleration. The margins introduced in development of the SSE design loads and the design of reinforced concrete members are sufficient to ensure that the uncertainties due to possible variation of concrete stiffness are bounded.

(b) The analyses that serve as basis for the preliminary design of the R/B complex structures do not consider the effects of concrete on the in-plane seismic response of the R/B complex and the load path redistribution in the reinforced concrete members. The primary lateral force resisting system for the R/B is comprised of shear walls, which are structurally designed such that the nominal shear stresses are low. Section and material properties for the shear walls in the lumped mass stick models used for seismic response analyses are developed using methods discussed in DCD Section 3.7.2.3.5. The implemented modeling approach that neglects to consider the cracking of the walls is deemed to be satisfactory for the determination of shear wall design forces since the concrete cracking does not significantly affect the in-plane response of the structures. The seismic forces used for structural design are defined by the maximum response values, which envelope the lumped mass stick model seismic response analysis results, considering all four generic subgrade soil conditions identified in DCD Subsection 3.7.2.2. Further, the enveloped maximum response values are multiplied by a factor no less than 1.2 for use as seismic forces in the structural design. The consideration of a wide range of generic subgrade conditions is deemed sufficient to address the possible variations of in-plane seismic response of the structures due to concrete cracking. The margins introduced in the structural design of the reinforced concrete members are considered to be sufficient to envelope the effects of load path redistribution due to cracking of the concrete. With respect to the above, MHI is to incorporate the effects of local vibration modes on ISRS, including the effects of potential concrete cracking on the out-of-plane stiffness of the slabs, as discussed in the responses to Questions RAI 3.7.2-8 and RAI 3.7.2-15 of RAI 212-1950, Revision 1.

(c) The detailed FE models that are used for calculating demands on the structural members of the PS/B and R/B complex use 3-D shell elements to represent the reinforced concrete walls and slabs and 3-D beam elements to model the stiffness of the columns, girders and beams. Brick FEs are used to model the foundation basemat with attached spring elements representing the subgrade stiffness. Rigid body elements are used to represent the geometric offsets in the detailed FE models.

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

7/03/2009

US-APWR Design Certification
Mitsubishi Heavy Industries
Docket No. 52-021

RAI NO.:	**NO. 342-2000 REVISION 0**
SRP SECTION:	**3.8.4 – Other Seismic Category I Structures**
APPLICATION SECTION:	**03.08.04**
DATE OF RAI ISSUE:	**4/21/2009**

QUESTION NO.: 3.8.4-13

In DCD Subsection 3.8.4.4.1.1, it (Page 3.8-57) states that South interior wall of R/B (Section 2) is one of the most highly stressed shear walls.

The applicant is requested to provide the following information:

a. What is the load or load combination that causes the high stress?
b. Is the south interior wall of R/B the highest stressed shear wall? If not, which is the highest stressed shear wall?
c. Has the stress exceeded the cracking stress of concrete in the highest stressed shear wall? If yes, was the model re-analyzed by using the cracked moment of inertia for that shear wall? If not, provide the reason for not doing it.

ANSWER:

a. The controlling load combination case corresponding to the provided reinforcement of typical shear walls will be added during a revision of the DCD to Tables 3.8.4-6 through 3.8.4-9, as shown in the following Tables 1 through 4. According to these tables, the controlling load combinations for provided reinforcement are 0.9D+1.0F+1.0E+T, 1.05D+1.3L+1.05F+1.2T, or 1.0D+1.0L+1.0F+1.0E+T, as applicable. It is noted that load combinations with thermal load are controlling. Therefore, load combinations with thermal loads cause the highest stress.

Table 1 West Exterior Wall, Section 1, Details of Wall Reinforcement

	Provided Reinforcement		
	Vertical	Horizontal	Shear
WALL ZONE 1 (Concrete Thickness 40 in.) El 3'-7" → El 25'-3"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Outside	#11@12"+#11@12" (0.65)	#11@6"+#11@12" (0.975)	–
Inside	#11@12" +#10@12" (0.59)	#11@12" +#10@12" (0.59)	
WALL ZONE 2 (Concrete Thickness 40 in.) El 25'-3" → El 50'-2"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Outside	#11@12"+#11@12" (0.65)	#11@6"+#11@12" (0.813)	–
Inside	#11@12" (0.325)	#11@12" (0.325)	
WALL ZONE 3 (Concrete Thickness 32 in.) El 50'-2" → El 76'-5"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Outside	#11@12" (0.406)	#11@6" (0.929)	–
Inside	#11@12" (0.406)	#11@12" (0.406)	
WALL ZONE 4 (Concrete Thickness 28 in.) El 76'-5" → El 101'-0"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$1.05D+1.3L+1.05F+1.2T_o$	NONE
Outside	#11@12" (0.464)	#11@6" (0.929)	–
Inside	#11@12" (0.464)	#11@12" (0.464)	

Note: Load Combination reflects the controlling load combination for the outside face required reinforcement. ( ) indicates reinforcement ratio.

Table 2 South Interior Wall, Section 2, Details of Wall Reinforcement

	Provided Reinforcement		
	Vertical	Horizontal	Shear
WALL ZONE 1 (Concrete Thickness 44 in.) El 3'-7" → El 25'-3"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Each Face	#11@12" +#11@12" (0.591)	#11@6" +#11@12" (0.886)	–
WALL ZONE 2 (Concrete Thickness 40 in.) El 25'-3" → El 50'-2"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Each Face	#11@12" (0.325)	#11@12" (0.325)	–
WALL ZONE 3 (Concrete Thickness 40 in.) El 50'-2" → El 76'-5"			
Load Combination	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	NONE
Each Face	#11@12" (0.325)	#11@12" (0.325)	–
WALL ZONE 4 (Concrete Thickness 40 in.) El 76'-5" → El 86'-4"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Each Face	#11@12" (0.325)	#11@12"+#11@12" (0.65)	–
WALL ZONE 5 (Concrete Thickness 40 in.) El 86'-4" → El 101'-0"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$1.05D+1.3L+1.05F+1.2T_o$	NONE
Each Face	#11@12" (0.325)	#11@12"+#11@12" (0.65)	–

Note: ( ) indicates reinforcement ratio.

Table 3 North Exterior Wall of Spent Fuel Pit, Section 3, Details of Wall Reinforcement

	Provided Reinforcement		
	Vertical	Horizontal	Shear
WALL ZONE 1 (Concrete Thickness 93 in.) El 30'-1"→ El 50'-2"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Outside	#14@6"+#14@6" (0.806)	#14@6"+#14@6" (0.806)	–
Inside	#14@12"+#14@12" (0.403)	#14@12"+#14@12" (0.403)	
WALL ZONE 2 (Concrete Thickness 93 in.) El 50'-2" → El 65'-0"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Outside	#14@6"+#14@6" (0.806)	#14@6"+#14@6" (0.806)	–
Inside	#14@12"+#14@12" (0.403)	#14@12"+#14@12" (0.403)	
WALL ZONE 3 (Concrete Thickness 152 in.) El 65'-0" → El 76'-5"			
Load Combination	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	$1.05D+1.3L+1.05F+1.2T_o$	NONE
Outside	#14@6"+#14@12" (0.370)	#14@6"+#14@6"+#14@12" (0.617)	–
Inside	#14@12"+#14@12" (0.247)	#14@12"+#14@12" (0.247)	

Note: Load Combination reflects the controlling load combination for the outside face required reinforcement. ( ) indicates reinforcement ratio.

Table 4 South Exterior Wall, Section 4, Details of Wall Reinforcement

	Provided Reinforcement		
	Vertical	Horizontal	Shear
WALL ZONE 1 (Concrete Thickness 44 in.) El 3'-7" → El 25'-3"			
Load Combination	1.0D+1.0L+1.0F+1.0E_{ss}+T_o	1.0D+1.0L+1.0F+1.0E_{ss}+T_o	NONE
Outside	#11@6"+#11@6" (1.182)	#11@6"+#11@12" (0.886)	–
Inside	#11@12"+#11@12" (0.591)	#11@12"+#11@12" (0.591)	
WALL ZONE 2 (Concrete Thickness 40 in.) El 25'-3" → El 50'-2"			
Load Combination	1.0D+1.0L+1.0F+1.0E_{ss}+T_o	1.0D+1.0L+1.0F+1.0E_{ss}+T_o	NONE
Outside	#11@6"+#11@12" (0.975	#11@6"+#11@12" (0.975)	–
Inside	#11@12"+#11@12" (0.65)	#11@12"+#11@12" (0.65)	
WALL ZONE 3 (Concrete Thickness 40 in.) El 50'-2" → El 76'-5"			
Load Combination	1.0D+1.0L+1.0F+1.0E_{ss}+T_o	1.0D+1.0L+1.0F+1.0E_{ss}+T_o	NONE
Outside	#11@12"+#11@12" (0.65)	#11@6"+#11@12" (0.975)	–
Inside	#11@12" (0.325)	#11@12" (0.325)	
WALL ZONE 4 (Concrete Thickness 40 in.) El 76'-5" → El 101'-0"			
Load Combination	1.0D+1.0L+1.0F+1.0E_{ss}+T_o	1.0D+1.0L+1.0F+1.0E_{ss}+T_o	NONE
Outside	#11@12"+#11@12" (0.65)	#11@12"+#11@12" (0.65)	–
Inside	#11@12" (0.325)	#11@12" (0.325)	
WALL ZONE 5 (Concrete Thickness 40 in.) El 101'-0" → El 115'-6"			
Load Combination	1.05D+1.3L+1.05F+1.2T_o	1.05D+1.3L+1.05F+1.2T_o	NONE
Outside	#11@12"+#11@12" (0.65)	#11@12"+#11@12" (0.65)	–
Inside	#11@12" (0.325)	#11@12" (0.325)	

Note: Load Combination shows the dominant load combination for the outside face required reinforcement. ( ) shows reinforcement ratio.

b. Although the south interior wall of R/B is one of the highest stressed interior shear walls, according to the reinforcement ratio shown in Table 1 to 4, the south exterior wall of R/B is the highest stressed shear wall among the shear walls in DCD.

c. Stress exceeds the cracking stress in some shear walls, including the south exterior wall of the R/B. As noted in the response to RAI 212-1950, Question 3.7.2-15, the effects of potential concrete cracking on structural stiffnesses are considered in the development of local vibration modes for the ISRS. As discussed in the response to RAI 212-1950, Question 3.7.2-8, the ISRS considering local vibration modes and the description of the analysis method will be provided in Revision 2 of the DCD. Further discussion is also provided in the response to related Question 3.8.4-12(b) of this RAI.

Impact on DCD

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

Item (a)

- Replace Table 3.8.4-6 with the following:

Table 3.8.4-6 West Exterior Wall, SECTION 1, Details of Wall Reinforcement

	Provided Reinforcement		
	Vertical	**Horizontal**	**Shear**
WALL ZONE 1 (Concrete Thickness 40 in.) El 3'-7" → El 25'-3"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Outside	#11@12"+#11@12" (0.65)	#11@6"+#11@12" (0.975)	—
Inside	#11@12" +#10@12" (0.59)	#11@12" +#10@12" (0.59)	
WALL ZONE 2 (Concrete Thickness 40 in.) El 25'-3" → El 50'-2"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Outside	#11@12"+#11@12" (0.65)	#11@6"+#11@12" (0.813)	—
Inside	#11@12" (0.325)	#11@12" (0.325)	
WALL ZONE 3 (Concrete Thickness 32 in.) El 50'-2" → El 76'-5"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Outside	#11@12" (0.406)	#11@6" (0.929)	—
Inside	#11@12" (0.406)	#11@12" (0.406)	
WALL ZONE 4 (Concrete Thickness 28 in.) El 76'-5" → El 101'-0"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$1.05D+1.3L+1.05F+1.2T_o$	NONE
Outside	#11@12" (0.464)	#11@6" (0.929)	—
Inside	#11@12" (0.464)	#11@12" (0.464)	

Note: Load Combination reflects the controlling load combination for the outside face required reinforcement. ( ) indicates reinforcement ratio.

- Replace Table 3.8.4-7 with the following:

Table 3.8.4-7 South Interior Wall, SECTION 2, Details of Wall Reinforcement

	Provided Reinforcement		
	Vertical	**Horizontal**	**Shear**
WALL ZONE 1 (Concrete Thickness 44 in.) El 3'-7" → El 25'-3"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Each Face	#11@12" +#11@12" (0.591)	#11@6" +#11@12" (0.886)	–
WALL ZONE 2 (Concrete Thickness 40 in.) El 25'-3" → El 50'-2"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Each Face	#11@12" (0.325)	#11@12" (0.325)	–
WALL ZONE 3 (Concrete Thickness 40 in.) El 50'-2" → El 76'-5"			
Load Combination	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	NONE
Each Face	#11@12" (0.325)	#11@12" (0.325)	–
WALL ZONE 4 (Concrete Thickness 40 in.) El 76'-5" → El 86'-4"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Each Face	#11@12" (0.325)	#11@12"+#11@12" (0.65)	–
WALL ZONE 5 (Concrete Thickness 40 in.) El 86'-4" → El 101'-0"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$1.05D+1.3L+1.05F+1.2T_o$	NONE
Each Face	#11@12" (0.325)	#11@12"+#11@12" (0.65)	–

Note: ( ) indicates reinforcement ratio.

- Replace Table 3.8.4-8 with the following:

Table 3.8.4-8 North Exterior Wall of Spent Fuel Pit, SECTION 3, Details of Wall Reinforcement

	Provided Reinforcement		
	Vertical	Horizontal	Shear
WALL ZONE 1 (Concrete Thickness 93 in.) El 30'-1"→ El 50'-2"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Outside	#14@6"+#14@6" (0.806)	#14@6"+#14@6" (0.806)	–
Inside	#14@12"+#14@12" (0.403)	#14@12"+#14@12" (0.403)	
WALL ZONE 2 (Concrete Thickness 93 in.) El 50'-2" → El 65'-0"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Outside	#14@6"+#14@6" (0.806)	#14@6"+#14@6" (0.806)	–
Inside	#14@12"+#14@12" (0.403)	#14@12"+#14@12" (0.403)	
WALL ZONE 3 (Concrete Thickness 152 in.) El 65'-0" → El 76'-5"			
Load Combination	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	$1.05D+1.3L+1.05F+1.2T_o$	NONE
Outside	#14@6"+#14@12" (0.370)	#14@6"+#14@6"+#14@12" (0.617)	–
Inside	#14@12"+#14@12" (0.247)	#14@12"+#14@12" (0.247)	

Note: Load Combination reflects the controlling load combination for the outside face required reinforcement. ( ) indicates reinforcement ratio.

- Replace Table 3.8.4-9 with the following:

Table 3.8.4-9 South Exterior Wall, SECTION 4, Details of Wall Reinforcement

	Provided Reinforcement		
	Vertical	Horizontal	Shear
WALL ZONE 1 (Concrete Thickness 44 in.) El 3'-7" → El 25'-3"			
Load Combination	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	NONE
Outside	#11@6"+#11@6" (1.182)	#11@6"+#11@12" (0.886)	–
Inside	#11@12"+#11@12" (0.591)	#11@12"+#11@12" (0.591)	
WALL ZONE 2 (Concrete Thickness 40 in.) El 25'-3" → El 50'-2"			
Load Combination	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	NONE
Outside	#11@6"+#11@12" (0.975	#11@6"+#11@12" (0.975)	–
Inside	#11@12"+#11@12" (0.65)	#11@12"+#11@12" (0.65)	
WALL ZONE 3 (Concrete Thickness 40 in.) El 50'-2" → El 76'-5"			
Load Combination	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	NONE
Outside	#11@12"+#11@12" (0.65)	#11@6"+#11@12" (0.975)	–
Inside	#11@12" (0.325)	#11@12" (0.325)	
WALL ZONE 4 (Concrete Thickness 40 in.) El 76'-5" → El 101'-0"			
Load Combination	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	NONE
Outside	#11@12"+#11@12" (0.65)	#11@12"+#11@12" (0.65)	–
Inside	#11@12" (0.325)	#11@12" (0.325)	
WALL ZONE 5 (Concrete Thickness 40 in.) El 101'-0" → El 115'-6"			
Load Combination	$1.05D+1.3L+1.05F+1.2T_o$	$1.05D+1.3L+1.05F+1.2T_o$	NONE
Outside	#11@12"+#11@12" (0.65)	#11@12"+#11@12" (0.65)	–
Inside	#11@12" (0.325)	#11@12" (0.325)	

Note: Load Combination reflects the controlling load combination for the outside face required reinforcement. ( ) indicates reinforcement ratio.

Item (c)

- Refer to RAI 212-1950, Question 3.7.2-8, for impact on 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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Docket No. 52-021

RAI NO.:	**NO. 342-2000 REVISION 0**
SRP SECTION:	**3.8.4 – Other Seismic Category I Structures**
APPLICATION SECTION:	**03.08.04**
DATE OF RAI ISSUE:	**4/21/2009**

QUESTION NO.: 3.8.4-14

In DCD Subsection 3.8.4.4.1.2, the first paragraph (Page 3.8-57) states that, "The shear walls are used as the primary system for resisting lateral loads, such as earthquakes."

The applicant is requested to provide the following information:

DCD Figures 3.8.4-4 to 3.8.4-7 (Pages 3.8-202 to 3.8.205) show vertical cross section views of the shear walls with re-bar layout. Provide the corresponding horizontal cross section views of these shear walls with the re-bar layout.

ANSWER:

The DCD will be revised to add horizontal cross section views of RB and PS/B shear walls with the vertical cross section views in Figures 3.8.4-4 to 3.8.4-7.

Impact on DCD

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

- Replace the title to Figure 3.8.4-4 with the following:

"**Vertical Cross Section**

Figure 3.8.4-4 Typical Reinforcement in West Exterior Wall – SECTION 1 (Sheet 1 of 2)"

- Replace the title to Figure 3.8.4-5 with the following:

"**Vertical Cross Section**

Figure 3.8.4-5 Typical Reinforcement in South Interior Wall – SECTION 2 (Sheet 1 of 2)"

- Replace the title to Figure 3.8.4-6 with the following:

"<u>Vertical Cross Section</u>

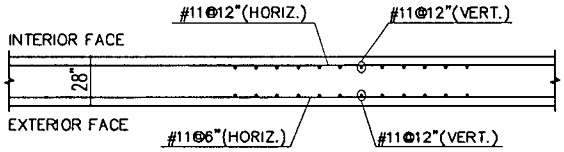
Figure 3.8.4-6 Typical Reinforcement in North Exterior Wall of Spent Fuel Pit – SECTION 3 (Sheet 1 of 2)"

- Replace the title to Figure 3.8.4-7 with the following:

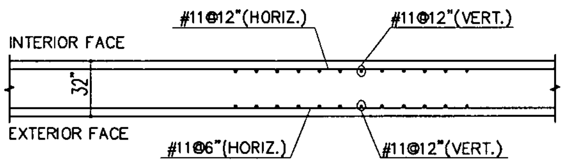
"<u>Vertical Cross Section</u>

Figure 3.8.4-7 Typical Reinforcement in South Exterior Wall – SECTION 4 (Sheet 1 of 2)"

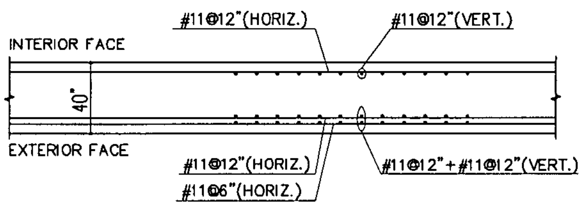
- Insert the following as Sheet 2 of Figure 3.8.4-4:



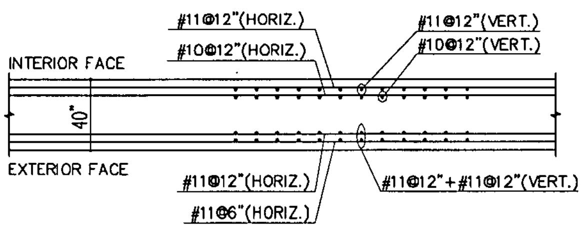
ZONE 4 [EL76'-5" to EL101'-0"]



ZONE 3 [EL50'-2" to EL76'-5"]



ZONE 2 [EL25'-3" to EL50'-2"]

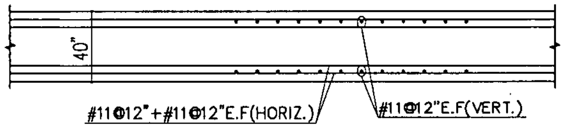


ZONE 1 [EL3'-7" to EL25'-3"]

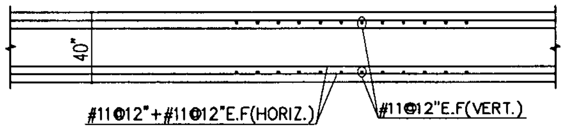
Horizontal Cross Section

Figure 3.8.4-4 Typical Reinforcement in West Exterior Wall – SECTION 1 (Sheet 2 of 2)

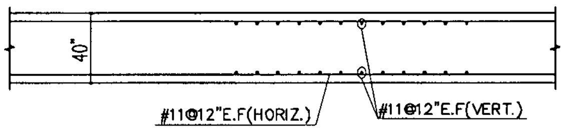
- Insert the following as Sheet 2 of Figure 3.8.4-5:



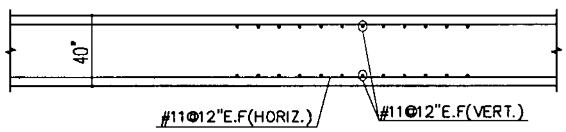
ZONE 5 [EL86'-4" to EL101'-0"]



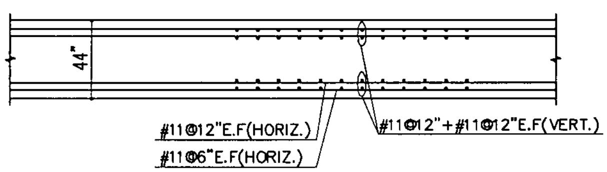
ZONE 4 [EL76'-5" to EL86'-4"]



ZONE 3 [EL50'-2" to EL76'-5"]



ZONE 2 [EL25'-3" to EL50'-2"]

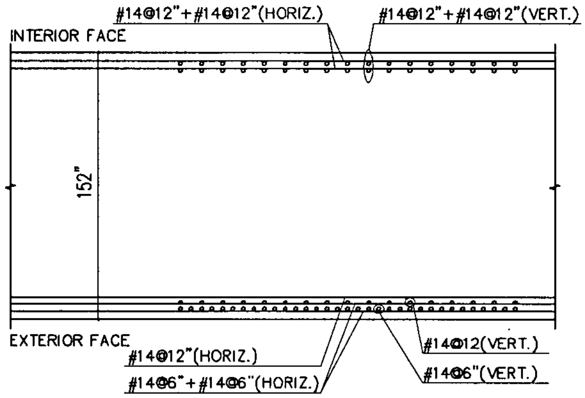


ZONE 1 [EL3'-7" to EL25'-3"]

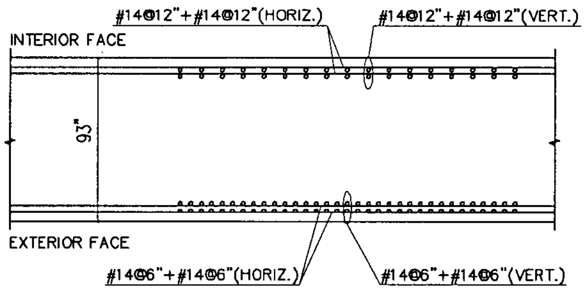
Horizontal Cross Section

Figure 3.8.4-5 Typical Reinforcement in South interior Wall – SECTION 2 (Sheet 2 of 2)

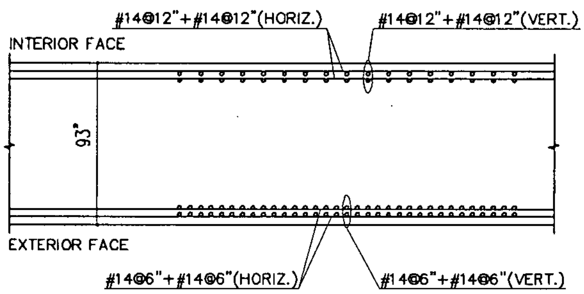
- Insert the following as Sheet 2 of Figure 3.8.4-6:



ZONE 3 [EL65'-0" to EL76'-5"]



ZONE 2 [EL50'-2" to EL65'-0"]

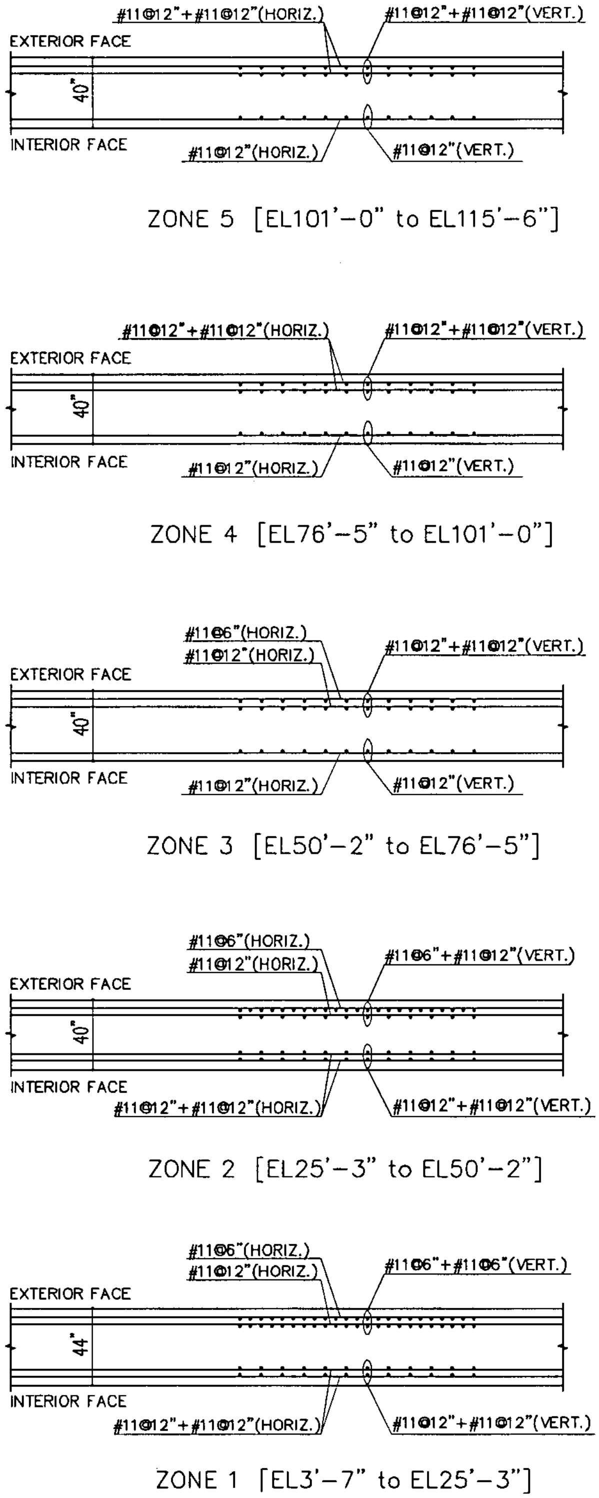


ZONE 1 [EL30'-1" to EL50'-2"]

<u>Horizontal Cross Section</u>

Figure 3.8.4-6 Typical Reinforcement in North Exterior Wall of Spent Fuel Pit – SECTION 3 (Sheet 2 of 2)

- Insert the following as Sheet 2 of Figure 3.8.4-7:



Horizontal Cross Section

Figure 3.8.4-7 Typical Reinforcement in South Exterior Wall – SECTION 4 (Sheet 2 of 2)

- Replace Figure 3.8.4-13 with the following:

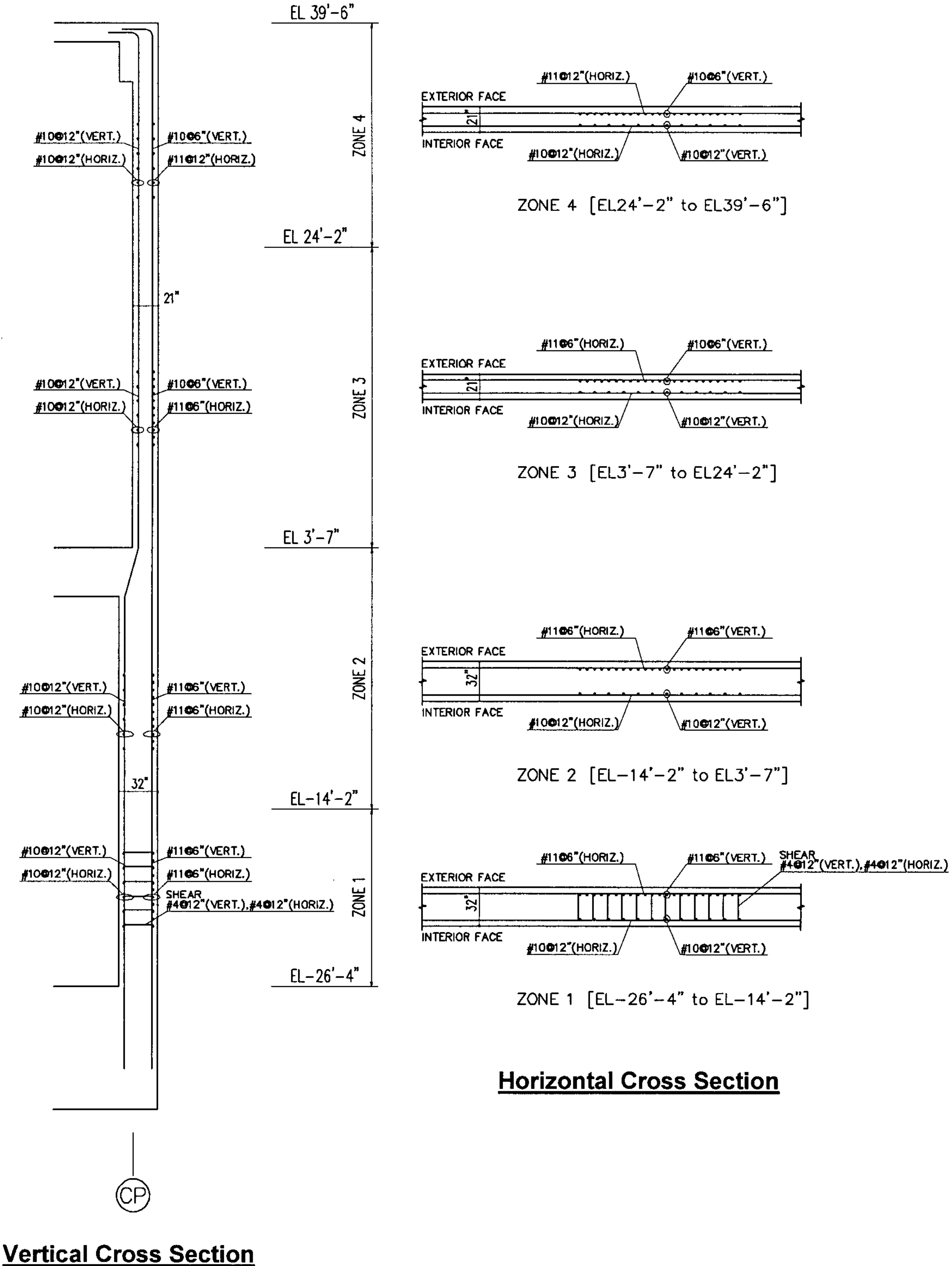
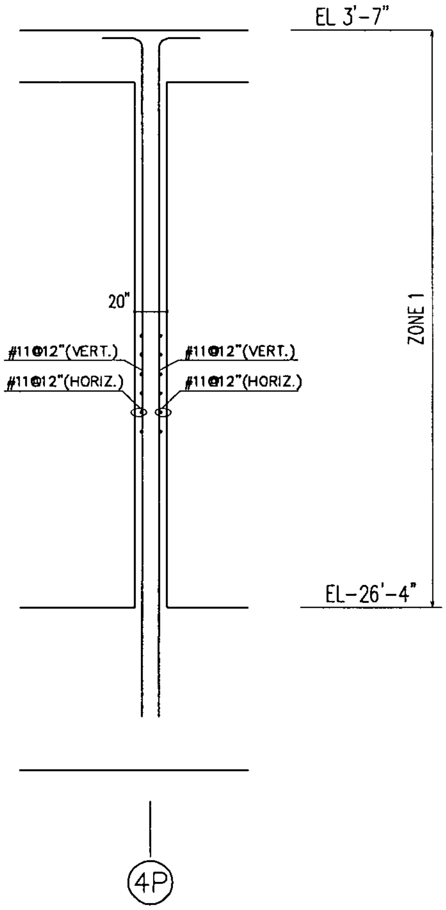
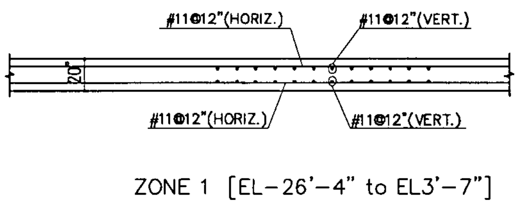


Figure 3.8.4-13 Typical Reinforcement in South Exterior Wall – SECTION 1 (On Column Line CP and Between Column Lines 1P & 2P)

- Replace Figure 3.8.4-14 with the following:



Vertical Cross Section



Horizontal Cross Section

Figure 3.8.4-14 Typical Reinforcement in Interior Wall – SECTION 2 (On Column Line 4P and Between Column Lines BP & CP)

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:	**03.08.04**
DATE OF RAI ISSUE:	**4/21/2009**

QUESTION NO.: 3.8.4-15

In DCD Subsection 3.8.4.4.2.1, the first paragraph (Page 3.8-59) states that the West PS/B is the worst case configuration and contains the most critical sections.

The applicant is requested to provide the following information:

Explain why the West PS/B is the worst configuration, and why the East PS/B is better. Why are they not the same? As shown in Figures 1.2-2 and 1.2-3, the East PS/B and West PS/B appear to be symmetric.

ANSWER:

In particular, the West PS/B is not worse configuration than the East PS/B. According to the simplified plane view in DCD Tier 2, Chapter 1, Figures 1.2-2 and 1.2-3, the West PS/B and the East PS/B are symmetrical structures, but the West PS/B has the tray space as shown in Figure 1 below. Accordingly, the weight of the West PS/B is larger than that of the East PS/B. The West PS/B also has unbalanced lateral soil pressures that are greater than those of the East PS/B. Therefore, the West PS/B is used as the representative building for the structure evaluation.

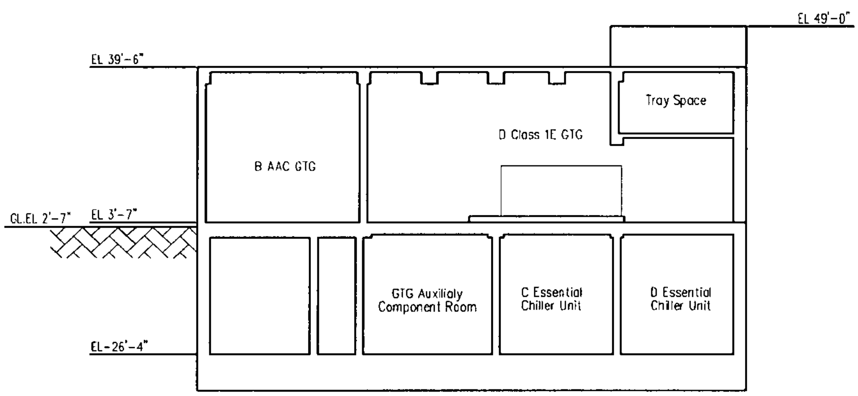

Figure 1 E-W Section of West

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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DATE OF RAI ISSUE:	**4/21/2009**

QUESTION NO.: 3.8.4-16

In DCD Subsection 3.8.4.4.3, the sixth paragraph states, "Lateral earth pressure is calculated in accordance with ASCE 4-98 (Reference 3.8-34) for both active and passive earth pressures."

The staff was unable to find the information for calculating the passive earth pressure in ASCE 4-98. Provide the section number of ASCE 4-98 where the guideline for calculating the passive earth pressure is given.

ANSWER:

ASCE 4-98 does not provide information for calculating passive earth pressures. As discussed in the response to Question 3.8.5-18 of RAI 340-2004, passive earth pressure is not relied upon in resisting overturning, sliding, or flotation forces for the US-APWR standard plant seismic category I buildings and structures. Therefore, the sixth paragraph of DCD Subsection 3.8.4.4.3 will be revised to delete the reference to ASCE 4-98 for calculating passive earth pressure. A grammatical correction is also made to the first sentence of that paragraph. Computation of lateral earth pressure is addressed further in the response to Question RAI 3.7.2-13 of RAI 212-1950, Revision 1.

Impact on DCD

See Attachment 1 for a mark-up of DCD Tier 2, Section 3.8 changes to be incorporated.

- Change the sixth paragraph of Subsection 3.8.4.4.3 to the following:

  "Exterior concrete walls below grade and basemats of seismic category I structures 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. Dynamic lateral earth pressure is calculated in accordance with ASCE 4-98 (Reference 3.8-34)."

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:	**4/21/2009**

QUESTION NO.: 3.8.4-17

In DCD Subsection 3.8.4.5, "Structural Acceptance Criteria", the first paragraph (Page 3.8-62) states, "Structural acceptance criteria are listed in Table 3.8.4-3 for concrete structures and in Table 3.8.4-4 for steel structures,……."

DCD Tables 3.8.4-3 and 3.8.4-4 are the load combinations for concrete structures and steel structures, respectively. These two tables were mentioned in DCD Subsection 3.8.4.3.9 (Page 3.8-55) with the title of "Load Combinations". The staff was unable to locate tables providing the structural acceptance criteria as stated in DCD subsection 3.8.4.5. The applicant is requested to provide information for the structural acceptance criteria.

ANSWER:

Tables 3.8.4-3 and 3.8.4-4 will be revised to include structural acceptance criteria.

Impact on DCD

See Attachment 1 for a mark-up of DCD Tier 2, Section 3.8 changes to be incorporated.

- Add the following row at the bottom of Table 3.8.4-3:

Acceptance Criteria(8)		U	U	U	U	U	U	U	U	U	U	U

- Add the following Note 8 at the end of Table 3.8.4-3 Notes:

  8. The required strength U shall be equal to or greater than the strength required to resist the factored loads and/or related internal moments and forces, for each of the load combinations shown in this table.

- Add the following sentence to the end of Table 3.8.4-4, Note 1:

"Calculated stresses shall not exceed allowable stresses for each of the load combinations shown in this table."

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:	**03.08.04**
DATE OF RAI ISSUE:	**4/21/2009**

QUESTION NO.: 3.8.4-18

In DCD Subsection 3.8.4.6.1.3, the second paragraph (Page 3.8-64) states, "Placement of concrete reinforcement is in accordance with ACI-349, Section 7.7 (Reference 3.8-8)."

The applicant is requested to provide the following information:

The title for ACI-349 Section 7.5 is "Placing reinforcement". The title for ACI-349 Section 7.7 is "Concrete protection for reinforcement". Which section, 7.5 or 7.7, was actually followed?

ANSWER:

After review of DCD Subsection 3.8.4.6.1.3 and ACI 349-01, it was found to be more appropriate to reference ACI 349 Sections 7.5, "Placing Reinforcement" and Section 7.6 "Spacing Limits for Reinforcement," instead of Section 7.7. DCD Section 3.8.4.6.1.3 has been revised to incorporate the appropriate reference.

Impact on DCD

See Attachment 1 for a mark-up of DCD Tier 2, Section 3.8 changes to be incorporated.

- Change the second sentence in the second paragraph of Subsection 3.8.4.6.1.3 to the following: "Placement of concrete reinforcement is in accordance with ACI-349 (Reference 3.8-8), Sections 7.5 and 7.6."

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:	**03.08.04**
DATE OF RAI ISSUE:	**4/21/2009**

QUESTION NO.: 3.8.4-19

In DCD Subsection 3.8.4.6.3, the paragraph (Page 3.8-68) states, "There are no special construction techniques utilized in the construction of other seismic category I structures."

The applicant is requested to describe what construction techniques and provisions are needed to address issues related to the use of the massive concrete pour of the basemat, such as the heat generated, the volume changes associated with the massive concrete pour, and the concrete cracking control.

ANSWER:

The thermal behavior of the basemat concrete pour is the most important characteristic that differentiates it from other concrete pours. Significant temperature differential between the interior and outside surface of the basemat could result due to heat of hydration caused by large concrete pours. When the temperature differential across the gradient is excessive, potential of cracking becomes a concern.

Standard provisions of ACI are anticipated to be applied where necessary to address issues related to the use of massive concrete pours. The following provisions will be needed for mass concrete pour of basemats to control heat generation, volume change effects and concrete cracking control:

(1) Use of low heat cement. For mass concrete, specify (a) ASTM C150 Type II cement with moderate heat of hydration and (b) Fly Ash (ASTM C618, Type F) up to 25% of cement content by weight.

    The temperature rise can be minimized by the use of minimal cement contents in the mixture, partial substitution of pozzolans for cement, and use of special type of cement with lower or delayed heat of hydration.

(2) Minimize change in the volume to the extent feasible. As reported in Section 1.3 of ACI 207.2R-07 (Reference 1), the change in volume can be minimized by such measures as reducing cement content, replacing part of the cement with pozzolans, precooling,

postcooling, insulating to control the rate of heat absorbed or lost and by other temperature control measures outlined in ACI 207.4R-05 (Reference 3).

(3) Use approaches for crack control as prescribed in Section 1.3 of ACI 207.2R-07 (Reference 1) and Section 7.2 of ACI 224R-01 (Reference 2). Mitigate concrete cracking by effective placement of reinforcement per provisions of ACI 349-01.

This approach can eliminate large cracks and replaces with many smaller cracks of acceptably smaller widths. However, this is achieved in the normal design practice since crack control is important for other issues such as leakage and corrosion.

Appropriate construction procedure can be used to meet the above provisions. The following are construction techniques commonly employed, either singularly or in combination, to mitigate the problems associated with massive concrete pours for basemats:

1. Limiting the size of concrete pour.
2. Use *checkerboard* pattern of concrete placement in a single lift. To avoid a weak horizontal shear plane, a double lift placement of concrete, in general, is avoided. However, when it is absolutely needed to have two lifts, there will be adequate design considerations and also, in general, shear stirrups will be provided.
3. Schedule pour for the most advantageous day and time to control temperature rise in the concrete.
4. Post-cooling can be performed by cooling the freshly placed concrete by running chilled water lines in the concrete.

The below references will be added to the DCD as indicated in "Impact on DCD".

REFERENCES

1. ACI 207.2R-07, "Report on Thermal and Volume Change Effects on Cracking of Mass Concrete," Reported by ACI Committee 207.

2. ACI 224R-01, "Control of Cracking in Concrete Structures," Reported by ACI Committee 224.

3. ACI 207.4R-05, "Cooling and Insulating Systems for Mass Concrete," Reported by ACI Committee 207.

Impact on DCD

See Attachment 1 for a mark-up of DCD Tier 2, Section 3.8 changes to be incorporated.

- Add ACI 224R-01 to the end of bulleted list in DCD Subsection 3.8.4.1.1 as follows:

  "• ACI-224R, Control of Cracking in Concrete Structures, American Concrete Institute, 2001 (Reference 3.8-54)."

- Add the following sentence at the end of the first paragraph of DCD Subsection 3.8.4.6.1.1: "During construction, volume changes in mass concrete are controlled where necessary by applying measures and provisions outlined in ACI 207.2R (Reference 3.8-52) and ACI 207.4R (Reference 3.8-53)."

- Add the following references to Subsection 3.8.5 of the DCD:

"3.8-52 Report on Thermal and Volume Change Effects on Cracking of Mass Concrete. ACI-207.2R, American Concrete Institute, 2007.

3.8-53 Cooling and Insulating Systems for Mass Concrete. ACI-207.4R, American Concrete Institute, 2005.

3.8-54 Control of Cracking in Concrete Structures. ACI-224R, American Concrete Institute, 2001."

Impact on COLA

There is no impact on the COLA.

Impact on PRA

There is no impact on the PRA.

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RAI NO.:	**NO. 342-2000 REVISION 0**
SRP SECTION:	**3.8.4 – Other Seismic Category I Structures**
APPLICATION SECTION:	**03.08.04**
DATE OF RAI ISSUE:	**4/21/2009**

QUESTION NO.: 3.8.4-20

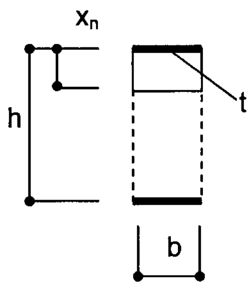
In DCD Subsection 3.8.3.4, the fifth paragraph (Page 3.8-38) states, "For thermal loads, design forces are calculated by multiplying the reduction ratio α, considering the reduction of stiffness by cracking to the result values of above analysis. The reduction ratio α is set to 0.5 as the reduction ratio of flexural stiffness caused by cracking for the typical member. For example, the flexural stiffness of cracked section for 48 in. wall with 0.5 in. plates assuming zero tensile strength of concrete is 22.2 by 10^9 lbs-in.^{2}/in., and the reduction ratio calculated by this value and elastic flexural stiffness (47.5 x 10^9 lbs-in.^{2}/in.) is 0.47."

The applicant is requested to provide the following information:

(a) The example given above indicated that α is 0.47 for 48-in walls. What are the values of α for the 56-in and 39-in walls? Provide justification for using 0.5 for all three walls.
(b) α is the reduction ratio for the flexural stiffness. DCD Table 3.8.3-4 (Page 3.8-92) indicates that the same value of α, 0.5, was applied to the axial stiffness and shear stiffness as well in the analyses. Provide the technical basis and data to show that the reduction factors for the axial stiffness and shear stiffness are 0.5.
(c) Provide test data that substantiate the values of the reduction factor α as stated.

ANSWER:

For parts (a) and (b), the ratio of the flexural stiffness for the cracked section EI_{cr} to elastic section EI_e of a 48-in steel-concrete (SC) module wall is calculated as shown below.



$x_n=t+2nt[-1+\sqrt{1+(h-2t)/2nt}]$, $n=E_s/E_c=29/3.605=8.04$

$EI_{cr}/E_c = bx_n^3/3 + (n-1)bt(x_n-t/2)^2 + nbt(h-x_n-t/2)^2$

$EI_e/E_c= bh^3/12 + 2(n-1)bt(h/2-t/2)^2$

$H = 48''$, $t = 0.5''$, $b = 1'' \rightarrow x_n = 13.5''$

$EI_{cr} = 3.605*10^6*(13.5^3/3 + 7.04*0.5*13.25^2 + 8.04*0.5*34.25^2)$

$=22.2*10^9$(psi)

$EI_e = 3.605*10^6*(48^3/12 + 2*7.04*0.5*23.75^2)$

$=47.5*10^9$(psi)

$\therefore\ \alpha = EI_{cr}/EI_e = 22.2/47.5 = 0.47 \rightarrow 0.5$

where b is the section width (unit width for the calculation), h is the section (wall) thickness, t is the faceplate thickness, x_n is the neutral axis, E is Young's modulus, I is moment of inertia, and each suffix s, c, cr, e is for steel, concrete, cracked section, and elastic section, respectively.

Using the same methodology, the calculation results for 56-in, 48-in, and 39-in thick walls are shown in the following Table 1.

Table 1 Stiffness Ratios

Thickness		Modulus Ratio	Neutral Axis Ratio	Ratio of Stiffness		
Section	Faceplate			Flexural	Axial	Shear
h(in.)	t(in.)	$n=E_s/E_c$	$x_{n1}=x_n/h$	EI_{cr}/EI_e	EA_{cr}/EA_e	$GA_{cr}/GA_e = x_{n1}$
56	0.5	8.04	0.267	0.43	0.35	0.27
48	0.5	8.04	0.281	0.47	0.38	0.28
39	0.5	8.04	0.300	0.52	0.42	0.30

Each ratio value of flexural stiffness, which is basis for α, proves to be nearly equal to or less than 0.5. Therefore the assumption of $\alpha = 0.5$ is reasonable. The value of EI_{cr}/EI_e for a reinforced concrete (RC) section of similar size would be less than that for the SC section because the reinforcement is located further away for the section edge than is the faceplate.

The calculation results of the reduction factor for axial stiffness EA_{cr}/EA_e are similarly shown in Table 1, and reflect values less than those for flexural stiffness. Reduction factors for shear stiffness reflect similar results.

Therefore, the ratio for flexural stiffness used for the basis of reduction factor α is reasonable.

Relating to part (c), Reference 1[1] shows the experimental results of heating tests using beam specimens which have 500 mm width (w) and 600 mm height (h), and show the relationship of residual ratio of flexural stiffness and tensile stress of reinforcement or plate (See Table 2 which is translated from original Japanese table shown below).

---

[1] Reference 1: Experimental Study on a Concrete Filled Steel Structure Part 12 Experiment for Thermal characteristics (Planning of Experiment and Results of Heating Test), and Part 13 Experiment for Thermal Characteristics (Results of Experiment), Kanda Sigeru, Michikoshi Shintaro, et al., pp. 1073-1076, the technical papers of annual meeting of AIJ (Architectural Institute of Japan), 1997.

Table 2 Comparison of Residual Ratio of Flexural Stiffness

Specimen \ Steel Stress	Tensile Steel Stress Level		Parameter
	Allowable Stress for Long Term	Allowable Stress for Short Term (Yield)	
S1.0-0T (Plate t=6.0mm)	0.60	0.56	Standard
S0.5-0T (Plate t=3.2mm)	0.47	0.43	Steel Ratio
S1.5-0T (Plate t=9.0mm)	0.70	0.66	Steel Ratio
R1.2-0T (RC Structure)	0.52	0.48	Structure
S1.030T (under Compression)	0.64	0.58	Axial Load
S1.0-0 (No Heating)	0.69	0.59	Thermal

Table 2 shows that, when the tensile steel stress level is "Allowable Stress for Short Term" and equal to yield stress, the SC section, which has 0.5%, 1.0%, 1.5% steel ratio, or the RC section, which has 1.2% steel ratio, has a residual stiffness ratio of 0.43, 0.56, 0.66, and 0.48, respectively.

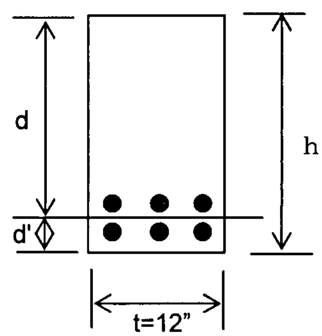
The US-APWR SC sections with 39", 48" or 56" section thicknesses and a 0.5" faceplate thickness result in steel-to-concrete ratios of 1.3%, 1.0% and 0.9%, respectively. The tests above show that SC sections with such steel ratios prove to be nearly 0.5 of residual ratio.

The ratios of flexural stiffness of the cracked section EI_{cr} to elastic section EI_e for the example of RC member (R/B, West Exterior Wall, Section 1) are shown Table 3.

Table 3 Ratios of Flexural Stiffness

R/B, West Exterior Wall, Section 1		Modulus Ratio	Ratio of Stiffness
Elevation	Thickness h(in.)		Flexural
		$n=E_s/E_c$	EI_{cr}/EI_e
El 3'-7" → El 25'-3" (ZONE 1)	40	8.04	0.34
El 25'-3" → El 50'-2" (ZONE 2)	40	8.04	0.33
El 50'-2" → El 76'-5" (ZONE 3)	32	8.04	0.21
El 76'-5" → El 101'-0" (ZONE 4)	28	8.04	0.22

The calculation example of ZONE 1 is shown below:



Tension reinforcement area (in^2)

$As := 3.12$

Compression reinforcement area (in^2)

$A's := 2.83$

Section properties

$h := 40$

$d' := (2.0 + 1.5 \cdot 1.410)$ $\qquad d' = 4.115$

$d := h - d'$ $\qquad d = 35.885$

$Ec := 57000 \cdot \sqrt{4000}$ $\qquad Ec = 3.605 \times 10^6$ psi

$Es := 2.9 \cdot 10^7$ psi

$n := \frac{Es}{Ec}$ $\qquad n = 8.044$

Moment of inertia of un-cracked concrete section neglecting reinforcement

$Ig := 12\frac{h^3}{12}$ $\qquad Ig = 6.4 \times 10^4$ in^4/ft

Neutral axis cracked reinforced concrete section

Given $\quad x := 1 \quad x > 0$

$$(n-1)\cdot A's\cdot(x-d') + n\cdot As\cdot(x-d) + \int_0^x 12y\,dy = 0$$

$xn := \text{Find}(x)$ $\qquad xn = 9.584$

Moment of inertia of cracked reinforced concrete section transformed to concrete

$$Icr := (n-1)\cdot A's\cdot(xn-d')^2 + n\cdot As\cdot(d-xn)^2 + \int_0^{xn} 12\cdot y^2\,dy \qquad Icr = 2.148\times 10^4 \ in^4/ft$$

Reduction factor for axial stiffness

$\alpha := \frac{Ec\cdot Icr}{Ec\cdot Ig}$ $\qquad \alpha = 0.336$ $\qquad$ conservatively rounded = 0.5

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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Mitsubishi Heavy Industries

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QUESTION NO.: 3.8.4-21

DCD Subsection 3.8.4.7,"Testing and Inservice Inspection Requirements", (Page 3.8-68) did not discuss any requirements for monitoring of settlement and differential displacements that are mentioned in SRP 3.8.4, Section I.7. In DCD Subsection 3.8.5.4.4,"Analysis of Settlement" (Page 3.8-73) states that, "The potential for foundation subsidence, or differential displacement, is designed for a maximum 2 in......"

The applicant is requested to provide the rationale why monitoring of settlement and differential displacement is not included in testing and inservice inspection requirements?

ANSWER:

The monitoring of settlement and differential displacement was inadvertently omitted from the testing and inservice inspection requirements, and will be added to DCD Subsection 3.8.4.7.

Impact on DCD

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

- Insert the following as the last sentence in the first paragraph of Subsection 3.8.4.7: "For seismic category I structures, monitoring is to include base settlements and differential displacements."
- Insert the following as the last sentence in COL 3.8(22) of Subsection 3.8.6: "For seismic category I structures, monitoring is to include base settlements and differential displacements."

See Attachment 5 for the mark-up of DCD Tier 2, Section 1.8, changes to be incorporated.

- Insert the following as the last sentence in COL 3.8(22) of Table 1.8-2 (sheet 13 of 44): "For seismic category I structures, monitoring is to include base settlements and differential displacements."

Impact on COLA

FSAR sections corresponding to the impacted DCD sections will need to be revised to be consistent with the DCD, including revising the COL Item statement in FSAR Table 1.8-201.

Impact on PRA

There is no impact on the PRA.

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QUESTION NO.: 3.8.4-22

In DCD Subsection 3.8.4.6.1.4,"Splices", the last sentence (Page 3.8-64) states, "Welding of reinforcing steel, other than in the PCCV, is performed in accordance with American Welding Society (AWS) D1.4 (Reference 3.8-46)."

The applicant is requested to provide the following information:

> In SRP Section 3.8.4, subsection I.6.B (Page 3.8.4-6 of SRP 3.8.4, Revision 2, March 2007), It is stated that, "If welding of reinforcing bars is proposed, it should comply with American Society of Mechanical Engineers (ASME), Boiler and Pressure Vessel Code (Code) Section III, Division 2. Any exception to compliance should be supported with adequate justification." Are the requirements of American Welding Society D1.4 the same as those of the ASME Code Section III, Division 2? If not, provide justification for this exception.

ANSWER:

SRP Section 3.8.4 covers "Other Seismic Category I Structures" and specifies in Subsections II.2 and II.3 the acceptance criteria of ACI 349 (Ref. 3.8-8) for concrete structures, supplemented with additional guidance from RG 1.142 (Ref. 3.8-19). ACI 349 Subsection 12.14.3.2 specifies that the welding of rebar welded splices shall conform to "Structural Welding Code – Reinforcing Steel" (ANSI/AWS D1.4).

SRP Section 3.8.1 covers "Concrete Containment" and specifies in Subsections II.2 and II.3 the acceptance criteria of ASME Code, Section II, Subsection CC (also known as ACI 359 (Ref. 3.8-2)) supplemented with additional guidance from RG 1.136 (Ref. 3.8-3). The NRC Staff provided supplementary guidance/clarification on (rebar) splices in the "Discussion" section of RG 1.136 on ASME, Section II, Subsection CC-4352: Splices, as follows:

> "Welded splices and other mechanical connections are allowed as long as they conform to ACI-349-01, Section 12.14.3 (Ref. 7). Regulatory Position C.8 gives guidance for splices."

The AWS code is a more recent industry standard and provides more welding requirements and details than the ASME BPV Code. AWS D1.4 is also a reference to the ASME BPV Code and the design processes presented within each code are very similar. For the welding of reinforcing steel, other than in the PCCV, the use of the American Welding Society D1.4 is more appropriate than the ASME BPV Code.

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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QUESTION NO.: 3.8.4-23

In DCD Subsection 3.8.4.6.1.1,"Concrete", only 4,000 psi concrete is included in the description. However, this subsection is referred by the DCD Section 3.8.5.6 for the material information used in the foundations. DCD Tier 2, Table 3.8.5-2 (page 3.8-108) indicates that in the basemat 7,000 psi concrete is used at the upper part of Tendon Gallery. The 7,000 psi concrete should be included in Subsection 3.8.4.6.1.1. Also include the codes and standards that 7,000 psi concrete needs to be in compliance with.

ANSWER:

DCD Subsection 3.8.4.6.1.1 will be revised to discuss 7,000 psi concrete and a cross-reference to Subsection 3.8.1.6 which addresses material requirements for 7,000 psi concrete. The 7,000 psi concrete of the PCCV and upper part of the tendon gallery, as well as the basemat directly under the PCCV as shown in Figure 3.8.5-4, is subject to the requirements of ASME III, Division 2.

Impact on DCD

See Attachment 1 for a mark-up of DCD Tier 2, Section 3.8 changes to be incorporated.

- Revise the first paragraph of Subsection 3.8.4.6.1.1 to the following:

  "Concrete utilized in standard plant seismic category I structures, other than PCCV and upper part of the tendon gallery in the basemat, has a compressive strength of f'_c = 4,000 psi. Concrete utilized in the PCCV and upper part of the tendon gallery in the basemat has a compressive strength of f'_c = 7,000 psi and is subject to the PCCV material requirements in Subsection 3.8.1.6, including the requirements in ASME III, Division 2 (Reference 3.8-2), as shown in Figure 3.8.5-4. The COL Applicant is to specify concrete strength utilized in non-standard plant seismic category I structures. A test of 28 days is used for normal concrete. Batching and placement of concrete is performed in accordance with ACI 349 (Reference 3.8-8), ACI 304R (Reference 3.8-38), and ASTM C 94 (Reference 3.8-42)."

Impact on COLA

There is no impact on the COLA.

Impact on PRA

There is no impact on the PRA.

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QUESTION NO.: 3.8.4-24

In DCD Subsection 3.8.4.6.1.7,"Masonry Walls", it (Page 3.8-67) states, "A non safety-related masonry wall exists in the spray pump room located at the lowest level of the R/B, which is not subjected to pressure loads and is restrained against seismic accelerations to preclude damage to safety-related SSCs."

The applicant is requested to provide the following information:

> Describe how the masonry is restrained against seismic accelerations. Provide information to show that the restraint works. What are the nearby safety-related SSCs?

ANSWER:

The masonry restraints are classified as seismic category II supports in order to prevent collapse of the non safety-related masonry walls in the containment spray pump rooms and in order to preclude interaction or damage to any nearby safety-related SSCs. The restraints will consist of structural steel plates and shapes that will be anchored to adjacent building concrete floors, ceilings, columns, and walls. The anchorage design will be in accordance with provisions and requirements of ACI-349 Appendix B, IE Bulletin 79-02, and RG 1.199 and will be consistent with anchorage design approaches described in the responses to Question 3.9.2-34 of RAI 214-1920, Revision 0, and Question 3.9.2-10 of RAI 205-1584, Revision 0. The restraints and anchorage are to be designed using the equivalent static method described in DCD Tier 2, Subsection 3.7.3.1 based on accelerations obtained from the applicable R/B in-structure response spectra. The detailed design of the restraints and anchorage is dependent on the revised in-structure response spectra which will be developed as discussed in the response to Question 3.7.2-8 of RAI 212-1950, Revision 1.

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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QUESTION NO.: 3.8.4-25

In DCD Subsection 3.8.4.2,"Applicable Codes, Standards, and Specifications", the first paragraph (Page 3.8-48) provides a list of the codes and standards that are applicable to other seismic category I structures. The staff compared this list with the codes, standards, guides, and specifications listed in SRP Section 3.8.4.II.2, and noted that RGs 1.69, 1.91, 1.115, 1.127, 1.142, 1.143, 1.160, and 1.199 are not included in the list. The applicant is requested to include these RGs in DCD Subsection 3.8.4.2 according to SRP 3.8.4.III.2.

ANSWER:

The DCD will be revised to incorporate the Regulatory Guides, listed within SRP Section 3.8.4.II.2, into the industry standards list contained in DCD Subsection 3.8.4.2.

Impact on DCD

See Attachment 1 for a mark-up of DCD Tier 2, Section 3.8 changes to be incorporated.

- Add the following to the end of first paragraph in Subsection 3.8.4.2, as twelfth through nineteenth bullet items:

  - RG 1.69, Concrete Radiation Shields for Nuclear Power Plants, U.S. Nuclear Regulatory Commission, December 1973 (Reference 3.8-20).
  - RG 1.91, Evaluations of Explosions Postulated to Occur on Transportation Routes Near Nuclear Power Plants, U.S. Nuclear Regulatory Commission, February 1978 (Reference 3.8-49).
  - RG 1.115, Protection Against Low-Trajectory Turbine Missiles, U.S. Nuclear Regulatory Commission, July 1977 (Reference 3.8-50).
  - RG 1.127, Inspection of Water-Control Structures Associated with Nuclear Power Plants, U.S. Nuclear Regulatory Commission, March 1978 (Reference 3.8-47).

- RG 1.142, Safety-Related Concrete Structures for Nuclear Power Plants (Other than Reactor Vessels and Containments), U.S. Nuclear Regulatory Commission, November 2001 (Reference 3.8-19).

- RG 1.143, Design Guidance for Radioactive Waste Management Systems, Structures, and Components Installed in Light-Water-Cooled Nuclear Power Plants, U.S. Nuclear Regulatory Commission, November 2001 (Reference 3.8-51).

- RG 1.160, Monitoring the Effectiveness of Maintenance at Nuclear Power Plants, U.S. Nuclear Regulatory Commission, March 1997 (Reference 3.8-30).

- RG 1.199, Anchoring Components and Structural Supports in Concrete, U.S. Nuclear Regulatory Commission, November 2003 (Reference 3.8-41).

- Add the following references to the end of Subsection 3.8.7 [Reference 3.8-48 was previously added by RAI 223-1996, Question 3.8.1-12]:

3.8-49 <u>Evaluations of Explosions Postulated to Occur on Transportation Routes Near Nuclear Power Plants</u>, RG 1.91, Rev. 1, U.S. Nuclear Regulatory Commission, Washington, DC, February 1978.

3.8-50 <u>Protection Against Low-Trajectory Turbine Missiles</u>, RG 1.115, Rev. 1, U.S. Nuclear Regulatory Commission, Washington, DC, July 1977.

3.8-51 <u>Design Guidance for Radioactive Waste Management Systems, Structures, and Components Installed in Light-Water-Cooled Nuclear Power Plants</u>, RG 1.143, Rev. 2, U.S. Nuclear Regulatory Commission, Washington, DC, November 2001.

Impact on COLA

There is no impact on the COLA.

Impact on PRA

There is no impact on the PRA.

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QUESTION NO.: 3.8.4-26

In DCD Subsection 3.8.4.4,"Design and Analysis Procedures", the last sentence of the first paragraph (Page 3.8-55) states, "Table 3.8.4-5 summarizes the modeling and analytical methods of R/B and PS/Bs." In DCD Table 3.8.4-5 (Page 3.8-99), the first column is "Computer Program and Model", and the first row is "Three-dimensional NASTRAN FE of R/B model fixed at elevation 3 ft, 7 in."; the second row is "Three-dimensional NASTRAN FE of R/B whole model".

The applicant is requested to provide the following information:

(a) Why not use the R/B whole model in the first row case?
(b) What is the boundary condition for the second row case? Does the R/B whole model include soil springs?

ANSWER:

(a) The first row case in Table 3.8.4-5 uses fixed boundary condition instead of the whole model for analysis. A similar issue was raised in this RAI, Question 3.8.4-11, and its response provides reasons for using fixed base conditions for the model. A design margin of 20% to 30% is added to members' seismic forces of R/B structure. Refer to Question 3.8.4-11 item (c) and (a), respectively for the previous two sentences. The fixed based model is more cost-effective compared to the whole model.

It is recognized that the whole model provides a more accurate analysis approach than that used in the first row case in DCD Table 3.8.4-5. However, the current design adds a design margin to the analysis results as described above. In view of staff suggestions to use the R/B whole model in the first row case, the US-APWR will conduct a confirmatory analysis using the whole model with soil springs for critical load case(s) that were determined based on the fixed base analysis. The whole model takes into considerations interaction between the subgrade, basemat and R/B superstructure. Thus, the whole model eliminates the assumption of the fixed base boundary conditions at the base of the model. The objective of the confirmatory analysis is to validate the fixed base analysis.

(b) The second row case represents a three-dimensional NASTRAN finite element model of the whole R/B model. The basemat is part of the whole model where boundary conditions caused by the subgrade to the basemat are utilized. Three linear translational springs at each node point of the basemat are used as the boundary condition to simulate subgrade effects on the basemat and supporting superstructure. Further description on the soil springs is provided in response to RAI 223-1996, Revision 0, Question 3.8.1-5.

In response to the second part of the question in Item (b), the whole R/B model employs soil springs as described in the above paragraph.

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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QUESTION NO.: 3.8.4-27

DCD Subsection 3.8.4.2,"Applicable Codes, Standards, and Specifications" (Page 3.8-47), lists ACI 318-99 and ACI 349-01 that are applicable for Other Seismic Category I Structures (Section 3.8.4). Also, DCD Subsection 3.8.4.2 is referred by DCD Subsection 3.8.3.2 (Page 3.8-34), the applicable codes, standards, and specifications for the Concrete and Steel Internal Structures of Concrete Containment (Section 3.8.3, Page 3.8-30).

The applicant is requested to provide the following information:

Identify the seismic Category I structure or structural elements that are designed in accordance with the requirements of ACI 318-99 Code, but not with ACI 349-01 Code. Provide the rationale for choosing the ACI 318-99 Code instead of ACI 349-01 Code.

ANSWER:

It was not intended that certain structures or structural elements be designed according to ACI 318-99 and others designed according to ACI 349-01. The reference of both ACI 318-99 and ACI 349-01 in DCD Subsection 3.8.4.2 was provided so that the two codes could be used in conjunction with each other, as well as all applicable Standard Review Plans and Regulatory Guides, based on the following statement in Regulatory Guide 1.142, Revision 2, Section B:

> "Some sections of ACI 318-1995 contain criteria on certain aspects on concrete structural design that are more current that those of ACI 349-97. When this is the case, ACI 318-1999 is recommended for use. ACI 318 has long been the basis for the design of concrete buildings in the United States, and it has been used by the NRC staff initially in the evaluating the adequacy of concrete structures in nuclear power plants."

The design of reinforced concrete structures discussed in Section 3.8 of the DCD is based on ACI 349-01, which includes updated provisions from ACI 318, and therefore the reference to ACI 318 is considered redundant and will be deleted from DCD Subsection 3.8.4.2 and from the references Subsection 3.8.7.

Impact on DCD

See Attachment 1 for a mark-up of DCD Tier 2, Section 3.8 changes to be incorporated.

- Delete the first bullet in the first paragraph of DCD Subsection 3.8.4.2: • ACI 318-99, Building Code Requirements for Structural Concrete, American Concrete Institute, 1999 (Reference 3.8-32).

- Replace Reference 3.8-32 in Subsection 3.8.7 with the following:

  "3.8-32 Deleted."

Impact on COLA

There is no impact on the COLA.

Impact on PRA

There is no impact on the PRA.

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SRP SECTION:	**3.8.4 – Other Seismic Category I Structures**
APPLICATION SECTION:	**03.08.04**
DATE OF RAI ISSUE:	**4/21/2009**

QUESTION NO.: 3.8.4-28

In DCD Subsection 3.8.4.4.3,"Other Seismic Category I Structures", the fourth paragraph (Page 3.8-61) states: "Members that are subject to torsion and combined shear and torsion are evaluated to the standards of Section 11.6 of ACI 318-99 (Reference 3.8-32) instead of the requirements of Section 11.6 of ACI 349 (Reference 3.8-8), as recommended by RG 1.142 (Reference 3.8-19)."

The above statement is confusing. DCD Reference 3.8-8 is ACI 349-01. The torsion requirements in Section 11.6 of ACI 349-01 are the same as those in Section 11.6 of ACI 318-99. The guidelines provided in RG 1.142 for torsion are meant for ACI 349-97 (not ACI 349-01) and ACI 318- 99. The design provisions for torsion were completely revised from ACI 349-97 to ACI 349-01.

Since the requirements for torsion in ACI 318-99 and ACI 349-01 are identical, the 4th paragraph in Subsection 3.8.4.4.3 quoted above does not appear to be needed. Explain the purpose of this paragraph.

ANSWER:

It is agreed that the reference to ACI 318-99 is redundant. The fourth paragraph of DCD 3.8.4.4.3 has been revised to reference only Section 11.6 of ACI 349 for the evaluation of concrete members subjected to torsion and combined shear and torsion.

Impact on DCD

See Attachment 1 for a mark-up of DCD Tier 2, Section 3.8 changes to be incorporated.

- Change the fourth paragraph of Subsection 3.8.4.4.3 to the following:

  "Concrete members that are subject to torsion and combined shear and torsion are evaluated to the standards of Section 11.6 of ACI 349 (Reference 3.8-8)."

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

7/03/2009

US-APWR Design Certification

Mitsubishi Heavy Industries

Docket No. 52-021

RAI NO.:	**NO. 342-2000 REVISION 0**
SRP SECTION:	**3.8.4 – Other Seismic Category I Structures**
APPLICATION SECTION:	**03.08.04**
DATE OF RAI ISSUE:	**4/21/2009**

QUESTION NO.: 3.8.4-29

In DCD Tier 2, Subsection 3.8.4.4.3, "Other Seismic Category I Structures", eighth paragraph, page 3.8-61, the applicant states, "The design and analysis procedures for seismic category I distribution systems, such as HVAC ducts, conduits, and cable trays including their respective seismic category I supports, are in accordance with AISC N690 (Reference 3.8-8) and AISI Specification for Design of Cold-Formed Steel Members (Reference 3.8-34). The following appendices provide additional discussion of the design and analysis of these subsystems.

- Appendix 3A Heating, Ventilation, and Air Conditioning Ducts and Duct Supports
- Appendix 3F Design of Conduits and Conduit Supports
- Appendix 3G Seismic Qualification of Cable Trays and Supports"

Some of the information in Appendix 3A is very general in nature.

The applicant is requested to provide more detailed information and answers to specific questions as follows:

1. The code, AMSE/ANSI AG-1-2003, "Code on Nuclear Air and Gas Treatment", provides minimum requirements for the performance, design, construction, acceptance testing, and quality assurance of equipment used as components in nuclear safety-related air and gas treatment systems in nuclear facilities. DCD Tier 2, Appendix 3A does not include this code in its list of codes and standards. Explain why ASME/ANSI is not used in the design of HVAC ducts and supports for the US-APWR plant.

2. In DCD Tier 2, Appendix 3A, "Heating, Ventilating, and Air Conditioning Ducts and Duct Supports", Subsection 3A.1, p. 3A-1, the applicant states that one of the actions taken in designing the HVAC ductwork and supports is to: "Qualify local stresses in ductwork at un-reinforced and reinforced openings". Describe what method(s) are used to accomplish these qualifying actions.

3. In DCD Tier 2, Appendix 3A, Subsection 3A.1.1, "Seismic Category I Ductwork", the applicant, in addressing the stress criteria used in the selection of duct member sizes and span lengths, states, "Typically stress criteria for ductwork and supports results in selection of standard member sizes and maximum span lengths. However, some HVAC

systems require a high degree of leak tightness, experience excessive pressures, or need to account for other external influences (such as tornados) that can require thicker members or closer support spacing." Describe what HVAC subsystems are included in this apparently more restrictive group (i.e., higher degree of leak tightness, etc.). Also, describe what stress criteria govern the design of these affected HVAC subsystems.

4. In DCD Tier 2, Appendix 3A, Subsection 3A.3, "Loads and Load Combinations", the applicant presents a general description of the loads and load combinations used in the design of the HVAC ductwork. Also, reference is made in 3A.3 of Appendix 3A to the use of DCD Table 3.8.4-4 for the load combinations used. Table 3.8.4-4 list several load combinations and associated allowable stress coefficients. Provide the specific loads and load combinations used in the HVAC ductwork and associated supports design, and confirm whether the values of the stress coefficients in Table 3.8.4-4 apply to the HVAC ductwork and supports. In addition, DCD Subsection 3.8.4.5 does not mention AISI. The DCD Table 3.8.4-4 mentioned in Subsection 3.8.4.5 for the load combinations and allowable stresses is based on AISC N690. Clarify whether AISI is also applicable to Subsection 3.8.4.5.

5. In DCD Tier 2, Appendix 3A, Subsection 3A.4, the applicant states, "Refer to Section 3.7 for seismic system analysis and qualification requirements of seismic category I and seismic category II SSCs and their supports." DCD Section 3.7 contains many detailed requirements for seismic design. Provide the specific subsection of DCD 3.7 that is used for the design and analyses procedures. In addition, the DCD presents two approaches for the design and analysis procedures: (1) Simplified Design Approach; and (2) Detailed Design Approach. Describe which HVAC subsystems are designed by either of these two approaches. In addition, clarify the first sentence in 3A.4.2, "For certain geometric and stiffness conditions, the seismic forces are more accurately analyzed for a duct subsystem, including supports." What are the "geometric and stiffness conditions"?

6. Appendix 3A, Subsection 3A.4.1 (Page 3A-3), "Simplified Design Approach" states, "A simplified analysis is applicable when the seismic accelerations are taken as 1.5 times peak of the support attachment spectrum and the system is isolated from any rod hung seismic category II duct." Provide the following information:

   Is "Simplified Design Approach" the same as "Equivalent Static Analysis"?
   If not, provide technical information for this approach. How is it performed?
   If yes, in SRP 3.9.2 Revision 3, March 2007, "Dynamic Testing and Analysis of Systems, Structures, and Components", the SRP Acceptance Criteria 2.A.(ii) states that, "An equivalent static load method is acceptable if:

   a. There is a justification that the system can be realistically represented by a simple model and the method produces conservative results in responses.
   b. The design and simplified analysis account for the relative motion between all points of supports.
   c. To obtain an equivalent static load of equipment or components which can be represented by a simple model, a factor of 1.5 is applied to the peak acceleration of the applicable floor response spectrum. A factor of less than 1.5 may be used with adequate justification."

   Provide detailed technical information to demonstrate that US-APWR design meets these criteria.

7. Appendix 3A, Subsection 3A.4.2 (Page 3A-3), "Detailed Design Approach" states that, "For certain geometric and stiffness conditions, the seismic forces are more accurately analyzed for a duct subsystem, including supports. This approach is considered when (a) the duct run is 3-dimensional, (b) the duct run contains a wye fitting, (c) the duct run

contains a branch tee fitting with dimensions within 6 inches of the main duct, (d) the duct run is not isolated from a rod hung category II duct, or (e) the duct and/or supports cannot be qualified using standard designs.
The detailed design approach utilizes an analytical model consisting of a duct run with multiple support points that also account for axial and lateral bracing. The subsystem is analyzed using the response spectrum analysis method for applicable operating and seismic loads, including any accessories and eccentricities that are present."

Provide the following information:

a. Explain what are the "standard designs" mentioned in the condition (e) of the first paragraph of the above quote.
b. Describe the "analytical model" mentioned in the first sentence of the second paragraph of the above quote.
c. Describe how the response spectrum analysis is carried out. How are the relative displacements at the support points considered?

8. Appendix 3A, Subsection 3A.1.2 (Page 3A-2), "Seismic Category II Ductwork", states that, "...structural steel in-plane stress limits are permitted to reach 1.0 F_y."
Provide the references for Codes, Standards and Specifications or the technical basis that permit structural steel in-plane stress to reach 1.0 F_y.

9. Appendix 3A, Subsection 3A.3.1 (Page 3A-2), "Loads", states that, "Supports are designed for dead, seismic, thermal loads, and airflow forces at duct elbows, as applicable. Ducts are also designed for the operational and accident pressure loads. Construction live load is considered, however, it is not present during design seismic events."
Provide the following information:
The values of construction live load, thermal loads, operational and accident pressure loads, and airflow forces at duct elbows considered. Also, are the overpressure transit loads due to rapid damper closure considered?

10. Appendix 3A, Subsection 3A.4.3 (Page 3A-3), "Axial Brace Spacing" states, "As a general rule, axial braces are spaced at intervals less than 50 feet for straight horizontal runs and less than 25 feet for straight vertical runs." Provide a reference for this "general rule".

11. Appendix 3A, Subsection 3A.6.5 (Page 3A-5), "Anchor Bolts", states "The flexibility of base plates is considered in determining the anchor bolt loads when expansion anchors are used for supports."
Provide the following information:
    a. Explain how the flexibility of base plates affects the anchor bolt loads?
    b. Explain how the anchor bolt loads are determined when cast-in-place anchor bolts are used for support. Is the base plate considered to be rigid?

ANSWER:

The responses will be provided in the order of the question outline.

1. MHI agrees to reference the requirements of ASME AG-1-2003, Code on Nuclear Air and Gas Treatment," including Addendum 1a and 1b, into the design of HVAC ductwork. See Item 4 below for further discussion.

2. Methods for qualification of local stresses at unreinforced and reinforced openings of ductwork are in accordance with the ASME Code on Nuclear Air and Gas Treatment and

the AISI Specification for the Design of Cold-Formed Steel Structural Members, as applicable. The ductwork may be analyzed by a simplified or detailed design approach. Either method of qualification evaluates the local stresses in ductwork at un-reinforced and reinforced openings.

The effective cross sections, reduction factors, and slenderness factors for rectangular ductwork are calculated and applied in relationship with the component of load. For example, longitudinal membrane stresses due to axial forces are determined considering the effective compression cross sectional area for shear lag and plate buckling of an unsupported length of thin metal ductwork, unless the opening is reinforced.

3. The qualification of all HVAC systems account for their associated loads and operating conditions. In most cases, HVAC systems are designed using pre-qualified standard member sizes and maximum lengths. However, there is the potential for HVAC systems to have loads and/or operating conditions that exceed the parameters used for the sizing of pre-qualified members and span lengths. The discussion in DCD Appendix 3A, Subsection 3A.1.1, is not intended to describe a subset of HVAC with special restrictions.

   DCD Appendix 3A, Subsection 3A.1.1, will be changed to indicate those HVAC systems that do not satisfy the parameters qualified for standard member sizes and maximum span lengths will be designed to satisfy their specific load and operating conditions.

4. Same questions were asked by the NRC Staff on the RAI 342-2000, Question 3.8.4-30, Item 4 and Item 8.

   The AISI specification specifies the type of loads (dead, live, impact, wind or seismic loads, etc.) that a structure using cold-formed steel structural members should be designed for. The AISI specification does not establish the dead, live, impact, wind or seismic, etc. loading requirements and does not provide load combinations. It makes the assumption that these loads and load combinations are adequately covered by the applicable building code or design standard. For HVAC subsystem design, the load combinations presented in Table 3.8.4-4 are applicable for the design of cold-formed steel structural members if they are used in the HVAC duct supports. The response to Question 3.8.4-30 adds Note 12 to Table 3.8.4-4 to state that load combinations and Stress Limit Coefficients are applicable for AISI design of cold-formed steel structural members used in subsystem supports, which includes HVAC duct supports.

   The AISI Specification is applicable for the design of cold-formed steel structural members if they are used in the subsystem HVAC (Appendix 3A) supports. The civil structural steel structures "normally" covered in DCD Section 3.8.4 typically use hot-rolled structural members and are designed and analyzed in compliance with the requirements of SRP 3.8.4 (i.e., AISC N690). DCD Appendix 3A was provided specifically to cover HVAC ducts and duct supports. This allows additional flexibility in the subsystem support design if cold-formed steel structural members are selected for use on the supports. Therefore, the AISI Specification is mentioned specifically in the Appendix Subsection 3.A.5.1.

   As documented in DCD Tier 2, Subsection 3.8.4 "Other Seismic Category I Structures", and DCD Appendices 3A, 3F and 3G on subsystem (HVAC ducts, conduit and cable tray) design, it was MHI's intention to analyze and design their supports to be in compliance with the NRC requirements of SRP 3.8.4. The NRC Staff has provided and documented their latest guidance and acceptance criteria for other Seismic Category I steel structures in SRP 3.8.4 Subsection II.3.B and Subsection II.5 which imposed the use of AISC N-

690-1994, including Supplement 2. This original MHI commitment in the DCD to use AISC N-690 for the HVAC duct supports has been maintained.

As noted in Item 1 above and to clarify the ductwork design, MHI agrees to reference the requirements of ASME AG-1-2003 into the design of the HVAC ductwork. Ductwork loads, load combinations, and acceptance criteria are being changed to align with ASME AG-1-2003, Code on Nuclear Air and Gas Treatment.

Subsection 3A.3.1 of Appendix 3A will be revised to reflect ductwork loads that are defined in AG-1-2003 for the ductwork. In addition, load combinations defined for Service Levels A, B, C, and D will be provided in Subsection 3A.3.2 for ductwork. As defined in AG-1-2003, stress criteria for ductwork is based on the AISI Specifications for the Design of Cold-Formed Steel Structural Members. Subsection 3A.5.1 of Appendix 3A will be revised accordingly.

5. DCD Subsection 3.7.2.1, "Seismic Analysis Methods," states that seismic subsystems are discussed in Subsection 3.7.3, and the modal response spectra and equivalent static load analysis methods are discussed in Subsection 3.7.3.1. However, Subsection 3.7.3.1, "Seismic Analysis Methods," states the methods are the same as those discussed in Subsection 3.7.2.1 and conform to the requirements of SRP 3.7.1 and SRP 3.7.2. Therefore, the applicable, referenced DCD Tier 2 Subsections are 3.7.2.1 and 3.7.3.

   The descriptions of design approaches within Section 3A.4 reflect a choice of methods used to qualify the HVAC subsystems that are implemented in the design criteria. A simplified design approach is used to determine standard member sizes and maximum span lengths provided for typical ductwork and support types. HVAC systems that satisfy the design parameters are qualified for these standard sizes and lengths are thereby designed using a "simplified design approach."

   The detailed design approach is used when the HVAC systems do not satisfy the design parameters of the simplified design approach, or for which a more accurate analysis of the duct system, including supports, is required. The phrase "certain geometric and stiffness conditions" in Subsection 3A.4.2 is in reference to configurations outside the parameters for duct run configurations described by conditions (a) through (e) that follow the statement, which includes those ducts and/or supports that cannot be qualified using the standard designs of the simplified design approach.

6. The "Simplified Design Approach" uses the principles of an equivalent static analysis. The US-APWR design and analysis of HVAC duct/duct support subsystems conforms to the guidance and criteria provided SRP 3.7.3.II.1, and SRP 3.7.2.II.1 (referenced by SRP 3.7.3.II.1) as presently stated in DCD Tier 2, Subsection 3.7.3. The "Equivalent Static Analysis" criteria in SRP 3.9.2, Acceptance Criteria 2.A.(ii) is also in SRP 3.7.2, Acceptance Criteria 1.B.

   The NRC Staff submitted similar questions relating to "Equivalent Static Analysis" methods in Questions 3.7.3-02, 3.7.3-03, 3.7.3-04 and 3.7.3-15 of RAI 213-1951. Please refer to the responses to these questions in RAI 213-1951 for further discussion.

7. Standard designs of HVAC ductwork and support configurations are to be utilized for duct routing, location of supports, and sizing of members when possible. In the context of the DCD, those ducts and/or supports that cannot be constructed using standard configurations require a detailed design analysis.

The analytical modeling of duct runs requiring a detailed design approach satisfies the requirements of DCD Subsection 3.7.3, which also conforms to the requirements of SRP 3.7.1 and SRP 3.7.2. The choice of the seismic analysis methods described in the subsections of DCD Subsection 3.7.3 is dependent on the desired level of precision, and the level of complexity of the particular HVAC segment being designed.

The methods of modal response spectra analysis available for the design of seismic category I and II HVAC duct runs are the envelope broadened response spectra method, the peak shifting method, the uniform support motion method and the independent support motion method. When there is more than one supporting structure, the independent support motion method for seismic response spectra may be used. In this case, each support group is considered to be in a random-phase relationship to the other support groups, and the responses caused by each support group are combined by the SRSS method. A support group is defined by supports that have the same time-history input, typically where all supports are located on the same floor of a structure.

8. MHI's responses to Question 3.9.2-40 in RAI 214-1920 and Question 3.8.1-14, Item 3 in RAI 223-1996 provide clarification and changes to the DCD relating to this question. Please refer to the MHI Answer and Impact on DCD for Question 3.8.1-14, Item 3 in RAI 223-1996.

9. Thermal loads, operational and accident pressure loads, and airflow forces at duct elbows are qualified using design values consistent with the operational parameters of the particular HVAC subsystem. Overpressure transient loads due to rapid damper closure result in fluid momentum loads (*FML*), and are evaluated for the applicable load combination provided in Appendix 3A, Table 3A-1.

   See Item 4 above for further discussion.

10. The provision of axial brace spacing "as a general rule" is intended as design guidance for the initial layout of ductwork subsystems. Actual brace configurations and spacing are established on design documents to satisfy the approved analytical qualification. Change "as a general rule" to "unless otherwise justified by analysis" in the second sentence of Subsection 3A.4.3.

11. Two similar, related questions were submitted by the NRC Staff on Question 3.9.2-10 in RAI 205-1584 and Question 3.9.2-34 in RAI 214-1920. As stated in MHI's responses to these two questions, it is intended that embedded plates, surface mounted plate with cast-in-place anchors, surface mounted plate with direct-bearing undercut expansion anchors, through-bolts, and/or grouted embedment and surface mounted plate with wedge-type or sleeve-type expansion anchors (where not excluded from use due to vibratory motion under normal operating conditions) be used as the anchorage for safety-related system, subsystem and components.

    Please refer to the responses to Questions 3.9.2-10 in RAI 205-1584 and 3.9.2-34 in RAI 214-1920 for further discussions on anchorage flexibility and stiffness.

Impact on DCD

For items 2, 5, 7, and 11, there is no impact to the DCD.

See Attachment 2 for the mark-up of DCD Tier 2, Appendix 3A, changes to be incorporated.

Item 1

- Insert the following as the first sentence in the first paragraph of Section 3A.2: "The design and construction of seismic category I HVAC systems conform to AG-1-2003, Code on Nuclear Air and Gas Treatment, including Addendums AG-1a and AG-1b (Reference 3A-8)."

- Revise the forth sentence in Section 3A.2 to the following:

  "Structural steel duct supports are designed and constructed in accordance with the American Institute of Steel Construction (AISC) Specification for the Design, Fabrication and Erection of Steel Safety Related Structures for Nuclear Facilities (Reference 3A-3) or AISI as applicable."

- Add the following at the end of references in Section 3A.7:

  "3A-8 Code on Nuclear Air and Gas Treatment. ASME AG-1-2003, Addendum AG-1a-2004, and AG-1b-2007, American Society of Mechanical Engineers, 2003."

Item 3

- Change the second sentence in the second paragraph of Subsection 3A.1.1 to the following: "Those HVAC systems that do not satisfy the parameters qualified for standard member sizes and maximum span lengths are designed to satisfy their specific load and operating conditions."

Item 4

- Refer to Impact on DCD for Question 3.8.4-30, Item 8 of this RAI

- Insert the following as the last paragraph in Subsection 3A.3.1:

  "The following loads are applicable for the ductwork load combinations:

  ADL Additional dynamic loads resulting from system excitations due to structural motion, such as that caused by safety relief valve actuation and other hydrodynamic loads due to the design basis accident (DBA), small pipe break accident (SBA), and intermediate pipe break accident (IBA).

  T Load from constraint of free end displacement resulting from thermal or other movements.

  DW Dead weight of equipment or ductwork including supports, stiffeners, insulation, all internally or externally mounted components or accessories, and any contained fluids.

  DPD Design pressure differential, resulting in dynamic pressure loads from DBA, IBA, or SBA.

  W Design wind loads due to design hurricane, design tornado, or other abnormal meteorological condition that could occur infrequently.

  EL External loads applied by attached piping, accessories, or other equipment.

  FML Fluid momentum loads other than those separately listed, such as the momentum and pressure forces due to fluid flow. Section SA-4211 of ASME AG-1-2003 contains additional clarification of applicable loads.

  L Live loads occurring during construction and maintenance, but may also be due to snow, ponded water, and ice. As a minimum, live load is equal to a construction manload of 250 pounds applied at the mid-span of the duct, midpoint of a stiffener, or within a duct panel. When applied on a panel, the load is distributed over a 10 square inch area.

NOPD Normal operating pressure differential, taken as the maximum positive or negative pressure differential that may occur during normal plant operation, including plant startup and test conditions. Included are pressures resulting from normal airflow and damper or valve closure.

SL Seismic loads resulting from the safe shutdown earthquake."

- Change the first paragraph in Subsection 3A.3.2 to the following:

  "Tables 3A-1 provides load combinations for ductwork. Refer to Subsection 3.8.4.3 for various load combinations applicable to seismic category I duct supports."

- Change the last sentence of the second paragraph in Subsection 3A.3.2 to the following: "Seismic category II duct supports are, therefore, qualified for the maximum seismic load combinations and associated allowable stresses as discussed in Subsection 3.8.4.3."

- Insert the following Tables 3A-1 at the end of Appendix 3A:

Table 3A-1

Ductwork Load Combinations

Component Service Level	Load Combinations
A	DW + NOPD + FML + EL + L + T + W
B	Not Required
C	DW + NOPD + FML + EL + SL + ADL + W
D	N + DPD + SSE + ADL, Not Required Unless DPD is Applicable

- Add the following as the first paragraph in Subsection 3A.5.1:

  "Allowable ductwork stresses are in accordance with AISI, Specifications for the Design of Cold-Formed Steel Structural Steel Members (Reference 3A-2) and ASME AG-1 (Reference 3A-8) Subarticles SA-4220, AA-4320 and AA-4330. Allowable ductwork support stresses are in accordance with AISC Specification for Structural Steel Buildings (Reference 3A-3) or AISI as applicable.

Item 6

- Refer to Impact on DCD for Questions 3.7.3-03 and 3.7.3-04 in RAI 213-1951.

Item 8

- Refer to Impact on DCD for Question 3.8.1-14, Item 3, in RAI 223-1996

Item 9

- Refer to Item 4, above, for related DCD Impact.

Item 10

- Revise the second sentence in Subsection 3A.4.3 to the following:

  "Unless otherwise justified by analysis, axial braces are spaced at intervals less than 50 feet for straight horizontal runs and less than 25 feet for straight vertical runs."

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

7/03/2009

US-APWR Design Certification

Mitsubishi Heavy Industries

Docket No. 52-021

RAI NO.:	**NO. 342-2000 REVISION 0**
SRP SECTION:	**3.8.4 – Other Seismic Category I Structures**
APPLICATION SECTION:	**03.08.04**
DATE OF RAI ISSUE:	**4/21/2009**

QUESTION NO.: 3.8.4-30

In DCD Tier 2, Subsection 3.8.4.4.3, "Other Seismic Category I Structures", eighth paragraph, page 3.8-61, the applicant states, "The design and analysis procedures for seismic category I distribution systems, such as HVAC ducts, conduits, and cable trays including their respective seismic category I supports, are in accordance with AISC N690 (Reference 3.8-8) and AISI Specification for Design of Cold-Formed Steel Members (Reference 3.8-34). Refer to the following appendices for additional discussion of the design and analysis of these subsystems.

- Appendix 3A Heating, Ventilation, and Air Conditioning Ducts and Duct Supports
- Appendix 3F Design of Conduits and Conduit Supports
- Appendix 3G Seismic Qualification of Cable Trays and Supports"

Some of the information in Appendix 3F is very general in nature.

The applicant is requested to provide more detailed information and answers to specific questions as follows:

1. In Appendix 3F, Section 3F.1 (Page 3F-1), "Description", it states, "Limit spacing of conduit supports to maintain conduit stresses within allowable stresses corresponding to the applicable load combinations." What is the maximum spacing limitation used for conduit supports? Provide the technical rationale for that value.

2. In Appendix 3F, Subsection 3F.1.2 (Page 3F-1), "Seismic Category II Conduit Systems", it states, "...structural steel in-plane stress limits are permitted to reach 1.0 *Fy*." Provide the references for Codes, Standards and Specifications or the technical basis that permit structural steel in-plane stress to reach 1.0 *Fy*. Is there an exception for cold formed steel?

3. Appendix 3F, Subsection 3F.3.1 (Page 3F-2), "Loads", states, "Conduit systems are designed for dead, live, and thermal loads, as applicable. Design dead load includes the working load (weight) of cables permitted in the conduit. In addition, any accessory loads to the conduit and conduit supports are included in the qualification of the conduit and conduit supports." Provide the following information:
    a. The values of live load and thermal loads considered.

b. Is there any construction live load?
c. Why is seismic load not included?
d. Specify the accessory loads included in the qualification of conduit and conduit supports.

4. Appendix 3F, Subsection 3F.3.2 (Page 3F-2), "Load Combinations", states, "Refer to Subsection 3.8.4.3 for various load combinations applicable to seismic category I SSCs.", and DCD Subsection 3.8.4.3.9 states, "Steel structures are designed using the allowable strength design method in accordance with AISC N690 (Reference 3.8-9) for the load combinations and allowable strength factors provided in Table 3.8.4-4." The load combinations presented in Tables 3.8.4-4 are those of AISC N690. Are the load combinations specified in AISI the same as those of AISC N690?

5. Appendix 3F, Subsection 3F.4 (Page 3F-2), "Design and Analysis Procedures" states, "Refer to Section 3.7 for seismic system analysis and qualification requirements of seismic category I and II SSCs and their supports." Provide the following information.
   a. Provide the exact subsection numbers where the information is presented.
   b. Table 3.7.3-1(a) and Table 3.7.3-1(b) (Pages 3.7-73 and -74) presented damping values for conduits and related supports for SSE and OBE, respectively. What is the conduit fill ratio assumed in the seismic analysis? Per ASCE 4-98, Section 3.5.5.2. (d), the damping values for conduit systems depend on the fill ratio.

6. Appendix 3F, Subsection 3F.4.1 (Page 3F-3), "Equivalent Static Analysis" states, "Equivalent static analysis determined seismic loads for conduit and conduit support systems as detailed 3.7.2.1. The masses considered included nominal size weights, concentrated weights, support members, cable, insulation, conduit (including cantilevers), flexible conduit, and other applicable components."
   Provide the following information:
   a. Explain what is "nominal size weights" and "flexible conduit". What is the difference between "conduit" and "flexible conduit" mentioned in the above quoted paragraph?
   b. In SRP 3.9.2, Revision 3, March 2007, "Dynamic Testing and Analysis of Systems, Structures, and Components", the SRP Acceptance Criteria 2.A.(ii) states, "An equivalent static load method is acceptable if:
      (1) There is a justification that the system can be realistically represented by a simple model and the method produces conservative results in responses.
      (2) The design and simplified analysis account for the relative motion between all points of supports.
      (3) To obtain an equivalent static load of equipment or components which can be represented by a simple model, a factor of 1.5 is applied to the peak acceleration of the applicable floor response spectrum. A factor of less than 1.5 may be used with adequate justification."
      Provide detailed technical information to demonstrate that US-APWR design meets these criteria above

7. Appendix 3F, Subsection 3F.4.2 (Page 3F-3), "Response Spectrum Modal Analysis", states, "For more exact results, conduit systems can be analyzed using the envelope broadened response spectra methods, considering uniform support motion, or the independent support motion method." Provide information for the following:
   The first approach mentioned in the above quote, the envelope broadened response spectra method, is referred to as the Uniform Support Motion (USM) method in SRP 3.7.3 Revision 3, March 2007, "Seismic Subsystem Analysis". It is required by SRP 3.7.3 that when USM is used, the relative displacements at the support points should be considered in addition to the USM calculation. The second approach mentioned in

the above quote is the independent support motion (ISM) method. SRP 3.7.3 specifies that if the ISM method is utilized, all of the criteria presented in NUREG-1061 related to the ISM method must be followed. The applicant is requested to provide technical information in the analysis that the SRP 3.7.3 requirements are met.

8. Appendix 3F, Subsection 3F.5.1 (Page 3F-3), "Allowable Stresses", states, "Allowable stress coefficients are applied in accordance with basic allowables of AISC or AISI. Refer to Subsection 3.8.4.5 for the combination of appropriate allowable stresses with the appropriate load combinations and material specifications."
   DCD Subsection 3.8.4.5 does not mention AISI. The DCD Table 3.8.4-4 mentioned in Subsection 3.8.4.5 for the load combinations and allowable stresses is based on AISC N690. Clarify whether AISI is also applicable to Subsection 3.8.4.5.

9. Appendix 3F, Subsection 3F.5.1.2 (Page 3F-3), "Conduit Supports", states, "Seismic category I and seismic category II supports are designed to withstand the combined effects of normal operating loads (dead weight) acting simultaneously with the seismic loadings."
   The applicant is requested to explain why the live loads and thermal loads are not included in the load combinations.

10. Appendix 3F, Subsection 3F.6.6 (Page 3F-4), "Anchor Bolts", states, "Anchor bolts used for conduit supports, seismic category I and II, are expansion anchors qualified in accordance with ACI 355.2 (Reference 3F-9). The flexibility of base plates was considered in determining the anchor bolt loads."
    Are any cast-in-place anchor bolts used? If not, explain why not? Explain how the flexibility of base plates is considered in determining the anchor bolt loads.

ANSWER:

The responses will be provided in the order of the question outline.

1. The methodology as discussed below is used to determine the standard (or maximum permissible) conduit spans (lengths) in the vertical, transverse and longitudinal directions during detailed design. It is an iterative process since the conduit is subjected to two direction bending and axial loading.

   The standard (or maximum permissible) conduit spans (lengths) or the conduit support spacing in the vertical and transverse directions are determined by manual hand calculations. It is controlled by the conduit's allowable bending stress for the governing SSE loading combination in each direction. The conduit is assumed as a simply supported conduit (or pipe) between two supports. The permissible conduit span for each specific type and size of conduit is then calculated from the equation of the determined conduit maximum bending moments (from the dead loads [conduit and cable fill] plus SSE) set equal to the allowable bending stress.

   The standard conduit spans (lengths) in the longitudinal direction are determined by manual hand calculations. It is controlled by the conduit's allowable axial (column) stress for the governing SSE loading combination. The conduit's dead load and cable fill is used to determine the maximum axial load on the conduit from the governing SSE loading combination. This is then set equal to the conduit's allowable axial stress and the longitudinal standard conduit span calculated.

As stated in the first paragraph, an iterative process is used to determine each size / type of conduit standard (or maximum permissible) spans (lengths) in the vertical, transverse and longitudinal directions during detailed design.

2. MHI's responses to Question 3.9.2-40 in RAI 214-1920 and Question 3.8.1-14, Item 3 in RAI 223-1996 provide clarification and changes to the DCD relating to this question. Please refer to the MHI Answer and Impact on DCD for Question 3.8.1-14, Item 3 in RAI 223-1996.

3.a The cable weight inside conduits, and any accessory loads to the conduit and conduit supports, are treated as part of the design dead load as stated in Subsection 3.F.3.1. No other loads, including construction loads, are applicable as live load to the conduit subsystem design. Subsection 3F.3.1 will be clarified by removing "live" as an applicable load.

As noted in Subsection 3F.3.2, the conduit systems are designed in accordance with Subsection 3.8.4.3 for the various load combinations applicable to seismic category I SSCs. Thermal loads due to an accident load case are considered in the conduit subsystem load combinations if applicable.

3.b As noted in DCD Subsection 3.8.4.3.4.6, construction loads are defined as a live load required by construction activities. Conduits and conduit supports are not appropriate for supporting additional loads required for construction activities, and construction loads are therefore not applicable to the design of conduits and conduit supports.

3.c Seismic loads are included in the load combinations defined in DCD Subsection 3.8.4.3. By reference to DCD Subsection 3.8.4.3, the conduit subsystem is designed for the seismic acceleration of the subsystem dead load.

3.d The statement within DCD Appendix 3F, Subsection 3F.3.1, recognizes accessories may exist that are applied loads to the conduit subsystem. When these loads exist, the accessory item is included as a dead load within the conduit subsystem, and is qualified on a case-by-case basis.

4. The AISI specification specifies the type of loads (dead, live, impact, wind or seismic loads, etc.) that a structure using cold-formed steel structural members should be designed. It does not establish the dead, live, impact, wind or seismic, etc. load loading requirements and does not provide load combinations. It makes the assumption that these loads and load combinations are adequately covered by the applicable building code or design standard. For conduit subsystem design, the load combinations presented in Table 3.8.4-4 are applicable for the design of cold-formed steel structural members if they are used in the conduit supports. The DCD will be revised to add Note 12 to Table 3.8.4-4 to state that load combinations and Stress Limit Coefficients are applicable for AISI design of cold-formed steel structural members used in conduit supports.

See the response to Item 8 below of this RAI for further discussion.

5.a DCD Subsection 3.7.2.1, "Seismic Analysis Methods," states that seismic subsystems are discussed in Subsection 3.7.3, and the modal response spectra and equivalent static load analysis methods are discussed in Subsection 3.7.3.1. However, Subsection 3.7.3.1, "Seismic Analysis Methods," states the methods are the same as those discussed in Subsection 3.7.2.1 and conform to the requirements of SRP 3.7.1 and SRP 3.7.2. Therefore, the applicable subsections referenced in DCD Tier 2 are Subsections 3.7.2.1 and 3.7.3.

5.b MHI's responses to Question 3.7.1-05 in RAI 211-1996 provided clarification and changes to the DCD relating to this question. Please refer to the MHI Answer and Impact on DCD for Question 3.7.1-05 in RAI 211-1996.

6.a "Nominal size weights" are the nominal weight of a conduit supplied by the conduit manufacturer based on the conduit size (diameter), nominal inside diameter and nominal outside diameter. "Flexible conduit" is a conduit which can "bend or flex", provide load isolation, and provide protection for wiring installations which are subjected to movement or vibration, oil, moisture, corrosive conditions, etc. For power plant applications, flexible metal liquid tight conduit manufactured by vendors such as Anamet, Inc. or Electro-Flex Company are normally used to provide protection and seismic load isolation between electrical equipment and rigid conduits.

6.b The US-APWR design and analysis of conduits and conduit support subsystems conforms to the guidance and criteria provided SRP 3.7.3.II.1, and SRP 3.7.2.II.1 (referenced by SRP 3.7.3.II.1) as presently stated in DCD Tier 2 Subsection 3.7.3. The "Equivalent Static Analysis" criteria in SRP 3.9.2 Acceptance Criteria 2.A.(ii) is also in SRP 3.7.2 Acceptance Criteria 1.B.

The NRC Staff submitted similar questions relating to "Equivalent Static Analysis" methods in Questions 3.7.3-02, 3.7.3-03, 3.7.3-04 and 3.7.3-15 of RAI 213-1951. Please refer to the responses to these questions in RAI 213-1951 for further discussion.

7. The NRC Staff submitted a similar question relating to USM and ISM methods in Question 3.9.2-41 of RAI 214-1920. Please refer to the responses to Question 3.9.2-41 of RAI 214-1920 for further discussion.

8. The AISI Specification is applicable for the design of cold-formed steel structural members if they are used in the conduit (Appendix 3F) and cable tray (Appendix 3G)) subsystems. The civil structural steel structures discussed in DCD Section 3.8.4 use hot-rolled structural members, and are designed and analyzed in compliance with the requirements AISC N690 in compliance with SRP 3.8.4. DCD Appendices 3F and 3G specifically address conduits and conduit supports, and cable trays and supports, respectively. This allows additional flexibility in the subsystem support design if cold-formed steel structural members are selected for use on the supports. Therefore, the AISI Specification is mentioned specifically in the Appendices (Subsections 3.F.5.1 and 3.G.5.1).

   See the above response to Item 4 of this RAI for further discussion.

9. As noted in the response to Question 3.8.4-30, Item 3a above, cable weight inside conduits, and any accessory loads to the conduit and conduit supports, are treated as part of the design dead load. No other loads, including construction loads, are applicable as live load to the conduit subsystem design.

   Also as noted in the response to Question 3.8.4-30, Item 3a above, the conduit systems are designed in accordance with Subsection 3.8.4.3 for the various load combinations applicable to seismic category I SSCs. Thermal loads due to an accident load case are considered in the conduit subsystem load combinations if applicable.

10. Two similar, related questions were submitted by the NRC Staff on Question 3.9.2-10 in RAI 205-1584 and Question 3.9.2-34 in RAI 214-1920. As stated in MHI's responses to these two questions, it is intended that embedded plates, surface mounted plate with cast-in-place anchors, surface mounted plate with direct-bearing undercut expansion anchors, through-bolts, and/or grouted embedment and surface mounted plate with wedge-type or sleeve-type

expansion anchors (where not excluded from use due to vibratory motion under normal operating conditions) be used as the anchorage for safety-related system, subsystem and components.

Please refer to the responses to questions 3.9.2-10 in RAI 205-1584 and 3.9.2-34 in RAI 214-1920 for further discussions on anchorage flexibility and stiffness.

Impact on DCD

For Items 1 and 10, there is no impact to the DCD.

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

Item 4

- Refer to Impact on DCD for Item 8 of this RAI.

Item 5

- Refer to Impact on DCD for Question 3.7.1-05 in RAI 211-1996.

Item 6

- Refer to Impact on DCD for Questions 3.7.3-03 and 3.7.3-04 in RAI 213-1951.

Item 7

- Refer to Impact on DCD for Question 3.9.2-41 in RAI 214-1920.

Item 8

- Replace the title of Table 3.8.4-4 with the following:

Table 3.8.4-4 Load Combinations and Load Factors for Seismic Category I Steel Structures <u>(Sheet 1 of 2)</u>

- Insert new Sheet 2 of 2 of Table 3.8.4-4, with the following title:

Table 3.8.4-4 Load Combinations and Load Factors for Seismic Category I Steel Structures <u>(Sheet 2 of 2)</u>

- Insert the following Note 12 at the end of Notes on Table 3.8.4-4 (Sheet 2 of 2):

"12. <u>Load combinations and stress limit coefficients are applicable for AISI design of cold-formed steel structural members used in subsystem supports. Allowable strengths per AISI may be increased by the stress limit coefficients shown, subject to the limits noted in this table. The allowable strength shall equal or exceed the required strength calculated, in accordance with AISI, for each of the load combinations shown in this table.</u>"

- Revise the first column of last row in Table 3.8.4-4 to the following:

"Stress Limit

Coefficient
(1)(2)(8)(12)"

See Attachment 3 for the mark-up of DCD Tier 2, Appendix 3F, changes to be incorporated.

Item 2

- Refer to Impact on DCD for Question 3.8.1-14, Item 3 in RAI 223-1996.

Item 3

- Change the first sentence in Subsection 3F.3.1 to the following: "Conduit systems are designed for dead, seismic, and thermal loads, as applicable."

Item 4

- Refer to Impact on DCD for Item 3 of this RAI.

Item 9

- Refer to Impact on DCD for Item 3 of this RAI.

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

7/03/2009

US-APWR Design Certification

Mitsubishi Heavy Industries

Docket No. 52-021

RAI NO.:	**NO. 342-2000 REVISION 0**
SRP SECTION:	**03.08.04 – Other Seismic Category I Structures**
APPLICATION SECTION:	**03.08.04**
DATE OF RAI ISSUE:	**4/21/2009**

QUESTION NO.: 3.8.4-31

In DCD Tier 2, Subsection 3.8.4.4.3, eighth paragraph, "Other Seismic Category I Structures", the applicant states, "The design and analysis procedures for seismic category I distribution systems, such as HVAC ducts, conduits, and cable trays including their respective seismic category I supports, are in accordance with AISC N690 (Reference 3.8-8) and AISI Specification for Design of Cold-Formed Steel Members (Reference 3.8-34). The following appendices provide additional discussion of the design and analysis of these subsystems.

- Appendix 3A Heating, Ventilation, and Air Conditioning Ducts and Duct Supports
- Appendix 3F Design of Conduits and Conduit Supports
- Appendix 3G Seismic Qualification of Cable Trays and Supports"

Some of the information in Appendix in 3G is very general in nature.

The applicant is requested to provide more detailed information and answers to specific questions as requested in the following:

1. In Appendix 3G, Section 3G.1 (Page 3G-1), "Description", it states, "Limit spacing of tray supports to maintain tray stresses within allowable stresses corresponding to the applicable load combination."
   Provide the maximum spacing limitation used for tray supports, and the technical rationale for that value.

2. In Appendix 3G, Subsection 3G.1.2 (Page 3G-1), "Seismic Category II Cable Tray Systems", the last sentence states, "....structural steel in-plane stress limits are permitted to reach 1.0 F_y." Provide the references for Codes, Standards, and Specifications or other technical basis that permit structural steel in-plane stresses to reach 1.0 F_y.

3. Appendix 3G, Section 3G.2 (Page 3G-2), "Applicable Codes, Standards and Specifications", lists the National Electric Manufacturers Association (NEMA) Standards VE-1 and VE-2, National Electric Code Article 392, 2002, AISI Specification for the Design of Cold-Formed Steel Members, and AISC N690. The applicant is requested to include the following code, specified in SRP 3.7.3,"Seismic Subsystem Analysis":

Institute of Electrical and Electronic Engineers (IEEE), Standard 344-1987, IEEE Recommended Practice for Seismic Qualification of Class 1E Equipment for Nuclear Power Generation Stations.

4. In Appendix 3G, Subsection 3G.3.1 (Page 3G-2), "Loads", it states, "Cable tray systems are designed for dead, live, seismic, and thermal loads, as applicable. Design dead load includes the working load (weight) of cables permitted in the tray (also known as "raceway"). Construction live load is considered in addition to the maximum weight of cables and trays." DCD Subsection 3.8.4.3.4.6,"Construction Loads", does not provide this information. The applicant is requested to provide information for the values of live load, thermal load, and Construction Live Load considered in Appendix 3G.

5. Appendix 3G, Subsection 3G.3.2 (Page 3G-2), "Load Combinations", refers to DCD Subsection 3.8.4.3 for the information, and DCD Subsection 3.8.4.3.9 states, "Steel structures are designed using the allowable strength design method in accordance with AISC N690 (Reference 3.8-9) for the load combinations and allowable strength factors provided in Table 3.8.4-4." The load combinations presented in Tables 3.8.4-4 are those of AISC N690. Are the load combinations specified in AISI the same as those of AISC N690? The applicant is requested to include AISI in the description of Subsection 3.8.4.3.

6. Appendix 3G, Subsection 3G.4 (Page 3G-2), "Design and Analysis Procedures" states, "Refer to Section 3.7 for seismic system analysis and qualification requirements of seismic category I and II SSCs and their supports." Provide the following information.
   a. Provide the exact subsection numbers where the information is presented.
   b. Table 3.7.3-1(a) and Table 3.7.3-1(b) (Pages 3.7-73 and -74) presented damping values for full cable trays and empty cable trays for SSE and OBE, respectively. What is the cable fill ratio assumed in the seismic analysis?
   c. Per ASCE 4-98, Section 3.5.5.2, the damping values for cable trays depend on the input acceleration level, cable fill ratio, and the ability of the cables to move within the trays during the seismic event. In US-APWR design, are cables restrained by spray-on fire protection materials? Is it a welded steel cable tray system or a bolted steel cable tray system?

7. Appendix 3G, Subsection 3G.4.1 (Page 3G-2), "Equivalent Static Analysis" states, "Using equivalent horizontal and vertical static forces applied at the center ofgravity of the various masses, the cable tray system is conservatively modeled to develop standard tray spans and support designs. The seismic accelerations are taken as 1.5 times peak of the support attachment spectrum during this analysis except when technical justification is provided for a lower factor unique to certain configuration." Provide the following information:
   a. Technical information for how the cable tray system is modeled to develop standard tray spans and support designs.
   b. In SRP 3.9.2 Revision 3, 2007,"Dynamic Testing and Analysis of Systems, Structures, and Components", the SRP Acceptance Criteria 2.A.(ii) states that, "An equivalent static load method is acceptable if:
      (1) There is a justification that the system can be realistically represented by a simple model and the method produces conservative results in responses.
      (2) The design and simplified analysis account for the relative motion between all points of supports.
      (3) To obtain an equivalent static load of equipment or components which can be represented by a simple model, a factor of 1.5 is applied to the peak acceleration of the applicable floor response spectrum. A factor of less than 1.5 may be used with adequate justification."

The DCD meets criterion (3) above only. The DCD also needs to meet criteria (1) and (2).

8. Appendix 3G, Subsection 3G.4.2 (Page 3G-3), "Modal Response Spectrum Analysis", states, "For more exact results, cable tray systems can be analyzed using the envelope broadened response spectra methods, considering uniform support motion, or the independent support motion method."
   The first approach mentioned in the above quote, the envelope broadened response spectra method, is referred to as the Uniform Support Motion (USM) method in SRP 3.7.3 Revision 3, 2007,"Seismic Subsystem Analysis". SRP 3.7.3 states that when USM is used, that the relative displacements at the support points should be considered in addition to the USM calculation. The second approach mentioned in the above quote is the independent support motion (ISM) method. SRP 3.7.3 specifies that if the ISM method is utilized, all of the criteria presented in NUREG-1061 related to the ISM method must be followed. The applicant is requested to state whether the analyses performed meet the guidance in SRP 3.7.3, regardless of which of the two methods are used, USM or ISM.

9. Appendix 3G, Subsection 3G.5.1 (Page 3G-3), "Allowable Stresses", states, "Allowable stress coefficients are applied in accordance with basic allowable of AISC or AISI. Refer to Subsection 3.8.4.5 for the combination of appropriate allowable stresses with the appropriate load combinations and material specifications."
   DCD Subsection 3.8.4.5 does not mention AISI. The DCD Table 3.8.4-4 mentioned in Subsection 3.8.4.5 for the load combinations and allowable stresses is based on AISC N690. Clarify whether AISI is also applicable to Subsection 3.8.4.5.

ANSWER:

The responses will be provided in the order of the question outline.

1. Refer to the response to Item 7A below.

2. MHI's responses to Question 3.9.2-40 in RAI 214-1920 and Question 3.8.1-14, Item 3 in RAI 223-1996 provide clarification and changes to the DCD relating to this question. Please refer to the MHI Answer and Impact on DCD for Question 3.8.1-14, Item 3 in RAI 223-1996.

3. Reference to the Institute of Electrical and Electronic Engineers (IEEE), Standard 344-1987 and Standard 344-2004 (for determination of the number of earthquake cycles for fatique analysis) has been appropriately made in DCD Tier 2, Subsection 3.7.1.1, page 3.7-5, "OBE" and Subsection 3.10, "Seismic and Dynamic Qualification of Mechanical and Electrical Equipment). It is not necessary to make reference to IEEE Standard 344 in Appendix G.

4. The cable weight inside trays, and any accessory loads to the tray and cable tray supports, are treated as part of the design dead load as stated in Subsection 3.G.3.1. Construction live load considered in Appendix 3G is a single concentrated design live load of 250 pounds from one person plus equipment carried by the person, assumed to be on top of the tray or on a horizontal member of the cable tray support. DCD Appendix 3G, Subsection 3G.3.1 will be revised to define the construction live load.

   As noted in Subsection 3G.3.2, the cable tray systems are designed in accordance with Subsection 3.8.4.3 for the various load combinations applicable to seismic category I SSCs. Thermal loads due to an accident load case are considered in the cable tray subsystem load combinations, if applicable.

5. Refer to the response to Question 3.8.4-30, Item 4, for further discussion on applicability of the AISI specification for the load combinations presented in Table 3.8.4-4.

6.a DCD Subsection 3.7.2.1, "Seismic Analysis Methods," states that seismic subsystems are discussed in Subsection 3.7.3, and the modal response spectra and equivalent static load analysis methods are discussed in Subsection 3.7.3.1. However, Subsection 3.7.3.1, "Seismic Analysis Methods," states the methods are the same as those discussed in Subsection 3.7.2.1 and conform to the requirements of SRP 3.7.1 and SRP 3.7.2. Therefore, the applicable subsections referenced in DCD Tier 2 are Subsections 3.7.2.1 and 3.7.3.

6.b The damping values for full and empty cable trays as presented in DCD Tier 2, Subsection 3.7.1.2, Tables 3.7.3-1(a) and 3.7.3-1(a) are in accordance with RG 1.6.1. In the US-APWR cable tray design, enveloped seismic (SSE) member forces / moments derived from the seismic 10% damping (for full cable tray) case and the seismic 7% damping (for empty cable tray) case are used in the member design. It is expected that the use of enveloped seismic member forces / moments will enveloped all cable tray fill ratios. This will be confirmed during final detailed design.

6.c Refer to the response on damping values in Part 6.b above. In the US-APWR design, cables in the cable trays are not restrained by any spray-on fire protection material and bolted steel cable trays are used.

7.a The methodology as discussed below is used to determine the standard (or maximum permissible) cable tray spans (lengths) in the vertical, transverse and longitudinal directions during detailed design. It is an iterative process since the tray is subjected to two directions of bending and axial loading.

The standard (or maximum permissible) cable tray spans (lengths) or the cable tray support spacing in the vertical and transverse directions are determined by manual hand calculations. It is controlled by the tray's allowable bending stress for the governing SSE loading combination in each direction. The cable tray is assumed as a simply supported tray between two supports. The permissible cable tray span for each specific type and size of tray is then calculated from the equation of the determined tray maximum bending moments (from the dead loads [cable tray weight and cable fill] plus SSE) set equal to the allowable bending stress.

The standard cable tray spans (lengths) in the tray longitudinal direction are determined by manual hand calculations. It is controlled by the tray's allowable axial (column) stress for the governing SSE loading combination. The tray's dead load and cable fill is used to determine the maximum axial load on the tray from the governing SSE loading combination. This is then set equal to the tray's allowable axial stress and the longitudinal standard cable tray span calculated.

As stated in the first paragraph, an iterative process is used to determine each size / type of cable tray standard (or maximum permissible) cable tray spans (lengths) in the vertical, transverse and longitudinal directions during detailed design.

7.b The US-APWR design and analysis of cable tray and cable tray support subsystems conforms to the guidance and criteria provided SRP 3.7.3.II.1, and SRP 3.7.2.II.1 (referenced by SRP 3.7.3.II.1) as presently stated in DCD Tier 2 Subsection 3.7.3. The "Equivalent Static Analysis" criteria in SRP 3.9.2 Acceptance Criteria 2.A.(ii) is also in SRP 3.7.2 Acceptance Criteria 1.B.

The NRC Staff submitted similar questions relating to "Equivalent Static Analysis" methods in Questions 3.7.3-02, 3.7.3-03, 3.7.3-04 and 3.7.3-15 of RAI 213-1951. Please refer to the responses to these questions in RAI 213-1951 for further discussion.

8. The same information request was made by the NRC Staff in Question 3.9.2-41 in RAI 214-1920 and Question 3.7.3-01 in RAI 213-1951. See MHI responses to Questions 3.9.2-41 and 3.7.3-01,

9. Refer to the response to Question 3.8.4-30, Item 8, for further discussion on applicability of the AISI specification for allowable stresses and stress limit coefficients presented in Table 3.8.4-4.

Impact on DCD

For Items 1, 3, 6 and 7, there is no impact on the DCD.

See Attachment 4 for the mark-up of DCD Tier 2, Appendix 3F, changes to be incorporated.

Item 2

- Refer to Impact on DCD for Question 3.8.1-14, Item 3 in RAI 223-1996.

Item 4

- Change the third sentence in Subsection 3G.3.1 to the following: "Construction live load, defined as 250 pounds from one person plus equipment carried by the person, is considered in addition to the maximum weight of cables and trays; however it is not present during design seismic events."

Item 5

- Refer to Impact on DCD for Question 3.8.4-30, Item 4 in this RAI.

Item 8

- Refer to Impact on DCD for Question 3.7.3-01 in RAI 213-1951.

Impact on COLA

There is no impact on the COLA.

Impact on PRA

There is no impact on the PRA.

This completes MHI's responses to the NRC's questions.

Discussion of design methodology, applicable loads, load combinations and acceptance criteria within this subsection is applicable for the R/B structures and the east and west PS/Bs, which are part of the US-APWR standard plant.

The COL Applicant is responsible for the seismic design of those seismic category I and seismic category II SSCs not part of the US-APWR standard plant, including the following non-standard seismic category I structures designed to the site-specific SSE:

- ESWPT
- UHSRS
- PSFSVs

Non-standard seismic category I SSCs are site-specific, and are designed for the site specific or more conservative SSE based on the ground motion response spectra, the site-specific foundation input response spectra, and the minimum response spectrum as described in Subsection 3.7.1.1.

3.8.4.1 Description of the Structures

Seismic category I buildings, except the R/B, PCCV, and containment internal structure, are free standing on separate concrete basemats and are primarily reinforced concrete structures. The R/B, PCCV, and containment internal structure share a common basemat; however, they are otherwise independent of each other. Adjoining building basemats 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 basemat subgrade for the interface between the R/B, and east and west PS/Bs. To be consistent with seismic modeling requirements of Section 3.7, no 4 in. gap is permitted in the fill concrete between these buildings.

The minimum gaps between building superstructures is two times the absolute sum of the maximum displacement of each building under the most unfavorable load combination, or a minimum of 4 in.

3.8.4.1.1 R/B

The R/B has five main floors. ~~The~~ In plan, the R/B ~~building~~ surrounds ~~contains~~ the PCCV and containment internal structure ~~at its center~~, and is founded with those structures on a common basemat. The outer perimeter of the R/B is ~~nearly square~~ basically rectangular, and is constructed of reinforced concrete walls, floors, and roofs. ~~The roof~~ In cross-section, the height of the R/B varies from elevation 101 ft, 0 in. to ~~124~~ 154 ft, ~~0~~ 6 in., ~~except~~ and the PCCV ~~dome which~~ extends above the R/B to elevation 232 ft, 0 in.

The R/B consists of the following ~~five~~ areas, defined by their functions.

- ~~PCCV and containment internal structure~~
- Safety system pumps and heat exchangers area
- Fuel handling area
- Main steam and feedwater area
- Safety-related electrical area

- ~~ACI 318-99, Building Code Requirements for Structural Concrete, American Concrete Institute, 1999 (Reference 3.8-32).~~
- ACI 349-01, Code Requirements for Nuclear Safety-Related Concrete Structures, American Concrete Institute, 2001 (Reference 3.8-8).
- ANSI/AISC N690-1994, Specification for the Design, Fabrication and Erection of Steel Safety-Related Structures for Nuclear Facilities, including Supplement 2 (2004), American National Standards Institute/American Institute of Steel Construction, 1994 & 2004 (Reference 3.8-9).
- ANSI/ANS-57.7 Design Criteria for an Independent Spent Fuel Storage Installation (Water Pool Type), American National Standards Institute/American Nuclear Society, 1997 (Reference 3.8-33).
- ASCE 4-98, Seismic Analysis of Safety-Related Nuclear Structures and Commentary on Seismic Analysis of Safety-Related Nuclear Structures, American Society of Civil Engineers, 1998 (Reference 3.8-34).
- ASCE 7-05, Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, 2005 (Reference 3.8-35).
- ASCE 37-02, Design Loads on Structures During Construction, American Society of Civil Engineers, 2002 (Reference 3.8-36).
- ASME BPVC-III, Rules for Construction of Nuclear Facility Components - Section III Division 1 - Subsection NF - Supports, American Society of Mechanical Engineers, 2001 Edition through the 2003 Addenda (Reference 3.8-2).
- ASME NQA-2-1983, Quality Assurance Requirements for Nuclear Power Plants, with ASME NQA-2a-1985, Addenda to ASME NQA-2-1983, American Society of Mechanical Engineers (Reference 3.8-37).
- Specification for the Design of Cold-Formed Steel Members. 1996 Edition and Supplement No 1, American Iron and Steel Institute, July 30, 1999 (Reference 3.8-38).
- ACI-304R, Guide for Measuring, Mixing, Transporting, and Placing Concrete, American Concrete Institute, 2000 (Reference 3.8-39).
- ACI-224R, Control of Cracking in Concrete Structures, American Concrete Institute, 2001 (Reference 3.8-54).
- RG 1.69, Concrete Radiation Shields for Nuclear Power Plants, U.S. Nuclear Regulatory Commission, December 1973 (Reference 3.8-20).
- RG 1.91, Evaluations of Explosions Postulated to Occur on Transportation Routes Near Nuclear Power Plants, U.S. Nuclear Regulatory Commission, February 1978 (Reference 3.8-49).
- RG 1.115, Protection Against Low-Trajectory Turbine Missiles, U.S. Nuclear Regulatory Commission, July 1977 (Reference 3.8-50).
- RG 1.127, Inspection of Water-Control Structures Associated with Nuclear Power Plants, U.S. Nuclear Regulatory Commission, March 1978 (Reference 3.8-47).

- RG 1.142, Safety-Related Concrete Structures for Nuclear Power Plants (Other than Reactor Vessels and Containments), U.S. Nuclear Regulatory Commission, November 2001 (Reference 3.8-19).

- RG 1.143, Design Guidance for Radioactive Waste Management Systems, Structures, and Components Installed in Light-Water-Cooled Nuclear Power Plants, U.S. Nuclear Regulatory Commission, November 2001 (Reference 3.8-51).

- RG 1.160, Monitoring the Effectiveness of Maintenance at Nuclear Power Plants, U.S. Nuclear Regulatory Commission, March 1997 (Reference 3.8-30).

- RG 1.199, Anchoring Components and Structural Supports in Concrete, U.S. Nuclear Regulatory Commission, November 2003 (Reference 3.8-41).

Appendix 3A, Section 3A.2, lists the applicable codes, standards and specifications for HVAC ducts and duct supports. Appendix 3F, Section 3F.2, lists the applicable codes, standards and specifications for conduit and conduit supports. Appendix 3G, Section 3G.2, lists the applicable codes, standards and specifications for cable trays and cable tray supports.

3.8.4.3 Loads and Load Combinations

Loads considered in the design are listed below. Not all loads listed are necessarily applicable to all structures and their elements. The loads for which each structure is designed are dependent on the applicable conditions.

The COL Applicant is to identify any applicable externally generated loads. Such site-specific loads include those induced by floods, potential non-terrorism related aircraft crashes, explosive hazards in proximity to the site, and projectiles and missiles generated from activities of nearby military installations. Loads that are due to malevolent vehicle assault, aircraft impact, and accidental explosion are taken as W_t in load combination 5 in accordance with RG 1.142 (Reference 3.8-19), Regulatory Position 7. Externally generated loads are not normally postulated to occur simultaneously with abnormal plant loads; however, the applicable loads and the related load combinations are determined on a case-by-case basis.

3.8.4.3.1 Dead Loads (D)

Dead loads are taken as the weight of all permanent construction/installations including fixed equipment and tanks. Uniform and/or concentrated dead loads are generally utilized for design of individual members. Equivalent dead loads are used during global analyses as conservative uniform load allowances of minor equipment and distribution systems, including small bore piping.

3.8.4.3.1.1 Dead Loads (Uniform and/or Concentrated)

Dead loads include the weight of structures such as slabs, roofs, decking, framing (beams, columns, bracing, and walls), and the weight of permanently attached major equipment, tanks, machinery, cranes, elevators, etc. The deadweight of equipment is based on its bounding operating condition including the weight of fluids. In addition,

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All other floors (ground floor and elevated floors)	200 lb/ft^2

In design reconciliation analysis if actual loads are established to be lower than the above loads, the actual loads may be used for reconciliation. Floor live loads for design are not reduced below 100 lb/ft^2, except for offices which are maintained as 50 lb/ft^2 minimum.

3.8.4.3.4.2 Roof Snow Loads and Roof Live Loads

The roof is designed for uniform snow live load as specified in Chapter 2. Normal winter precipitation roof loads are added to all other live loads that may be expected to be present at the time to determine the design live load on the roof, and include appropriate load factors in applicable loading combinations. The extreme winter precipitation roof load is included as live load in extreme loading combinations using the applicable load factor. Other extreme environmental loads (e.g., seismic and tornado loads) are not considered as occurring simultaneously. Slope roof snow loads, partially loaded, unbalanced roof snow loads, and drifts (including sliding snow) on lower roofs, as applicable, are determined in accordance with ASCE 7-05 (Reference 3.8-35).

The roof design accommodates a roof live load of 40 psf to account for loads produced by workers, equipment, and materials. Roof live load is not added to roof snow load when evaluating the design load combinations.

~~The roof is designed for uniform snow live load as specified in Chapter 2. The snow load is not additive with other roof live loads, except as noted in Subsection 3.8.4.3 below. Roof snow loads are calculated in accordance with ASCE 7-05 (Reference 3.8-35), accounting for snow drift where appropriate. The importance factor is taken as 1.2 for category I and II SSCs (essential facilities). Roof snow load is considered as live load for seismic analysis, as defined in Subsection 3.8.4.3.~~

3.8.4.3.4.3 Roof Rain Loads

Roof rain load is accounted for in accordance with Chapter 8 of ASCE 7-05 (Reference 3.8-35), and applied as applicable in load combinations. Roof rain load is included in live load in applicable load combinations, including additive effects with roof snow load as identified in Section 7.10 of ASCE 7-05. Subsection 3.4.1.2 provides additional discussion of design features to limit ponding of rain on the roofs of plant buildings.

3.8.4.3.4.4 Concentrated Loads for the Design of Local Members

Concentrated load on beams and girders (in load combinations that do not include seismic load)	5,000 lbs to be applied as to maximize moment or shear. This load is not carried to columns. It is not applied in office or access control areas (1)
Concentrated load on slabs (to be considered with dead load only)	5,000 lbs to be so applied as to maximize moment or shear. This load is not cumulative and is not carried to columns. It is not applied in office or access control areas (1)

(1) Area where no heavy equipment is located or transported.

In the design reconciliation analysis, if actual loads are established to be lower than the above loads, the actual loads may be used for reconciliation.

3.8.4.3.4.5 Temporary Exterior Wall Surcharge

When applicable, the most critical of either a minimum subsurface wall surcharge of 250 lb/ft^2 (wheel load converted to equivalent uniform vertical load) or a railroad surcharge is applied.

3.8.4.3.4.6 Construction Loads

In the load combination for the construction case, the live load is defined as the additional construction loads produced by cranes, trucks, or any type of vehicle with its pick-up load, as required by construction. ASCE 37-02 (Reference 3.8-36) provides additional guidance. For steel beams supporting concrete floors, the weight of the concrete plus 100 lb/ft^2 uniform load or 5,000 pounds concentrated load, distributed near points of maximum shear and moment, are applied. A one third increase in allowable stress is permitted in this case.

Metal decking and precast concrete panels used as formwork for concrete floors are designed for the wet weight of the concrete plus a construction live load of 20 lb/ft^2 uniform or 150 pound concentrated. The deflection for these items used as a form is limited to the lesser of 0.75 in. or the span length (in inches) divided by 180. For relatively high construction loads, temporary supports may be used to prop floor beams without increasing their size.

3.8.4.3.4.7 Crane Loads

Crane and equipment supplier's information are used to determine wheel loads, equipment loads, weights of moving parts, and reactions of clamps (if any). Construction loads are considered where applicable.

Impact allowance for traveling crane supports and runway horizontal forces are in accordance with AISC N690 (Reference 3.8-9) for seismic category I and II structures, unless the crane manufacturer's design specifies higher impact loads. The vertical live load is increased by 25% to account for vertical impact of cab-operated traveling cranes and 10% of pendant-operated traveling cranes. A lateral force, equal to 20% of the lifted load and crane trolley are applied at the top and perpendicular to the crane rails. A longitudinal force equal to 10% of the maximum wheel load is applied at the top of the rails. <u>Crane runways are also designed for crane stop forces.</u>

Crane lift loads are not combined with wind loads. During construction; however, wind effects on the crane are considered. For load combinations, including SSE, all cranes in seismic category I areas are considered with a "most probable lift load" or heaviest load to be lifted over seismic category I SSCs/fuel, whichever is greater. Impact and seismic forces are not applied simultaneously.

3.8.4.3.4.8 Elevator Loads

Impact allowance for supports of elevators is 100%, applied to design capacity and weight of car plus appurtenances, or as specified by the equipment supplier.

3.8.4.3.4.9 Equipment Laydown and Major Maintenance

Floors are designed for planned refueling and maintenance activities as defined on equipment laydown drawings. Plans are developed for major equipment removal (such as SGs) and laydown. Temporary supports can be included in these plans provided such supports are easy to install and the installation of such supports is described in the plans.

3.8.4.3.5 Wind Load

3.8.4.3.5.1 Design Wind (W)

The design wind is determined as discussed in Subsection 3.3.1 for values specified in Chapter 2. Wind loads are not combined with seismic loads.

3.8.4.3.5.2 Tornado Load (Wt)

The design for tornado loads is in accordance with Subsection 3.3.2 for values specified in Chapter 2. In addition, extreme winds such as hurricanes and tornadoes have the potential to generate missiles. Missiles generated by tornadoes and extreme winds are listed in Subsection 3.5.1.4 and barrier design for missiles is discussed in Subsection 3.5.3. These subsections describe the determination of tornado loads applicable to the protection of safety-related equipment.

3.8.4.3.6 Seismic Loads

3.8.4.3.6.1 Operating Basis (E_{ob})

For seismic category I SSCs whose design is site-specific, that is, not included in the seismic design of the US-APWR standard plant, OBE loading has to be considered only if the value of site-specific OBE is set higher than 1/3 of the site-specific SSE. Therefore, the site-specific seismic design does not have to consider OBE loads if the OBE spectra are enveloped by 1/3 of the site-specific foundation input response spectra and ground motion response spectra.

3.8.4.3.6.2 Safe Shutdown (E_{ss})

E_{ss} is defined as the loads generated by the SSE specified for the plant, including the associated hydrodynamic loads and dynamic incremental soil pressure (based on three-dimensional SSI analysis results). Earthquake loads (E_{ss}), are derived for evaluation of seismic category I structures using ground motion accelerations in accordance with Section 3.7.

Seismic dynamic analyses of the buildings consider the dead load and the equivalent dead loads as the accelerated mass. In addition to the dead load, 25% of the floor live load during normal operation or 75% of the roof snow load, whichever is applicable, is also considered as accelerated mass in the seismic models.

For the local design of members loaded individually, such as the floors and beams, seismic member forces include the vertical response due to masses equal to 50% of the specified floor live loads instead of 25% of floor live load, as follows: ~~or roof snow load,~~

~~whichever is applicable. These seismic loads are combined with 100% of the specified live loads, or 75% of the roof snow load, whichever is applicable, as shown below.~~

~~$1.0\,D + (1.0\,L\ or\ 0.75\,S) + a_v(D + 0.5\,(L\ or\ S))$~~ $a_v(0.5L)$

where

a_v	=	Vertical seismic acceleration obtained from the seismic dynamic analysis results
~~D~~	~~=~~	~~Dead load, including the equivalent dead load~~
L	=	Floor live load per Subsection 3.8.4.3.4
~~S~~	~~=~~	~~Roof snow load as per Subsection 3.8.4.3~~

In locations where live loads are expected to always be present, the percentage of live load acting as accelerated mass is increased up to 100% of the live load for the affected members.

For the seismic load combination, the containment operating deck is designed for a live load of 200 lb/ft^2 which is appropriate for plant operating conditions, and 25% of this live load is included as mass in the seismic analyses. The mass of equipment and distributed system are included in both the dead and seismic loads.

3.8.4.3.7 Normal Operating Loads

3.8.4.3.7.1 Operating Thermal Loads (T_o)

The normal operating environment inside and outside the R/B is specified in Table 3.8.4-1. Temperature Gradients of the PS/Bs are provided in Table 3.8.4-2 and Figure 3.8.4-1. Normal thermal loads for the exterior walls and roofs are caused by positive and negative temperature variations through the concrete wall. The temperature in the concrete is based on one-dimensional steady state heat transfer analysis, which considers the surface heat transfer between the environment and the concrete. All exterior walls of the R/B are designed for these thermal loads, even if the exterior surface is protected by an adjacent building. The thermal gradient is also applied to the portion of the R/B between the PCCV upper annulus and the auxiliary building (A/B).

The COL Applicant is to specify normal operating thermal loads for site-specific structures, as applicable.

3.8.4.3.7.2 Operating Pipe Reactions (R_o)

Pipe and equipment reactions during normal operation or shutdown conditions are based on the most critical transient or steady state condition.

3.8.4.3.8 Effects of Pipe Rupture (Y) and other Accidents (P_a, T_a, R_a)

3.8.4.3.8.1 Accident Pressure Load (P_a)

Accident pressure loads are considered within or across a compartment and/or building due to a differential pressure generated by postulated pipe rupture. Dynamic effects due to pressure time-history are also included in the design.

Seismic category I structures are modeled globally using applicable loads, including equivalent dead and live loads, in load combinations that include design-basis earthquake accelerations as described in Section 3.7. Computer modeling utilizes three-dimensional FE models to globally analyze the beams, columns, slabs, and shear walls. Individual structural members are further analyzed for localized loading as described in specific load cases.

Concrete components such as walls, slabs, and foundations are evaluated for the effects of frame interaction when the flexural moment from seismic loads is a large percentage of the flexural capacity. When at least two-thirds of the flexural capacity of a component is from seismic loads alone, the component is designed as a frame to assure design capacity even under a seismic margin earthquake equal to 150% of the SSE, in accordance with RG 1.142 (Reference 3.8-19), Regulatory Position 3.

Concrete ~~M~~members that are subject to torsion and combined shear and torsion are evaluated to the standards of Section 11.6 of ACI 349 (Reference 3.8-8). ~~318-99 (Reference 3.8-32) instead of the requirements of Section 11.6 of ACI 349 (Reference 3.8-8), as recommended by RG 1.142 (Reference 3.8-19).~~

Design and analysis of the spent fuel pit, the spent fuel racks, and the fuel handling system is in accordance with Appendix D of NUREG-0800, SRP 3.8.4 (Reference 3.8-40). Additional general information is provided by ANSI/ANS-57.7 (Reference 3.8-33). Subsection 9.1.2 describes the design bases and layout of the spent fuel pit, the spent fuel racks, and the fuel handling system.

Exterior concrete walls below grade and basemats of seismic category I structures 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. Dynamic ~~L~~lateral earth pressure is calculated in accordance with ASCE 4-98 (Reference 3.8-34)~~ for both active and passive earth pressures~~.

Structural steel framing in seismic category I structures is primarily for the support of distribution systems, access platforms, and other plant appurtenances. Steel members are sized and detailed based on maximum stresses and reactions determined through conservative manual calculations and computer models based on pinned-end connections, including slotted hole clip angle connections, to relieve thermal expansion forces where appropriate, unless detailed to develop end moments in accordance with AISC N690 (Reference 3.8-9). The design of the support anchorage to the concrete structure is in accordance with ACI 349 Appendix B (Reference 3.8-8), RG 1.142 (Reference 3.8-19), and RG 1.199 (Reference 3.8-41).

The design and analysis procedures for seismic category I distribution systems, such as HVAC ducts, conduits, and cable trays including their respective seismic category I supports, are in accordance with AISC N690 (Reference 3.8-8) and AISI Specification for Design of Cold-Formed Steel Members (Reference 3.8-34). The following appendices provide additional discussion of the design and analysis of these subsystems.

- Appendix 3A Heating, Ventilation, and Air Conditioning Ducts and Duct Supports
- Appendix 3F Design of Conduits and Conduit Supports

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- Appendix 3G Seismic Qualification of Cable Trays and Supports

The COL Applicant is to provide design and analysis procedures for the ESWPT, UHSRS, and PSFSVs.

3.8.4.4.4 Seismic Category II Structures

Seismic category II structures need not remain functional during and after an SSE. However, such structures must not fall or displace to the point they could damage seismic category I SSCs.

Seismic Category II structures and subsystems are analyzed and designed using the same methods and stress limits specified for seismic Category I structures and subsystems, except structural steel in-plane stress limits are permitted to reach 1.0 F_y.

3.8.4.5 STRUCTURAL ACCEPTANCE CRITERIA

Structural acceptance criteria are listed in Table 3.8.4-3 for concrete structures and in Table 3.8.4-4 for steel structures, and are in accordance with ACI-349 (Reference 3.8-8) and AISC N690 (Reference 3.8-9), except as provided in the table notes.

The deflection of the structural members is limited to the maximum values as specified in ACI-349 (Reference 3.8-8) and AISC N690 (Reference 3.8-9), as applicable.

Subsection 3.8.5.5 identifies acceptance criteria applicable to additional basemat load combinations.

3.8.4.6 Materials, Quality Control, and Special Construction Techniques

The following information pertains to the materials, quality control programs, and any special construction techniques utilized in the construction of the seismic category I structures for the US-APWR.

3.8.4.6.1 Materials

The major materials of construction in seismic category I structures are concrete, grout, steel reinforcement bars, splices of steel reinforcing bars, structural steel shapes, and anchors.

3.8.4.6.1.1 Concrete

Concrete utilized in standard plant seismic category I structures, other than PCCV and upper part of the tendon gallery in the basemat, has a compressive strength of f'_c = 4,000 psi. Concrete utilized in the PCCV and upper part of the tendon gallery in the basemat has a compressive strength of f'_c = 7,000 psi and is subject to the PCCV material requirements in Subsection 3.8.1.6, including the requirements of ASME III, Division 2 (Reference 3.8-2), as shown in Figure 3.8.5-4. The COL Applicant is to specify concrete strength utilized in non-standard plant seismic category I structures. A test age of 28 days is used for normal concrete. Batching and placement of concrete is performed in accordance with ACI 349 (Reference 3.8-8), ACI 304R (Reference 3.8-38), and ASTM C 94 (Reference 3.8-42). During construction, volume changes in mass concrete are controlled where necessary by applying measures and provisions outlined in ACI 207.2R (Reference 3.8-52) and ACI 207.4R (Reference 3.8-53).

Portland cement is used in the concrete conforms to ASTM C 150, Type II (Reference 3.8-43) standards. The confirmation of the chemical composition of the cement properties is validated by certified copies of test reports showing the chemical composition of each Portland cement shipment.

Aggregates used in the concrete conform to ASTM C 33 (Reference 3.8-44). Aggregate and source acceptance is based on documented test results for each source and random sampling of shipments based on MIL-STD-1916 (Reference 3.8-45).

Water and ice used in the concrete conform to the requirements of ACI-349 (Reference 3.8-8).

Admixtures include an air entraining admixture, pozzolans, and a water reducing admixture. The admixtures, except the pozzolans, are stored in a liquid state.

Admixtures and concrete mix conform to the following requirements:

Pozzolans	ASTM C 618
Sampling and Testing of Pozzolans	ASTM C 311
Air Entraining Admixtures	ASTM C 260
Water Reducing Admixtures	ASTM C 494
Concrete Mix	ACI 211.1 and ASTM C 94 (Reference 3.8-45)
Concrete Mix Testing	ASTM C 172, ASTM C 192, and ASTM C 39
Minimum Number of Strength Tests(1)	ACI 349 (Reference 3.8-7) and ASME NQA-2 (Reference 3.8-37)

Note 1: In lieu of frequency of compressive strength testing specified by Section 5.6.1.1 of ACI 349-97 (Reference 3.8-8) or that specified by ASME NQA-2 (Reference 3.8-37), the following is acceptable per RG 1.142, Regulatory Position 5 (Reference 3.8-19).

Samples for strength tests of concrete should be taken at least once per day for each class of concrete placed or at least once for each 100 cubic yards of concrete placed. When the standard deviation for 30 consecutive tests of a given class is less than 600 psi, the amount of concrete placed between tests may be increased by 50 cubic yards for each 100 psi the standard deviation is below 600 psi, except that the minimum testing rate should not be less than one test for each shift when the concrete is placed on more than one shift per day or not less than one test for each 200 cubic yards of concrete placed. The test frequency should revert to once for each 100 cubic yards placed if the data for any 30 consecutive tests indicate a higher standard deviation than the value controlling the decreased test frequency.

3.8.4.6.1.2 Grout

Grout is used to transfer load from machinery, equipment, and column bases to their foundations, and to anchor the reinforcing bars, dowels, and anchor rods into hardened concrete. Grout generally consists of Portland cement, sand, water, and admixtures. Epoxy grout is only used in areas where radiation levels and temperature levels are compatible with epoxy use.

Portland cement used in the concrete conforms to ASTM C 150, Type II (Reference 3.8-43). Sand must be clean with gradation and fineness in accordance with ASTM C33 (Reference 3.8-44). Water and ice used in the grout conforms to the requirements of ACI 349 (Reference 3.8-8). Water-reducing and/or retarding admixtures conform to ASTM C494.

3.8.4.6.1.3 Steel for Concrete Reinforcement

Steel bars for concrete reinforcement are deformed bars conforming to ASTM A 615, Grade 60, or ASTM A 706, Grade 60 (minimum yield strength of 60,000 psi). For each heat (batch) of reinforcing steel bars, certified mill test reports are provided. Additionally, for each 50 tons/bar size/heat, a minimum of one tensile test is performed. Where mechanical anchorage can not be achieved through the use of deformed bars, headed steel bars conforming to ASTM A 970 are used.

Coated reinforcing steel is not used. Placement of concrete reinforcement is in accordance with ACI-349 Reference 3.8-8), Sections 7.5 and 7.6 ~~7.7 (Reference 3.8-8)~~.

3.8.4.6.1.4 Splices

Reinforcement splices comply with ACI-349, Chapter 12 (Reference 3.8-8). All bars are sheared or cut to the correct length shown on the bar bending schedules from continuous rolled bar stock. In general, all splices are made with a wire-tied lap of length in accordance with ACI 408R. Mechanical splices used are in conformance with ACI 493.3R. Mechanical splices develop 125% of the specified yield strength of the spliced bar. Welding of reinforcing steel, other than in the PCCV, is performed in accordance with American Welding Society (AWS) D1.4 (Reference 3.8-46).

3.8.4.6.1.5 Structural Steel Shapes

Structural steel shapes used in other seismic category I structures conform to the following standards:

Standard	Description
ASTM A 1	Carbon Steel Rails
ASTM A 3	Standard Specification for Steel Joint Bars, Low, Medium, and High Carbon (Non-Heat Treated)

3.8.4.6.2 Quality Control

Chapter 17 details the quality assurance program for the US-APWR.

3.8.4.6.3 Special Construction Techniques

There are no special construction techniques utilized in the construction of other seismic category I structures.

3.8.4.7 Testing and Inservice Inspection Requirements

The COL Applicant is to address monitoring 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.

For water control structures, ISI programs are acceptable if in accordance with RG 1.127 (Reference 3.8-47). Water control structures covered by this program include concrete structures, embankment structures, spillway structures, outlet works, reservoirs, cooling water channels, canals and intake and discharge structures, and safety and performance instrumentation.

For seismic category I structures, it is important to accommodate ISI of critical areas. Monitoring and maintaining the condition of other seismic category I structures are essential for plant safety. Any special design provisions (e.g., providing sufficient physical access, providing alternative means for identification of conditions in inaccessible areas that can lead to degradation, remote visual monitoring of high-radiation areas) to accommodate ISI of other seismic category I structures are to be provided on a case-by-case basis.

For plants with nonaggressive ground water/soil (i.e., pH greater than 5.5, chlorides less than 500 ppm, and sulfates less than 1,500 ppm), an acceptable program for normally inaccessible, below-grade concrete walls and foundations is to (1) examine the exposed portions of the below-grade concrete, when excavated for any reason, for signs of degradation; and (2) conduct periodic site monitoring of ground water chemistry, to confirm that the ground water remains nonaggressive.

For plants with aggressive ground water/soil (i.e., it exceeds any of the limits noted above), an acceptable approach is to implement a periodic surveillance program to monitor the condition of normally inaccessible, below-grade concrete for signs of degradation.

3.8.4.7.1.1 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).

COL 3.8(22) *The COL Applicant is to address monitoring 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 lean concrete under any basemat above the frost line so that the bottom of lean 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.*

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.

3.8-28 Industry Guideline for Monitoring the Effectiveness of Maintenance at Nuclear Power Plants. NUMARC 93-01, Rev.2, Nuclear Energy Institute, April 1996.

3.8-29 Requirements for Monitoring the Effectiveness of Maintenance at Nuclear Power Plants, Domestic Licensing of Production and Utilization Facilities, Energy. Title 10 Code of Federal Regulations Part 50.65, U.S. Nuclear Regulatory Commission, Washington, DC, January 1, 2007.

3.8-30 Monitoring the Effectiveness of Maintenance at Nuclear Power Plants, RG 1.160, Rev.2, U.S. Nuclear Regulatory Commission, Washington, DC, March 1997.

3.8-31 Visual Weld Acceptance Criteria: Volume 1, Visual Weld Acceptance Criteria for Structural Welding at Nuclear Power Plants, NCIG-01, Rev.3, Nuclear Construction Issues Group/Electric Power Research Institute, July 28, 1999.

3.8-32 Deleted. ~~Building Code Requirements for Structural Concrete, ACI 318-99, American Concrete Institute, 1999.~~

3.8-33 Design Criteria for an Independent Spent Fuel Storage Installation (Water Pool Type), ANSI/ANS-57.7, American National Standards Institute/American Nuclear Society, 1997.

3.8-34 Seismic Analysis of Safety Related Nuclear Structures and Commentary on Seismic Analysis of Safety Related Nuclear Structures, ASCE Standard 4-98, American Society of Civil Engineers, 1998.

3.8-35 Minimum Design Loads for Buildings and Other Structures. ASCE 7-05, American Society of Civil Engineers, 2005.

3.8-36 Design Loads on Structures During Construction. ASCE 37-02, American Society of Civil Engineers, 2002.

3.8-37 Quality Assurance Requirements for Nuclear Power Plants. ASME NQA-2-1983, with ASME NQA-2a-1985 addenda to ASME NQA-2-1983, American Society of Mechanical Engineers, 1983.

3.8-38 Specification for the Design of Cold-Formed Steel Members. 1996 Edition and Supplement No 1, American Iron and Steel Institute, July 30, 1999.

3.8-39 Guide for Measuring, Mixing, Transporting, and Placing Concrete. ACI-304R, American Concrete Institute, 2000.

3.8-40 Other Seismic Category I Structures, Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants. NUREG-0800 SRP Section 3.8.4, Rev.2, U.S. Nuclear Regulatory Commission, Washington, DC, March, 2007.

3.8-41 Anchoring Components and Structural Supports in Concrete, RG 1.199, Rev. 0, U.S. Nuclear Regulatory Commission, November 2003.

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3.8-42 Standard Specification for Ready-Mixed Concrete, C94-07, American Standards Testing and Materials (ASTM), 2007.

3.8-43 Standard Specification for Portland Cement, C150-07, Type II, American Society of Testing and Materials.

3.8-44 Standard Specification for Concrete Aggregates, C33-03, ASTM, 2003.

3.8-45 DOD Preferred Methods for Acceptance of Product, MIL-STD-1916, Department of Defense Test Method Standard, April 1, 1996.

3.8-46 Structural Welding Code – Reinforcing Steel, D1.4, American Welding Society, 2005.

3.8-47 Inspection of Water-Control Structures Associated with Nuclear Power Plants, RG 1.127, Rev. 1, U.S. Nuclear Regulatory Commission, March 1978.

3.8-49 Evaluations of Explosions Postulated to Occur on Transportation Routes Near Nuclear Power Plants, RG 1.91, Rev. 1, U.S. Nuclear Regulatory Commission, Washington, DC, February 1978.

3.8-50 Protection Against Low-Trajectory Turbine Missiles, RG 1.115, Rev. 1, U.S. Nuclear Regulatory Commission, Washington, DC, July 1977.

3.8-51 Design Guidance for Radioactive Waste Management Systems, Structures, and Components Installed in Light-Water-Cooled Nuclear Power Plants, RG 1.143, Rev. 2, U.S. Nuclear Regulatory Commission, Washington, DC, November 2001.

3.8-52 Report on Thermal and Volume Change Effects on Cracking of Mass Concrete. ACI-207.2R, American Concrete Institute, 2007.

3.8-53 Cooling and Insulating Systems for Mass Concrete. ACI-207.4R, American Concrete Institute, 2005.

3.8-54 Control of Cracking in Concrete Structures. ACI-224R, American Concrete Institute, 2001.

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Table 3.8.4-3 Load Combinations and Load Factors for Seismic Category I Concrete Structures

LOAD COMBINATIONS AND FACTORS[1],[2]												
ACI 349 Load Combination:		1	2	3	4	5[7]	6[6]	7[6], [7]	8[6], [7]	9	10	11
Load Type												
Dead	D	1.4	1.4	1.4	1.0	1.0	1.0	1.0	1.0	1.05	1.05	1.05
Liquid	F	1.4	1.4	1.4	1.0	1.0	1.0	1.0	1.0	1.05	1.05	1.05
Live	L	1.7	1.7	1.7	1.0	1.0	1.0	1.0	1.0	1.3	1.3	1.3
Earth	H	1.7	1.7	1.7	1.0	1.0	1.0	1.0	1.0	1.3	1.3	1.3
Design pressure	P_d											
Normal pipe reactions	R_o	1.7	1.7	1.7	1.0	1.0				1.3	1.3	1.3
Normal thermal	T_o				1.0	1.0				1.2[5]	1.2[5]	1.2[5]
Wind	W			1.7								1.3
OBE	E_{ob}		1.7[3]					1.15[3]			1.3[3]	
SSE	E_{ss}				1.0[4]				1.0[4]			
Tornado	W_t					1.0						
Accident pressure	P_a						1.4[5]	1.15	1.0			
Accident thermal	T_a						1.0	1.0	1.0			
Accident thermal pipe reactions	R_a						1.0	1.0	1.0			
Pipe rupture reactions	Y_r							1.0	1.0			
Jet impingement	Y_j							1.0	1.0			
Pipe Impact	Y_m							1.0	1.0			
Acceptance Criteria[8]		U	U	U	U	U	U	U	U	U	U	U

Notes:

1. Design per ACI-349 Strength Design Method for all load combinations
2. Where any load reduces the effects of other loads, the corresponding coefficient for that load is taken as 0.9 if it can be demonstrated that the load is always present or occurs simultaneously with the other loads. Otherwise the coefficient is taken as zero.
3. OBE loading is applicable for site-specific seismic category I SSCs, only if the value of site-specific OBE is set higher than 1/3 of the site-specific SSE.
4. SSE includes all seismic related hydrodynamic loads and percentage of live loads
5. Load factor adjusted in accordance with RG 1.142, Regulatory Position 6.
6. The maximum values of P_a, T_a, R_a, Y_j, Y_r, and Y_m including an appropriate dynamic load factor are used, unless an appropriate time history analysis is performed to justify otherwise.
7. Satisfy the load combination first without W_t, Y_r, Y_j, and Y_m. When considering concentrated loads, exceedences of local strengths and stresses may be considered in analyses for impactive or impulsive effects in accordance with ACI 349-97, Appendix C, except as noted in RG 1.142 Regulatory Positions 10 and 11.
8. The required strength U shall be equal to or greater than the strength required to resist the factored loads and/or related internal moments and forces, for each of the load combinations shown in this table.

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Table 3.8.4-4 Load Combinations and Load Factors for Seismic Category I Steel Structures (Sheet 1 of 2)

ALLOWABLE STRESS DESIGN (ASD) LOAD COMBINATIONS AND APPLICABLE STRESS LIMIT COEFFICIENTS													
AISC N690 Load Combination: (6)		**1**	**2**	**3**(9)	**4**(9)	**5**(9)	**6**(9)	**7**	**8**	**9**(4)	**9a**(4)(10)	**10**(4)(5)	**11**(4)(5)
Load Type													
Dead	***D***	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Live	***L***	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Normal pipe reactions	R_o		1.0			1.0	1.0	1.0	1.0				
Normal thermal	T_o		1.0			1.0	1.0	1.0	1.0				
Wind	***W***			1.0		1.0							
OBE	E_{ob}				1.0		1.0						
SSE	E_{ss}								1.0				1.0
Tornado	W_t							1.0					
Accident pressure	P_a									1.0		1.0	1.0
Accident thermal	T_a									1.0	1.0	1.0	1.0
Accident thermal pipe reactions	R_a									1.0	1.0	1.0	1.0
Pipe rupture reactions	Y_r											1.0	1.0
Jet impingement	Y_j											1.0	1.0
Pipe Impact	Y_m											1.0	1.0
Stress Limit Coefficient (1)(2)(8)(12)		1.0(3)	1.0(3)	1.0(3)	1.0(3)	1.0(3)	1.0(3)	1.6(7)(11)	1.6(7)(11)	1.6(7)(11)	1.6(7)(11)	1.6(7)(11)	1.7(7)(11)

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Table 3.8.4-4 Load Combinations and Load Factors for Seismic Category I Steel Structures (Sheet 2 of 2)

Notes:

1. Coefficients are applicable to primary stress limits given in ANSI/AISC N690-1994 Sections Q1.5.1, Q1.5.2, Q1.5.3, Q1.5.4, Q1.5.5, Q1.6, Q1.10, and Q1.11. Calculated stresses shall not exceed allowable stresses for each of the load combinations shown in this table.
2. In no instance shall the allowable stress exceed $0.7F_u$ in axial tension nor $0.7F_u$ times the ratio Z/S for tension plus bending.
3. For primary plus secondary stress, the allowable limits are increased by a factor of 1.5.
4. The maximum values of P_a, T_a, R_a, Y_j, Y_r, and Y_m, including an appropriate dynamic load factor, is used in load combinations 9 through 11, unless an appropriate time history analysis is performed to justify otherwise.
5. In combining loads from a postulated high-energy pipe break accident and a seismic event, the SRSS may be used, provided that the responses are calculated on a linear basis.
6. All load combinations is checked for a no-live-load condition
7. In load combinations 7 through 11, the stress limit coefficient in shear shall not exceed 1.4 in members and bolts.
8. Secondary stresses which are used to limit primary stresses are treated as primary stresses.
9. Consideration is also given to snow and other loads as defined in ASCE 7.
10. This load combination is to be used when the global (non-transient) sustained effects of T_a are considered.
11. The stress limit coefficient where axial compression exceeds 20% of normal allowable, is 1.5 for load combinations 7, 8, 9, 9a, and 10, and 1.6 for load combination 11.
12. Load combinations and stress limit coefficients are applicable for AISI design of cold-formed steel structural members used in subsystem supports. Allowable strengths per AISI may be increased by the stress limit coefficients shown, subject to the limits noted in this table. The allowable strength shall equal or exceed the required strength calculated, in accordance with AISI, for each of the load combinations shown in this table.

Table 3.8.4-6 West Exterior Wall, SECTION 1, Details of Wall Reinforcement

	Provided Reinforcement		
	Vertical	Horizontal	Shear
WALL ZONE 1 (Concrete Thickness 40 in.) El 3'-7" → El 25'-3"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Outside	#11@12"+#11@12" (0.65)	#11@6"+#11@12" (0.975)	–
Inside	#11@12" +#10@12" (0.59)	#11@12" +#10@12" (0.59)	
WALL ZONE 2 (Concrete Thickness 40 in.) El 25'-3" → El 50'-2"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Outside	#11@12"+#11@12" (0.65)	#11@6"+#11@12" (0.813)	–
Inside	#11@12" (0.325)	#11@12" (0.325)	
WALL ZONE 3 (Concrete Thickness 32 in.) El 50'-2" → El 76'-5"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Outside	#11@12" (0.406)	#11@6" (0.929)	–
Inside	#11@12" (0.406)	#11@12" (0.406)	
WALL ZONE 4 (Concrete Thickness 28 in.) El 76'-5" → El 101'-0"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$1.05D+1.3L+1.05F+1.2T_o$	NONE
Outside	#11@12" (0.464)	#11@6" (0.929)	–
Inside	#11@12" (0.464)	#11@12" (0.464)	

Note: Load Combination reflects the controlling load combination for the outside face required reinforcement. ( ) indicates reinforcement ratio.

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Table 3.8.4-7 South Interior Wall, SECTION 2, Details of Wall Reinforcement

	Provided Reinforcement		
	Vertical	**Horizontal**	**Shear**
WALL ZONE 1 (Concrete Thickness 44 in.) El 3'-7" → El 25'-3"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Each Face	#11@12" +#11@12" (0.591)	#11@6" +#11@12" (0.886)	–
WALL ZONE 2 (Concrete Thickness 40 in.) El 25'-3" → El 50'-2"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Each Face	#11@12" (0.325)	#11@12" (0.325)	–
WALL ZONE 3 (Concrete Thickness 40 in.) El 50'-2" → El 76'-5"			
Load Combination	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	NONE
Each Face	#11@12" (0.325)	#11@12" (0.325)	–
WALL ZONE 4 (Concrete Thickness 40 in.) El 76'-5" → El 86'-4"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Each Face	#11@12" (0.325)	#11@12"+#11@12" (0.65)	–
WALL ZONE 5 (Concrete Thickness 40 in.) El 86'-4" → El 101'-0"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$1.05D+1.3L+1.05F+1.2T_o$	NONE
Each Face	#11@12" (0.325)	#11@12"+#11@12" (0.65)	–

Note: ( ) indicates reinforcement ratio.

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Table 3.8.4-8 North Exterior Wall of Spent Fuel Pit, SECTION 3, Details of Wall Reinforcement

	Provided Reinforcement		
	Vertical	**Horizontal**	**Shear**
WALL ZONE 1 (Concrete Thickness 93 in.) El 30'-1"→ El 50'-2"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Outside	#14@6"+#14@6" (0.806)	#14@6"+#14@6" (0.806)	–
Inside	#14@12"+#14@12" (0.403)	#14@12"+#14@12" (0.403)	
WALL ZONE 2 (Concrete Thickness 93 in.) El 50'-2" → El 65'-0"			
Load Combination	$0.9D+1.0F+1.0E_{ss}+T_o$	$0.9D+1.0F+1.0E_{ss}+T_o$	NONE
Outside	#14@6"+#14@6" (0.806)	#14@6"+#14@6" (0.806)	–
Inside	#14@12"+#14@12" (0.403)	#14@12"+#14@12" (0.403)	
WALL ZONE 3 (Concrete Thickness 152 in.) El 65'-0" → El 76'-5"			
Load Combination	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	$1.05D+1.3L+1.05F+1.2T_o$	NONE
Outside	#14@6"+#14@12" (0.370)	#14@6"+#14@6"+#14@12" (0.617)	–
Inside	#14@12"+#14@12" (0.247)	#14@12"+#14@12" (0.247)	

Note: Load Combination reflects the controlling load combination for the outside face required reinforcement. ( ) indicates reinforcement ratio.

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Table 3.8.4-9 South Exterior Wall, SECTION 4, Details of Wall Reinforcement

	Provided Reinforcement		
	Vertical	**Horizontal**	**Shear**
WALL ZONE 1 (Concrete Thickness 44 in.) El 3'-7" → El 25'-3"			
Load Combination	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	NONE
Outside	#11@6"+#11@6" (1.182)	#11@6"+#11@12" (0.886)	–
Inside	#11@12"+#11@12" (0.591)	#11@12"+#11@12" (0.591)	
WALL ZONE 2 (Concrete Thickness 40 in.) El 25'-3" → El 50'-2"			
Load Combination	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	NONE
Outside	#11@6"+#11@12" (0.975	#11@6"+#11@12" (0.975)	–
Inside	#11@12"+#11@12" (0.65)	#11@12"+#11@12" (0.65)	
WALL ZONE 3 (Concrete Thickness 40 in.) El 50'-2" → El 76'-5"			
Load Combination	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	NONE
Outside	#11@12"+#11@12" (0.65)	#11@6"+#11@12" (0.975)	–
Inside	#11@12" (0.325)	#11@12" (0.325)	
WALL ZONE 4 (Concrete Thickness 40 in.) El 76'-5" → El 101'-0"			
Load Combination	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	$1.0D+1.0L+1.0F+1.0E_{ss}+T_o$	NONE
Outside	#11@12"+#11@12" (0.65)	#11@12"+#11@12" (0.65)	–
Inside	#11@12" (0.325)	#11@12" (0.325)	
WALL ZONE 5 (Concrete Thickness 40 in.) El 101'-0" → El 115'-6"			
Load Combination	$1.05D+1.3L+1.05F+1.2T_o$	$1.05D+1.3L+1.05F+1.2T_o$	NONE
Outside	#11@12"+#11@12" (0.65)	#11@12"+#11@12" (0.65)	–
Inside	#11@12" (0.325)	#11@12" (0.325)	

Note: Load Combination reflects the controlling load combination for the outside face required reinforcement. ( ) indicates reinforcement ratio.

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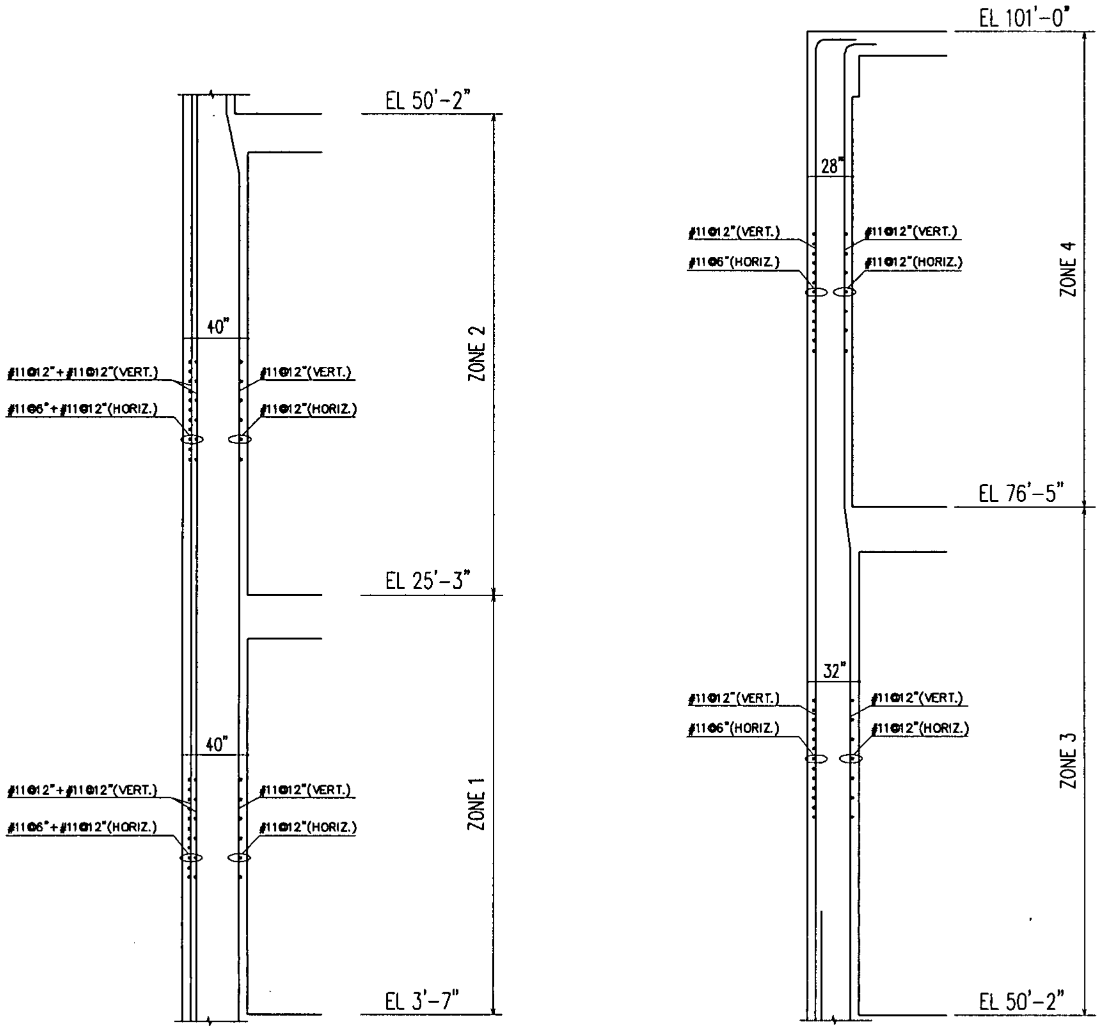

LOWER-SECTION UPPER-SECTION

Vertical Cross Section

Figure 3.8.4-4 Typical Reinforcement in West Exterior Wall – SECTION 1 (Sheet 1 of 2)

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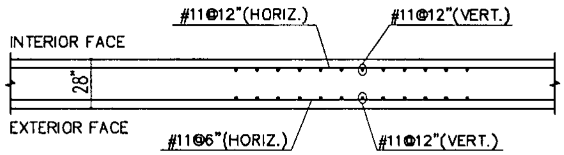
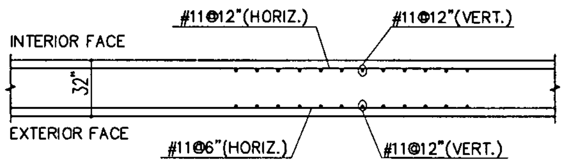
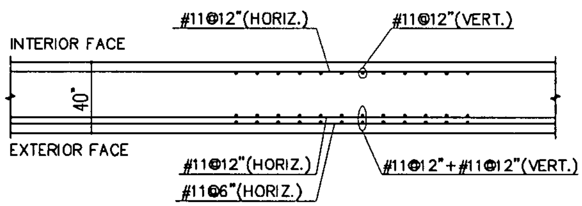


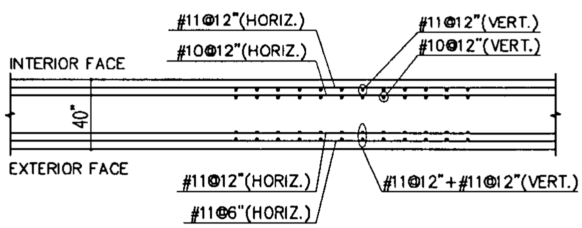
ZONE 4 [EL76'–5" to EL101'–0"]



ZONE 3 [EL50'–2" to EL76'–5"]



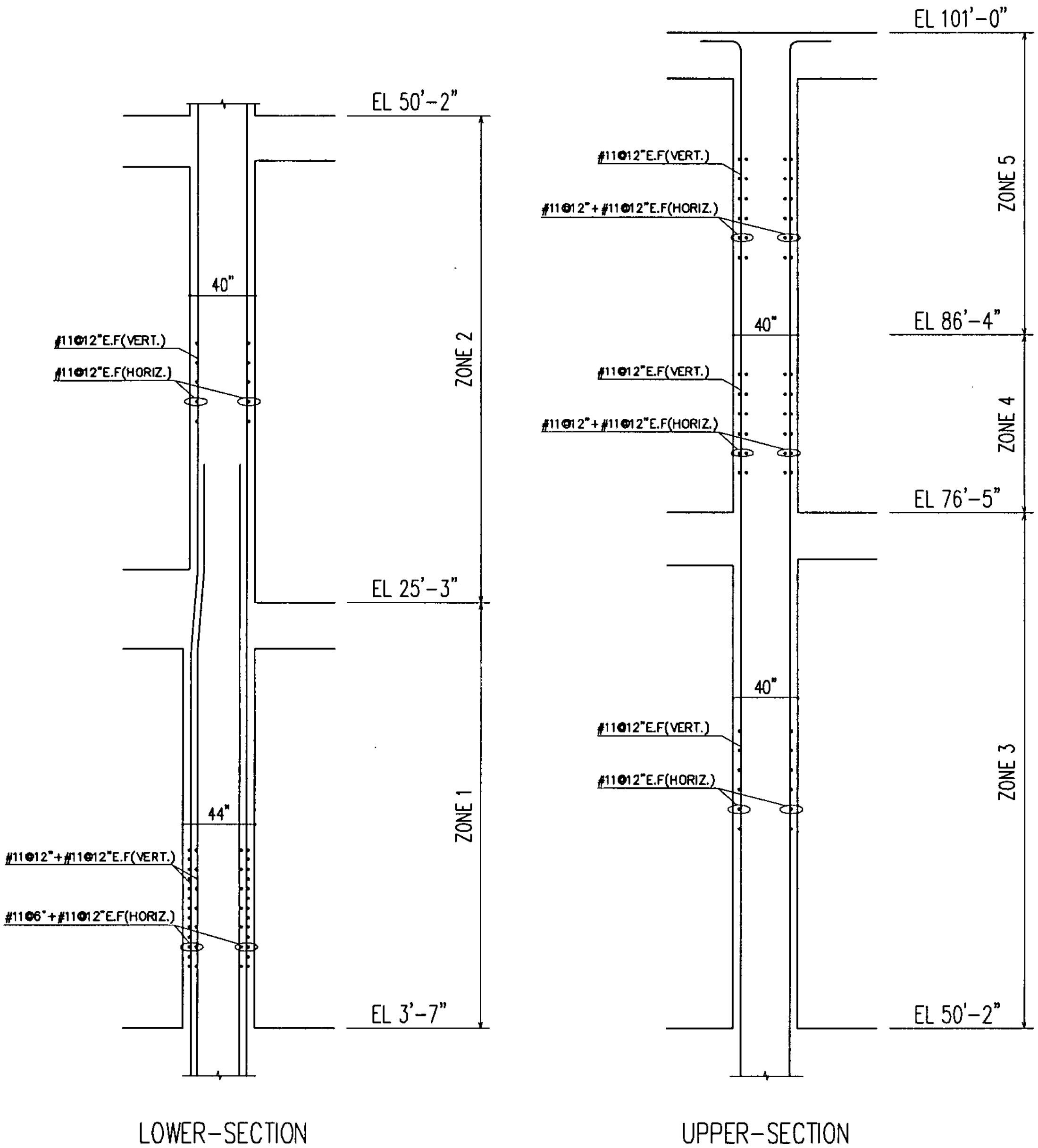
ZONE 2 [EL25'–3" to EL50'–2"]



ZONE 1 [EL3'–7" to EL25'–3"]

Horizontal Cross Section

Figure 3.8.4-4 Typical Reinforcement in West Exterior Wall – SECTION 1 (Sheet 2 of 2)



Vertical Cross Section

Figure 3.8.4-5 Typical Reinforcement in South interior Wall – SECTION 2 (Sheet 1 of 2)

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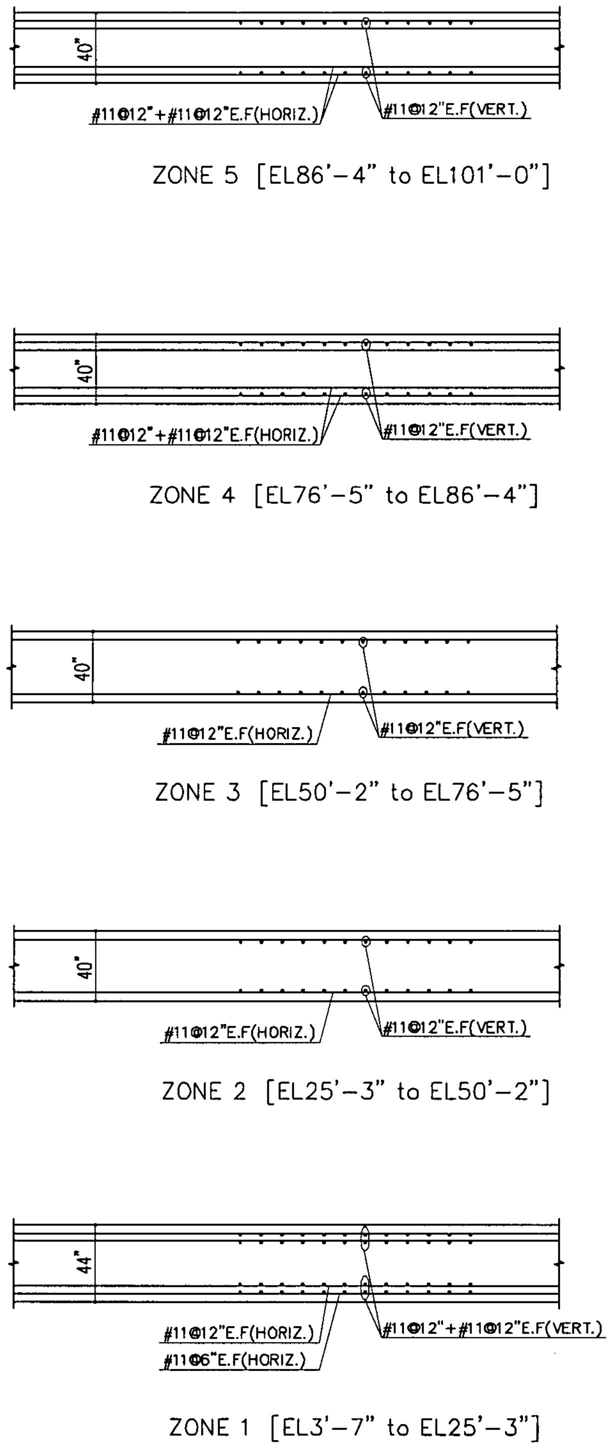
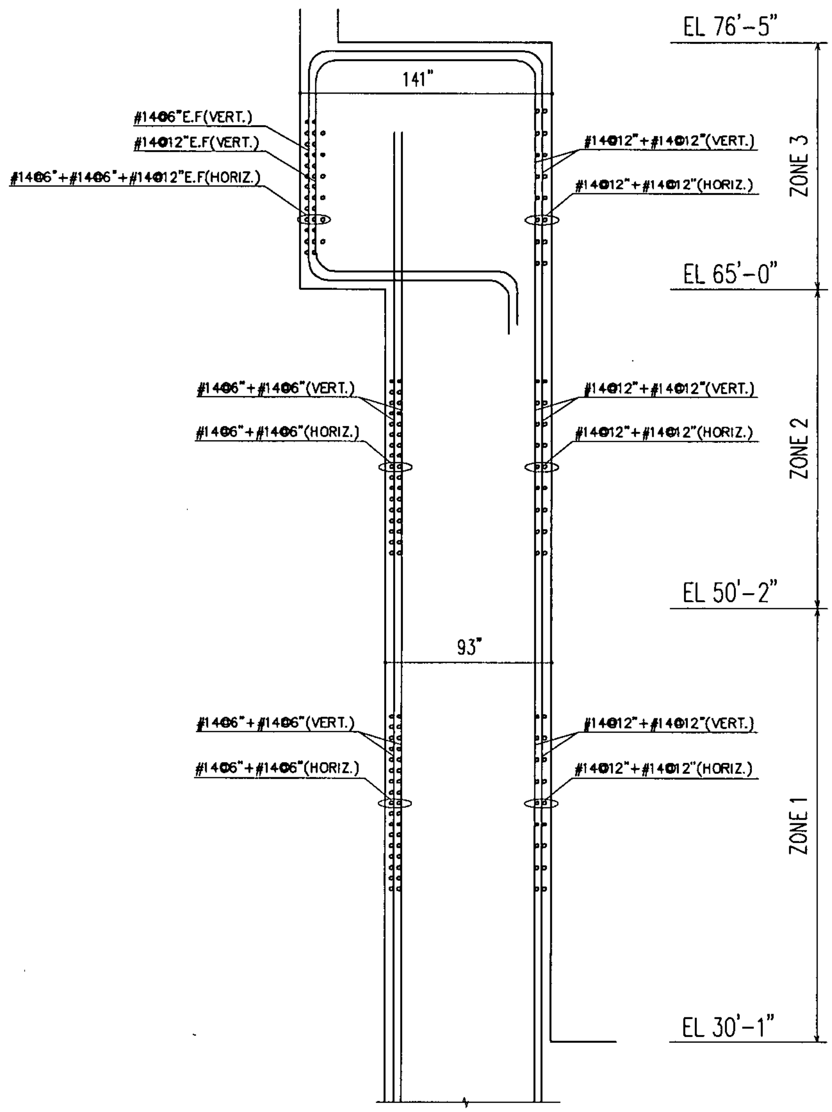
to RAI 342-2000



Horizontal Cross Section

Figure 3.8.4-5 Typical Reinforcement in South interior Wall – SECTION 2 (Sheet 2 of 2)

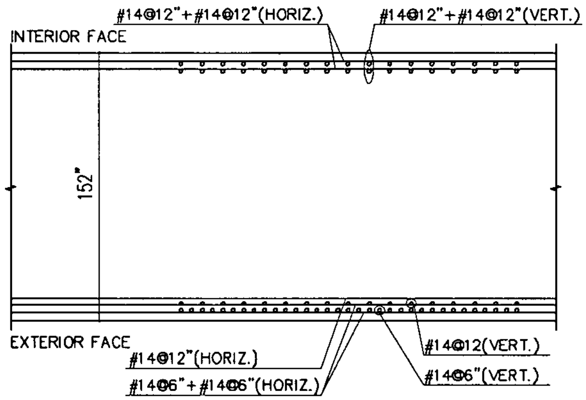
ATTACHMENT 1
to RAI 342-2000



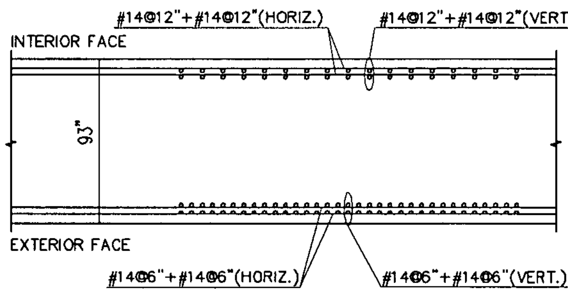
Vertical Cross Section

Figure 3.8.4-6 Typical Reinforcement in North Exterior Wall of Spent Fuel Pit – SECTION 3 (Sheet 1 of 2)

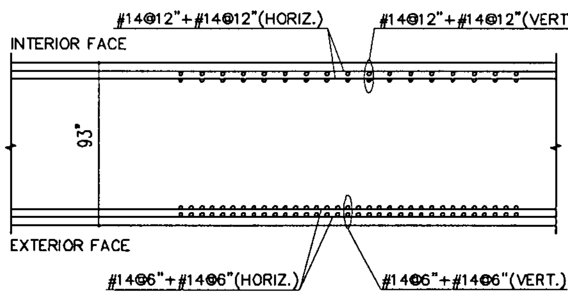
ATTACHMENT 1
to RAI 342-2000



ZONE 3 [EL65'–0" to EL76'–5"]



ZONE 2 [EL50'–2" to EL65'–0"]



ZONE 1 [EL30'–1" to EL50'–2"]

Horizontal Cross Section

Figure 3.8.4-6 Typical Reinforcement in North Exterior Wall of Spent Fuel Pit – SECTION 3 (Sheet 2 of 2)

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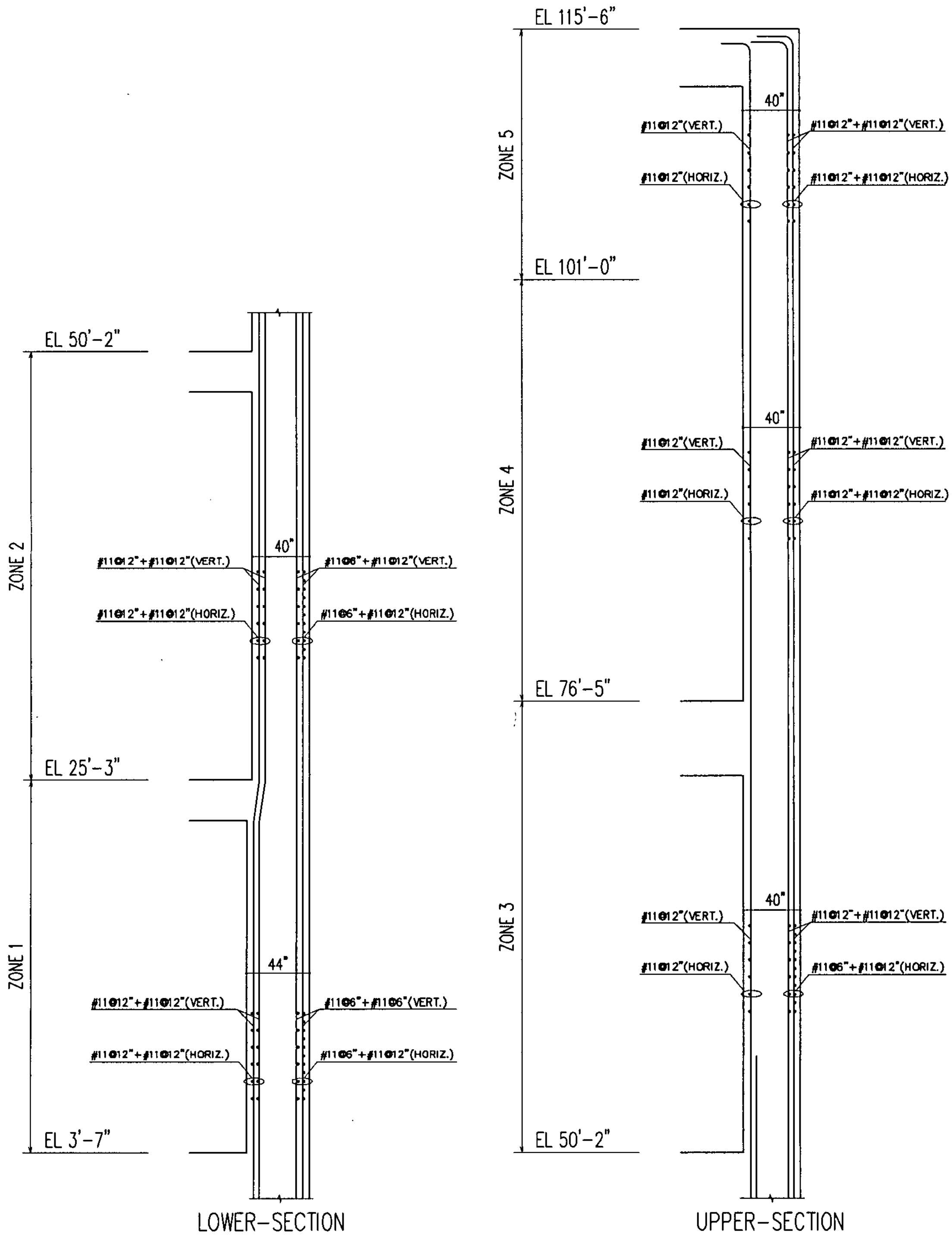
to RAI 342-2000



Vertical Cross Section

Figure 3.8.4-7 Typical Reinforcement in South Exterior Wall – SECTION 4 (Sheet 1 of 2)

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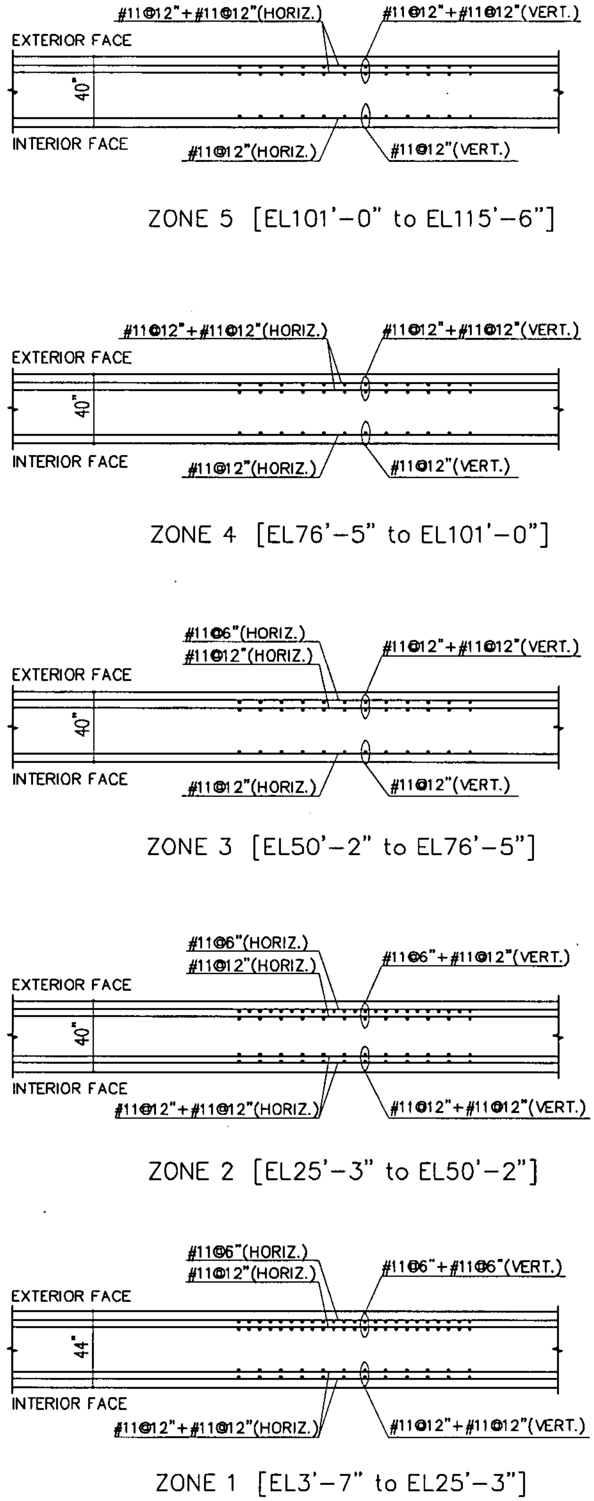
to RAI 342-2000



Horizontal Cross Section

Figure 3.8.4-7 Typical Reinforcement in South Exterior Wall – SECTION 4 (Sheet 2 of 2)

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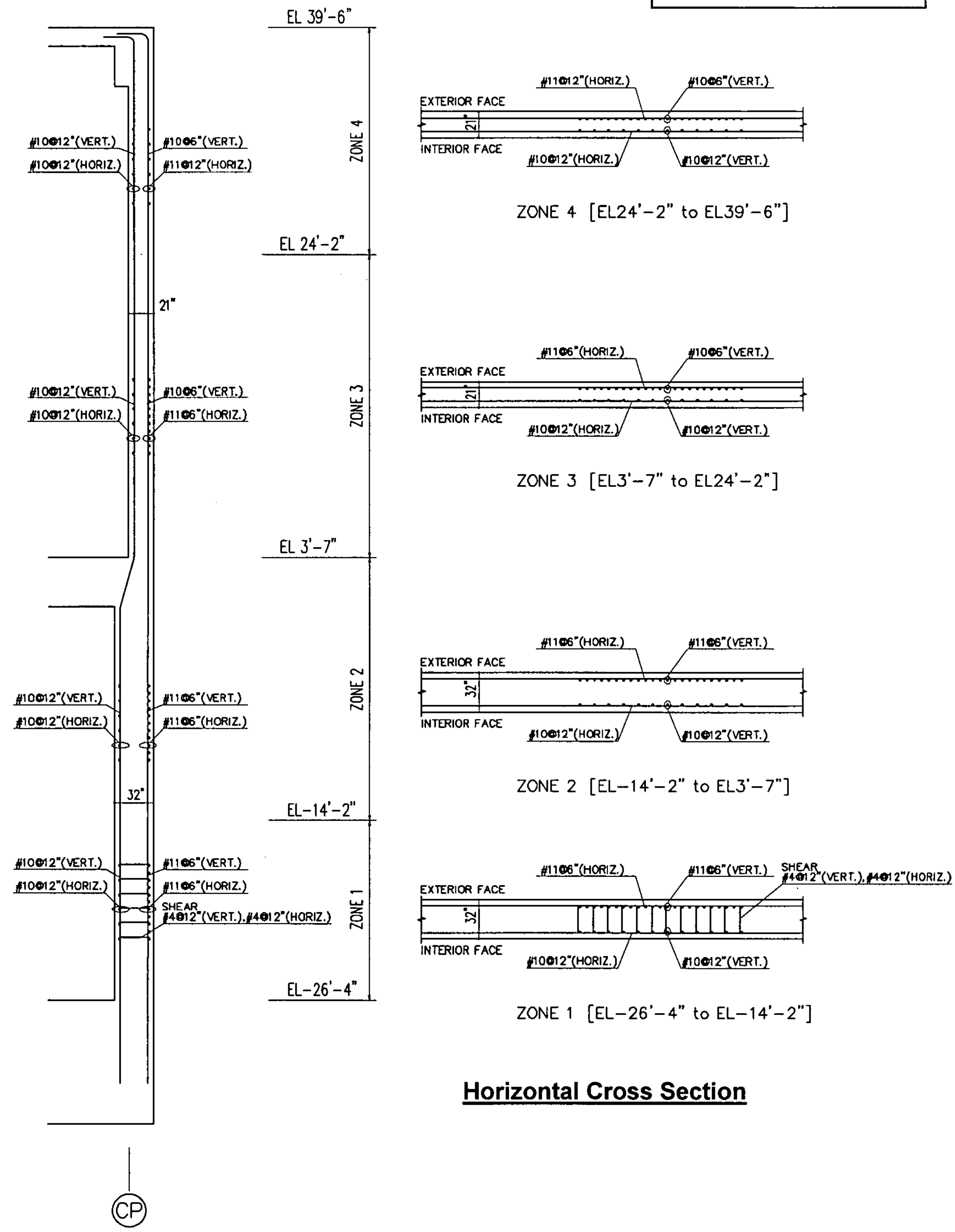
to RAI 342-2000



Horizontal Cross Section

Vertical Cross Section

Figure 3.8.4-13 Typical Reinforcement in South Exterior Wall – SECTION 1 (On Column Line CP and Between Column Lines 1P & 2P)

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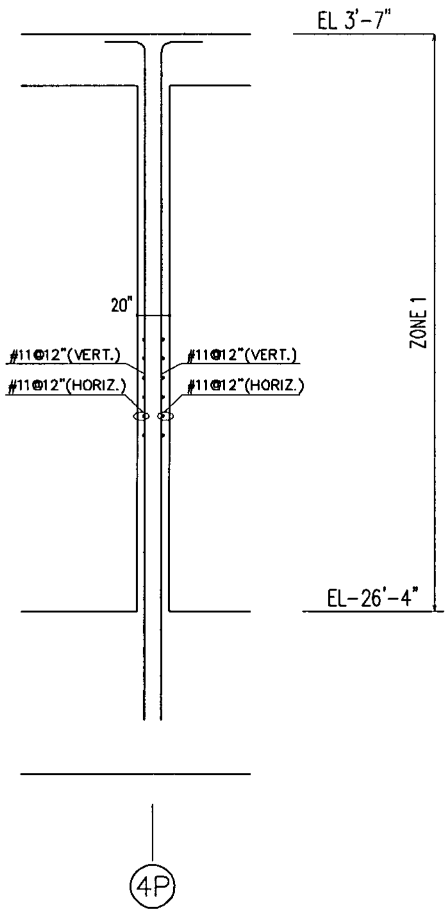
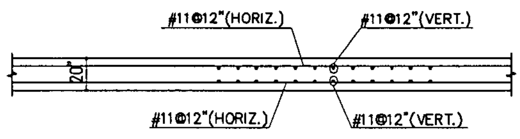


Vertical Cross Section



ZONE 1 [EL-26'-4" to EL3'-7"]

Horizontal Cross Section

Figure 3.8.4-14 Typical Reinforcement in Interior Wall – SECTION 2 (On Column Line 4P and Between Column Lines BP & CP)

Typically stress criteria for ductwork and supports results in selection of standard member sizes and maximum span lengths. ~~However, some HVAC systems require a high degree of leak tightness, experience excessive pressures, or need to account for other external influences (such as tornados) that can require thicker members or closer support spacing.~~ Those HVAC systems that do not satisfy the parameters qualified for standard member sizes and maximum span lengths are designed to satisfy their specific load and operating conditions. Pressures due to flow velocity are based on the operability requirements of each HVAC system.

3A.1.2 Seismic Category II Ductwork

Seismic category II ductwork is not essential for the safe shutdown of the plant and need not remain functional during, and after, a SSE. However, such ductwork and supports must not fall or displace excessively where it could damage any seismic category I structures, systems, and components (SSCs). Seismic category II ductwork and supports, including support anchorages, are therefore analyzed and designed using the same methods and stress limits specified for seismic category I structures and subsystems, except structural steel in-plane stress limits are permitted to reach 1.0 F_y.

3A.2 Applicable Codes, Standards and Specifications

The design and construction of seismic category I HVAC systems conform to AG-1-2003, Code on Nuclear Air and Gas Treatment, including Addendum AG-1a and AG-1b (Reference 3A-8). Sheet metal ducts are constructed in accordance with the American National Standards Institute (ANSI)/Sheet Metal and Air Conditioning Contractors National Association (SMACNA), HVAC Duct Construction Standards – Metal and Flexible (Reference 3A-1). The American Iron and Steel Institute (AISI), Specification for the Design of Cold-Formed Steel Members (Reference 3A-2), provides the methodology for evaluating the effects of shear lag and plate buckling appropriate for this type of duct construction. Structural steel duct supports are designed and constructed in accordance with the American Institute of Steel Construction (AISC) Specification for the Design, Fabrication and Erection of Steel Safety Related Structures for Nuclear Facilities (Reference 3A-3) or AISI as applicable.

Schedule round pipe used as ductwork is not discussed within this Appendix. Codes, standards, and specifications applicable to schedule pipe is in accordance with piping and pipe support criteria in Sections 3.9 and 3.12.

3A.3 Loads and Load Combinations

3A.3.1 Loads

Supports are designed for dead, seismic, thermal loads, and airflow forces at duct elbows, as applicable. Ducts are also designed for the operational and accident pressure loads. Construction live load is considered, however, it is not present during design seismic events. In addition, any accessory loads to the duct or supports are included in the qualification of the duct and duct supports.

The following loads are applicable for the ductwork load combinations:

ADL Additional dynamic loads resulting from system excitations due to structural motion, such as that caused by safety relief valve actuation and other hydrodynamic loads due to the design basis accident (DBA), small pipe break accident (SBA), and intermediate pipe break accident (IBA).

T Load from constraint of free end displacement resulting from thermal or other movements.

DW Dead weight of equipment or ductwork including supports, stiffeners, insulation, all internally or externally mounted components or accessories, and any contained fluids.

DPD Design pressure differential, resulting in dynamic pressure loads from DBA, IBA, or SBA.

W Design wind loads due to design hurricane, design tornado, or other abnormal meteorological condition that could occur infrequently.

EL External loads applied by attached piping, accessories, or other equipment.

FML Fluid momentum loads other than those separately listed, such as the momentum and pressure forces due to fluid flow. Section SA-4211 of ASME AG-1-2003 contains additional clarification of applicable loads.

L Live loads occurring during construction and maintenance, but may also be due to snow, ponded water, and ice. As a minimum, live load is equal to a construction manload of 250 pounds applied at the mid-span of the duct, midpoint of a stiffener, or within a duct panel. When applied on a panel, the load is distributed over a 10 square inch area.

NOPD Normal operating pressure differential, taken as the maximum positive or negative pressure differential that may occur during normal plant operation, including plant startup and test conditions. Included are pressures resulting from normal airflow and damper or valve closure.

SL Seismic loads resulting from the safe shutdown earthquake.

3A.3.2 Load Combinations

Table 3A-1 provides load combinations for ductwork. Refer to Subsection 3.8.4.3 for various load combinations applicable to seismic category I ~~SSCs~~ duct supports.

Seismic category II ducts and duct supports are to be qualified for the applicable SSE to assure that they do not damage any seismic category I SSCs by falling or displacing excessively under any seismic loads. Seismic category II duct supports are, therefore, qualified for the maximum seismic load combinations and associated allowable stresses as discussed in Subsection 3.8.4.3.

3A.4 Design and Analysis Procedures

Refer to Section 3.7 for seismic system analysis and qualification requirements of seismic category I and seismic category II SSCs and their supports.

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3A.4.1 Simplified Design Approach

The duct and duct support designs can be simplified and performed separately. A simplified analysis is applicable when the seismic accelerations are taken as 1.5 times peak of the support attachment spectrum and the system is isolated from any rod hung seismic category II duct.

3A.4.2 Detailed Design Approach

For certain geometric and stiffness conditions, the seismic forces are more accurately analyzed for a duct subsystem, including supports. This approach is considered when (a) the duct run is 3-dimensional, (b) the duct run contains a wye fitting, (c) the duct run contains a branch tee fitting with dimensions within 6 inches of the main duct, (d) the duct run is not isolated from a rod hung category II duct, or (e) the duct and/or supports cannot be qualified using standard designs.

The detailed design approach utilizes an analytical model consisting of a duct run with multiple support points that also account for axial and lateral bracing. The subsystem is analyzed using the response spectrum analysis method for applicable operating and seismic loads, including any accessories and eccentricities that are present.

3A.4.3 Axial Brace Spacing

Axial bracing resist loads in the axial direction of a duct run. Axial braces are strategically located near directional changes in the duct run to avoid adverse load distribution due to axial effects. ~~As a general rule~~ Unless otherwise justified by analysis, axial braces are spaced at intervals less than 50 feet for straight horizontal runs and less than 25 feet for straight vertical runs. A lateral brace on one leg of a 90-degree elbow bend can serve jointly as an axial brace to the other leg of the bend when the axial load is appropriately distributed.

3A.4.4 Lateral Brace Spacing

Lateral bracing resist loads perpendicular to the axial direction of a duct run. The lateral directions for design correspond to the two principal axes of bending for the duct cross-section. For horizontal runs, one lateral direction is horizontal, the other is vertical. For vertical runs, both lateral directions are horizontal.

In determining the placement of braces, a wall (or floor) penetration is not considered a point of lateral support except as specifically designed on a case-by-case basis and shown to have the capacity to provide support. The spacing of lateral braces is based on level of stress in the duct.

3A.5 Structural Acceptance Criteria

3A.5.1 Allowable Stresses

Allowable ductwork stresses are in accordance with AISI, Specifications for the Design of Cold-Formed Steel Structural Steel Members (Reference 3A-2) and ASME AG-1 (Reference 3A-8) Subsubarticles SA-4220, AA-4320 and AA-4330. Allowable ductwork support stresses are in accordance with AISC Specification for Structural Steel Buildings

(Reference 3A-3) or AISI as applicable.

Allowable stress coefficients are applied in accordance with basic allowables of AISC or AISI. Refer to Subsection 3.8.4.5 for a combination of appropriate allowable stresses with the appropriate load combinations and material specifications.

3A.5.2 Deflection Limitations

Seismic category I ducts and duct supports satisfy deflection limits intended to control interface loads and flexible connector requirements. Where flexible connectors are not possible, attached accessories or commodities are designed for these deflections to prevent excessive interaction with the duct or duct support.

No specific requirements for seismic category II duct and duct supports are necessary. These components are designed not to fall during a seismic event. However, displacements are limited to prevent potential adverse interactions with adjacent commodities. Refer to Subsection 3.7.2.8 for criteria relating to seismic interaction of non-category I structures with seismic category I structures.

When HVAC ducts cross between adjacent buildings, the potential for differential movements is accommodated through flexible connectors. Differential displacements caused by seismic motion are obtained at the duct elevation using seismic analysis reports for each building.

3A.6 Materials

The principal materials for fabrication of HVAC ducts and duct supports are thin gauge sheet metal, cold formed steel shapes, and structural steel shapes.

3A.6.1 Thin Gauge Sheet Metal

Sheet metal ducts are welded constructed in accordance with ANSI/SMACNA (Reference 3A-1). The AISI (Reference 3A-2) provides an appropriate methodology for evaluating this type of duct construction.

3A.6.2 Cold Formed Steel Shapes

Cold formed steel shapes that may be used as support members satisfy the requirements specified in Reference 3A-2.

3A.6.3 Structural Steel Shapes

The design, fabrication, and installation of structural steel supports, and structural shapes and plates used in duct construction, complies with AISC (Reference 3A-3).

3A.6.4 Steel Bolts

Bolts of American Society for Testing and Materials (ASTM) A307 Type A (Reference 3A-4) with lockwashers are used for ductwork fit-up and support connections.

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3A.6.5 Anchor Bolts

Anchor bolts are ASTM A307 (Reference 3A-4) or F1554 (Reference 3A-5), 36 x 1,000 pounds per square inch yield strength material. Higher strength F1554 material is used, as necessary, and noted on the design drawings. The flexibility of base plates is considered in determining the anchor bolt loads when expansion anchors are used for supports.

3A.6.6 Welds

Welding electrodes is minimum American Welding Society (AWS) E70 (References 3A-6, and 3A-7) for structural steel, and AWS E60 for sheet steel (less than or equal to 3/16th inch thick).

3A.7 References

3A-1 HVAC Duct Construction Standards – Metal and Flexible. American National Standards Institute/Sheet Metal and Air Conditioning Contractors National Association, 1995.

3A-2 North American Specification for the Design of Cold-Formed Steel structural Members. 2001 Edition and 2004 Supplement, American Iron and Steel Institute, 2001.

3A-3 Specification for the Design, Fabrication and Erection of Steel Safety Related Structures for Nuclear Facilities AISC-N690-1994, 1994 and Supplement 2, American Institute of Steel Construction, 1994.

3A-4 Standard Specification for Carbon Steel Bolts and Studs, 60 000 PSI Tensile Strength. ASTM A307-04E1, American Society for Testing and Materials, 2004.

3A-5 Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength. ASTM F1554-04E1, American Society for Testing and Materials, 2004.

3A-6 Structural Welding Code – Steel. AWS D1.1/D1.1M:2006, American Welding Society, 2006.

3A-7 Structural Welding Code – Sheet Steel. AWS D1.3/D1.3M:2007, American Welding Society, 2007.

3A-8 Code on Nuclear Air and Gas Treatment. ASME AG-1-2003, Addendum AG-1a-2004, and AG-1b-2007, American Society of Mechanical Engineers, 2003.

Table 3A-1

Ductwork Load Combinations

Component Service Level	Load Combinations
A	DW + NOPD + FML + EL + L + T + W
B	Not Required
C	DW + NOPD + FML + EL + SL + ADL + W
D	N + DPD + SSE + ADL, Not Required Unless DPD is Applicable

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3F Design of Conduit and Conduit Supports

3F.1 Description

Conduit is a means of routing electrical and fiber optic cable to and from termination points in equipment and cable tray. The conduit assures electrical and fiber optic cables are protected from various means of damage. Conduit supports are the means by which conduit is supported and protected from seismic events and other postulated loads. The term conduit and conduit supports includes electrical rigid and flexible conduit of various material types and diameters, a variety of conduit support configurations, junction boxes and their supports, and conduit fittings (hereafter referred as conduit systems). Conduit containing non-Class 1E cable in seismic category II and non-seismic structures are not required to satisfy the requirements of this appendix.

In general, the design of conduit and conduit supports is accomplished through the following steps:

- Determine applicable load combinations and corresponding allowable stresses for conduit and conduit supports.
- Limit spacing of conduit supports to maintain conduit stresses within allowable stresses corresponding to the applicable load combinations.
- Assure maximum stresses are within allowable stresses corresponding to the applicable load combination.
- Provide system bracing to control seismic movement and interaction with other category I, structures, systems, and components (SSCs).

3F.1.1 Seismic Category I Conduit Systems

Seismic category I conduit systems, electrical conduit containing 1E cable, are designed for all applicable load combinations to maintain structural integrity within stress limits. This is achieved by analyzing the conduit system and limiting support spacing to maintain critical stresses to acceptably low levels. The seismic qualification of conduit systems, including supporting brackets, is to satisfy the safe-shutdown earthquake (SSE) requirements of the plant system(s) for which it is associated. Seismic category I conduit systems, including support anchorages, in the US-APWR, standard plant seismic category I structures are analyzed and designed for a SSE which is equivalent to the in-structure response spectra developed from the certified seismic design response spectra (CSDRS). Site-specific seismic category I structures are analyzed and designed using as a minimum the site-specific SSE developed from the site-specific ground motion response spectra (GMRS) and foundation input response spectra (FIRS).

3F.1.2 Seismic Category II Conduit Systems

Seismic category II conduit systems, electrical conduit containing non-1E cable in seismic category I buildings, are not essential for safe shutdown of the plant and need not remain functional during, and after, a SSE. However, such conduit systems must not fall or displace excessively where they could damage any seismic category I SSCs. Seismic category II conduit systems, including support anchorages, are therefore analyzed and designed using the same methods and stress limits specified for seismic category I structures and subsystems, except structural steel in-plane stress limits are permitted to reach 1.0 F_y.

3F.2 Applicable Codes, Standards, and Specifications

Conduits are manufactured to satisfy the American National Standard Institute (ANSI) C80.1 American Standard for Electrical Rigid Steel Conduit (ERSC), (Reference 3F-1) or ANSI C80.5, American Standard for Electrical Rigid Aluminum Conduit (ERAC), (Reference 3F-2), as applicable. Junction boxes are manufactured to satisfy the National Electrical Manufacturer Association (NEMA) Standards Publication 250 Enclosures for Electrical Equipment (1000 Volts Maximum) (Reference 3F-3). Installation of the conduit system conforms to the requirements of the National Fire Protection Associations (NFPA) 70, National Electric Code (NEC), (Reference 3F-4).

The American Iron and Steel Institute (AISI) Specification for the Design of Cold-Formed Steel Members (Reference 3F-5) provides the methodology for structurally evaluating cold formed steel shapes, as applicable. Structural steel shapes used for supports are designed and constructed in accordance with the American Institute of Steel Construction (AISC) Specification for the Design, Fabrication, and Erection of Steel Safety Related Structures for Nuclear Facilities (Reference 3F-6). Welding is evaluated and performed in accordance with the American Welding Society (AWS) Standard D1.1 Structural Welding Code, (Reference 3F-7).

3F.3 Loads and Load Combinations

3F.3.1 Loads

Conduit systems are designed for dead, ~~live,~~ seismic, and thermal loads, as applicable. Design dead load includes the working load (weight) of cables permitted in the conduit. In addition, any accessory loads to the conduit and conduit supports are included in the qualification of the conduit and conduit supports.

3F.3.2 Load Combinations

Refer to Subsection 3.8.4.3 for various load combinations applicable to seismic category I SSCs.

Seismic category II conduit and conduit supports are qualified for the applicable SSE to assure they do not damage any seismic category I SSCs by falling or displacing excessively under any seismic loads. Seismic category II conduit supports are, therefore, qualified for maximum seismic load combinations and associated allowable stresses as discussed in Subsection 3.8.4.3.

3F.4 Design and Analysis Procedures

Refer to Section 3.7 for seismic system analysis and qualification requirements of seismic category I and seismic category II SSCs and their supports.

3G Seismic Qualification of Cable Trays and Supports

3G.1 Description

This appendix provides the methodology used to qualify the structural integrity of seismic category I and seismic category II electrical cable trays and cable tray supports (hereafter referred to as "cable tray systems"). Cable tray systems containing non-Class 1E cable in non-seismic structures are not required to be qualified to the requirements of this appendix.

In general, the design of cable trays and cable tray supports is accomplished through the following steps:

- Determine applicable load combinations and corresponding allowable stresses for trays and supports
- Limit spacing of tray supports to maintain tray stresses within allowable stresses corresponding to the applicable load combination
- Assure that the maximum stresses of tray supports are within allowable stresses corresponding to the applicable load combination
- Provide system bracing to control seismic movement and interaction with other seismic category I structures, systems, or components (SSCs).

3G.1.1 Seismic Category I Cable Tray Systems

Seismic category I cable tray systems are designed for all applicable load combinations to maintain structural integrity within stress limits. This is achieved by analyzing the cable tray system (tray, fittings, connectors, fasteners, supports, etc.) and limiting the support spacing to maintain critical stresses to acceptably low levels. The seismic qualification of cable tray systems is to satisfy the safe-shutdown earthquake (SSE) requirements of the structure in which they are contained. Seismic category I cable tray systems, including support anchorages, in US-APWR standard plant seismic category I structures are analyzed and designed for a SSE which is equivalent to the in-structure response spectra developed from the certified seismic design response spectra (CSDRS). Site-specific seismic category I structures are analyzed and designed using as a minimum the site-specific SSE developed from the site-specific ground motion response spectra (GMRS) and foundation input response spectra (FIRS).

3G.1.2 Seismic Category II Cable Tray Systems

Seismic category II cable tray systems are designed to verify that the items will not fall or displace excessively where it could damage any seismic category I SSCs during, and after, a SSE. Seismic category II cable tray systems including support anchorages are, therefore, analyzed and designed for the applicable SSE, such as in-structure response spectra developed from the CSDRS within the standard plant Reactor Building and the East and West Power Source Buildings using the same methods and stress limits specified for seismic category I cable tray systems except structural steel in-plane stress limits are permitted to reach 1.0 F_y.

3G.2 Applicable Codes, Standards and Specifications

Cable trays are manufactured to satisfy the National Electrical Manufacturers Association (NEMA) Standard VE-1, Metal Cable Tray Systems (Reference 3G-1), and consist of thin gauge steel channel side rails on ladder-type or solid-bottom trays, with or without covers. The installation of the cable tray system conforms to the requirements of NEMA Standard VE 2, Cable Tray Installation Guidelines (Reference 3G-2), and National Electric Code (NEC), Article 392, Cable Trays (Reference 3G-3).

The American Iron and Steel Institute (AISI) Specification for the Design of Cold-Formed Steel Members (Reference 3G-4) provides the methodology for evaluating cold formed shapes, as applicable. Structural steel shapes used for supports are designed and constructed in accordance with the American Institute of Steel Construction (AISC) Specification for the Design, Fabrication, and Erection of Steel Safety Related Structures for Nuclear Facilities (Reference 3G-5).

3G.3 Loads and Load Combinations

3G.3.1 Loads

Cable tray systems are designed for dead, live, seismic, and thermal loads, as applicable. Design dead load includes the working load (weight) of cables permitted in the tray (also known as "raceway"). Construction live load, defined as 250 pounds from one person plus equipment carried by the person, is considered in addition to the maximum weight of cables and trays; however, it is not present during design seismic events. In addition, any accessory loads to the cable trays and supports are included in the qualification of the cable tray and cable tray supports.

3G.3.2 Load Combinations

Refer to Subsection 3.8.4.3 for various load combinations applicable to seismic category I, SSCs. When determining dynamic loading for wall mounted supports, envelope the response spectra curves for the floors immediately above and below the support location.

Seismic category II cable tray systems are qualified for the applicable SSE to assure that they do not damage any seismic category I SSCs by falling or displacing excessively under any seismic loads. Seismic category II cable tray systems are, therefore, qualified for maximum seismic load combinations, and associated allowable stresses as discussed in Subsection 3.8.4.3.

3G.4 Design and Analysis Procedures

Refer to Section 3.7 for seismic system analysis and qualification requirements of seismic category I and II SSCs and their supports.

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3G.6.3 Structural Steel Shapes

The design, fabrication, and installation of structural steel supports, and structural shapes and plates used in support construction, comply with AISC-N690-1994, Specification for the Design, Fabrication, and Erection of Steel Safety Related Structures for Nuclear Facilities (Reference 3G-5).

3G.7 References

3G-1 Metal Cable Tray Systems. NEMA Standard VE-1, National Electrical Manufacturer Association, 1998.

3G-2 Cable Tray Installation Guidelines. NEMA VE-2, National Electrical Manufacturer Association, 2006.

3G-3 Cable Trays. NEC Article 392, National Electric Code, 2002.

3G-4 Specification for the Design of Cold-Formed Steel Members. 1996 Edition and Supplement No 1, American Iron and Steel Institute, July 1999.

3G-5 Specification for the Design, Fabrication and Erection of Steel Safety Related Structures for Nuclear Facilities. AISC-N690-1994, American Institute of Steel Construction, 1994.

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Table 1.8-2 Compilation of All Combined License Applicant Items for Chapters 1-19 (sheet 13 of 44)

COL ITEM NO.	COL ITEM
COL 3.8(19)	*The design and analysis of the ESWPT, UHSRS, PSFSVs, and other site-specific structures are to be provided by the COL Applicant based on site-specific seismic criteria.*
COL 3.8(20)	*The COL Applicant is to identify any applicable externally generated loads. Such site-specific loads include those induced by floods, potential non-terrorism related aircraft crashes, explosive hazards in proximity to the site, and projectiles and missiles generated from activities of nearby military installations.*
COL 3.8(21)	*Deleted*
COL 3.8(22)	*The COL Applicant is to address monitoring 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 lean concrete under any basemat above the frost line so that the bottom of lean 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.*