3.7 Seismic Design

The SSCs of the US-APWR are designed as required by the GDC 2 of 10 CFR 50, Appendix A (Reference 3.7-1), to withstand the effects of natural phenomena, including earthquakes, without jeopardizing the plant safety. The US-APWR SSCs are assigned to one of three seismic categories (seismic category I, seismic category II, or non-seismic [NS]) depending on the nuclear safety function of the particular SSC, as discussed in Subsection 3.2.1. The US-APWR standard plant seismic design is based on the SSE and the OBE as discussed in Subsection 3.7.1.1. The OBE defines the magnitude of the ground motion that if exceeded would require that the plant be shut down.

The values of peak ground accelerations (PGAs) and the response spectra of the seismic ground motion in horizontal and vertical directions ~~characterize~~define the magnitude of the design basis earthquake. Certified seismic design response spectra (CSDRS) ~~define~~are used as the site-independent SSE for the seismic design of standard plant structures, and the ground motion response spectra (GMRS) define the horizontal and vertical response spectra of the site-dependent ~~seismic~~SSE design motion. MIC-03-03-00066 MIC-03-03-00054 MIC-03-03-00054

The COL Applicant is to validate the site-independent seismic design of the standard plant for the site-specific conditions, including geological, seismological, and geophysical characteristics, and to develop the site-specific GMRS and foundation input response spectra (FIRS). MIC-03-03-00054

The COL Applicant is responsible for the seismic design of those seismic category I and seismic category II SSCs that are not part of the US-APWR standard plant~~. Spectra appropriately derived from the GMRS can be used to define the~~ using site-specific SSE design ground motion~~ for the design of those seismic category I and II buildings and structures that are not part of the US-APWR standard plant~~. The response spectra of site-specific SSEs are developed following the requirements of RG 1.208 (Reference 3.7-3).~~, and represent the envelope of the foundation input response spectra (FIRS) and a minimum response spectra as discussed in Subsection 3.7.1.1.The COL Applicant is to develop site-specific GMRS and FIRS by an analysis methodology, which accounts for the upward propagation of the GMRS~~. The FIRS are compared to the CSDRS to assure that the US-APWR standard plant seismic design is valid for a particular site. If the FIRS are not enveloped by the CSDRS, the US-APWR standard plant seismic design is modified as part of the COLA in order to validate the US-APWR for installation at that site. MIC-03-03-00066 MIC-03-03-00054 MIC-03-03-00066

3.7.1 Seismic Design Parameters

3.7.1.1 Design Ground Motion

The Peak Ground Acceleration (PGA) of the design ground motion used for the purpose of the site-independent design of the seismic category I SSCs of the US-APWR standard plant is 0.3 g ~~ground acceleration~~ for the two horizontal directions and the vertical direction. The COL Applicant is to confirm that the site-specific PGA at the basemat level control point of the CSDRS is less than or equal to 0.3 g. MIC-03-03-00054 MIC-03-03-00054

Design Ground Motion Response Spectra

Horizontal and vertical response spectra define the design seismic ground motion used for the US-APWR standard plant seismic design. The SSE, CSDRS, Site Specific GMRS, FIRS and OBE, and the spectra, which are used to characterize these earthquake motions, are discussed in the following paragraphs. MIC-03-03-00054

SSE

The SSE is the earthquake which produces the maximum vibratory ground motion for which certain SSCs are designed to remain functional and within applicable stress, strain, and deformation limits.

The SSCs that must remain functional are those necessary to assure the following:

1. The integrity of the RCPB.

2. The capability to shut down the reactor and maintain it in a safe-shutdown condition.

3. The capability to prevent or mitigate the consequences of accidents which could result in potential offsite exposures comparable to the guideline exposures of 10 CFR 100 (Reference 3.7-4).

The CSDRS ~~define the site-independent SSE used~~are used as the SSE for the site-independent design of the US-APWR standard plant seismic category I and seismic category II SSCs. The major seismic category I buildings and structures of the US-APWR standard plant ~~include the R/B, PCCV and containment internal structure, and the east and west PS/Bs~~is the R/B complex which includes the R/B, PCCV, containment internal structure, (CIS), east PS/B, west PS/B, essential service water pipe chase (ESWPC), and seismic category II A/B all on a common basemat. MIC-03-03-00054 MIC-03-03-00054

For the seismic design of seismic category I and seismic category II SSCs that are not part of the US-APWR standard plant, and for the detailed design of the US-APWR standard plant structures that are modified for the site-specific conditions ~~which can affect their integrity~~, a site-~~dependent~~specific SSE ~~that is derived from the site-specific GMRS~~ can be used. ~~Refer to Subsection 3.8.4 for discussion relating to the seismic design of seismic category I and seismic category II buildings and structures that are not part of the US-APWR standard plant~~The site-specific SSE is developed following the requirements of RG 1.208 (Reference 3.7-3). MIC-03-03-00054 MIC-03-03-00066

CSDRS

The CSDRS are presented ~~herein to be approved under 10 CFR 52, Subpart B (Reference 3.7-5)~~ as the site-independent seismic design response spectra for an approved certified design of the US-APWR standard nuclear power plant. The CSDRS ~~characterize the site-independent SSE design ground motion that is defined at a control point located at the bottom of each US-APWR standard plant building basemat~~are identified as an outcrop motion in the free field at the same level as the bottom of the foundation of the R/B complex. MIC-03-03-00054 MIC-03-03-00054

~~The in-structure response spectra (ISRS), which are used to design the seismic category I and II SSCs contained within or mounted to the US-APWR standard plant seismic category I buildings and structures, are computed from the CSDRS using methodology and approaches discussed in Subsection 3.7.2.5.~~ MIC-03-03-00054

The site-independent CSDRS that are employed for the seismic category I design of the US-APWR standard plant are shown for ~~0.5~~2%, ~~2~~3%, 5%, 7%, and 10% damping values in Figures 3.7.1-1 and 3.7.1-2 for the horizontal and vertical components, respectively. The CSDRS are derived from RG 1.60 (Reference 3.7-6) spectra by scaling the spectra contained in RG 1.60 from 1.0 g to 0.3 g zero period acceleration (ZPA) values, and by modifying the RG 1.60 control points to broaden the spectra in the higher frequency range. The RG 1.60 spectral values are based on deterministic values for western United States earthquakes. ~~However, recent seismic research including recently published attenuation relations~~NUREG/CR-6728 (Reference 3.7-14) indicates that earthquakes in the central and eastern United States (CEUS) have more energy content in the higher frequency range than earthquakes in the western United States. Thus, the RG 1.60 (Reference 3.7-6) spectra control points have been modified by shifting the control points at 9 Hz and 33 Hz to 12 Hz and 50 Hz, respectively, for both the horizontal and the vertical spectra. Therefore, for the US-APWR CSDRS, the horizontal spectra control points are at 0.25, 2.5, 12, and 50 Hz and the vertical response spectra control points are at 0.25, 3.5, 12, and 50 Hz. The modified RG 1.60 (Reference 3.7-6) spectra used for the CSDRS are expected to envelope many sites in the central and eastern United States in order to maximize the applicability of the US-APWR standard plant design; however, it is anticipated that there are some site-specific instances, particularly on hard rock sites in high seismic areas, where high-frequency exceedances of the CSDRS may occur. In these cases, the COL Applicant is required to perform site-specific seismic analyses, including a soil-structure interaction (SSI) analysis which may consider seismic wave transmission incoherence ~~and analysis of the cumulative absolute velocity (CAV)~~ of the seismic input motion, in order to determine if high-frequency exceedances of the CSDRS could be transmitted to SSCs in the plant superstructure with potentially damaging effects.

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Consistent with RG 1.60 (Reference 3.7-6), the CSDRS representing the vertical accelerations is obtained by scaling the horizontal acceleration response spectra (ARS) by a factor of 2/3 for frequencies less than 0.25 Hz. The scaling factor that varies from 2/3 to 1.0 is applied for the frequency range between 0.25 and 3.5 Hz. The horizontal and vertical acceleration spectra are kept identical above frequency 3.5 Hz and, consequently, the vertical PGA is taken as the same as the horizontal PGA.

The US-APWR design response spectral accelerations for each of the spectral control points are presented in Tables 3.7.1-1 and 3.7.1-2. The US-APWR site-independent CSDRS as defined herein meet the requirements of 10 CFR 50, Appendix S(IV)(a)(1)(i) (Reference 3.7-7), which require that the horizontal component of the SSE ground motion in the free-field at the basemat level of the structures must be an appropriate response spectra with a PGA of at least 0.1 g.

Site-Specific GMRS

In accordance with NUREG-0800, SRP 2.5.2 (Reference 3.7-8), the site-specific GMRS, developed by the COL Applicant, define the site-specific SSE through a horizontal and MIC-03-03-00054

vertical response spectra of the free-field motion that is specified either on the ground surface or at an outcrop (real or hypothetical) of the uppermost in-situ competent material that will exist after excavation. The competent material is defined as having a shear wave velocity of 1,000 ft/s or greater. Free-field ground motion is defined as the seismic motion of the ground that is not influenced by the presence of any basemats and structures.

Site-specific GMRS are developed at a sufficient number of frequencies (at least 25) that adequately represent the local and regional seismic hazards using the site-specific geological, seismological, and geophysical input data. A probabilistic seismic hazard analysis is performed that is based on the performance-based approach outlined in RG 1.208 (Reference 3.7-3). Horizontal GMRS are developed using a site amplification function obtained from site response analyses performed on site-specific soil profiles that include the layers of soil and rock over the generic rock conditions defined by the attenuation relationships used in the probabilistic seismic hazard analysis (PSHA). For example, attenuation relationships for the CEUS typically define generic rock as the rock with shear wave velocity exceeding 9,200 ft/s. ~~The~~Randomized site-specific soil profiles are used to account for the uncertainties and variations of the site soil and rock properties. The site response analysis will address probable effects of non-linearity due to strain-dependence of the subgrade materials' response. Equivalent linear methodology can be utilized with soil stiffness and damping degradation curves that represent the stiffness and damping properties of the subgrade materials as a function of strain. However, the strain-compatible soil material damping shall not exceed 15% as stipulated in SRP 3.7.1 (Reference 3.7-10). MIC-03-03-00054

With respect to determining the site-specific GMRS, note that Section 2.5.4 requires site-specific characterization of subsurface materials and investigation of the associated engineering properties to assure consistency with Section 3.7.2. Further, vertical GMRS are developed by combining the horizontal GMRS and the most up-to-date vertical/horizontal response spectral ratios appropriate for the site obtained from the most up-to-date attenuation relationships.

FIRS

~~The site-specific GMRS serves as the basis for the development of~~The site-specific FIRS ~~that~~ define the horizontal and vertical response spectra of the outcrop ground motion at the bottom elevation of the seismic category I and II basemats. ~~Free-field outcrop spectra of site-specific horizontal ground motion are derived from the horizontal GMRS using site response analyses that consider only the wave propagation effects in materials that are below the control point elevation at the bottom of the basemat. The material present above the control point elevation can be excluded from the site response analysis.~~Free-field outcrop spectra of site-specific horizontal ground motion are developed consistent with the horizontal GMRS using site response analyses which employ a suite of randomized soil profiles to account for uncertainties and variations in the site soil and rock properties. The profiles also include materials present above the input ground motion control point elevation in order to account for their effect on soil and rock properties. MIC-03-03-00066 DCD_03.07.01-14 MIC-03-03-00054

Appendix S (IV)(a)(1)(i) of 10 CFR 50 (Reference 3.7-7) requires that the SSE ground motion in the free-field at the basemat level must be represented by an appropriate response spectra with a PGA of at least 0.1 g. This requirement is met on a site-specific

basis by considering minimum horizontal response spectra that are tied to the shapes of the US-APWR CSDRS and anchored at 0.1g. Since the CSDRS are based on modified RG 1.60-spectra, this assures that there is sufficient energy content in the low-frequency range. The COL Applicant is to assure that the horizontal FIRS defining the site-specific SSE ground motion at the bottom of seismic category I or II basemats envelope the minimum response spectra required by 10 CFR 50, Appendix S (Reference 3.7-7), and the site-specific response spectra obtained from the response analysis. The same requirements apply to the vertical FIRS, which are developed from the horizontal FIRS by using vertical/horizontal response spectral ratios appropriate for the site.

The COL Applicant is to perform an analysis of the US-APWR standard plant seismic category I design to verify that the site-specific FIRS at the basemat level control point of the CSDRS are enveloped by the site-independent CSDRS. If the verification analysis proves the site-independent seismic design to be inadequate, a reanalysis of the affected SSCs is performed based on a site-specific SSE defined by the site-specific FIRS. In this case, the scoping re-design analysis may focus on affected SSCs rather than a complete analysis of all SSCs. ~~The scoping analysis may determine that the CSDRS (as defined at their basemat level control point) need to be adjusted or modified. One possible example would be if the site-specific FIRS exhibited a significant peak at a higher frequency than the peak of the CSDRS. In this case it might be impractical to broaden the peak of the CSDRS, and instead the seismic design could be modified based on a design for two separate spectra (the site-specific FIRS in addition to the standard plant CSDRS), subject to further review for adequacy as discussed in NUREG-0800, SRP 3.7.1 (Reference 3.7-10).~~ MIC-03-03-00054

OBE

The OBE specifies the magnitude of ground motion that requires the shutdown of the plant operations. Appendix S of 10 CFR 50 (Reference 3.7-7) stipulates that the magnitude of an OBE can be adopted either as (A) 1/3 or less of the SSE; or (B) a value greater than 1/3 of the SSE. For Option A, the Applicant is not required to perform explicit response or design analyses. If Option B is chosen, an explicit analysis and design must be performed to demonstrate that all SSCs necessary for the continued operation without undue risk to the health and safety of the public will remain functional within applicable stress, strain, and deformation limits. For the US-APWR standard plant, the OBE is defined as 1/3 of the SSE (which is the CSDRS). Therefore, no specific analysis is required for the standard plant. MIC-03-03-00054

The COL Applicant is to set the value of the OBE that serves as the basis for defining the criteria for shutdown of the plant, according to the site-specific conditions. ~~The site-specific seismic design does not have to consider OBE loads when Option A is maintained by setting the OBE spectra as enveloped by 1/3 of the site-specific FIRS and GMRS.~~ Subsection 3.7.4 describes the criteria and the seismic instrumentation used to determine whether the OBE has been exceeded. ~~By limiting the value of the OBE to 1/3 of the site-independent SSE, Option A is also maintained for the site-independent seismic design of the US-APWR standard plant, and no design analysis is required to address the OBE loads for the seismic category I SSCs that are designed using the site-independent SSE.~~ MIC-03-03-00054

It is recognized that during the life of the plant, the site may be subjected to seismic excitations of lower levels than the SSE. This can have an effect of reducing the "life expectancy" of those items sensitive to fatigue (i.e., piping, electrical, and mechanical equipment). Earthquake cycles are considered in the fatigue evaluation of the ASME Code, Section III, Class 1, 2, and 3. Components and Core Support Structures (Reference 3.7-11) (when required by the ASME Code) are discussed further in Sections 3.9 and 3.12, and in Section 3.10 for qualification testing of equipment. For fatigue evaluations, based on the OBE defined as less than or equal to 1/3 of the SSE, the guidance for determining the number of earthquake cycles for use in fatigue calculations is the same as the guidance provided in the ~~NRC staff requirements memorandum for Secretary of the Commission Letter (SECY) 93-087~~U.S. NRC Staff Requirements Memorandum SECY-93-087 (Reference 3.7-12) for piping systems. The number of earthquake cycles to consider is two SSE events with 10 maximum stress cycles per event. Alternatively, the number of fractional vibratory cycles equivalent to that of 20 full SSE vibratory cycles may be used (but with an amplitude not less than 1/3 of the maximum SSE amplitude) when derived in accordance with Institute of Electrical and Electronic Engineers (IEEE), Standard 344-2004, Appendix D (Reference 3.7-13). MIC-03-03-00054 MIC-03-03-00054

Design Ground Motion Time History

~~As described in Technical Report MUAP-10001 (Reference 3.7-47), one set of three statistically independent time histories of seismic motion is synthesized from seed recorded earthquake ground motions for use as the input motion in the earthquake response analysis of the US-APWR standard plant including the R/B, PCCV, containment internal structure, and PS/Bs. The three time histories are developed to represent the ground motion for the three orthogonal earthquake components, two horizontal ("H1" and "H2") and vertical ("V") following the requirements and conditions set in Section II of SRP 3.7.1 (Reference 3.7-10) for the development of a single set of time histories Option 1, Approach 2.one set of three statistically independent artificial ground motion time histories is generated in accordance with guidance of SRP 3.7.1 (Reference 3.7-10), Subsection 3.7.1.II.1B, Option 1 Approach 2, for use in US-APWR standard plant seismic analysis. These time histories represent ground motion for the three orthogonal directions, two horizontal ("H1" in the north-south [NS] direction, and "H2" in the east-west [EW] direction) and one vertical ("V").~~ DCD_03.07.02-35 MIC-03-03-00054

~~The three orthogonal directions may be alternately referred toreferenced within Section 3.7 using the following differentequivalent designations:~~

~~H1 = Direction 1 = NS = Plant north-south= Global -axis~~

~~H2 = Direction 2 = EW = Plant east-west = GlobalY-axis~~

~~V = Direction 3 = Vertical = UD = Up-Down = Gloal Z-axis~~

~~Approach 2 is utilized withselected for the objective of generating artificial acceleration time histories whose response spectra achieve approximately mean based fits to the target CSDRS presented in Figures 3.7.1-1 and 3.7.1-2. Tables 3.7.1-1 and 3.7.1-2 show horizontal and vertical CSDRS control points for damping factors of 0.5%, 2%, 5%, 7% and 10%. CSDRS control points are based on modified RG 1.60 (Reference 3.7-6) response spectra, as described above. The average ratio of the ARS calculated from the~~ DCD_03.07.02-35

~~artificial time histories to the corresponding target CSDRS is kept only slightly greater than one. The spectral acceleration ratio is calculated frequency by frequency.~~ MIC-03-03-00054

~~The artificial time histories plots for the ground accelerations, velocity, and displacements in three orthogonal directions ("H1," "H2," and "V") are shown in Figures 3.7.1-3, 3.7.1-4, and 3.7.1-5, respectively. The time history plots of the ground acceleration, velocity, and displacement are shown together to demonstrate their non-stationary process.~~ DCD_03.07.02-35 MIC-03-03-00066

~~Figures 3.7.1-6, 3.7.1-7, and 3.7.1-8 show the ARS of the US-APWR artificial time histories for 5% damping for the three orthogonal directions H1, H2, and V, respectively. The plots of the CSDRS, which are based on the modified RG 1.60 (Reference 3.7-6) response spectra as described in Subsection 3.7.1.1, are also included in the figure to demonstrate that the ARS of the synthesized time histories envelope those of the CSDRS for 5% damping values. The figures demonstrate that the synthesized acceleration time histories do not have significant gaps in the Fourier amplitude spectra but are also not biased high with respect to the target CSDRS.The CSDRS with 5% damping is shown in these figures for comparison with the ARS from artificial time histories. This demonstrates uniform energy distribution by showing that the artificial acceleration time histories are not biased high, do not have significant response gaps, and show general consistency with the target CSDRS.~~

~~The three US-APWR artificial time histories are discussed further with respect to the requirements specified in NUREG-0800, SRP 3.7.1 (Reference 3.7-10) for Approach 2 in the following, steps (a) through (d):The artificial time histories conform to NUREG-0800, SRP 3.7.1 (Reference 3.7-10) Section II for generation of a single set of time histories in accordance with Option 1, Approach 2, as summarized in Table 3.7.1-4 and described in the following, steps (a) through (d):~~

a. ~~The US-APWR artificial time histories have a sufficiently small time increments (Δt =0.005 seconds) and a total duration of 22.005 seconds. The time history data records have a Nyquist frequency of $N_f = 1/(2\Delta t)$ =100 Hz, and meet the NUREG-0800 SRP 3.7.1 requirement of a total duration of at least 20 seconds. The time increment of 0.005 seconds is lower than the maximum time increment of 0.01 seconds permitted by SRP 3.7.1. The Nyquist frequency of 100 Hz is considered to be above the range of frequencies important for the design of the US-APWR plant and assures that the seismic analysis capture the responses of SSCs in the high frequency range. This is particularly important for site-specific subgrade conditions where seismic category I structures are founded on a hard rock subgrade.~~

b. ~~The 5% damped ARS of the US-APWR artificial time history components, shown in Figures 3.7.1-6, 3.7.1-7, and 3.7.1-8, are computed at 300301 frequency points that are divided such that 100 frequency points are uniformly spaced over the log frequency scale from 0.1 Hz to 1 Hz, 1 Hz to 10 Hz, and 10 Hz to 100 Hz. Each ARS obtained from the three artificial ground motion time history components is compared with the target response spectra at each frequency computed in the frequency range from 0.1 Hz to 100 Hz.~~ DCD_03.07.02-35 MIC-03-03-00066

c. ~~The 5% damped ARS computed for each of the three US-APWR artificial time history components do not fall more than 10% below the corresponding CSDRS~~ DCD_03.07.02-35

~~target response spectra at any particular frequency. In addition, within a frequency window no larger than ±10% centered at any frequency data point, none of the three ARS (H1, H2, and V) falls below its corresponding target CSDRS. Meeting the requirements of SRP 3.7.1 is confirmed by assuring that, for each of the spectra derived from the artificial time history components, no more than nine adjacent frequency points fall below the CSDRS target response spectra for frequencies between 0.1 Hz and 100 Hz. This ensures that the response spectra resulting from the artificial time history components do not fall below the corresponding target response spectra in large frequency windows. Table 3.7.1-4 demonstrates that these requirements are met by showing a summary of the frequency non-exceedances.~~ MIC-03-03-00054 DCD_03.07.02-35

d. ~~In lieu of the power spectral density requirement of Option 1 Approach 1 in NUREG-0800, SRP 3.7.1 (Reference 3.7-10), Approach 2 specifies that the computed 5% damped response spectra of each artificial ground motion time history component does not exceed its target response spectra at any frequency by more than 30% (a factor of 1.3) in the frequency range of interest. For the US-APWR, the response spectra derived from the artificial time histories are checked to ensure that they do not exceed the corresponding target spectra (CSDRS) by more than 30% at any frequency range measured as described in item (b) above. The results of this check are presented in Table 3.7.1-4~~.

~~The cross-correlation coefficients between the three components of the design time histories are as follows:~~ MIC-03-03-00066

~~ρ_{12} = -0.0892, ρ_{23} = -0.0654, and ρ_{31} = -0.0836~~

~~The artificial time histories also conform to NUREG-0800, SRP 3.7.1 (Reference 3.7-10), Acceptance Criteria 1B, guidance as summarized in Table 3.7.1-7 and further described below:~~

~~Cross-Correlation between Components~~

~~Cross-correlation coefficients between the three artificial ground motion time histories are as follows:~~

~~ρ_{12} = 0.0892, ρ_{23} = -0.0836, and ρ_{31} = -0.0654~~

~~where 1, 2, and 3 are the three global directions corresponding to north-south, east-west, and vertical directions for the US-APWR standard plant.~~

~~Since the absolute values of the cross-correlation coefficients of the US-APWR artificial time histories are less than 0.16, as demonstrated above, in accordance with NUREG/CR-6728 (Reference 3.7-14), the time histories are considered statistically independent of each other.~~ DCD_03.07.02-35

~~Duration of Motion~~

~~Each time history of the set of three statistically independent time histories which are developed for design of the US-APWR seismic category I buildings has a strong duration~~

~~of motion greater than 7 seconds and a total duration of motion greater than 22 seconds. The strong duration of motion meets the acceptance criterion of 6 seconds minimum for strong motion duration as given in SRP 3.7.1 (Reference 3.7-10) for design time histories. The duration of motion has been determined usingto be long enough to capture the random phase characteristics of the earthquake motion. The total duration of motion meets the acceptance criterion of 20 seconds minimum as given in SRP 3.7.1 (Reference 3.7-10) design time histories, Option 1, Approach 2 Part (a).~~ MIC-03-03-00054 DCD_03.07.01-15

~~For the linear structural analyses, which are based on the synthesized time histories presented in Technical Report MUAP-10001 (Reference 3.7-47) that are used to design US-APWR standard plant seismic category I buildings and structures, the total duration of the ground motion time histories has been demonstrated to be long enough such that adequate representation of the Fourier components at low frequency is included in the time history.~~

~~The corresponding stationary phase strong-motion duration is consistent with the longest duration of strong motion from the earthquakes defined in SRP 2.5.2 (Reference 3.7-8) at low and high frequency and as presented in NUREG/CR-6728 (Reference 3.7-14). The strong-motion duration is defined as the time required for the Arias Intensity to rise from 5% to 75% in accordance with SRP 3.7.1 (Reference 3.7-10). The uniformity of the growth of this Arias Intensity has been examined and is acceptable. The duration of motion of the US-APWR artificial time histories with respect to the time duration needed to achieve 5% and 75% Arias intensities is summarized in Table 3.7.1-5.~~

A set of three statistically independent artificial ground motion time histories is generated in accordance with guidance of SRP 3.7.1 (Reference 3.7-10), Subsection 3.7.1.II.1B, Option 1 Approach 1, for use in US-APWR standard plant seismic analysis. These time histories represent ground motion for the three orthogonal directions, two horizontal ("H1" in the north-south [NS] direction, and "H2" in the east-west [EW] direction) and one vertical ("V"). Five additional sets of artificial ground motion time histories are developed as described in Section 3.8.5.5.2 to address sliding.

SRP 3.7.1 (Reference 3.7-10), Subsection 3.7.1.II SRP Acceptance Criteria 1B, Option 1 Approach 1 provides methodology used to generate a design basis time history with three components compatible with the CSDRS from seed recorded earthquake ground motions. The seed used to develop the design basis time history is a segment including the strong motion portion of the BAL (Mount Baldy, CA) recordings, i.e., the January 17th, 1994, Magnitude 6.7 Northridge Earthquake, obtained from the Pacific Earthquake Engineering Research (PEER) Center's digital ground motion library (Reference 3.7-56) recorded at the Mt. Baldy Station.

The BAL recordings of the Northridge earthquake are selected because they have the required durations and correlations (statistical independence among the three components) and because their spectral shapes, when scaled, are a reasonably good match to the CSDRS in the 2-20 Hz range for all three orthogonal components. The recorded time histories contain 4,000 digitized data points using a 0.01 second time step. The strong motion portion of the recorded time histories between t=11.0 to t=33.08 seconds, i.e., duration of 22.08 seconds, are extracted as the seeds which are developed to be compatible with the CSDRS. The digital acceleration records are linearly

interpolated to obtain accelerations at every 0.005 seconds to enable the time histories to account for higher frequency content after adjustment such that their Nyquist frequency is 100 Hz. MIC-03-03-00054

The goal of the artificial time history development process is to produce modified time histories whose response spectra envelop the CSDRS for the US-APWR. In order to achieve this goal, the Fourier amplitudes of the seed acceleration time histories are modified to generate three new acceleration time histories. This Fourier amplitude modification process is iterated until the response spectra calculated from the modified Northridge time histories envelop the target CSDRS at damping ratios of 2%, 3%, 5%, 7%, and 10%.

Once the response spectra of the time histories envelop the CSDRS, the PSD envelope requirements are assessed. This development of PSD targets, and development of the PSD curves from the time histories, is done in conformance with guidance in NUREG/CR-5347 (Reference 3.7-59) Appendix B and SRP 3.7.1 (Reference 3.7-10) Appendix A. This process is described in more detail in MUAP-10006 (Reference 3.7-48). The target PSDs are shown in Figure 3.7.1-9. At frequencies with PSD lower than 80% of the target PSD, the Fourier amplitudes of the time histories are adjusted to satisfy the PSD requirements. Then a baseline correction is applied to the time histories.

Next, the resulting time histories are verified for their compliance with the SRP 3.7.1, Option 1, Approach 1 Acceptance Criteria. When necessary, the baseline corrected time histories are scaled to comply with the enveloping criteria for the spectra at the 2%, 3%, 5%, 7%, and 10% damping ratios, and the envelope requirements for the target power spectral density functions.

Finally, the time histories are checked for the requirements of strong motion duration, correlation coefficients, and V/A and AD/V^2 ratios, where A is the maximum ground acceleration, V is the maximum ground velocity, and D is the maximum ground displacement. The final modified Northridge time histories are the design basis time histories used as input ground motions for the SSI and SSSI analyses.

The final design basis time histories are shown in Figure 3.7.1-3, Figure 3.7.1-4, and Figure 3.7.1-5. The corresponding velocity and displacement time histories have also been computed and are plotted in the same set of Figures. Each of these component time histories meets the criteria of SRP 3.7.1 Option 1, Approach 1. Compliance to these is summarized in Table 3.7.1-3.

Table 3.7.1-3 provides statistical independence values of the three components of the design basis time histories, which satisfies the pertinent SRP 3.7.1 criterion that the absolute value of correlation coefficients between the components must be less than 0.16.

As demonstrated in Table 3.7.1-3 the total durations of the design basis time histories meet the SRP guidance criteria that the durations exceed 20 seconds. The table also shows the rise time, strong motion duration, and decay time of each component. These values are computed based on the definition of strong motion duration in SRP 3.7.1, using the normalized Arias Intensity (AI). Figure 3.7.1-13 shows the normalized AI plots of cumulative energy for each component. The time history components show an initial

time interval of gradual energy buildup, followed by a ramp of rapid energy accumulation, and then followed by a gradual tapering of energy accumulation. The strong motion duration should be at least six seconds according to SRP 3.7.1 and in compliance with duration criteria for earthquake magnitude and distance bins listed in Table 3.7.1-4. The strong motion durations of the design basis time history satisfy both duration criteria. MIC-03-03-00054

Table 3.7.1-3 also shows the V/A and AD/V^2 ratios for mean ratios ± one standard deviation for the earthquakes of magnitude bins of **M**6.5+ with distance bins from 10 to 100 km, using data provided in Table 3-6 of NUREG/CR-6728 (Reference 3.7-14). The V/A and AD/V^2 ratios of the design basis time histories are within the limits in Table 3.7.1-3.

Figure 3.7.1-6 through Figure 3.7.1-8 graphically demonstrate that the response spectra derived from the design basis time histories are developed in accordance with SRP 3.7.1 Option 1, Approach 1, for time history components 180 (H1), 090 (H2), and Vertical (UP), respectively. The response spectra of each component envelopes the CSDRS at 2%, 3%, 5%, 7%, and 10% damping values.

Figure 3.7.1-10 through Figure 3.7.12 show that the smoothed PSDs of the design basis time histories are greater than 80% of the horizontal and vertical target PSDs at all frequencies between 0.3 Hz and 50 Hz, for the three time history components.

Adequate representation of the Fourier components at low frequency is achieved by ensuring the artificial time history matches the CSDRS at all damping values and meets the PSD targets. As demonstrated above, the time histories developed from the Northridge Mt. Baldy seeds satisfy all the requirements described in the Option 1, Approach 1 of SRP 3.7.1 (Reference 3.7-10).

For site-specific design, the applicant will develop ground motion time histories that are compatible with the site-specific FIRS. The COL Applicant is to verify that the site-specific ratios V/A and AD/V^2 (A, V, D, are PGA, ground velocity, and ground displacement, respectively) are consistent with characteristic values for the magnitude and distance of the appropriate controlling events defining the site-specific uniform hazard response spectra. These parameters are examined to assure that they are consistent with the values determined for the low and high frequency events described in Appendix D of RG 1.208 (Reference 3.7-3).

The COL Applicant is to provide site-specific design ground motion time histories and durations of motion.

3.7.1.2 Percentage of Critical Damping Values

The site independent SSI analyses that are performed separately for each of the three directional earthquake excitations use the same generic profiles representing the total unit weight and strain compatible shear wave velocity, compression wave velocity and hysteretic damping ratio for the soil layer media.The strain compatible damping values assigned to these six generic profiles are well below the 15% limit set by SRP on shear wave damping and 10% limit on compression wave damping recommended by the correlation studies in Reference 3.7-62. MIC-03-03-00054

The same median values of shear wave damping are used for both shear and compression wave damping in order to account, in a more realistic manner, for the dissipation of energy in the soil under the wave propagation pattern present in the SSI model. The seismic SSI analyses for horizontal and vertical seismic input motions assume that the input motions are caused by different horizontal and vertical seismic wave field excitations even though the seismic input environment is always 3-D consisting of simultaneous 3-component seismic input motions. Due to the simplified wave propagation assumption made in the SSI vertical motion analyses where the motion is applied as vertically propagating compression waves, strain iterated shear wave damping is assumed for compression wave damping to avoid unrealistic vertical motions at high frequencies. MIC-03-03-00054

Correlation studies of the vertical site response motions recorded in actual earthquakes with the vertical motions predicted from vertical One-Dimensional wave propagation site response analyses have been made (Reference 3.7-10).These studies conclude that, using the strain compatible soil damping values derived from the horizontal site response analyses as the damping values for vertical site response analysis, but limiting their values to no more than 10%, produces reasonably good correlation between the predicted and recorded vertical site response motions. Consistent with this conclusion, the soil damping values used for the horizontal and vertical SSI analyses are the shear strain compatible soil damping values derived from the horizontal free field site response analyses. The horizontal SSI response analyses are performed assuming vertically propagating plane shear wave field excitations. The vertical SSI response analysis is performed assuming vertically propagating plane compression wave field excitation, the shear strain compatible damping values derived from the horizontal site response analyses are used but with their values limited to no more than 10%, as recommended by the correlation studies reported in Reference 3.7-62.

The site response analyses use very low values for the material damping of hard base rock in order to model the low dissipation of energy in the deep hard rock strata. In order to improve the numerical stability of SSI results, the damping of the base rock material when included in the site profile is set to a low nominal value of 0.1%. This modification does not affect the SSI response because, unlike in the site response analyses, the thickness of the modeled hard rock strata in the SSI site model has a finite thickness so the use of higher damping values realistically projects the actual dissipation of energy in the base rock.

Damping coefficient values representing percentages of critical damping are assigned to the linear-elastic models to quantify the dissipation energy in the dynamic system. ~~Table 3.7.3-1(a) presents the values of damping coefficients used for the SSE seismic analysis of seismic category I and II systems and subsystems. The specified damping coefficients are in accordance with RG 1.61 (Reference 3.7-15), and are based on consideration of the material, load conditions, and type of construction used in the structural system.~~ Two levels of stiffness and damping are developed and assigned to structural models used for seismic response analyses in order to capture structural stiffness and damping variations caused by concrete cracking: (1) full stiffness (uncracked concrete) corresponding to low stress levels; and (2) reduced stiffness (cracked concrete) corresponding to high stress levels. DCD_03.07.02-35

In accordance with RG 1.61 (Reference 3.7-15) guidance and associated stress levels and industry standards, OBE structural damping values are used with the full stiffness (uncracked concrete) and SSE structural damping values are used with the reduced stiffness (cracked concrete). ~~To ensure that the standard seismic designs ISRS of the R/B complex and PS/B properly address concrete cracking effects, SSI analyses were performed using both full and reduced stiffness.~~CIS and PCCV stiffness and damping are based on loading conditions as described in Section 3.7.2.3.5. MIC-03-03-00066 MIC-03-03-00054

OBE structural damping values shown in Table 3.7.3-1(b) are for reinforced concrete, prestressed concrete and steel concrete modules assigned to the full stiffness (uncracked) model to calculate the effects from lesser dissipation of energy in the structures when they are subjected to low stress levels. SSE structural damping values shown in Table 3.7.3-1(a) are for reinforced concrete, prestressed concrete and steel concrete modules assigned to the reduced stiffness (cracked concrete) model to calculate the effects of greater dissipation of energy in the structures when they are subjected to higher stress levels. MIC-03-03-00066 MIC-03-03-00066 DCD_03.07.02-35

~~In order to address the variations of cracking patterns under normal operating and accident thermal conditions, SSE loads for Containment Structures (PCCV and CIS) are based on envelope of SSI responses obtained from two levels of stiffness. Responses obtained from reduced stiffness (cracked concrete) models with SSE damping are used to develop SSE loads for structural design of R/B and PS/B reinforced concrete structures since SSE loads create the maximum design stress condition. This approach is consistent with RG 1.61 Section 1.2 guidance. Models based on reduced stiffness (cracked concrete) properties produce the bounding structural displacements for use in seismic structural interaction analysis.~~ MIC-03-03-00054

~~In oder to capture the effects of concrete cracking on design of Category I and II SSC's, ISRS are developed using the envelope of responses obtained from SSI analyses of the models with the two bounding levels of stiffness.~~ MIC-03-03-00066

~~The damping values in Table 3.7.3-1(a) and Table 3.7.3-1(b) are applicable to all modes of vibration of a structure constructed of the same material.~~ DCD_03.07.02-35

~~The values of the SSE damping coefficients specified in Table 3.7.3-1(a) are based on the expectation that the response of the linear elastic structure attributed to load combinations that include the SSE is close to applicable stress limits. This is considered acceptable for the US-APWR standard plant seismic design where, as described in RG 1.61 (Reference 3.7-15) Section 1.2, the design-basis ISRS represent the envelope of the in-structure responses obtained from multiple analyses conducted to consider a range of expected site soil conditions. However, this does not apply for the site-specific seismic analysis that use site-specific site properties since it is possible that the predicted structural response to the load combinations that include an SSE is significantly below the stress limits. In these cases, the SSE values in Table 3.7.3-1(a) may overestimate the actual dissipation of energy in the linear dynamic system and, thus, result in a non-conservative estimate of the structural response for frequencies close to the resonant frequencies.~~ To prevent non-conservative results, the COL Applicant is to review the resulting level of seismic response and determine appropriate damping values for the site-specific calculations of ISRS that serve as input for the seismic analysis of seismic category I and seismic category II subsystems. ~~In accordance with Section 1.2 of RG~~ DCD_03.07.02-35

~~1.61 (Reference 3.7-15), no verifications of seismic response are required if the lower damping values listed in Table 3.7.3-1(b) are used as input for computation of ISRS. In accordance with RG 1.61 (Reference 3.7-15), the damping values in Table 3.7.3-1(b) are also intended for use in site-specific OBE analyses, if the site-specific OBE is higher than 1/3 of the site-specific SSE.~~ DCD_03.07.02-35

~~The damping values in Table 3.7.3-1(a) and Table 3.7.3-1(b) are applicable to all modes of vibration of a structure constructed of the same material.~~

The frequency domain SSI analyses of the R/B complex~~ and the PS/Bs~~, discussed in Section 3.7.2.4, use complex damping formulation to represent the dissipation of energy in the combined dynamic system due to material damping. The global stiffness and damping of the dynamic system are represented by a global complex matrix that is assembled from the element complex matrices modeling the stiffness and material damping of different subsystems and/or structural components. Damping values associated with site-specific SSI analyses are addressed in Subsection 3.7.2.4.1. MIC-03-03-00054

~~Generally, the~~The damping values for systems that include two or more substructures, such as a concrete and steel composite structure, ~~can~~may also be obtained using the strain energy method. ~~The strain energy dependent modal damping values are computed based on Reference 3.7-18, which~~This is the same as the stiffness weighted composite modal damping method~~, and acceptable~~ as provided in to SRP 3.7.2 (Reference 3.7-16).

The stiffness weighted modal damping ratio h_j of the j^{th} mode is obtained from the following equation:

$$h_j = \frac{\vec{\phi}_j^{\,T}[\overline{K}]\vec{\phi}_j}{\vec{\phi}_j^{\,T}[K]\vec{\phi}_j}$$

where

$[K]$ = the stiffness matrix of the combined soil-structure system

$\vec{\phi}_j$ = the j^{th} normalized mode shape vector

$[\overline{K}] = \sum [k_i] \cdot \xi_i$ = the modified stiffness matrix constructed from the products of the element stiffness matrices $[k_i]$ and the applicable damping ratio ξ_i

~~The frequency domain SSI analyses of the R/B complex and the PS/Bs, discussed in Section 3.7.2.4, use complex damping formulation to represent the dissipation of energy in the combined dynamic system due to material damping. The global stiffness and damping of the dynamic system are represented by a global complex matrix that is assembled from the element complex matrices modeling the stiffness and material damping of different subsystems and/or structural components. Damping values~~ DCD_03.07.02-35

~~associated with site-specific SSI analyses are addressed in Subsection 3.7.2.4.1.~~ DCD_03.07.02-35

3.7.1.3 Supporting Media for Seismic Category I Structures

A range of soil parameters of the basemat supporting media are considered in the seismic design of seismic category I building structures for the US-APWR standard plant. ~~The overall basemat dimensions, basemat embedment depths, and maximum height of the US-APWR R/B, PCCV, and containment internal structure on their common basemat are given in Table 3.7.1-3 and as updated by the COL Applicant to include site-specific seismic category I structures.~~The R/B complex is approximately 336 ft 4 in. in the north-south (NS) direction and 409 ft 8 in. in the east-west (EW) direction. The total footprint area is 127,016 ft^2. The nominal bottom elevation is -39 ft 8 in. Embedment depth is approximately 42 feet from grade which is at 2 ft 7 in. The basemat is nominally 13 ft 4 in. thick, however it is 30 ft 6 in. thick under the PCCV and 43 ft 3 in. thick under the CIS. See the figures in Section 1.2 for detailed elevation and plan views of the structure. MIC-03-03-00054

The ~~required~~minimum allowable static bearing capacity for ~~seismic category I building structure basemats, including the R/B-PCCV-containment internal structure on their common basemat,~~the R/B complex is 15 ksf. The minimum allowable dynamic bearing capacity for the R/B complex is 60 ksf. These values are developed in Subsection 3.8.5.4.1. The dynamic bearing loads for seismic category I structure basemats are dependent upon the magnitude of the seismic loads that can be obtained from a site-specific seismic analysis that considers FIRS. The COL Applicant is to determine the allowable static and dynamic bearing capacities based on site conditions, including the properties of fill concrete placed to provide a level surface for the bottom of foundation elevations,and to evaluate the bearing loads to these capacities. A minimum factor of safety of 2.5 is suggested for the ultimate bearing capacity versus the allowable static bearing capacity; however, a different value may be justified based on site-specific geotechnical conditions. A minimum factor of safety of 2 is suggested for the ultimate bearing capacity versus the allowable dynamic bearing capacity; however, a different value may be justified based on site-specific geotechnical conditions.

~~The range of soil parameters in the site-independent seismic design includes use of basemat supporting media that are comprised of generic layered soil profiles. The profiles have been selected from an exhaustive suite of soil profiles developed from a set of small strain generic profiles that cover the entire a reasonable range of generic site conditions from soft soil to firm rock that may exist across the central and eastern North Americacontinental US. The initial profile development recognized that for the softer conditions, the shallow materials would be either removed or improved for appropriate foundation conditions. From the exhaustive suite of profiles of measured properties at candidate sites, a final subset of eight generic profiles representing strata consisting of soil and soft, medium, and hard rock was selected.~~ MIC-03-03-00054 MIC-03-03-00066

~~The profiles are presented in Table 3.7.1-6. The generic profiles are classified using V_s (30m), the average shear wave velocity in the top 30 meters. This classification is consistent with current practice for building codes and is a convenient metric with which to distinguish the profiles, prior to soil removal or improvement for installation of the power block. StrainThe input for the SSI analyses are starain-compatible properties, consistent with the CSDRS input ground motion, including shear wave and compression~~ MIC-03-03-00066

~~wave velocities, corresponding hysteretic damping values, and Poisson ratios are developed for the profilesthat are developed from site response analyses of the selected subset of profiles and baserock depths. The dynamic soil/rock properties used as input for site-independent SSI analyses are compatible to the strains generated by the seismic ground motions for which the corresponding spectra at the ground surface are enveloped by US-APWR CSDRS. Note that for purposes of development of compression wave velocities, the profiles are analyzed in the saturated condition.~~ MIC-03-03-00054

~~The resulting generic layered supporting media provide a wide variation of properties to address potential ranges in dynamic soil properties.The generic layered profiles provide dynamic strain-compatible subgrade properties that capture a wide range of SSI responses expected at candidate sites across the continental US The development of the supporting soil media profiles are described further in Sections 4.2 and 5.2 of Technical Report MUAP-10001 (Reference 3.7-47).~~ MIC-03-03-00066

To select the soil profiles to use for design and analysis of the US-APWR, a database of soil profiles and depths to basement was evaluated as described in MUAP-10006. (Reference 3.7-48). Six small strain profiles were selected for the development of strain compatible properties. These six profiles include soft and hard soil profiles (nominal shear wave velocity of 270 m/s and 560 m/s respectively) and soft and firm rock profiles (nominal shear wave velocity of 900 m/s and 2,032 m/s, respectively). The development of softer soil strain compatible profiles considers additional soil removal if necessary to maintain a minimum strain compatible shear wave velocity of at least 800 ft/s near the plant grade surface.

There are two 270 m/s profiles where the top 68 ft of soil is replaced. These two profiles representing layers of dense cohesionless soil and/or over-consolidated stiff clay are considered with depths of 200 ft and 500 ft above rock foundations consisting of sedimentary or weathered rock section overlying Precambrian basement material.

The third soil profile considered is a 500 ft thick layer of stiff 560 m/s soil representative of glacial till sites consisting of highly consolidated mixtures of fine and course grained soils over 1000 ft deep strata of sedimentary or weathered rock resting on the rock basement.

Two soft rock profiles (900 m/s) are considered with depths of 100 ft and 200 ft.

The sixth profile is a firm rock profile with a nominal shear wave velocity of 2,032 m/s and depth of 100 ft is selected to represent a residual soil (saprolite) over weathered rock and underlain by hard rock. This profile is intended to reflect hard rock foundation depths after removal of the soft surficial residual soils.

The six generic layered profiles reflect range of realistic site conditions and provide a wide range of SSI responses to ensure the broad applicability of the design for the CEUS. The final soil profile categories are summarized in Table 3.7.1-6.

The small strain shear wave velocity (V_s) and compression wave velocity (V_p) are plotted in Figure 3.7.1-14 and Figure 3.7.1-15. The shear strain damping is plotted in Figure 3.7.1-16. The nomenclature for the final soil profiles gives both the shear wave velocity and the depth to bedrock. For example, soil profile 560-500 designates the soil with shear wave velocity of 560 m/s with a depth of 500 ft.

The generic profiles are representative of saturated soil properties and a water table located at the plant grade elevation. Generic soil 270-200, 270-500 and 560-500 profiles representative of unsaturated soil properties were developed and analyzed in Technical Report MUAP-11007 (Reference 3.7-52). MUAP-11007 concluded that the use of saturated soil profiles as a site independent analysis parameter results in a standard plant design that envelops the seismic demands at a majority of candidate sites within the CEUS. MIC-03-03-00054

The site-specific SSI analyses will use ~~site-dependent input control motion that is derived from GMRS and~~site-specific input soil/rock properties that are compatible to the site-specific ground motion compatible to site-specific FIRS discussed in Subsection 3.7.1.1. The primary non-linear material behavior of the soil must be considered and may be approximated by using equivalent linear material properties that are compatible to the free-field strains generated by the site-specific design ground motion. ~~If the earthquake-induced strains in the soil remain below 2%, t~~The strain-compatible soil properties are obtained from a 1-dimensional wave propagation analysis by using equivalent-linear methodology and site-specific soil stiffness and damping degradation curves. ~~The site-specific SSI analyses of the R/B–PCCV–containment internal structure on their common basemat uses the finite element (FE) analysis program ACS SASSI (Reference 3.7-17) that provides a frequency domain solution of the SSI model response by using a sub-structuring technique and, when applicable, is capable of addressing site-specific effects such as the properties and layering of the soil, as well as effects ofembedment and flexibility of the basemat, scattering, and incoherence of the input control motion. Based on successful comparison of ISRS derived from the CSDRS to those derived from site-specific SASSI analysis, other standard plant structures designed using lumped-parameter models with lumped SSI parameters subject to the CSDRS can be validated by direct comparison to demonstrate their site-specific FIRS are enveloped. A SASSI analysis can be performed to consider incoherency to reduce high frequency response.~~ MIC-03-03-00066 MIC-03-03-00054 MIC-03-03-00066

The site-independent SSI analyses include the subgrade as horizontally infinite layers resting on the surface of an elasto-viscous half-space, representing the stiffness, material, and ~~radiation~~ damping of geological hard rock stratum with shear wave velocity equal or greater than 9,200 ft/s. The soil material damping values used in conjunction with the shear and compression wave profiles in the SSI analysis models are identical. The seismic models used in the SSI analyses are discussed further in Section 3.7.2.3. The site-independent SSI analyses are discussed further in Section 3.7.2.4, as well as the suggested methodologies for analyzing the effect of site-specific conditions on the SSI response. MIC-03-03-00066

Site response analyses using the equivalent linear Random Vibration Theory (RVT) approach described in MUAP-10006 (Reference 3.7-48) are performed to develop the CSDRS strain-compatible soil properties used as input for the SSI analyses. The site response analyses to develop the strain-compatible properties use the point-source model to generate both the input horizontal and vertical motions. A Magnitude **M**7.5 earthquake is used since its broad spectral shape is consistent with that of the CSDRS. A smaller magnitude would result in higher short period motions and higher strains. Distances to the **M**7.5 control earthquake are adjusted such that the median spectrum as full column outcrop spectrum at foundation level computed for each profile approaches, but does not exceed, the horizontal and vertical CSDRS. The distances and median MIC-03-03-00054

estimates of the horizontal and vertical peak accelerations are listed in Table 3.7.1-7 and the median spectrum computed for each profile is compared to the CSDRS as described in MUAP-10006 (Reference 3.7-48). MIC-03-03-00054

3.7.2 Seismic System Analysis

Seismic system analysis is discussed in the following Subsections, 3.7.2.1 through 3.7.2.15. Following the guidance of the acceptance criteria in section II.3(a) of SRP 3.7.2 (Reference 3.7-16), two categories of seismic category I SSCs are defined: (1) seismic systems that include major seismic category I buildings and structures that are analyzed in conjunction with their basemats and supporting media (subgrade); and (2) seismic subsystems that include other seismic category I SSCs that are not analyzed in conjunction with basemats and subgrade. ~~This subsection discusses the following major seismic category I and II buildings and structures that are classified as seismic systems requiring SSI analysis:~~The details of the seismic system analysis is provided in Technical Report MUAP-10006 (Reference 3.7-48). MIC-03-03-00054 MIC-03-03-00054

- ~~R/B complex consisting of the R/B, PCCV containment internal structure on their common basemat (seismic category I)~~ DCD_03.07.02-35
- ~~East and west PS/Bs (seismic category I)~~
- ~~A/B (seismic category II)~~
- ~~T/B (seismic category II)~~

All standard plant seismic category I structures are part of the R/B complex. The R/B complex consists of the R/B, PCCV, CIS, East PS/B, West PS/B, ESWPC, and the seismic category II A/B all on a common basemat. MIC-03-03-00054

The T/B, consisting of the Turbine Building and the Electrical Room on their common basemat, has been classified as a seismic category II structure, and is in close proximity to the R/B complex. The T/B is analyzed in the same manner as the R/B complex as described in MUAP-11002 (Reference 3.7-61). The structure-soil-structure interaction (SSSI) effect of the T/B on the R/B complex is discussed in Section 3.7.2.4, and the potential for physical interaction with the R/B complex is discussed in Section 3.7.2.8.

The seismic responses of the major seismic category I and seismic category II structures are ~~required to be~~ obtained from frequency domain time history analysis of seismic models considering a frequency-dependent SSI system.~~, including a set of eight generic layered soil profiles representing~~ These site-independent analyses are performed with the set of generic layered soil profiles described in Section 3.7.1.3, which represent a wide range of site conditions. Subsections 3.7.2.1 and 3.7.2.3, respectively, describe the analysis and modeling methods used for the seismic analyses, and Subsection 3.7.2.2 discusses the natural frequencies and results obtained from the seismic analyses. To address effects of concrete cracking on the standard seismic design, seismic responses obtained from SSI analyses of models with two bounding levels of stiffness and damping are considered as discussed in Subsection 3.7.2.3.5. ~~The results from the seismic analyses serve as the basis for the development of equivalent static seismic loads that~~ MIC-03-03-00054 MIC-03-03-00054 DCD_03.07.02-35

~~are applied in conjunction with other design loads on the detailed three-dimensional shell FE model in order to obtain the design stresses in the structural members and components.~~ MIC-03-03-00054

~~The results obtained from the site-independent SSI analyses for maximum nodal accelerations serve as basis for development of the seismic loads used for the standard design of the R/B complex and the PS/B structural members. The overall process for development of seismic loading for the R/B complex (excluding the containment internal structure) and the PS/B is summarized as follows:~~

1. ~~Obtain maximum accelerations in each of the 3 response directions due to each of the 3 direction input motions from the SSI analyses of R/B complex and PS/B models.~~

2. ~~Apply the square root sum of the squares (SRSS) rule to combine the nodal maximum accelerations due to the three directions of the earthquake calculated in Step 1. Envelope the results of the SRSS combined maximum acceleration responses from the SSI analyses of the eight generic soil conditions and the two structural stiffness levels.~~

3. ~~For each floor elevation, record the enveloped maximum response accelerations in the three directions calculated in Step 2.~~

4. ~~Calculate weighted average accelerations at floor elevation "f" as follows:~~

$$\cancel{A_i^f = \frac{\sum a_i^k \cdot w_k}{W_f}}$$

~~Where~~

~~W_f = total weight of the floor "f"~~

~~W_k = weight paticipating to node "k"~~

~~A_i^f = weighted average acceleration of floor "f" in the "i" direction~~

~~a_i^k = eneveloped and SRSS combined maximum acceleration at node "k" in the "i" direction as calculated in Step 3 above.~~

5. ~~Determine equivalent seismic quasi-static load at each floor elevation based on the maximum acceleration determined in Step 3 and the weighted average accelerations calculated in Step 4.~~ DCD_03.07.02-35

6. ~~Determine the out-of-plane seismic demands on slabs and walls with large unsupported areas based on the maximum acceleration plots created in Steps 3 and 5.~~

~~The overall process for development of seismic loading is summarized for the R/B complex as follows:~~ DCD_03.07.02-35

1. ~~Develop shear forces, axial force, bending moments and torsional moment diagrams from the results of SSI analyses of each generic soil profile for maximum member forces and moments in the stick elements.~~

2. ~~Develop equivalent horizontal loads F_{Hi} in NS and EW direction from the shear force diagrams for each floor elevation.~~

3. ~~Develop equivalent vertical loads F_{Vi} from the axial force diagram for each floor elevation.~~

4. ~~Develop equivalent floor rocking moment M_{Ri} in NS and EW direction from the bending moments diagrams for each floor elevation.~~

5. ~~Develop equivalent floor torsional moment M_{Ti} from the torsional moments diagrams for each floor elevation.~~

6. ~~Use the SRSS method to combine the seismic demands that are due to three components of the earthquake.~~

7. ~~Envelope the SRSS seismic demands from the SSI analyses of the eight generic soil profiles.~~

8. ~~Adjust the magnitude of the vertical load F_V to include the effect of the floor rocking using the following equation:~~

$$\cancel{F'_V = [(F_v)^2 + (M_{NS}/L_{NS})^2 + (M_{EW}/L_{EW})^2]^{1/2}}$$

~~where M_{NS} and M_{EW} are the floor rocking moments in NS and EW direction and the L_{NS} and L_{EW} are the NS and EW length of the floor. Note that this increase is applied to the vertical design SSE loads for the detailed finite element model in calculating demands on structural members.~~

9. ~~Adjust the magnitude of the horizontal floor loads F_H to include the effect of the floor torsional response M_T as shown in the equations below.~~

$$\cancel{F'_{NS} = F_{NS} + \frac{M_T}{L_{EW}} \qquad F'_{EW} = F_{EW} + \frac{M_T}{L_{NS}}}$$

~~The design also includes the accidental torsion discussed in Subsection 3.7.2.11.~~

10. ~~Conservative story seismic loads are selected based on F'_V, F'_{NS}, and F'_{EW}.~~

11. ~~Determine the out-of-plane seismic loads on flexible slabs and walls based on results for maximum accelerations of single degree of freedom oscillators.~~

~~The overall process for development of seismic loading is summarized for the PS/Bs as follows:~~ DCD_03.07.02-35

~~The results obtained from the site-independent SSI analyses for maximum nodal accelerations serve as basis for development of the seismic loads used for the standard design of PS/B structural members. These seismic design loads are developed in the form of equivalent static accelerations as follows:~~

1. ~~Calculate maximum accelerations in each of the 3 response directions due to each of the 3 direction input motions from the site-independent SSI analyses of PS/B.~~

2. ~~Envelope the maximum nodal accelerations results obtained from the site-independent SSI analyses of eight generic site conditions.~~

3. ~~Create nine (9) plots of the envelope maximum accelerations in three directions representing the response of the PS/B at particular floor elevation due to the three components of the earthquake.~~

4. ~~Apply the SRSS rule to combine the enveloped nodal maximum accelerations due to the three directions of the earthquake and plot the SRSS of the maximum acceleration responses in the three directions.~~

5. ~~Calculate weighted average accelerations at floor elevation “f” as follows:~~

$$A_i^f = \frac{\sum a_i^k \cdot w_k}{W_f}$$

~~where W_f is the total weight of the floor “f”, W_k is the weight participating to node “k”, A_i^f is the weighted average acceleration of floor “f” in “i” direction, and a_i^k is the maximum SRSS acceleration at node “k” in “i” direction.~~

6. ~~Determine equivalent seismic demand at each floor elevation based on the maximum acceleration plots and the weighted average accelerations obtained from items 3, 4 and 5. The design also includes the accidental torsion discussed in Subsection 3.7.2.11.~~

7. ~~Determine the out of plane seismic demands on slabs and walls with large unsupported areas based on the plots of the maximum acceleration plots obtained in step 4.~~

7. ~~Calculate story shear diagram for the R/B complex and the PS/B structure. The total shear at each floor elevation is obtained as the sum of the all nodal inertial forces at and above that elevation. The nodal inertial forces are the product of the nodal mass and the equivalent static acceleration acting on the node.~~ MIC-03-03-00054

~~The overall process for development of the seismic loading for the containment internal structures is as follows:~~

1. ~~Develop acceleration response spectra at the base of the containment internal structure. Base response spectra are generated for the containment internal structure in accordance with US NRC RG 1.122 (Reference 3.7-26). The ACS SASSI analyses provide results for the response of the containment internal structure due to the three components of the design earthquake for each of the eight soil profiles. Refer to Subsection 3.7.2.4 for discussion on the methodology used to consider SSI. At representative base node locations, 5% damping ARS in the three orthogonal directions are calculated for each of the three directions of the input ground motion. The ARS are calculated at frequency points equally distributed on the logarithmic scale at the range of frequency from 0.1 Hz to 100 Hz. The responses obtained for the three directions of the input ground motion are combined using the SRSS method as follows:~~ MIC-03-03-00054

$$\cancel{ARS_x = \sqrt{ARS_{xx}^2 + ARS_{yx}^2 + ARS_{zx}^2}}$$

$$\cancel{ARS_y = \sqrt{ARS_{xy}^2 + ARS_{yy}^2 + ARS_{zy}^2}}$$

$$\cancel{ARS_z = \sqrt{ARS_{xz}^2 + ARS_{yz}^2 + ARS_{zz}^2}}$$

~~where~~

~~*ARS_{mn}* =~~ ~~the ACS SASSI ARS results for the response in "*n*" direction due to earthqauke in "*m*" direction~~

~~*ARSx, ARSy and ARSz* =~~ ~~combined ARS of the structural response in NS (x), EW (y), and vertical (z) direction.~~

~~The results for SRSS-combined ARS at each representative node location obtained from the results of the ACS SASSI analyses of each of the eight generic soil conditions (presented in Section 3.2) are then enveloped. The enveloped ARS are then combined as described below to obtain the combined ARS for the response at different locations within the R/B complex and PS/B. The ISRS are developed by broadening the combined ARS by 15% to account for uncertainties in the analysis parameters.~~

2. ~~Two sets of response spectra are created as described in step one, one for cracked and one for the uncracked condition.~~

3. ~~Containment internal structure seismic response spectra loads are compared to ACS SASSI results for confirmation of the response analysis.~~

~~Discussed in~~ Subsection 3.7.3 ~~are~~discusses the seismic analyses applicable to seismic category I civil structure subsystems housed within or supported by the major seismic category I structures. Seismic and dynamic qualification of mechanical and electrical equipment and subsystems performed by testing is discussed in Section 3.10 and

Appendix 3D. Mechanical subsystems include mechanical equipment, piping, vessels, tanks, heat exchangers, valves, and instrumentation tubing and tubing supports. The seismic analysis of mechanical subsystems is addressed in Sections 3.9 and 3.12. The mass inertia properties of the major civil structural, mechanical, and all other seismic subsystems are ~~accounted for~~addressed in the seismic system analyses, as explained further in Subsection 3.7.2.3. MIC-03-03-00054

3.7.2.1 Seismic Analysis Methods

The methods used for the seismic analysis of the US-APWR seismic category I systems conform to the requirements of SRP Subsections 3.7.1 (Reference 3.7-10) and 3.7.2 (Reference 3.7-16). Table 3.7.2-1, as updated by the COL Applicant to include site-specific seismic category I structures, presents a summary of dynamic analysis and combination techniques including types of models and computer programs used, seismic analysis methods, and method of combination for the three directional components for the seismic analysis of the US-APWR standard plant seismic category I buildings and structures. MIC-03-03-00054

~~The seismic response of standard plant seismic category I and II structures is obtained from site-independent analyses performed using three-dimensional SSI models with the program ACS SASSI (Reference 3.7-17) which utilizes time history analysis in the frequency domain with the sub-structuring technique and complex stiffness representation of stiffness and damping properties of the structures and subgrade. The global complex stiffness matrix of the structure is assembled from the stiffness matrices of the finite elements. Table 3.7.2-3 provides stiffness and damping values for seismic Category I structures for both uncracked and cracked conditions. With the sub-structuring technique, impedance, load vector, and complex dynamic stiffness matrices are developed separately for the structure and the soil. The input ground motion is transformed into the frequency domain using Fast Fourier Transformation. The equations of motion for the complex SSI system are then developed by combining the equations of motion for the structure with those of the soil in the frequency domain. The seismic response is obtained in the frequency domain from solution of complex algebraic equations for a selected set of frequencies of analysis. The solutions obtained for the selected set of frequencies of analysis are then interpolated and transformed into the time domain using Inverse Fast Fourier Transformation. The entire process is described in Section 2 of the user's guide for ACS SASSI (Reference 3.7-17).~~ MIC-03-03-00054 DCD_03.07.02-35

~~As described above, the ACS SASSI program (Reference 3.7-17) analysis method requires solution in the frequency domain of the coupled equations of motion that represent the SSI system. As an alternative option for seismic category I systems and subsystems, it is also acceptable to utilize the composite modal damping method associated with the modal superposition of time history analysis when the equations of motion can be decoupled in accordance with SRP 3.7.2 (Reference 3.7-16), Section II.13.~~ DCD_03.07.02-108 (9)

~~Analyses of seismic category I and II subsystems are primarily performed using equivalent static load analysis or modal response spectra analysis. The input seismic loads are defined by ISRS that are obtained from the time history analyses of the major seismic category I buildings and structures. 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.~~ MIC-03-03-00066

~~Seismic anchor motions are taken into consideration for all seismic analysis methods used in the design of seismic category I and seismic category II SSCs. All analysis approaches have been based on linear elastic analysis of SSCs, with allowable stresses within the elastic limits for seismic loads and load combinations as delineated in Section 3.8. Except in limited cases where permitted by code, inelastic behavior is not considered for seismic loads and load combinations in performing the plant design, however, limited inelastic and nonlinear behavior for seismic loading conditions may be used for site-specific COL designs, future operability analyses or as-built evaluations, as permitted in SRP 3.7.2 (Reference 3.7-16). Nonlinear and inelastic behavior is considered for certain loads and load combinations involving impact and impulsive loading, as discussed in Subsection 3.8.4.~~ MIC-03-03-00054

SSI & SSSI Analyses Approach

The ACS SASSI computer program (Reference 3.7-17) is used for the SSI and SSSI analyses. The program employs the complex response method and FE technique to solve for the seismic response of the soil structure system in the frequency domain. The response is calculated at selected frequencies and then interpolated for the range of frequencies of interest. The Fast Fourier Transformation (FFT) and inverse FFT technique are used to transform the input motion and the nodal responses of the system between frequency and time domains. The following are the guidelines regarding the application of SASSI in the SSI and SSSI analyses.

Number of Points in FFT

The number of FFT points is set to 8,192 (or 2^{13}) for the R/B complex model. This number is acceptable since the input excitation duration is about 22 seconds (4,417 time steps of 0.005 seconds) in addition to a quiet zone for free vibration of 20 seconds. This quiet zone ensures that the structure will be at rest after the entire 42 second duration.

Three Directional Seismic Excitations

The three components of the earthquake are applied to the model separately and the solutions are superimposed to provide the solution for combined S- and P-wave excitations to all nodes. The vertically propagating S-waves represent the two horizontal components of the design earthquake motion H1 and H2 that are applied in NS and EW direction, respectively. The vertical component of the design earthquake (V) is represented by vertically propagating P-waves. Seismic input motions are considered in both the SSI and SSSI analyses. The SSI and SSSI analyses use within motions at the bottom of the R/B complex as control motions. Refer to Section 3.7.2.4 for details.

Modified Subtraction Method

There are several modeling approaches that can be used with SASSI, the Flexible Volume Method (also known as the Direct Method), the Subtraction Method, and the Modified Subtraction Method (MSM). Each method has different computational demands.

The R/B complex SSI and SSSI analysis use an improved subtraction method, the MSM. In addition to the interaction nodes at the interface of the excavated soil volume and surrounding free field soil interface defined in the Subtraction Method, MSM includes the

nodes at the ground surface of the excavated soil as interaction nodes. By adding interaction nodes at the ground surface,the scattered surface waves that occur at the ground surface in the mid and high frequency range are captured much more accurately than in the Subtraction Method. In addition, the inclusion of the ground surface nodes as interaction nodes provides significantly improved boundary conditions for simulating the excavated soil dynamic behavior. The scattered waves manifest primarily by Rayleigh and Love surface waves. MIC-03-03-00054

A review of ATFs at various locations throughout the R/B complex indicates that the MSM model reasonably captures the seismic responses for the analyzed frequency. MSM is an accurate and robust method for the R/B complex, which has a large footprint shallow embedded foundation for the frequency range analyzed. Refer to MUAP-10006 (Reference 3.7-48) for additional discussion on the use of the MSM.

Cut-off Frequency of the Analyses

The cut-off frequency is the highest frequency used in the dynamic analysis of the soil structuresystem. It sets an upper limit on the number of frequencies to be analyzed, and controls the maximum allowable element size which in turn controls the size of the problem to be solved. The maximum frequency of analysis is determined from the wave passage frequency (f_{pass}) of the soil layer and soil element size. The wave passage frequency is the maximum wave frequency that the soil layer can accurately transmit. It is determined using the Equation below (Reference 3.7-17):

$$f_{pass} = \frac{V_s}{5 \cdot d}$$

where V_s is the shear wave velocity of the soil and d is either the thickness of the soil layer or the maximum size of the FE mesh of the structural model at the SSI interface or the excavated soil volume mesh size.

Based on the calculated wave passage frequencies for each generic soil profile, the cut-off frequencies in the analyses are set to 40 Hz for 270-200 and 270-500 soil profiles, and 50 Hz for 560-500, 900-100, 900-200 and 2032-100 soil profiles.

Based on the maximum frequencies and intervals of frequency points, for both SSI and SSSI analysis, a total of 132 frequencies are analyzed for soil profiles 270-200 and 270-500, and a total of 152 frequencies are analyzed for soil profiles 560-500, 900-100, 900-200 and 2032-100.

3.7.2.2 Natural Frequencies and Responses

The seismic analysis of the R/B complex is based on a seismic model of the entire structure.~~, PCCV, and containment internal structure and their common basemat is based on a seismic model consisting of three lumped mass stickFE models (one for each of the three structures) that are all integrated with a three-dimensional finite element model of the R/B basement~~. The ~~lumped mass stick~~FE model of the containment internal structure is coupled with the lumped mass stick model of the RCL, ~~that~~which represents~~ models representing~~ the dynamic properties of the reactor vessel, ~~reactor coolant loop,~~ and other MIC-03-03-00054 DCD_03.07.02-35 MIC-03-03-00054

major equipment and piping. The dynamic model is discussed further ~~below~~ in Subsection 3.7.2.3 and in detail in Technical Report MUAP-1000~~1~~6 (Reference 3.7-4~~7~~8). ~~The lumped mass stick models of R/B, PCCV, and containment internal structure are coupled with a detailed RCL lumped mass stick model.~~ MIC-03-03-00054 DCD_03.07.02-35

The seismic model analyses results are obtained considering the potential effects of SSI. The site-independent SSI analyses, which are discussed further in Section 3.7.2.4, consider generic subgrade conditions previously described in Section 3.7.1.3. For each one of the subgrade conditions considered, the analyses are performed with the ACS SASSI program (Reference 3.7-17) where each one of the three directional components of the earthquake is applied separately to the model. The member and element forces results obtained from these analyses are then enveloped and used for development of seismic loads for design of structural members as previously described.

Subsection 3.7.2.5 discusses development of ISRS based on the results of the site-independent seismic analyses for the US-APWR standard plant.

3.7.2.3 Procedures Used for Analytical Modeling

3.7.2.3.1 General Discussion of Analytical Models

~~The procedures used for analytical modeling of the major standard plant seismic category I and seismic category II structures are discussed herein.~~ MIC-03-03-00054

The procedures used for development of analytical models for seismic analysis are consistent with the procedures and guidelines of SRP 3.7.2, Section II.3 (Reference 3.7-16). Structural element mass and stiffness characteristics, as well as load and tributary masses, and damping characteristics, are incorporated into the models.

~~The mass inertia and stiffness properties of the basemat and basement exterior walls of the R/B, and the entire PS/Bs, are characterized through the use of finite element models. Lumped mass stick models, interconnected by stiffness elements, are used to discretize the mass inertia and stiffness properties of the remaining portions of the R/B, the PCCV, and containment internal structure. The mass is lumped in selected nodes in a way that provides an adequate representation of the mass distribution considering the high-stress concentration points of the system. In general, lumped mass inertia is assigned at the selected locations in all six DOF corresponding to translations along three orthogonal axes, and rotations about these axes. The number of DOF should be reduced by the number of constraints, where applicable.Integrated dynamic FE models of the R/B complex, PS/Bs and A/B are developed and validated using ANSYS (Reference 3.7-21) and then translated into an ACS SASSI (Reference 3.7-17) format. Refer to Technical Report MUAP-10001 (Reference 3.7-47) for the detailed approach taken for development of the dynamic FE models for the R/B, PCCV and containment internal structures as well as the PS/Bs. Similarly, refer to Technical Report MUAP-11001 (Reference 3.7-50) for detailed approach taken for development of the dynamic FE models for the A/B. The following general steps are followed for the dynamic FE model of the R/B complex:~~ DCD_03.07.02-35 MIC-03-03-00054

~~Step 1: Develop the R/B Complex Dynamic FE Model.~~

~~ANSYS preprocessor and ANSYS Parametric Design Language are used to develop an integrated 3-D FE model that includes the R/B, PCCV, and containment internal structure that is coupled with the lumped-mass-stick model representing the dynamic properties of the RCL. The numbering of the nodes is adjusted following the guidelines of the ACS SASSI manual in order to optimize the computational effort.~~ MIC-03-03-00054

~~Step 2:~~ ~~Validate the R/B complex Dynamic FE Model to ensure that the model adequately captures the dynamic behavior of the structures.~~

~~For validation purposes, the dynamic FE model is separated into three parts: R/B-FH/A (including the common basemat), PCCV, and containment internal structure. Static and dynamic analyses using ANSYS solvers are performed on each of the three components of the dynamic FE model by establishing fixed boundary conditions at the base of each structure, respectively. An identical set of fixed base analyses are also performed on the detailed FE models of the R/B, PCCV, and containment internal structure. The results obtained from the dynamic FE models and detailed FE models are compared to demonstrate the ability of the less refined dynamic FE model to adequately capture the dynamic behavior of the corresponding detailed FE models.~~

~~Step 3:~~ ~~Translate the Dynamic FE Model into ACS SASSI format and verify the accuracy of the translation.~~

~~The translator built into the ACS SASSI code serves as the platform for the translation of the dynamic FE model from ANSYS to ACS SASSI house module format. In order to validate the translation of the model, validation SSI analyses are performed on the ACS SASSI dynamic FE model resting on a very rigid elastic half space. The dynamic properties of the model revealed by the resulting amplification transfer functions and 5% or 7% damping ARS at selected locations are compared to the fixed base dynamic properties and responses obtained from ANSYS modal and time history analyses to ensure the translation is competed correctly.~~

~~The R/B complex dynamic FE model consists of beam, shell, solid, and spring elements. The use of FEs provides an accurate representation of the dynamic properties of the structures and the foundation that enables an accurate modeling of dynamic interaction with the flexible foundation and the surrounding soil. Shell elements are used to model the reinforced concrete shear walls and slabs. 3-D beam elements model the reinforced concrete or steel columns and beams. Solid elements are used to model the basemat foundation and the massive structural members of the containment internal structure. Spring and beam elements are used to model the supports and connection of the RCL lumped-mass-stick model with the containment internal structure FE mesh. The ANSYS FE types used in the model are compatible with the ACS SASSI built in converter.~~

~~The development of the model provides that the connection between two different FE types transfers forces and/or moments from one structural component to the other. The nodes of the ACS SASSI solid elements have only three translational DOF and can therefore not transfer the moments from shell or beam elements. In order to enable the~~

~~transfer of bending moments from the walls modeled by shell elements to the basemat and containment internal structure massive structural members modeled by solid elements, the shell elements are extended into the solid elements so that the moments can be transferred to at least 2 layers of nodes belonging to the solid elements.~~ MIC-03-03-00054

~~In addition, each node of the ACS SASSI shell elements has five DOF that enable beam elements to transfer forces and bending moments to shell elements but not torsional moments. Therefore, massless beam elements are generated on the surface of the shell or solid elements in order to provide adequate transfer of moments from beams in all three rotational DOF. For beams or columns connecting to slabs or walls in the R/B model, the effect of adjusting the stiffness of the slab and wall shell elements is evaluated and the impact on the results is found to be negligible.~~

The Dynamic FE model of the R/B complex is developed and validated using ANSYS (Reference 3.7-21) and then translated into SASSI (Reference 3.7-17) format. The dynamic model is a simplified, coarsely meshed model that is validated against a more refined, detailed model. The translation process is described in the following steps:

Step 1: Develop the R/B Complex Dynamic FE Model

ANSYS Workbench and ANSYS Parametric Design Language (APDL) are used to develop an integrated 3-D FE model that includes the R/B, PCCV, CIS, A/B, East and West PS/Bs, and ESWPC coupled with the model representing the dynamic properties of the RCL. The numbering of the nodes is adjusted following the guidelines of the SASSI manual in order to optimize the computational effort.

Step 2: Validate the R/B Complex Dynamic FE Model to ensure that the model adequately captures the dynamic behavior of the structures

The Dynamic FE model is separated into six parts for the purpose of validation: R/B-FH/A, PCCV, CIS (with RCL), A/B, East PS/B, and West PS/B. The ESWPC is split and included in the R/B-FH/A, East PS/B, and West PS/B models. Static, modal, harmonic response, and stiffness analyses using ANSYS solvers are performed on each of the six parts of the dynamic model by establishing fixed boundary conditions at the base of each structure. An identical set of fixed base analyses are also performed on detailed FE models of each structure. The results obtained from the dynamic and detailed models are compared to demonstrate the ability of the less refined dynamic models to adequately capture the dynamic behavior of the corresponding detailed models. After all six parts are validated independently; the same process is used to validate the combined Dynamic FE model.

Step 3: Translate the Dynamic FE Model into SASSI format and verify the accuracy of the translation

The translator built into the SASSI code serves as the platform for the translation of the Dynamic FE model from ANSYS to SASSI house module format. In order to validate the translation of the model, a validation SSI

analysis is performed on the SASSI Dynamic FE model resting on a very stiff elastic half space. The dynamic properties of the model, revealed by the resulting ATFs at selected locations, are compared to the fixed base dynamic properties and responses obtained from ANSYS modal analyses to ensure the translation is completed correctly. MIC-03-03-00054

The R/B complex Dynamic FE model consists of beam, shell, solid, and spring elements. The use of finite elements provides an accurate representation of the dynamic properties of the structures and the foundation that enables an accurate modeling of dynamic interaction with the flexible foundation and the surrounding soil. Shell elements are used to model the reinforced concrete shear walls and slabs. Three-dimensional (3-D) beam elements model the reinforced concrete or steel columns and beams. Solid elements are used to model the basemat foundation and the massive structural members of the CIS. Spring and beam elements are used to model the supports and connection of the RCL model with the CIS mesh. The finite element types used in the ANSYS model are compatible with the SASSI built in converter.

The Dynamic FE model is presented in Figure 3.7.2-1. This model has a total of 33,564 nodes, 47,580 elements and has an average mesh size of approximately 9 ft. The Dynamic FE model is based on the Detailed FE model presented in Figure 3.7.2-2. The Detailed FE model has a total of 62,252 nodes, 74,961 elements, with an average mesh size of approximately 5 ft. Figure 3.7.2-3 and Figure 3.7.2-4 present the detailed PCCV and CIS finite element models, respectively.

The development of the model ensures that the connection between two different element types is such that an adequate transfer of forces and/or moments from one structural component to the other is enabled. The nodes of the solid elements have only three translational degrees of freedom and can therefore not transfer the moments from shell or beam elements. In order to enable the transfer of bending moments from the walls modeled by shell elements to the basemat and massive concrete sections of the CIS modeled by solid elements, the shell elements are extended into or overlaid on the solid elements. A special layer of transitional rigid shell elements is created between the CIS reactor cavity top flange solid elements and the adjacent surrounding SC walls.

In addition, each node of the SASSI shell elements has five degrees of freedom that enable beam elements to transfer forces and bending moments to shell elements but not torsional moments. Therefore, massless beam elements are generated on the surface of the shell or solid elements to provide adequate transfer of moments from beams in all three rotational degrees of freedom. For beams or columns connecting to slabs or walls in the R/B model, the effect of adding torsional stiffness to the slab and wall shell elements is evaluated and the impact on the results is found to be negligible.

Refer to MUAP-10006 (Reference 3.7-48) for additional discussion on the development of the Dynamic FE model.

When the subsystem analysis is performed, reduced degrees of freedom (DOF)~~s~~ can be used to represent the dynamic behavior at locations needed for equipment qualification, provided that they can provide an adequate and conservative prediction of the response of the equipment.

The seismic analyses of the US-APWR standard plant are performed on three-dimensional seismic models representing seismic category I and seismic category II structures. The basic dimensions of these buildings and structures as considered in the seismic analyses are presented in the general arrangement drawings in Section 1.2. ~~The models consider all six DOF (three rotational and three translational) and incorporate mass and stiffness eccentricities to assure that torsional and rocking/swaying effects, and any cross-directional coupling, are captured.~~The 3-D FE models have an adequate number of discrete mass DOF to capture the global and local translational, rocking, and torsional responses of the structures. Torsional and rocking/swaying effects are also captured at the basemat/subgrade interface by taking into account SSI, including effects related to the flexibility of the basemat foundation. The SSI analyses are addressed in Subsection 3.7.2.4. DCD_03.07.02-35

It is the responsibility of the COL Applicant to develop analytical models appropriate for the seismic analysis of buildings and structures that are designed on a site-specific basis including, but not limited to, the following:

- PSFSVs (seismic category I)
- ESWPT (seismic category I)
- UHSRS (seismic category I)

~~Since there will not be any seismic category I SSCs contained within seismic category II buildings, the development of ISRS is not necessary.Technical Report MUAP-11001 (Reference 3.7-50) documents the seismic evaluation of the MHI US-APWR Standard Plant A/B, which is classified as a seismic Category II building. This report presents the development and validation of structural models and results of the seismic response analyses of the A/B.~~ DCD_03.07.02-35 MIC-03-03-00054

~~The site-independent SSI analyses provide the seismic response of the A/B in terms of maximum seismic displacements relative to free field motion, absolute accelerations, ISRS at the top of the basemat, and base force and moment reactions. Member stress demands calculated from the static and response spectrum analyses are used for evaluation of the structural integrity of the A/B under load combinations associated with seismic design loads. The structural seismic demands obtained from the response spectrum analyses are compared to seismic demands computed from the SSI analysis to confirm the adequacy of results of the seismic structural evaluation of the A/B. The results of the SSI analyses for base force and moment reactions serve as input for the evaluation of seismic stability of the building.~~

~~Seismic evaluation of the A/B is based on two types of seismic response analyses: (1) SASSI SSI analyses that provide seismic response for generic subgrade conditions (described in Technical Report MUAP-10001, Reference 3.7-47); and (2) ANSYS response spectrum analyses that provide seismic demands on A/B structural members. The CSDRS and CSDRS compatible time histories developed in Section 5.2 of Technical Report MUAP-10001 define the ground motion for the standard design of the A/B. In addition to serving as input for the seismic stability evaluation of the structure, maximum~~

~~relative displacements computed in the A/B SSI analyses are used for evaluation of the adequacy of the gap between the A/B and adjacent structures.~~ MIC-03-03-00054

~~Using the computer program ANSYS (Reference 3.7-21), detailed FE models are developed for the major seismic category I and seismic category II structures, primarily to be utilized as static analysis models for structural design based on loads and load combinations as described in Section 3.8. However, the ANSYS FE models are also used for validation of the dynamic lumped mass stick models and the seismic analysis results, as discussed later in this section.~~

3.7.2.3.2 R/B Complex~~, PCCV, and Containment Internal Structure Lumped Mass Stick~~Dynamic Finite Element Model~~s~~

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~~Technical Report MUAP-10001 (Reference 3.7-47) presents the approach taken for development of the R/B, PCCV and containment internal structures dynamic FE models. Built using the ANSYS preprocessor, the R/B complex dynamic FE model is an integrated 3-D model of the R/B-FH/A, PCCV and containment internal structure resting on top of a common 9'-11" thick basemat. Typical element size in the basemat and the slabs is about 9 ft, while the element size for the walls in the vertical direction is approximately 1.5 times larger. Figures 3.7.2-4 and 3.7.2-5 show an overview of the R/B complex dynamic FE model, while Figures 3.7.2-6 and 3.7.2-7 reveal the interior structures with section views. Figure 3.7.2-8 through Figure 3.7.2-10 show the PCCV and containment internal structure (including the RCL). Note that the global origin is located at the center of the PCCV and top of the basement with X pointing North, Y pointing West, and Z pointing upward.~~

~~The R/B complex dynamic FE model is developed incrementally using the ANSYS postprocessor in the following seven steps:~~

~~Step 1:~~ ~~R/B complex structures geometry is created in a manner that allows control of the model FE mesh.~~

~~Step 2:~~ ~~Attributes are assigned and additional mass are applied on each of the structures.~~

~~Step 3:~~ ~~Mesh controls are set and the model excluding the RCL is meshed.~~

~~Step 4:~~ ~~Modifications are implemented as needed to make the model more consistent with the detailed FE model.~~

~~Step 5:~~ ~~The nodes are renumbered sequentially in the order of their X, Y, and Z coordinates as recommended by ACS SASSI Manual in order to enhance computational efficiency.~~

~~Step 6:~~ ~~The lumped mass stick model used for ANSYS analyses of RCL is translated into format that can be translated into ACS SASSI.~~

~~Step 7: RCL structure is added to the FE model of R/B complex structures and proper connections are implemented to represent the physical supports attached to the containment internal structure.~~ MIC-03-03-00054

~~In order to accurately capture the effects of foundation geometry for SSI, the basemat footprint in the dynamic FE model is extended to the external face of perimeter walls instead of their centerline. For simplicity without compromising accuracy, slab elevations in the dynamic FE model are slightly shifted upward or downward such that the middle planes of nearby upper or lower slabs fall into a major common horizontal plane. Also, only large openings in slabs and walls are included in the model. Wall properties are adjusted to account for the presence of small openings not included in the model.~~

~~Special attention is given in the dynamic FE model as to how wall and slab shell elements are connected to the basement/mat solid elements. Wall shell elements are extended into the basemat solid elements to ensure a proper transfer of bending moment between them. Likewise, slab shell elements joining the basement walls are extended one element to overlay the basement top surface. These connecting elements are also assigned a zero density not to increase the overall mass of the basemat.~~

~~Also for simplicity, only the main steel frame in the Fuel Handling crane support system, including a simplified rail truss girder, is modeled in the dynamic FE model. The steel sections are modeled as beam elements within, but unattached to concrete shell elements representing the Fuel Handling exterior walls and slab. As a result, the composite behavior of the crane support system is not directly included in the FE model. To compensate for such a deficiency, all steel sections are assigned an increased moment of inertia in their strong axis to account for their composite behavior. An adjusted moment of inertia is also assigned to the embedded sections of the crane support steel columns between elevation 65’-0” and 76’-5”, encased in 7’-8” by 4’-0” concrete columns.~~

~~The thickness of the PCCV is also simplified for ease of modeling of the dynamic FE model. Only the large equipment hatch is modeled and the elements modeling the buttresses on the East and West sides of the structure are not offset with respect to adjacent elements. Also, the personnel airlocks as well as the main steam and feed water penetrations are not modeled in the dynamic FE model. Figure 3.7.2-8 shows the dynamic PCCV model.~~

~~The dynamic containment internal structure model consists of a combination of shell, solid and beam elements. The solid elements shown in Figure 3.7.2-9 make up the containment internal structure base, which starts at elevation 2’-7” and the reactor support which extends up to elevation 35’-10.87”. The shell elements model the remaining walls and slabs of the structure and begin at the same elevation as the containment internal structure solid elements, but extend to the top of the pressurizer compartment at 138’-7”. The beam elements model the supports for the RCL components, and the columns between the 2nd and 4th floors.~~

~~The dynamic containment internal structure model utilizes numerous simplifications to preclude meshing difficulties. The geometry of some of the containment internal structure components are simplified, and the various small openings throughout the structure are neglected. In order to maintain the dynamic properties of the detailed containment~~

~~internal structure model, the material properties of some of the containment internal structure members are modified.~~ MIC-03-03-00054

~~The standard plant R/B complex seismic response analyses utilize lumped mass stick models representing the dynamic properties of the PCCV, containment internal structure, and the R/B superstructure above its basement. To account for the effects of dynamic coupling of the containment internal structure with the equipment and the piping, the analysis of the R/B complex includes lumped mass stick models representing the stiffness and mass inertia properties of the major equipment and piping, including the reactor vessel and reactor coolant loop. The methodology initially used to develop the stick models and the stick model properties is presented in Technical Report MUAP-10001 (Reference 3.7-47), and in the following Subsections 3.7.2.3.4 through 3.7.2.3.9. The methodology in Subsections 3.7.2.3.4 through 3.7.2.3.9 is enhanced by: adjusting member properties where necessary to account for the effects of concrete cracking, incorporating single degree of freedom models representing the out-of-plane flexibility of slabs and walls, and integrating the stick models with a finite element model of the R/B complex basement to form the overall SSI model of the R/B complex. Technical Report MUAP-10001 (Reference 3.7-47) presents the methodology used for these enhancements and describes the overall enhanced R/B complex model in further detail.~~

~~The lumped-mass stick models used for the seismic analysis of US-APWR R/B, PCCV, and containment internal structure and their basemat consider the eccentricities between the center of rigidity and the center of mass of structures. The models represent the actual locations of masses and centers of rigidity, thus, accounting for the torsional effects caused by the eccentricity. The modeling approach accounts for the differences between the vertical and horizontal centers of rigidity by using two stick elements to model the stiffness of the structural members at each story. A truss element located at the vertical center of rigidity represents the vertical stiffness of the floor, and a beam element located at the shear center of rigidity represents the shear and bending stiffness of the floor. Both stick elements are rigidly connected to the common center of mass at each major floor elevation. This modeling approach helps eliminate the errors in computation of the seismic responses that are due to the rocking SSI effects caused by an inaccurate representation of the vertical center of rigidity. See Subsection 3.7.2.11 for discussion of accidental torsion.~~

~~To model the interaction with the underlying subgrade, the standard plant R/B complex model is analyzed in conjunction with a set of eight generic layered profiles using the program ACS SASSI (Reference 3.7-17) as discussed in Subsection 3.7.2.4.~~

~~The structural elements of the R/B, which includes the fuel handling area, are concentrated and reduced to one set of stick models below the operating floor level. The part of the R/B above the operation floor level, except for the fuel handling area, is represented by several stick models that are interconnected by horizontal rigid links representing the floor diaphragm. The rigid links restrain only the in-plane translational displacements of the floor without affecting the deformations in the other DOF. The containment internal structure and the PCCV are modeled separately, and they are rigidly connected to the R/B stick model at the surface of the common basemat. The R/B, PCCV, and containment internal structure are all structurally separated from each other above their common basemat by expansion joints, which are discussed further in Subsection~~ DCD_03.07.02-35

~~3.7.2.8. Detailed descriptions of the R/B, PCCV, and containment internal structure and their common basemat are presented in Section 3.8, where the structural design of those buildings and structures is addressed.~~ MIC-03-03-00054

~~A set of static and dynamic analyses is performed on the detailed three-dimensional FE model that is developed for computation of internal forces and stresses in the structural members and components of R/B-PCCV-containment internal structure subject to design loads and load combinations that are discussed further in Section 3.8. The FE model combines the R/B, PCCV, and containment internal structure on their common basemat. For clarity of presentation, the combined FE model is shown in Figures 3.7.2-1, 3.7.2-2, 3.7.2-3, and 3.7.2-4 as fouand 3.7.2-3 as three separate isometric views showing the PCCV, containment internal structure, and R/B on the common basemat. The results from the static and dynamic analyses performed on the detailed FE model are used to validate the dynamic properties of the stick models as described in Subsection 3.7.2.3. The lumped mass stick models of R/B, PCCV, and containment internal structures are developed independently of the detailed FE model.~~

Technical Report MUAP-10006 (Reference 3.7-47) presents a detailed discussion of the approach taken for development and validation of the R/B complex FE model.

The R/B complex Dynamic FE model is an integrated 3-D model of the R/B, PCCV, CIS, East and West PS/B, A/B, and ESWPC structures sharing common shear walls and resting on top of a common 13'-4" to 43'-3" thick basemat. Figure 3.7.2-1 shows an overview of the R/B complex model, while Figure 3.7.2-5 and Figure 3.7.2-6 reveal the interior structures with section views. Figure 3.7.2-7 through Figure 3.7.2-13 show the PCCV, CIS shell elements (excluding the RCL), CIS solid elements, CIS beam elements (excluding RCL), East PS/B with ESWPC, West PS/B with ESWPC, and A/B as individual structures, respectively. The global origin is located at the center of the PCCV and top of the basement with the X axis pointing north, Y axis pointing west, and Z axis pointing upward. Once the model is translated into SASSI format, the global coordinate system is rotated 180 degrees about the Z axis so that the X axis is pointing south and the Y axis pointing east. Typical element size in the basemat and the slabs is approximately 9 ft. The element mesh used in the dynamic model is selected to provide sufficient modeling to capture the dynamic properties of the structure. The validation discussed in Subsection 3.7.2.3.10 show that no further refinement of the Dynamic FE model is necessary.

The R/B complex Dynamic FE model is developed incrementally using ANSYS in the following seven steps:

Step 1. R/B complex structures geometry is created in a manner that allows control of the model FE mesh.

Step 2. Attributes are assigned and additional masses are applied on each of the structures.

Step 3. Mesh controls are set and the model, excluding the RCL, is meshed.

Step 4. Modifications are implemented as needed to make the model more consistent with the Detailed FE model.

Step 5. The nodes are renumbered sequentially in the order of their X, Y, and Z coordinates as recommended by the SASSI Manual in order to enhance computational efficiency. MIC-03-03-00054

Step 6. The model used for ANSYS analyses of RCL is translated into a format that can be translated into SASSI.

Step 7. RCL structure is added to the FE model of R/B complex structures and proper connections are implemented to represent the physical supports attached to the CIS.

For simplicity without compromising accuracy, slab elevations in the Dynamic FE model are slightly shifted upward or downward such that the middle planes of nearby upper or lower slabs fall into a major common horizontal plane. Also, only large openings in slabs and walls are included in the model.

Special attention is given in the Dynamic FE model as to how wall and slab shell elements are connected to the basement/mat solid elements. Wall shell elements are extended into the basemat solid elements to ensure a proper transfer of bending moment between them. Likewise, slab shell elements joining the basement walls are extended one element to overlay the basement top surface. These connecting elements are also assigned a zero density not to increase the overall mass of the basemat.

Also for simplicity, only the main steel frame in the Fuel Handling crane support system, including a simplified rail truss girder, and the main steel framing in the CIS are modeled in the Dynamic FE model. The steel sections are modeled as beam elements to share nodes with the concrete shell elements representing the Fuel Handling exterior walls and slab. Thus, the composite behavior of the crane support system would not be fully represented in the FE model without further adjustment. The adjustment made is that all steel sections are assigned an increased moment of inertia in their strong axis to account for their composite behavior. An adjusted moment of inertia is also assigned to the embedded sections of the crane support steel columns between elevation 65'-0" and 76'-5", encased in 7'-8" by 4'-0" concrete columns.

The thickness of the PCCV is also simplified for ease of modeling. For the Dynamic FE model, only the large equipment hatch is modeled and the elements modeling the buttresses on the East and West sides of the structure are not offset with respect to adjacent elements. Also, the personnel airlocks as well as the Main Steam and Feed Water penetrations are not modeled in the Dynamic FE model. The stiffness and weight of the PCCV Dynamic model are not adjusted to account for the openings since they were found to have a negligible impact on the overall response of the structure. Figure 3.7.2-7 shows the PCCV Dynamic model.

To account for the effects of dynamic coupling with the building structures, the PCCV polar crane and the R/B fuel handling crane are incorporated into the standard plant design by using typical mass and stiffness properties anticipated for the cranes in the R/B complex dynamic and detailed structural FE models. The cranes are modeled in their parked positions as occupied during normal plant operations. The parked position for the polar crane is parallel to the centerline of the PCCV running between azimuth 0° and

azimuth 180° with the hoist trolley located over the roof slab above the pressurizer. The fuel handling crane is stored above the truck access area on the east side of the fuel handling area when not in service. The building models include the crane's lifted mass, mass of the trolley, crane bridge girders, and end trucks. The building models include the stiffness of the crane bridge girders, end trucks, and the local stiffness of the supporting structural steel at the end truck locations. This is a generic crane design intended solely to be used for seismic analyses. Therefore, the polar crane is modeled to approximate the design weight. MIC-03-03-00054

The requirements of NOG-1 (Reference 3.7-22) require that the crane design analyses be performed by coupling the crane models with the building models. The PCCV polar crane and R/B fuel handling crane are procured on a site-specific basis. It is the responsibility of the COL Applicant to confirm the masses and frequencies of the PCCV polar crane and fuel handling crane and to determine if coupled site-specific analyses are required. If found that this is required, the site-specific seismic analysis of the US-APWR standard plant must be performed on models that incorporate the PCCV polar crane and the fuel handling crane, as appropriate in the site-specific SSI analyses and site-specific crane analyses.

The CIS portion of the Dynamic FE model, excluding the Reactor Coolant Loop (RCL), contains approximately 4,631 elements and 3,876 nodes with nominal mesh size of 7.2 ft in the vertical direction and 9 ft in the horizontal direction. It consists of a combination of shell, solid and beam elements. The solid elements shown in Figure 3.7.2-1 make up the CIS base which starts at elevation 2'-7" and the reactor support which extends up to elevation 35'-10.87". The shell elements make up the remaining walls and slabs of the structure and begin at the same elevation as the CIS solid elements, but extend to the top of the pressurizer compartment at elevation 139'-6". The beam elements shown in Figure 3.7.2-10 represent the steel frames and the supports for the RCL components.

The lumped mass stick model used for dynamic analyses of the RCL includes several parts representing the dynamic properties of the Nuclear Steam Supply System (NSSS) components and the main coolant piping. Appendix 3C discusses the RCL model.

The model of the RCL and major pipe components used for seismic analyses of the NSSS are translated into elements acceptable to SASSI format and then coupled with the dynamic CIS model. The translation included changes of ANSYS modeling features such as pipe element types, rigid links and constraint equations that can be supported by the SASSI translator. The pipe elements are replaced by 3-D beam elements with stiffness values equivalent to those of the straight and curved pipe sections. The rigid links and constraint equations are replaced by rigid beams. The coupling of the RCL to the CIS is accomplished such that there are no local effects from the CIS imparted upon the RCL. The validation of the model in Subsection 3.7.2.3.10 demonstrates that these modifications do not affect the overall stiffness of the model and thus the dynamic response of the RCL components.

3.7.2.3.3 ~~East and West PS/Bs Models~~Not Used

~~The seismic model for the PS/Bs is a finite element model in ACS SASSI (Reference 3.7-17) which represents the dynamic properties of the building in all important modes of vibration and is able to capture SSI effects related to the flexibility of the basemat~~

~~foundation. The model is developed initially in ANSYS (Reference 3.7-21) for purposes of equivalent static structural design as explained further below, and is translated into the format of ACS SASSI for purposes of seismic analysis by using the built-in converter in ACS SASSI. The ACS SASSI FE model is generated only for the West PS/B of the US-APWR standard plant, since the East and West PS/Bs are nearly identical structurally.~~ MIC-03-03-00054

~~The structural walls and slabs, including the floor slabs and roof slabs are modeled using shell elements, the beams and columns are modeled as beam elements and the basemat is modeled using solid elements. Mesh in the dynamic FE model varies in size from 4' to 8'. To reduce the size of the dynamic FE model and to permit a coarser mesh size, minor structural details and minor wall/slab openings are not included in the model. The use of FEs provides an accurate representation of the dynamic properties of the structures and accurate modeling of dynamic interaction of the flexible foundation and surrounding soil. At the connections between the basemat and the walls, the shell elements are extended into the basemat between the solid elements to transmit nodal rotations to the solid elements. The extended elements share nodes with the corresponding face of the solid elements but have no mass. The mass density properties of the finite elements are adjusted to account for the weight of the equipment and additional miscellaneous loading described in Section 3.8. The meshing of the seismic model ensures that the discretized structure is able to capture the local responses and the responses of the significant modes of vibration which corresponds to frequencies equal or below the highest frequency of interest, 50Hz. The seismic model is analyzed with ACS SASSI in conjunction with the generic soil profiles described in Subsection 3.7.1.3 to obtain SSI results and input loading for equivalent static analysis and structural design. The PS/B seismic model and the seismic analysis results are discussed in further detail in Technical Reports MUAP-10001 and MUAP-10006 (References 3.7-47 and 3.7-48), respectively.~~ DCD_03.07.02-35

~~A set of static analyses are performed on the detailed three-dimensional FE model that is developed for computation of internal forces and stresses in the structural members and components of PS/Bs. Applicable design loads and load combinations are discussed further in Section 3.8.~~

3.7.2.3.4 Subsystem Coupling Requirements

For purposes of modeling the R/B-PCCV-containment internal structure on their common basemat, large seismic subsystems contained within these structures are evaluated against the mass and frequency ratio criteria given in SRP 3.7.2, Section II.3(b) (Reference 3.7-16), as follows:

- If R_m ~~is less than~~ $\leq$ 0.01, decoupling can be done for any R_f DCD_03.07.02-35
- If 0.01 ~~less than or equal to~~ $\leq$ R_m and ~~less than or equal to~~ $\leq$ 0.1, decoupling can be done if 0.8 ~~is greater than or equal to~~ $\geq$ R_f ~~and greater than or equal to~~ $\geq$ 1.25
- If R_m ~~is greater than~~ $\geq$ 0.1, a subsystem model should be included in the primary system model

where

R_m = (total mass of supported subsystem)/(total mass of supporting system)

R_f = (fundamental frequency of supported system)/(dominant frequency of support motion)

If these criteria require the subsystem to be coupled with the primary seismic model, both the stiffness and the mass of the subsystem are included in the overall model to assure the accuracy of the calculated frequencies. This is the approach used for ~~including~~integrating the RCL seismic subsystem with the R/B complex dynamic FE model ~~in the coupled RCL-R/B-PCCV-containment internal structure lumped mass stick model~~ discussed in Technical Report MUAP-1000~~1~~6 (Reference 3.7-4~~7~~8). To account for the effects of dynamic coupling of the containment internal structure with the equipment and the piping, the dynamic FE model of the R/B complex also includes a lumped mass stick model of the RCL representing the stiffness and mass inertia properties of the major equipment and piping located in the PCCV. Spring elements are used to model the stiffness of the supports of the components and piping. The lumped mass stick model of the RCL and major piping components used for seismic analyses of nuclear steam supply system are translated into an acceptable ACS SASSI format and then coupled with the dynamic containment internal structure model. DCD_03.07.02-35 MIC-03-03-00054

When it has been determined through investigation of the above criteria that a subsystem is not required to be coupled with the primary seismic model, then the subsystem is assumed absolutely rigid and only its mass is included at appropriate node points of the global seismic model. ~~The PCCV polar crane and fuel handling crane are incorporated into the overall lumped mass stick model in this manner. In addition, the requirements of NOG-1 (Reference 3.7-22) for the design of cranes may require that the crane design analysis be performed by coupling the crane model with the overall building model. Therefore, it is the responsibility of the COL Applicant to confirm the masses and frequencies of the PCCV polar crane and fuel handling crane to determine if coupled site-specific analyses are required. If found that is required, the site-specific seismic analysis of the US-APWR standard plant must be performed on models that incorporate the PCCV polar crane and the fuel handling crane, as appropriate.~~ MIC-03-03-00054 DCD_03.07.02-35 DCD_03.07.02-102 MIC-03-03-00054

3.7.2.3.5 Section and Material Properties ~~for Lumped Mass Stick Models~~ DCD_03.07.02-35

The values of the modulus of elasticity and Poisson's ratio (ν) for concrete and steel used in ~~the lumped mass stick~~the dynamic models are discussed below. The values are for materials at or near ambient temperatures. DCD_03.07.02-35

a. Concrete

The concrete modulus of elasticity E_c, and shear modulus G_c corresponding to the compressive strengths of normal weight concrete used in the R/B, PCCV, and containment internal structure and their common basemat are summarized in Table 3.7.2-2 and are computed as follows:

E_c (~~k~~psi)= $57{,}000\ \sqrt{f'_c}$ MIC-03-03-00054

G_c (~~k~~psi)= $E_c / 2\ (1 + \nu_c)$

where

f'_c = specified 28-day compressive strength of concrete (psi)

ν_c = 0.17 (Poisson's ratio for concrete)

b. Steel

The properties of ferritic structural steel and non-prestressed reinforcement~~,~~: Young's modulus of elasticity E_s and Poisson's ratio for steel ν_s are as follows: MIC-03-03-00054

E_s = 29,000 ksi and ν_s = 0.3

Effects of Concrete Cracking on Reinforced Concrete Structures MIC-03-03-00054

Reinforced concrete structures include the R/B, East and West PS/Bs, A/B and ESWPC. In accordance with ASCE 4-98 (Reference 3.7-9), Section 3.1.3, traditional reinforced concrete members and elements are to be modeled as either cracked or uncracked sections. For the uncracked sections/elements, the stiffness is directly obtained from the concrete linear elastic properties and the section or element geometric dimensions. For the cracked concrete, a reduction to the uncracked concrete stiffness included. A 50% reduced value of the concrete modulus of elasticity is used in linear elastic analysis to address the effects of concrete cracking on the seismic response.

The design of the reinforced concrete structures is based on the ultimate capacity of the reinforced concrete sections. Therefore, the design of reinforced concrete members addresses code stress limits corresponding to reduced cracked concrete stiffness properties and higher SSE material damping levels as discussed in Section 1.2 of RG 1.61 (Reference 3.7-15). However, there is a possibility that the response of the structure under lower stress levels at certain frequency ranges will be higher than the response corresponding to the higher stress state under cracked conditions. In order to ensure that the structural integrity and functionality of the components and the equipment is not compromised under seismic loading conditions, the development of ISRS and seismic loads and displacement also considers the responses of the reinforced concrete structure with full (uncracked concrete) stiffness properties and lower OBE damping levels.

The seismic response analyses of reinforced concrete structures consider two stiffness and damping values in order to address the possible variations in the extent of concrete cracking:

1. Full stiffness representing low stress levels corresponding to uncracked concrete properties where the stiffness of the members are represent by gross cross sectional properties.

2. Reduced stiffness representing higher stress levels resulting in cracking of the concrete where the stiffness of the members are reduced in accordance with guidelines provided in Table 3-1 of ASCE/SEI 43-05. The stiffness of the composite members made of reinforced concrete and steel beams, such as the walls and the roof of Fuel Handling Area (FH/A), are also reduced accordingly to

represent 50% reduction in stiffness in the reinforced concrete part of the composite sections. MIC-03-03-00054

The structural material damping values used for these two different stress levels are OBE damping of 4% for the full (uncracked concrete) stiffness condition and SSE damping of 7% for the reduced (cracked concrete) stiffness condition, are obtained from RG 1.61 (Reference 3.7-15) and are shownin Table 3.7.2-3.

Effects of Concrete Cracking on the CIS

The CIS is comprised of different types of structural members including composite SC walls, massive reinforced concrete sections, and reinforced concrete slabs. The members can experience varying levels of stress resulting in different patterns of concrete cracking under the different loading conditions that can occur. Depending on the plant conditions, the CIS members can be subjected to design seismic loads in combination with normal operating or design basis accidental thermal loads resulting in different levels of stiffness reduction due to concrete cracking. Table 3.8.3-4 shows the summary of CIS stiffness and damping. The CIS members are classified in six categories, two stiffness levels corresponding to:

1. Loading Condition A: (SSE Seismic, plus operating temperatures): conditions characterized with insignificant reduction of stiffness and concrete cracking; and;

2. Loading Condition B: (SSE Seismic, plus accident temperatures): conditions characterized with significant reduction of stiffness due to cracking of the concrete under high design basis accidental thermal loads and SSE seismic.

Different material damping values are assigned to the different members depending on the level of stresses and corresponding concrete cracking.

Effects of Concrete Cracking on the PCCV

Similar to the CIS, the level of stress in the PCCV during a seismic design event depends on the plant conditions. The design of the PCCV structure is based on the premise that during normal operating conditions the pre-stressed concrete cross sections remain in compression. During the normal operating conditions, the earthquake design loads can cause only limited cracking having insignificant effect on the overall stiffness of the PCCV. Accordingly, the dissipation of energy due to material damping of the PCCV structure under normal operating conditions is low. The accident loading conditions include high temperatures and pressure loads in the reactor containment that can generate high stresses in the pre-stressed concrete accompanied with cracking that can result in a reduction of the global stiffness of the PCCV structure and higher dissipation of energy due to the material damping. The stress evaluations provided in Appendix 2-A of MUAP-10006 (Reference 3.7-48), indicate that the reduction of the overall stiffness of PCCV structure under seismic design loads in combination with accident loads can be up to 50%.

Two stiffness levels are considered for the seismic response analyses of PCCV:

1. Normal operating conditions corresponding to insignificant concrete cracking and full (uncracked concrete) stiffness of the pre-stressed concrete structure, using 3% damping, and; MIC-03-03-00054

2. Accident conditions when the high thermal and pressure loads generate high stresses that can result in significant cracking of the pre-stressed concrete and a 50% reduction of the stiffness, using 5% damping.

The structural material damping values used for these two different stiffness and stress levels are also provided in Table 3.7.2-3.

3.7.2.3.6 ~~Masses~~Modeling of Mass

~~The inertial properties include all tributary mass expected to be present at the time of the earthquake. This mass includes the effects of dead load, stationary equipment, piping, and the appropriate part of the live and snow load (see Subsection 3.7.2.33.7.2.3.11 for further discussion of equivalent live load). The mass properties of stick modelfor RCL stick model components consist of the total weight~~ ~~W~~~~, the weight moment of inertia (~~~~J_{xx}, J_{yy}, J_{zz}~~~~), and the center of mass are presented in Appendix 3H. They are in principle evaluated by hand calculation as described below.~~ DCD_03.07.02-35 MIC-03-03-00066

The mass included in the R/B complex Dynamic FE model includes contributions from the structural mass in addition to that of equipment, dead loads, and live loads.

Generally, the structural mass is assigned as a density to the finite elements based on the material properties of the components of the structures. The density is then increased to account for equipment, live, snow and other applicable loads. A mass equivalent to 25% of floor design live load and 75% of roof design snow load, as applicable, is included in the model in accordance with SRP 3.7.2 Acceptance Criteria II.1.D (Reference 3.7-17). Each load is applied over a particular area and the density of the elements in that area is increased such that the total increase in mass matches the mass of the applied loads.

Equipment load also includes a 50 psf dead load to account for miscellaneous pipe, minor equipment, and raceway loads applied on slabs in the R/B complex model, with the exception of a few locations where a heavier pipe load is used instead (e.g., main steam and feedwater pipe).

The above process is not applicable for the NSSS and major pipe that constitutes the RCL. The RCL dynamic mass is included directly in the RCL model.

The mass is applied to the Dynamic FE model in two steps. First, a mass density equal to the sum of the structural self-weight and pipe load is calculated and assigned to each of the shell elements modeling the R/B complex slabs. Where mass is carried by grating not explicitly modeled, the total mass supported is evenly distributed on the supporting walls and slabs.The remaining loads are applied as either additional mass densities on slab shell elements or concentrated lumped masses on wall and slab key points.

The density and thickness of the elements are further modified to account for stiffness reductions due to minor openings and cracking, but it is done in such a way as to not change the mass of the elements. Refer to Subsection 3.7.2.3.2 for further discussion.

The PCCV Polar Crane and Fuel Handling cranes are modeled in their respective parked locations with trolley masses and lifted load masses included. MIC-03-03-00054

The mass used for the New Fuel Storage Pit (NFSP) and Spent Fuel Pit (SFP) includes the mass of the fuel and the fuel storage racks contained within the pits. This is accomplished by adding the masses as lumped masses to the concrete slabs of the pits (pools). The dynamic characteristics of the racks are not modeled or coupled with the structure. Liquid masses contained in the SFP, Emergency Feed Water Pits (EFWP), and Refueling Water Storage Pit (RWSP) are modeled as directional masses using mass elements attached to walls and slabs. In accordance with ANSI/AISC 360-05 (Reference 3.7-57), the hydrodynamic responses of the water in the SFP and the EFWPs are evaluated, and the hydrodynamic masses are separated into the lower impulsive and the upper convective parts. Figure 3.7.2-14 shows a representative dynamic model for the SFP when it is subject to seismic motion in the north-south direction. The impulsive mass is rigidly fastened to the walls as it moves with the walls responding to the seismic excitation. Depending on the ratio of the water depth to the pit width, the lower portion of the impulsive part can be considered as fully constrained which responds as a rigid body. The impulsive pressure is evenly and uniformly divided into a pressure force on the wall accelerating into the fluid, and a suction force on the opposite wall accelerating away from the fluid. The convective mass is also included in the dynamic FE model and is fastened to the walls by springs that produce the period of vibration corresponding to the period of fluid sloshing. The stiffnesses of these connecting springs are calculated based on the sloshing period.

~~3.7.2.3.6.1 Mass Points and Associated Weights (*W*)~~

~~The mass points are, in principle, established at the major floor levels represented by nodes in the lumped mass stick model.~~ DCD_03.07.02-35

~~Figure 3.7.2-5 depicts how the mass moments of inertia and weights associated with the lumped masses are computed.~~

~~In addition to the structural mass, mass equivalent to a floor load of 50 pounds per ft^2 at least 50 psf miscellaneous dead load is considered to represent miscellaneous dead weights such as minor equipment, piping, and raceways. The mass equivalent to 25% of the floor design live load and 75% of the roof design snow load is also included. The mass of major equipment is considered to be distributed over a representative floor area or included as concentrated lumped masses at the equipment locations.~~ MIC-03-03-00066

~~Vertical amplification effects on the masses of floor slab systems due to out-of-plane flexibility are addressed as part of Technical Reports MUAP-10001 and MUAP-10006 (References 3.7-47 and 3.7-48).~~ MIC-03-03-00066

~~3.7.2.3.6.2 Mass Moment of Inertia (J_{xx}, J_{yy}, J_{zz})~~ DCD_03.07.02-35

~~Mass moments of inertia are considered at all of the mass points for all three rotational DOF.~~

3.7.2.3.7 ~~Shear Stiffness~~Adjustment of Stiffness and Mass Properties

MIC-03-03-00054

The coarse mesh of the dynamic FE model has limited resolution for modeling of openings in the walls. The elastic modulus and thickness of shell elements are adjusted to accurately model the reduction of shear stiffness of the wall due to openings. The density of shell elements is also adjusted to accurately represent the mass of the wall accounting for openings and the adjusted wall thickness.

~~A set of~~ Finite Element analyses ~~is~~are performed using ANSYS to obtain the stiffness reduction factors needed to adjust the material properties and account for the reduced stiffness of the shear wall openings. The correction factors are obtained by comparing the results from the static analyses of two detailed solid FE models. Model A represents the actual geometry of the wall with openings, and Model B represents the wall without openings. Unit displacements are applied at the top of each model in both the in-plane and the out-of-plane directions, to generate the reactions at the bottom, which can then be used to calculate the in-plane and out-of-plane wall stiffness. The ratio between the reaction obtained from Model A and Model B is used to determine out-of-plane stiffness reduction factors (*m*) and the in-plane stiffness reduction factor (*n*) that are then used to determine the adjusted elastic modulus (E_o), thickness (t_o), and density (γ_o) of the wall. Further details on the development and implementation of stiffness reduction of elements in the FE model are described in Technical Report MUAP-10006 (Reference 3.7-48).

~~The effect of in-plane shear deformation is included in the model. For the lumped mass stick model, effective shear area is computed from the sum of the component shear areas of the individual walls parallel to the direction of the applied force.~~ DCD_03.07.02-35

~~The stiffness of the stick model is evaluated by hand calculation as shown in Figure 3.7.2-6, which summarizes the approximate methods used to compute the section areas, geometrical moments of inertia, and axial areas for the PCCV and R/B structures, under the following assumptions:~~

- ~~Walls continuously built up from the basemat, whose thickness is more than 40 in. are treated as seismic walls.~~
- ~~Openings with area more than 2,880 in~~^{2} ~~(80 in. x 36 in.) are considered in evaluating the stiffness of walls as discussed in Subsection 3.7.2.3.~~

~~Generally, in accordance with ASCE 4 (Reference 3.7-9) Subsection C3.1.8.3, if the shear wall has no flange elements at its ends, the shear area is equal to the total web area divided by 1.2. If flanges are present, the shear area is equal to the total web area. The effective flange width of each perpendicular wall may be calculated using the following reduction due to shear lag effects:~~

$$\cancel{W_e - \frac{H}{3} \le \frac{W}{2}}$$

~~where~~

~~W_e = effective flange width on each side of the wall~~

~~H = height of the wall~~ DCD_03.07.02-35

~~W = actual width of the flange on each side of the wall~~

~~The above guidance on shape factors and reductions to flange width to account for shear lag effects is adjusted where necessary for some shear walls based on detailed local analysis in order to assure that the overall lumped mass stick model contains accurate shear stiffness values. For example, for the containment internal structure the effective shear area of the stick elements is calculated by a FE model as follows:~~

i. ~~A FE model of the containment internal structure above the upper level of the basemat, considering the walls, columns and floor slabs, is developed using brick, shell and beam elements.~~

ii. ~~By fixing the containment internal structure where it intersects with the upper level of the basemat, a set of vertically distributed horizontal loads, which is established considering the earthquake excitation, is applied at each main floor level and the resulting horizontal displacements are evaluated at the top level of each floor. To determine which portion of the resulting displacement at each floor is attributable to shear stiffness and which portion is related to bending stiffness, another analytical model in which the vertical DOF is constrained is also prepared separately. The flexibility coefficients for the equivalent beam are evaluated from the results of these analyses.~~

iii. ~~The equivalent stiffness properties (the equivalent shear stiffness, bending stiffness, etc) are evaluated from (ii) above.~~

~~3.7.2.3.7.1 Effective Shear Area (A_x, A_y)~~

~~Two effective shear areas A_x and A_y are calculated for each floor by considering the seismic walls that are parallel to each two horizontal earthquake directions. For the walls having openings, an equivalent shear area (A_e) is considered that is calculated using the equal shear deformation methodology depicted in Figure 3.7.2-7. From the requirement that the shear deformation δ_s of the wall with openings be equal to the shear deformation δ_s of a wall without openings with height equal the story height (H), the effective cross section area of the wall A_e is obtained from the following equation:~~

$$\cancel{A_e = \frac{H}{\sum_{i=1}^{n} \frac{\kappa_i h_i}{A_i}}} \qquad \text{~~(Eq. 3.7.2-5)~~}$$

~~where~~

~~A_i = the shear area~~

~~h_i = height of wall segment~~

~~κ_i = a shape factor~~

~~3.7.2.3.7.2 Bending Moment of Inertia (I_{yy}, I_{xx})~~ DCD_03.07.02-35

~~Bending moment of inertia of the shear walls are calculated around the horizontal axes established at the centroid of the floor shear walls area. The effective flange width of each perpendicular wall is calculated using the following reduction due to shear lag effects~~

$$\cancel{W_e = \frac{H}{3} \le \frac{W}{2}}$$

~~where~~

~~W_e = effective flange width on each side of the wall~~

~~H = total height of the wall~~

~~W = actual width of the flange on each side of the wall~~

~~Equivalent moments of inertia (I_e) are calculated for the walls with openings using the equal bending rotation methodology as shown in Figure 3.7.2-7. The bending rotation θ of the wall with openings loaded with unit bending moment M is calculated as follows:~~

$$\cancel{\theta = \int_0^{h_1} \frac{M}{EI_1} dh + \cdots + \int_{h_1+\cdots+h_{n-1}}^{h_1+\cdots+h_{n-1}+h_n} \frac{M}{EI_n} dh}$$

~~where I_i and h_i are the moment of inertia and height of wall segment as shown in Figure 3.7.2-7. The bending rotation θ' of equivalent wall without openings with height equal the story height (H) and cross section with equivalent moment of inertia I_e is:~~

$$\cancel{\theta' = \int_0^{H} \frac{M}{EI_e} dh = \frac{MH}{EI_e}}$$

~~From the requirement that the bending rotation of the two walls to be equal ($\theta = \theta'$) the effective moment of inertia is obtained as follows:~~

$$\cancel{I_e = \frac{H}{\sum_{i=1}^{n} \frac{h_i}{I_i}}} \qquad \text{~~(Eq. 3.7.2-6)~~}$$

3.7.2.3.8 ~~Torsional Stiffness~~Stiffness of ~~Composite~~ Steel- Reinforced Concrete Beams and Columns MIC-03-03-00054

~~In the FH/A, the crane supporting steel columns and girts are continuously anchored to the exterior concrete walls with headed steel studs. The steel roof beams and girders are also continuously anchored to the concrete roof slabs. The concrete/steel composite moment of inertia is based on the effective width of concrete and the degree of composite~~

~~action. A composite action of 75% of the composite transformed moment of inertia is used in calculating the effective moment of inertia (I_{eff}):~~ MIC-03-03-00054

$$\cancel{I_{eff} = \min\left[0.75 \cdot I_{tr}, I_x + \sqrt{Q_n / C_f} \cdot (I_{tr} - I_x)\right]}$$

~~Where~~

~~Q_n =~~ ~~shear capacity of the studs~~

~~C_f =~~ ~~smaller of steel yield force or concrete ultimate compressive force~~

~~I_x =~~ ~~moment of inertia of the steel column or beam~~

~~I_{tr} =~~ ~~composite transformed moment of inertia, calculated as follows:~~

$$\cancel{I_{tr} = I_X + \left(t_c + t_d + \frac{d}{2} - y_{bar}\right)^2 \cdot A_s + \frac{b_{eff} \cdot t_c}{12} + \left(y_{bar} - \frac{t_c}{2}\right)^2 \cdot b_{eff} \cdot t_c}$$

~~Where~~

~~t_c =~~ ~~slab or wall thickness~~ DCD_03.07.02-35

~~t_d =~~ ~~steel deck thickness, if any~~

~~d =~~ ~~depth of the steel member~~

~~y_{bar} =~~ ~~centroidal distance of the transformed section, measured from the top of concrete~~

~~A_s =~~ ~~area of the steel member~~

~~b_{eff} =~~ ~~effective width of concrete, after transforming to steel = b_e/n~~

~~b_e =~~ ~~effective width of concrete, before transforming to steel~~

~~n =~~ ~~modular ration Es/Ec~~

~~E_s =~~ ~~Young's modulus for steel~~

~~E_c =~~ ~~Young's modulus for concrete~~

~~In order to incorporate the composite stiffness of the steel beams and the reinforced concrete slabs, the moments of inertia of the beams are increased. This modeling approach provides an accurate representation of the actual out-of-plane bending stiffness~~

~~of the composite concrete-steel cross-sections, which is validated through comparison of responses obtained from the detailed and dynamic models.~~ MIC-03-03-00054

~~The effective width of concrete (before transformation to steel section) is based on AISC 360-05, Section I3. (Reference 3.7-57).~~

~~The centroidal distance of the transformed section, measured from the top flange of concrete in compression is calculated as:~~

$$y_{bar} = \frac{0.5 \cdot b_{eff} \cdot t_c^2 + A_s(t_c + t_d + 0.5 \cdot d)}{b_{eff} \cdot t_c \cdot A_s}$$

~~In the dynamic FE model, shell elements and beams are used to represent the individual members. The locations of the centerlines of the beam and shell elements are coincident so the effective bending stiffness of the section (*EI*) in the dynamic FE model is the sum of the individual moments of inertia:~~

$$EI = \frac{tc^3 \cdot be}{12} \cdot E_c + I_X + E_s$$

~~Therefore, the moment of inertia of the beam element that results in bending stiffness of the section (*EI*) that is equivalent to the stiffness of the actual composite section is calculated as follows:~~ DCD_03.07.02-35

$$I_S = I_X \cdot \alpha = I_{eff} - \frac{tc^3 \cdot be}{12} \cdot \frac{E_c}{E_s}$$

~~The factor "α" is used to adjust the bending moment of inertia of the beam element in the FE model in order to simulate the actual composite stiffness of the reinforced concrete-steel beam cross sections. The factor "α" is calculated as follows:~~

$$\alpha = I_S / I_X$$

~~The stick elements modeling the stiffness of the floor shear walls and columns are to be located at the center of rigidity of the floor, and an appropriate torsional stiffness must be assigned if the center of mass is not coincident with the center of rigidity. The torsional rigidity, K_p, can be computed from the following equations:~~

$$K_p = \sum_{i=1}^{N}\left(K_{yi}\bar{X}_i^2 + K_{xi}\bar{Y}_i^2\right) - X_{cr}^2\sum_{i=1}^{N}K_{yi} - Y_{cr}^2\sum_{i=1}^{N}K_{xi} \qquad \text{(Eq. 3.7.2-7)}$$

~~where~~

$\bar{X}_i, \bar{Y}_i$ = ~~coordinated of i^{th} wall or column elements~~

~~K_{xi}, K_{yi} = stiffness of i^{th} wall or column effective for shear, assuming rigid connection to the floor, in x and y directions, respectively~~ MIC-03-03-00054

~~X_{cr}, Y_{cr} = coordinates of center of rigidity~~

$$X_{cr} = \frac{\sum_{i=1}^{N}\left(\overline{X}_i K_{yi}\right)}{\sum_{i=1}^{N}\left(K_{yi}\right)}, \quad Y_{cr} = \frac{\sum_{i=1}^{N}\left(\overline{Y}_i K_{yi}\right)}{\sum_{i=1}^{N}\left(K_{xi}\right)} \qquad \text{(Eq. 3.7.2-8)}$$

~~Alternatively, the torsional stiffness of the floor can be obtained from the FE analysis of the floor model, as described in Subsection 3.7.2.3. Multiple stick elements may be used to represent the stiffness of the floor by locating the stick stiffness elements at the centers of rigidity of the respective groups of structural members (shear walls or beams). The multiple floor stick stiffness elements must be properly interconnected among each other and with the node(s) where the mass inertia of the floor is lumped.~~

~~3.7.2.3.8.1 Torsional Constant (I_{zz})~~

~~Torsional constant of seismic shear walls is calculated around the vertical axis that goes through the center of the floor shear rigidity.~~ DCD_03.07.02-35

In the fuel handling area (FH/A), the crane supporting steel columns and girts are continuously anchored to the exterior concrete walls with headed steel studs. The steel roof beams are also continuously anchored to the concrete roof slabs. The concrete/steel composite moment of inertia is based on the effective width of concrete and the degree of compositeness provided by the studs. The shear force that can be transferred between steel and concrete depends on the capacity of the studs, the yield strength of the steel section and the ultimate compressive strength of the effective concrete.

Based on AISC 360-05 Commentary (Reference 3.7-57), 75% of the composite transformed moment of inertia is used in calculating the effective moment of inertia of the composite section (I_{eff}):

$I_{eff} = 0.75 \cdot I_{tr}$ for a fully composite member

$I_{eff} = I_x + \sqrt{Q_n/C_f} \cdot (I_{tr} - I_x)$ for a partially composite member, with $Q_n/C_f \geq 0.25$

Where: Q_n = shear capacity of the studs between the points of inflection (zero moment)

C_f = smaller of steel yield force or concrete ultimate compressive force

I_x = moment of inertia of the steel column or beam

I_{tr}= composite transformed moment of inertia, calculated as follows: MIC-03-03-00054

$$I_{tr} = I_x + \left(t_c + t_d + \frac{d}{2} - y_{bar}\right)^2 \cdot A_s + \frac{b_{eff} \cdot t_c^3}{12} + \left(y_{bar} - \frac{t_c}{2}\right)^2 \cdot b_{eff} \cdot t_c$$

Where: t_c = slab or wall thickness

t_d = steel deck thickness, if any

d = depth of the steel member

y_{bar} = centroidal distance of the transformed section, measured from the top of concrete:

$$y_{bar} = \frac{0.5 \cdot b_{eff} \cdot t_c^2 + A_s(t_c + t_d + 0.5 \cdot d)}{b_{eff} \cdot t_c + A_s}$$

A_s = area of the steel member

b_{eff}= effective width of concrete, after transforming to steel = b_e/n

b_e = effective width of concrete, before transforming to steel

n = modular ratio = E_s/E_c

E_s = Young's Modulus for steel

E_c = Young's Modulus for concrete

In order to incorporate the composite stiffness of the steel beams and the reinforced concrete slabs the moments of inertia of the beams are increased. This modeling approach provides an accurate representation of the actual out-of-plane bending stiffness of the composite concrete-steel cross-sections which is validated through comparison of responses obtained from the detailed and dynamic models.

In the above, the effective width of concrete (before transformation to steel section) is based on AISC 360-05, Section I3 as shown below.

In the Dynamic FE model, beam and shell elements are used to represent the individual members. The locations of the centerlines of the beam and shell elements are coincident so the effective bending stiffness of the section (EI) in the Dynamic FE model is the sum of the individual moments of inertia:

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$$EI = \frac{t_c^3 \cdot b_e}{12} \cdot E_c + I_x + E_s$$

Therefore, the moment of inertia of the beam element that results in bending stiffness of the section (EI) that is equivalent to the stiffness of the actual composite section is calculated as follows:

$$I_s = I_x \cdot \alpha = I_{eff} - \frac{t_c^3 \cdot b_e}{12} \cdot \frac{E_c}{E_s}$$

Where $\alpha = I_s / I_x$ is a factor used to adjust the bending moment of inertia of the beam element in the FE model in order to simulate the actual composite stiffness of the reinforced concrete-steel beam cross sections.

3.7.2.3.9 ~~Axial Stiffness~~Dynamic Properties of R/B Slabs and SC Modules DCD_03.07.02-35

~~Axial stiffness is calculated considering all walls that contribute to shear stiffness and from vertically acting supports such as columns.~~

3.7.2.3.9.1 ~~Vertical Axial Area (A_a)~~Dynamic Properties of R/B Slabs DCD_03.07.02-35

~~The development of the dynamic FE model requires simplifications of the model geometry in order to produce regular FE mesh and to minimize the size of the model. This simplified model is suitable for time history frequency domain SSI analyses using ACS SASSI. In order to accurately capture the SSI effects, the R/B footprint dimensions represent the actual configuration of the basemat. The dynamic R/B model has the shell elements of the exterior walls placed at the perimeter of the building unlike the detailed R/B model where the shear walls are modeled at the centerline of the walls. These differences in modeling the building geometry affect the spans of some of the floor slabs in the dynamic R/B model and their local out-of-plane response. The stiffness and mass properties of these flexible slabs are adjusted to mimic the actual mass and dynamic properties of the slab.~~

~~The dynamic properties of the slabs at each major floor elevation are obtained by isolating each elevation. Boundary conditions are defined to restrain horizontal displacements of the walls and accurately mimic the bending stiffness at the wall/slab interfaces. The horizontal and vertical displacements of the wall are also restrained at slab elevation in order to eliminate the effects of the axial stiffness of the walls on the modal analyses results and to disregard the slab horizontal modes as well. Where the slab is supported by columns, the vertical displacement is constrained.~~

~~Modal analysis using ANSYS is performed on the isolated elevations of the detailed FE model and the dynamic FE model to obtain the dynamic properties. If the frequency of the first dominant mode of the slab obtained from the detailed FE model with full (uncracked concrete) stiffness properties is greater than 70 Hz, the slab is considered rigid. There is no need to adjust the stiffness of the shell elements modeling rigid slabs.~~

~~For the flexible slabs with frequencies below 70 Hz, the stiffness is adjusted as needed by tuning the modulus of elasticity of the slab shell elements in the dynamic FE model to match the frequency obtained from the detailed FE model. The matching criterion is the difference in the first dominant frequency of vibration of the slab obtained from the modal analyses of the two FE models being measured within 5%.~~ MIC-03-03-00054

The development of the Dynamic FE model requires simplifications of the model geometry in order to produce a regular FE mesh and to minimize the size of the model to be suitable for SSI analyses using SASSI. These simplifications in modeling the building geometry affect the spans of some of the floor slabs in the R/B model and their local out-of-plane response. The stiffness and mass properties of these flexible slabs are adjusted to model the actual mass and stiffness properties of the slab.

The dynamic stiffness properties of the slabs at each of the major floor elevations are obtained by isolating each elevation. Figure 3.7.2-15 shows an FE model of a R/B floor slab that is extracted from the Detailed FE model. Boundary conditions are established as shown in Figure 3.7.2-16 at the upper and lower border of the model to restrain horizontal displacements of the walls and accurately model the bending stiffness at the wall/slab interfaces. Figure 3.7.2-16 is meant to show representative boundary conditions, not the exact support conditions of all the individual slabs, which may be supported on three or four sides with walls. The horizontal and vertical displacements of the slab at the junctions of the slab with the supporting walls are also restrained in order to eliminate the effects of the axial stiffness of the walls on the modal analyses results and to ignore the slab horizontal modes as well. Where the slab is supported by columns, the vertical displacement is constrained.

Modal analysis using ANSYS is performed on the isolated elevations of Detailed FE model and Dynamic FE model to obtain the dynamic properties. If the frequency of the first dominant mode of the slab obtained from Detailed FE model with full (uncracked concrete) stiffness properties is greater than 70 Hz, the slab is considered rigid. There is no need to adjust the stiffness of the shell elements modeling rigid slabs.See Subsection 3.7.2.3.10 for additional discussion about the 70 Hz cutoff frequency.

For the flexible slabs with frequency below 70 Hz, the stiffness is adjusted as needed by tuning the modulus of elasticity of the slab shell elements in the Dynamic FE model to match the frequency obtained from the Detailed FE model. The difference in first dominant frequency of vibration of the slabs obtained from the modal analyses of the two FE models is minimized through an iterative process. This process is iterated until the difference in dominant frequencies for slabs at a given elevation are at a minimum. The largest difference in slab frequency after the completion of the above process is 6%. See MUAP-10006 (Reference 3.7-48) for additional discussion about the development and validation of the dynamic model.

3.7.2.3.9.2 Dynamic Properties of SC Modules

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Simplifications in the geometry of the dynamic containment internal structure model are made to produce a coarser FE mesh in order to be suitable for SSI analyses using ACS SASSI. Stiffness and mass properties of elements modeling some of the SC walls of the containment internal structure are adjusted in order to calibrate the dynamic response of

the simplified dynamic FE model to match the actual response of the containment internal structure as represented in the detailed FE model. The adjustments of the unit density and the elastic moduli of the shell elements are introduced to capture the actual distribution of mass and stiffness. The calibration of the model properties is performed based on the results of a 1g static analysis, and then verified using the results of modal and time history analyses. DCD_03.07.02-35

~~Vertical axial area of each element of the lumped mass stick model is equal to the summation of the effective shear areas for the two horizontal directions and axial area contributed by other vertically acting supports such as columns. However, overlapping areas such as the corner areas of box walls are subtracted from the summation.~~

~~The axial stiffness of the dome of PCCV is evaluated as follows:~~

i. ~~A FE model of PCCV as shown in Figure 3.7.2-8 is developed.~~

ii. ~~The static analysis is performed by applying to the FE model vertical loads with magnitude equivalent to the weight at the corresponding mass point weights at locations which correspond to the locations of the mass points in the stick model.~~

iii. ~~Vertical displacements are calculated from the static FE analysis and used to evaluate the effective vertical axial area as follows:~~

$$\cancel{K_i = \frac{A_i E}{L_i} = \frac{\sum W_i}{\Delta_i - \Delta_{i-1}}}$$

~~where~~

$\cancel{K_i}$ = ~~effective vertical axial stiffness of the element *i*~~

$\cancel{A_i}$ = ~~effective vertical axial area of the element *i*~~

$\cancel{L_i}$ = ~~length of the element *i*~~

$\cancel{W_i}$ = ~~weight of the mass *i*~~

$\cancel{\Delta_i}$ = ~~vertical displacement of node *i*~~

$\cancel{E}$ = ~~Young's modulus of the concrete material~~

3.7.2.3.10 Validation of the Seismic Models

~~The seismic response of R/B and containment internal structure is obtained from the analyses of three-dimensional seismic models, portions of which are lumped mass stick models whose stiffness and mass properties are evaluated by hand calculation as described in the previous section. The seismic response of the PS/Bs is obtained from analyses of three-dimensional finite element seismic models. As described herein,~~ DCD_03.07.02-35 MIC-03-03-00054

~~detailed FE mathematical models that are initially prepared for static design analysis of these structures are used to verify that the seismic models realistically represent the dynamic properties of the R/B complex and PS/Bs. The validation of the US-APWR seismic Category I, dynamic FE models of the R/B complex and the PS/B is performed following the methodology described in Sections 4.3.2, 4.4.3, 5.3.3, 5.3.4 and 5.4.5 of Reference 3.7-47. The validation ensures that modeling assumptions and simplifications do not affect the ability of the dynamic FE model to accurately represent the dynamic response of the R/B complex structures as mandated by SRP Sections 3.7.2.II.1 and 3.7.2.II.3 and by ISG-01, Section 3.1(Reference 3.7-54).~~ MIC-03-03-00054

~~The lumped mass stick models are validated as follows:~~

- ~~The static deformations results obtained from the analysis of the detailed FE models are compared to those obtained from the analysis of the stick models to verify that the models exhibit closely matching results.~~
- ~~The 5% damping ISRS results obtained from the fixed base analysis of the stick model are compared to those obtained from the fixed base analysis of the detailed FE model for various locations within the structures. The results are considered acceptable if the stick model results envelope (or reasonably match in terms of peaks and amplitude) those of the FE model.~~

~~All computer programs used for modeling are also verified and validated as in accordance with ANSI/ASME NQA-1-2004 (Reference 3.7-23) requirements.~~

The development of the R/B complex Dynamic FE Model is based on a number of adjustments in geometry and load configurations in order to minimize the size of the model and make it suitable for SSI analysis using SASSI. The validation ensures that these modeling adjustments do not affect the ability of the Dynamic FE Model to accurately represent the dynamic response of the R/B complex structures as described by SRP Sections 3.7.2.II.1 and 3.7.2.II.3 (Reference 3.7-16) and by ISG-01, Section 3.1 (Reference 3.7-54).

The R/B complex Dynamic FE Model is divided into six parts: R/B-FH/A-ESWPC, CIS coupled with RCL, PCCV, East PS/B, West PS/B, and A/B. The integrated model is divided into the individual components such that each structure is independent of the others. Common walls in the integrated model are included in each individual model for the purpose of validation. A series of fixed base analyses are performed on the six separate models using ANSYS and the results are compared to the ones obtained from corresponding analyses on the Detailed FE models of the R/B-FH/A-ESWPC, CIS, PCCV, East PS/B, West PS/B, and A/B structures. Once validation of the individual models is complete, confirmatory validation analyses on the integrated R/B complex model are performed.

The validation of the Dynamic FE Model of the R/B complex, with the exception of the CIS, that are carried out on models with full (uncracked concrete) stiffness are also valid for the models with reduced (cracked concrete) stiffness. The result of the global stiffness reduction is manifested by a shift of the response of the structure to lower frequencies. Hence, a 50% stiffness reduction corresponds to a shift of frequencies by $\sqrt{2} = 1.4$ times. Therefore, the dynamic validation analyses consider responses for frequencies up to

1.4X50 = 70 Hz and higher in order to ensure that the model with reduced (cracked concrete) stiffness properties can also meet the requirement of ISG-1, Section 3.1 (Reference 3.7-54) to accurately capture responses with frequencies up to 50 Hz. MIC-03-03-00054

Due to the complexity of the CIS, different stiffness and damping values are assigned to different types of structural components for the two bounding stiffness and damping conditions. As shown in Table 3.8.3-4, the reduction of stiffness applied to the CIS to account for cracking of the concrete of SC modules, reinforced concrete slabs and massive concrete portions is not uniform. Therefore, unlike the other structures, two sets of validation analyses are performed for the CIS to ensure the adequacy of the CIS Dynamic FE Model with full (uncracked concrete) stiffness and reduced (cracked concrete) stiffness.

The FE analysis computer program ANSYS (Reference 3.7-21) serves as the platform for three different types of analyses performed to validate the dynamic properties of the R/B complex Dynamic FE Model.

Sets of static analyses are performed on both the Dynamic FE Models and Detailed FE Models by applying 1-g quasi-static acceleration on the models with fixed boundary conditions established at the bottom of the model to calculate nodal displacements and reaction forces. The reaction force results are compared to ensure that the mass assigned to the Dynamic FE Model and Detailed FE Model are similar. For the R/B, the masses assigned to each major floor elevation are also compared in order to check the correlation of the mass distribution in the two models. The global distribution of mass and stiffness of the structure is checked further by comparison of the deflection results from the 1-g static analyses of the two FE models.

The comparison of the results for deflection along the corners of the structures under 1-g quasi-static acceleration in the two horizontal and the vertical directions, respectively, are used to determine if the number of discrete mass degrees of freedom are sufficient to capture accurately the dynamic response of the structures. The check is performed to ensure that the deflection shapes calculated from the analyses of the Dynamic FE Model correlate well with those obtained from the analyses of the Detailed FE Models. The Dynamic FE Model is considered to adequately represent the stiffness and mass distribution if the differences in the displacements results obtained from 1-g static analyses of the Dynamic FE Model and Detailed FE Model are small.

Modal analyses are performed using ANSYS on the Dynamic FE Models and the Detailed FE Models of R/B, CIS, PCCV, East and West PS/Bs, and A/B with fixed conditions at the bottom of the models. The analyses provide the fixed base dynamic properties of the models, such as the natural frequencies, mode shapes, modal mass participation and the total effective mass (mobilized mass) of all of the extracted natural modes of vibration of the structures.

In order to depict the global dynamic response of the structures and determine the dominant frequencies of vibration, the results of the modal analyses of the Dynamic FE Model and Detailed FE Model, the cumulative mass versus frequency are plotted together and compared. The Dynamic FE Model is considered to have sufficient accuracy if the cumulative mass versus frequency plots are consistent with those obtained from the Detailed FE Model. Additionally, the individual models are analyzed for frequencies and

mode shapes up to 100 Hz and the modal data for each direction are extracted. A comparative analysis of the modes between individual buildings of the Detailed FE Model dynamics and the Dynamic FE Model dynamics is performed. Parameters and discussion are provided that demonstrate that the models are dynamically equivalent. MIC-03-03-00054

After the models are developed the mass statistics are extracted and presented. The mode shapes were normalized to mass. Thus there was no direction given to force the maximum displacement of a shape to be positive. Consequently, some plots will show that a mode shape of the Detailed FE Model will appear as a mirror image, i.e., reversal of sign, of the Dynamic FE Model. There is no impact on the results since the signs of the participation factors will also be reversed.

In addition to the 1g static and modal analyses performed above which only provide a global comparison between the Dynamic FE Model and the Detailed FE Model, a series of full harmonic analyses are performed in ANSYS on the Dynamic FE Models and the Detailed FE Models of R/B, CIS, PCCV, East and West PS/Bs, and A/B with fixed condition at the bottom of the models. The harmonic analysis calculates the response of the structure to cyclic loads over a frequency range. To model the fundamental concept of Acceleration Transfer Function (ATF) in SASSI which directly relates the input motion to the structural response, a 1-g ground (global) acceleration is applied in each of the three orthogonal directions, respectively, from which the ATFs at selected locations are derived based on the displacement response at the specified range of frequencies. A constant damping ratio of 5% is applied in all the harmonic analyses.

3.7.2.3.10.1 Validation Method

~~***Static Loading Analysis***~~ DCD_03.07.02-35

~~To verify whether the stick model has stiffness properties that conform to those of the FE model, a static loading analysis is performed as follows:~~

i. ~~A FE model consisting of the portion of the building above the upper level of the basemat, including the walls, columns, and floor slabs, is developed using brick, shell, and beam elements.~~

ii. ~~By fixing the upper level of the basemat, a set of vertically distributed horizontal loads, which is established considering the earthquake excitation, is applied at each of the main floor levels of the FE model and the resulting horizontal displacements are evaluated at the top level of each floor.~~

iii. ~~The same analysis as described above in (ii) is performed on the seismic stick model and the set of vertically distributed horizontal displacements from the stick model analysis is compared with that obtained from the analysis of the FE model.~~

iv. ~~If the difference of displacement distribution between the FE model and the seismic stick model is considered to be large, the stiffness properties of the stick model are adjusted so that the difference between them becomes small, applying reasonable engineering judgment.~~

~~The adjustments to the stiffness properties of the stick model are as follows:~~

- ~~The flange width of the seismic walls of NS direction under the operation floor level is reduced from H/3 to H/6 (H: total height of the wall).~~ DCD_03.07.02-35
- ~~The flange widths of the seismic walls above the operation floor are not taken into account.~~
- ~~The shape factor (=1.2) is taken into account for the seismic walls in the NS direction above the operation floor, except for the fuel handling area of the R/B.~~

~~**Dynamic Analysis**~~

~~To verify whether dynamic properties of the stick model conform to those of the detailed FE model, the 5% damping ISRS are calculated at several arbitrarily selected node points in the lumped mass stick model that represent main floor levels. The ISRS derived for those node points in the lumped-mass stick model are then compared with ISRS developed for the corresponding locations in the FE model.With the exception of the containment internal structure, validation of the dynamic FE models of the R/B complex and the PS/B is performed on the models with full (uncracked concrete) stiffness. In order to ensure that the model with reduced (cracked concrete) stiffness properties can meet the requirement of ISG-01 Section 3.1 (Reference 3.7-54) to accurately capture responses with frequencies up to 50 Hz, the validation of the model with full stiffness properties compares dynamic responses up to frequencies of at least 70 Hz.~~ MIC-03-03-00054

~~The FE analysis computer program ANSYS (Reference 3.7-21) serves as the platform for three different types of analyses performed to validate the dynamic properties of the dynamic FE Modes. The validation steps include:~~

1. ~~1g Static Analysis~~
2. ~~Modal Analysis~~
3. ~~Modal Superposition Time History Analysis~~

Validation of the R/B complex Dynamic FE model is carried out in accordance with the methodology described above. Three sets of analysis using ANSYS are performed for the validation. They are:

- 1g analysis to check and validate element connectivity, mass and stiffness distributions represented by displacement distribution at various locations
- Modal analysis to validate global dynamic properties of the model represented by the fundamental frequencies and associated mode shapes and modal masses
- Harmonic analysis to obtain Acceleration Transfer Function at various locations to validate the global and local dynamic properties of the model.

The R/B complex Dynamic FE Model is divided into six sub-models that represent the portions of R/B, PCCV, CIS, East PS/B, West PS/B, and A/B. The sub-models are

developed by selecting the elements and associated properties within each building footprint area, including the common walls, from the completed R/B complex Dynamic FE Model. The ESWPC is divided into three sections and they are included in the sub-models of the East PS/B, R/B, and West PS/B accordingly. The three sets of validation analyses are performed on each of the sub-models. As a confirmatory validation, the completed R/B complex Dynamic FE Model is also validated by performing 1g static analysis and modal analysis. The PCCV and CIS are free standing structures. Their dynamic properties represented by the corresponding sub-models are similar to their dynamic properties that are represented in the R/B complex model. The four sub-models, R/B, East PS/B, West PS/B and A/B, are more flexible than their corresponding portions within the R/B complex dynamic model when they are integrated. Therefore, validating the sub-models up to a certain frequency results in that the R/B complex model is capable of capturing the response for the frequencies up to that frequency and even higher. MIC-03-03-00054

The six sub-models and the completed R/B complex Dynamic FE Model are checked and validated against the corresponding detailed models. The dynamic models have nominal mesh size of 9 ft and the detailed models have nominal mesh sizes of 4 to 6 ft, 6 ft for the PCCV, 5 ft for the R/B and 4 ft for the remainder of the buildings. The wall and floor slab thickness of the R/B complex structure are generally about 3 to 5 feet. The 4 to 6 ft mesh size is the reasonable lower limit for finite element simulations to capture the dynamic properties of the R/B complex structures. Comparison of the dynamic properties of the dynamic models with the detailed models therefore is considered as a "convergence study" of the dynamic properties in terms of mesh size selection as required by SRP 3.7.2, stated as "the element size should be selected on the basis that further refinement has only a negligible effect on the solution results."

The validation results presented in Section 02.5 of MUAP-10006 (Reference 3.7-48) conclude that the dynamic model and detailed model represent approximately the same mass and stiffness distribution, the same dynamic properties in terms of the fundamental frequencies and associated mode shapes and modal masses, and comparable ATFs at various locations. Therefore the R/B complex Dynamic FE Model adequately represents the building mass stiffness and dynamic properties for soil-structure interaction analysis.

~~3.7.2.3.10.2 R/B~~

~~In order to reduce the computational effort and speed up the validation, the R/B complex dynamic FE model is divided into three parts: R/B-FH/A (including common basemat), containment internal structure coupled with RCL, and PCCV. A series of fixed-base analyses are performed on the three separated models using ANSYS and the results are compared to the ones obtained from corresponding analyses on the detailed FE models of the R/B, containment internal structure and PCCV structures. The stiffness and mass attributes that are assigned to the dynamic FE model and detailed FE models are consistent.~~ DCD_03.07.02-35

~~The results of the R/B dynamic FE model validation are as follows:~~

i. ~~Fixed-base FE model~~

~~Figure 3.7.2-9 shows the fixed-base FE model for the R/B, which is compared with the three-dimensional stick model. The response of the stick model is compared with that of the FE model to account for the effect of a spatially extended structure. The torsional and rocking motion is included in both models.1g Static Analysis – The total weight and displacement results for the model indicates that the dynamic FE model stiffness is comparable to that of the detailed model.~~ MIC-03-03-00054

ii. ~~Rigidity estimation by static analysis~~

~~Comparisons of static deformations are made between the three-dimensional stick model and the FE model, as previously discussed.Modal Analysis - Fixed-base modal analyses using ANSYS are performed on both the detailed and dynamic FE models. The results of the modal analysis indicate that the dynamic FE model adequately captures the dynamic properties of the detailed FE model.~~

iii. ~~Comparison of ISRS~~

~~Comparisons of ISRS are made between the three-dimensional stick model and the FE model at various points in various elevations, as previously discussed.Modal Superposition Time History Analysis - Fixed base dynamic time history analyses using modal superposition are performed on both dynamic and detailed FE models. ARS with 5% damping are generated for each model at various locations. The ARS indicate that the dynamic R/B model properly captures the structure response to dynamic loads in all directions.~~

~~3.7.2.3.10.3 Containment Internal Structure~~

~~Due to the complexity of the containment internal structure, different stiffness and damping values are assigned to different types of structural components for the two bounding stiffness and damping conditions. As described in Subsection 3.8.3.4, the reduction of stiffness applied to the containment internal structure to account for cracking of the concrete of SC modules, reinforced concrete slabs and massive concrete portions, is non-uniform. Therefore, unlike the other US-APWR standard plant seismic Category I structures, two sets of validation analyses are performed for the containment~~ DCD_03.07.02-35

~~internal structure to ensure the adequacy of the containment internal structure dynamic FE model with full (uncracked concrete) stiffness and reduced (cracked concrete) stiffness.~~

~~The results of the containment internal structure dynamic FE model validation are as follows:~~

i. ~~Fixed-base FE model~~

~~Figure 3.7.2-10 shows the fixed-base FE model for the containment internal structure, which is compared with the three-dimensional stick model. To verify the three-dimensional stick model, the FE model is used to estimate its rigidity by both static and dynamic analyses.1g Static Analysis – The total weight and displacement results for the cracked and uncracked models indicates that the dynamic FE model stiffness is comparable to that of the detailed model.~~

ii. ~~Rigidity estimation by static analysis~~ MIC-03-03-00054

~~Comparisons of static deformations are made between the three-dimensional stick model and the FE model, as previously discussed.Modal Analysis - Fixed base modal analyses using ANSYS are performed on both the detailed and dynamic FE models. The results of the modal analysis for cracked and uncracked concrete indicate that the dynamic FE model adequately captures the dynamic properties of the detailed FE model.~~

iii. ~~Comparison of ISRS~~

~~Comparisons of ISRS are made between the three-dimensional stick model and the FE model at various points in various elevations as previously discussed.Modal Superposition Time History Analysis - Fixed base dynamic time history analyses using modal superposition for both the cracked and uncracked version are performed on both dynamic and detailed FE models. ARS with 5% damping are generated for each model at various locations. The ARS indicate that the dynamic containment internal structure model properly captures the structure response to dynamic loads in all directions.~~

~~3.7.2.3.10.4 PCCV~~

~~A series of fixed-base analyses are performed on the PCCV dynamic FE model using ANSYS and the results are compared to the ones obtained from corresponding analyses on the detailed FE models of the PCCV structures. The stiffness and mass attributes that are assigned to the dynamic FE model and detailed FE models are consistent.~~ DCD_03.07.02-35

~~The results of the PCCV dynamic FE model validation are as follows:~~

i. ~~Fixed-base FE model~~

~~Figure 3.7.2-1 shows the fixed-base FE model for the PCCV, which is compared with the three-dimensional stick model of the superstructure. To verify the three-dimensional stick model, the FE model is used to estimate its rigidity by both static and dynamic analyses.1g Static Analysis – The total weight and displacement results for the uncracked models indicates that the dynamic FE model stiffness is comparable to that of the detailed model.~~

ii. ~~Rigidity estimation by static analysis~~

~~Comparisons of static deformations are made between the three-dimensional stick model and the FE model, following the general approach discussed above.Modal Analysis - Fixed base modal analyses using ANSYS are performed on both the detailed and dynamic FE models. The results of the modal analysis indicate that the dynamic FE model adequately captures the dynamic properties of the detailed FE model.~~

iii. ~~Comparison of ISRS~~

~~Comparisons of ISRS are made between the three-dimensional stick model and the FE model at various nodes.Modal Superposition Time History Analysis - Fixed base dynamic time history analyses using modal superposition are performed on both dynamic and detailed FE models. ARS with 5% damping are generated for each model at various locations. The ARS indicate that the dynamic PCCV model properly captures the structure response to dynamic loads in all directions.~~ MIC-03-03-00054

iv. ~~Modal analysis~~

~~A modal analysis is performed for the three-dimensional finite element model and the SASSI stick model to compare the structure's dominant natural frequencies.~~

~~3.7.2.3.10.5 PS/Bs~~

~~As previously discussed, the PS/B seismic model is developed from a detailed FE model initially prepared for the detailed static analysis. The PS/Bs seismic model is validated by performing various static and dynamic structural analyses on both the static and dynamic models considering a "fixed base condition," i.e., full constraints on translation and rotation at the bottom of the basemat. The analyses include a 1g static analysis performed to confirm static load/deformation distributions and dynamic properties, modal analysis, and mode-superposition transient dynamic analysis using the corresponding ANSYS Solver. Transfer functions and ISRS obtained from the SASSI validation analyses at selected nodes are compared to the results obtained from the modal and modal superposition time history analyses on the detailed static model to ensure that SASSI model is accurately representing dynamic properties of the PS/B.The development of the PS/B dynamic FE model is based on a number of adjustments in geometry and load configurations in order to minimize the size of the model and make the model suitable for SSI analysis. The validation ensures that these modeling assumptions and simplifications do not affect the ability of the dynamic FE model to accurately represent the dynamic response of the PS/B structure as mandated by SRP Sections 3.7.2.II.1 and 3.7.2.II.3 (Reference 3.7-16) and of ISG-01 Section 3.1 (Reference 3.7-54).~~ DCD_03.07.02-35

~~A series of fixed-base analyses are performed on both the PS/B dynamic FE and detailed FE models using ANSYS and the results are compared. The stiffness and mass attributes assigned to the PS/B dynamic FE model with full stiffness and reduced stiffness are consistent. Therefore, the validation of the PS/B dynamic FE model is performed with full (uncracked concrete) stiffness. In order to ensure that the model with reduced (cracked concrete) stiffness properties can meet the requirement of ISG-01 Section 3.1 (Reference 3.7-54) to accurately capture responses with frequencies up to 50 Hz, the validation of the dynamic model with full stiffness properties compares dynamic responses up to frequencies of at least 70 Hz.~~

~~The three different types of analyses performed to validate the dynamic properties of PS/B dynamic FE model are the same as those used in the validation of the R/B complex dynamic FE model. They consist of a:~~

1. ~~1g Static Analysis~~

2. ~~Modal Analysis~~

3. ~~Modal Superposition Time History Analysis~~ MIC-03-03-00054

~~The results of the PS/B dynamic FE model are as follows:~~

i. ~~1g Static Analysis – The total weight and displacement results for the model indicates that the dynamic FE model stiffness is comparable to that of the detailed model.~~

ii. ~~Modal Analysis - Fixed base modal analyses using ANSYS are performed on both the detailed and dynamic FE model. The results of the modal analysis indicate that the dynamic FE model adequately captures the dynamic properties of the detailed FE model.~~

iii. ~~Modal Superposition Time History Analysis - Fixed base dynamic time history analyses using modal superposition are performed on both dynamic and detailed FE models. ARSs with 5% damping are generated for each model at various locations. The ARS indicate that the dynamic PS/B model properly captures the structure response to dynamic loads in all directions.~~

~~3.7.2.3.11 Equivalent Masses due to Dead and Live Loads~~

~~In the design of seismic category I and seismic category II buildings and structures, dead loads and various portions of live loads are treated as equivalent masses for consideration in the global seismic analysis models. For example, 25% of the design floor live loads during normal operation (ASCE 7, Subsection 12.7.2 [Reference 3.7-24]) and 75% of the roof snow load, whichever is applicable depending on the specific location in the building or structure, have been considered in computing tributary mass at node points in the seismic modelsthe models. Each load is distributed over a representative floor area and the density of the elements in that area is increased such that the total increase in dynamic mass matches the applied load magnitude. This is consistent with SRP 3.7.2, Section II.3(dD) (Reference 3.7-16). For the containment operating deck in the PCCV, the design floor live load for maintenance and refueling is 950 lb/ft^2 and the floor live load for normal operation is 200 lb/ft^2. Therefore, 50 lb/ft^2 (25% of 200 lb/ft^2) has been used as an equivalent live load (mass) for the seismic analysis models.~~ DCD_03.07.02-35

~~Equivalent dead loads used in the seismic analysis models also include the weight of SSCs not specifically identified or included as dead loads in the models such as the weight of minor piping systems, cables and cable trays, ducts, and all related supports. Similarly, equivalent live loads include fluid contained within the minor piping and equipment under operating conditions. The weight of permanently attached tanks (uniformly distributed over the room floor area) is included as equivalent dead load (mass) in the seismic models. For the seismic analysis models, an equivalent dead load of a minimum of 50 lb/ft^2 uniform load is applied to cover these conditions. This is consistent with SRP 3.7.2, Section II(3)(d) (Reference 3.7-16).~~

~~For floors with a significant number of small pieces of equipment (e.g., electrical cabinet rooms), their total weight divided by the floor area that effectively supports the equipment within the room, plus an additional 50 lb/ft^2, is used as the equivalent dead load.~~

~~The equivalent dead loads (mass) are appropriately increased in areas such as main piping corridors, and cable tray and HVAC ductwork runs where such loads exceed the value of 50 lb/ft^2.~~ MIC-03-03-00054 DCD_03.07.02-35

~~Equipment load also includes a 60 psf miscellaneous pipe load which is applied on all walls and slabs on the R/B model, with the exception of a few locations where a pipe load of 100 psf is used instead.~~

~~The above process is not applicable for the nuclear steam supply system and major piping that constitutes the RCL. The RCL dynamic mass is included directly in the RCL lumped-mass-stick model provided in Reference 3.7-49.~~

~~In the case of the R/B, the equivalent dynamic mass is applied to the dynamic FE model in two steps. First, a mass density equal to the sum of the structural self-weight and pipe load is calculated and assigned to each of the shell elements modeling the R/B walls and slabs.~~

~~The remaining loads are applied as either additional mass densities on slab shell elements or concentrated lumped masses on wall and slab key points. The density and thickness of the elements are further modified to account for stiffness reductions due to openings and cracking, but it is done is such a way as to not change the mass of the elements.~~ MIC-03-03-00066

~~Liquid masses contained are applied as impulsive mass to walls and slabs. The direction of the mass is perpendicular to the surface of the walls or slabs.~~

~~The elastic modulus and thickness of shell elements are adjusted to accurately model the reduction of shear stiffness of the wall due to openings. The density of shell elements is also adjusted to accurately represent the mass of the wall accounting for openings and the adjusted wall thickness.~~

3.7.2.4 Soil-Structure Interaction

~~In accordance with the requirements of SRP 3.7.2, Section II.4 (Reference 3.7-16), SSI effects are considered in the seismic response analysis of all major seismic category I and seismic category II buildings and structures that are part of the US-APWR standard and non-standard plant. The SSI analyses use seismic models that are described above in Section 3.7.2.3 to represent the dynamic properties of the structures. The ACS SASSI (Reference 3.7-17) PS/Bs model and enhanced R/B lumped mass stick model, combined in ACS SASSI with the FE model of the basement, provide adequate degrees of freedom to ensure that the modeling requirements of SRP 3.7.2, Section II A (iv) are met, and that the seismic response in the high frequency range is captured. An FE structural model for SSI analyses of the PS/B configuration has been developed and converted into ACS SASSI format. Similarly, an FE model has been developed representing the dynamic properties of the R/B complex, consisting of the PCCV, containment internal structure, the R/B, and the common foundation basemat. To account for the effects of dynamic coupling of the containment internal structure with the equipment and the piping, the dynamic FE model of the R/B complex also includes a model of the RCL, representing the stiffness and mass inertia properties of the major equipment and piping located in the PCCV. Technical Report MUAP-10001 (Reference 3.7-47) presents the dynamic FE structural~~ MIC-03-03-00054 DCD_03.07.02-35

~~models and results of the validation analyses performed to demonstrate their ability to accurately represent the dynamic properties of the PS/B and R/B complex at all important modes of vibration.~~ DCD_03.07.02-35 MIC-03-03-00054

~~Two different levels of stiffness and damping properties are assigned to the dynamic FE models of the R/B complex and the PS/B in order to capture the possible variations of the material properties of the structural members due to cracking. The first level represents the full (uncracked concrete) stiffness of the structures corresponding to low stress levels, and the second level represents the reduced stiffness (cracked concrete) of the structures corresponding to high stress levels. The OBE damping values are assigned to the models with full (uncracked concrete) stiffness in order to model the lower level of dissipation of energy in the structure when subjected to lower stresses. The SSE damping values are assigned to the models with reduced (cracked) concrete properties in order to model the higher dissipation of energy in the structure when subjected to high stresses.~~

~~For each soil case considered, two sets of SSI analyses are performed to obtain the seismic response of the R/B complex and the PS/B, corresponding to each of the two bounding levels of stiffness and damping. The responses obtained from the two SSI analyses for the two different stress levels are enveloped in order to address the possible range of effects of concrete cracking on the seismic response of the buildings. The CSDRS compatible ground motion time histories, the generic layered profiles, and the structural dynamic models define the input for the SSI analyses of the US-APWR standard plant seismic Category I structures. Refer to Technical Report MUAP-10006, "Soil-Structure Interaction Analyses and Results for the US-APWR Standard Plant" (Reference 3.7-48), for the results of these SSI analyses.For the R/B complex and PS/B model, site-specific SSI analyses are also performed using the computer program ACS SASSI in order to confirm that site-specific effects are enveloped by the site-independent analyses.~~

~~The site-independent SSI analyses take into account the flexibility of the basemat, frequency-dependence of the SSI impedance, layering of the subgrade, elevation of the water table, and scattering of the input motion. The SSI analyses are performed with ACS SASSI (Reference 3.7-17) in the frequency domain utilizing the substructuring technique and complex stiffness representation of stiffness and damping properties of the structures and the subgrade. The subgrade media and SSI system damping to model the dissipation of energy due to material damping of the structural members and the soil are also discussed in Subsections 3.7.1.3 and 3.7.1.2. The response of the system at selected frequencies of analyses is obtained as the solution of a set of complex algebraic equations. The frequencies of analyses are selected to accurately capture the response of the structure at all important frequency ranges. The amplitudes of the interpolated transfer functions are plotted and investigated to ensure the accuracy of the interpolation of the response for the required range of frequencies.Plots of transfer functions for various locations throughout the R/B complex and PS/B are presented in Appendix L of Technical Report MUAP-10006 (Reference 3.7-48). Approaches and methods used for the SSI analyses are discussed further in Technical Report MUAP-10001 (Reference 3.7-47).~~ DCD_03.07.02-103

~~Table 3.7.2-3 provides the percent of stiffness reduction and damping values used for the different structural components for the site-independent SSI analyses of US-APWR R/B complex and PS/B.~~ DCD_03.07.02-35 MIC-03-03-00054

~~The ratio of basemat depth-to-equivalent-radius for the R/B-PCCV basemat is approximately 0.27. ASCE 4-98 Subsection 3.3.4.2 (Reference 3.7-9) considers that a basemat depth-to-equivalent-radius ratio of less than 0.3 is an indication of a shallow embedment foundation, for which effects of the embedment on the seismic response of the building are generally not significant. SSI analysis performed as part of the site-independent US-APWR standard plant design neglects the effects of embedment of the common R/B and PCCV basemat. Therefore, the R/B-PCCV seismic models are not coupled with any subgrade or backfill material at the sides of the basemat or along the faces of below-grade exterior walls, and no credit is taken in the seismic analysis for reduction in amplitude of the response due to foundation embedment in the subgrade or backfill materials. Embedment effects, including shifts in the structural frequencies, are considered to be small enough to be enveloped by the variations of subgrade stiffness considered in the standard design seismic response analyses of a surface foundation. However, the effects of the embedment are required to be analyzed on a site-specific basis as discussed in Subsection 3.7.2.4.1 to confirm suitability of design. Technical Report MUAP-11007 (Reference 3.7-52) describes the methodologies used to perform US-APWR Standard Plant SSI analyses. This approach performs studies to assess embedment depth and groundwater level effects on the R/B complex and the PS/B structural seismic responses and to confirm SSI analysis validity and conservatism. These studies use existing dynamic FE and lumped mass stick models. Sensitivity studies are performed with the R/B complex dynamic FE model and the PS/B dynamic FE model and assume full stiffness and OBE related damping.~~ DCD_03.07.02-35

~~*Effects of Groundwater Level* - The US-APWR Standard Plant design assumes groundwater table level to be one foot below finished grade. The water level evaluation uses sensitivity studies of groundwater table fluctuation effects on R/B complex and PS/B seismic response. Technical Report MUAP-10001 (Reference 3.7-47) describes site-independent SSI analyses used for standard plant seismic Category I structures based on the eight generic soil cases. These generic soil cases model both saturated and unsaturated (dry) soil conditions. SSI analyses evaluate the R/B complex and PS/B as surface mounted structures that rest on each of the saturated and unsaturated soil cases. Evaluation of water table fluctuations compare saturated soil versus unsaturated soil case results at multiple soil profile depths.~~

~~*Results of Groundwater Level Studies* - Sensitivity studies of groundwater level effects generate strain compatible soil properties for selected soil cases and ARS at selected locations. The ARS obtained from analyses of unsaturated soil profiles is compared to the ISRS and broadened envelope of the ARS for the two generic saturated soil profiles. These comparisons are made at ground floor and roof levels of the subject structures, and at other selected locations such as RV supports.~~

~~*Effects of Embedment* - Structural embedment studies are performed to assess embedment effects on R/B complex seismic response. US-APWR standard plant seismic design is based on enveloped seismic response parameters generated from seismic SSI analyses of major plant structures surface-founded on eight generic layered site shear-~~

~~wave-velocity soil cases. In the standard design, major plant structures are embedded to a depth of approximately 40 ft. Structural embedment studies in this methodology explore the following effects of variations in surrounding soil media:~~ DCD_03.07.02-35 MIC-03-03-00054

(1) ~~changes in frequency-dependent foundation impedances;~~

(b) ~~changes in free-field seismic input motion at the base due to overburden soil layers above the base; and~~

(c) ~~effect on structure seismic response due to seimic response of the free-field side soil layer~~

~~The effects from (a), described above, can result in shifts in the SSI system frequencies and changes in associated SSI system damping values. The effect from (b) above can result in changes in the free-field seismic input motion to the SSI system. This can be examined by comparing the response spectra for the free-field "outcrop" seismic input motion with the response spectra for the free-field "in-layer" seismic response motion at the depth of the basemat base. The effect from (c) above can result in variations in the SSI system seismic response transfer functions reflecting the dynamic response characteristics of the side soil layer.~~

~~*Results of Embedment Studies* - Embedment study results are used to generate the following:~~

1) ~~PCCV and Reactor Pressure Vessel 5%-damped ISRS, used for comparison with surface-founded structure ISRS envelopes to determine whether such envelopes bound ISRS calculated using embedded foundations;~~

2) ~~dynamic soil pressures on exterior basemat walls, used to confirm conservatism represented in structural design; and~~

3) ~~global seismic response force and moment time histories at the bottom of the R/B stick model, used for seismic stability analyses of the R/B complex.~~

~~The SSI analyses are performed as previously described by analyzing the seismic models in ACS SASSI (Reference 3.7-17) in conjunction with the generic soil profiles with depths to rock as discussed in Subsection 3.7.1.3, including a profile which simulates a site with hard rock at a depth of 100 ft. The profiles have dynamic properties that are strain-compatible with the CSDRS input ground motion as discussed in Subsection 3.7.1.3, and the input ground motion time histories are applied separately in each direction in the SASSI analyses.~~

~~The site-specific SSI analyses take into account site-specific conditions such as soil layering, location of water table and embedment of the basemat and, thus, validate the results of the site-independent SSI analysis and assumptions contained in the US-APWR standard plant design. This is accomplished through site-specific SSI analysis as explained belowin Subsection 3.7.2.4.1.~~ DCD_03.07.02-35

~~*Structure-Soil-Structure Interaction* - Technical Report MUAP-11011 (Reference 3.7-53) describes coupled dynamic Structure-Soil-Structure Interaction (SSSI) analysis methodology. SSSI analysis evaluates dynamic coupling of seismic ground motion response between adjacent structures. US-APWR SSSI is performed based on eight generic soil cases and uses a combined ACS SASSI dynamic model to capture SSSI effects. Seismic Category I buildings include the R/B complex and the East and West PS/Bs. Seismic Category II buildings included in the standard plant design are the T/B and the A/B.~~ DCD_03.07.02-35 MIC-03-03-00054

~~US-APWR seismic Category I structures are separated from seismic Category II structures by gaps of at least four inches and seismic interactions between structures occur only through building foundations and their supporting soil (i.e., structure-to-soil-to-structure interaction). The most significant SSSI effects tend to be those exerted by larger, heavier structures upon smaller, lighter adjacent structures. For the standard plant, the largest, heaviest structure is the R/B complex and the smallest, lightest adjacent structures are the PS/B structures and the A/B. Initial SSSI effects are assessed between the R/B complex and the West PS/B, and between the R/B complex and the A/B. Where initial assessments indicate that SSSI effects are significant, SSSI effects among the R/B complex, T/B, and East PS/B are also assessed. Significance criteria are specified in MUAP-11011 (Reference 3.7-53).~~

~~R/B-West PS/B-A/B SSSI analyses are performed to assess the effect on ISRS used in design of seismic Category I and II systems, components, and equipment located in the West PS/B. SSSI analyses use the PS/B dynamic FE model that is used in site-independent SSI analyses to calculate seismic responses for that structure. To keep SSSI model size manageable, the R/B complex and A/B lumped mass stick model are coupled through soil to the PS/B dynamic FE model, to represent the global dynamic properties of adjacent buildings. R/B complex, PS/B and A/B SSSI analyses use surface-founded models that do not credit foundation embedment effects.~~

~~SSSI study initially focuses on generic soil cases that represent softer subgrade conditions, which are most likely to have significant SSSI. Results of site-independent PS/B SSI analyses are reviewed to determine the limiting structural stiffness level, which then becomes the PS/B stiffness used in the combined SSSI model. SSSI analyses use R/B complex and A/B stiffness properties that correspond to their respective generic soil cases. Reduced structural stiffness properties corresponding to cracked concrete and full stiffness properties corresponding to uncracked concrete are used in the SSSI analyses. Two SSSI effects are assessed; kinematic SSSI effect and coupled dynamic SSSI effect. Kinematic SSSI effect is used to determine ground motion at adjacent structure foundations in order to assess the effect of a stand-alone heavy building on the ground motions of adjacent structures. Coupled dynamic SSSI effect is determined by a coupled SSSI model formed as an integrated dynamic system to generate coupled seismic responses of adjacent structures, taking into account their mutual interaction through the supporting subgrade.~~

~~Results of kinematic and dynamic SSSI assessments become the basis for identification of generic soil cases for which SSSI effects are significant. PS/B responses obtained from the soil case analyses that indicate significant SSSI effects are enveloped with those of the stand-alone PS/B model SSI analyses. Enveloping ISRS loads from SSSI analysis~~

~~results and stand-alone structure SSI analysis results become the revised seismic design basis loads for considering SSSI effects.~~ DCD_03.07.02-35 MIC-03-03-00054

~~Technical Report MUAP-11006 (Reference 3.7-51) describes methodology used to develop and validate the R/B complex lumped mass stick model. The R/B complex model includes lumped mass stick model representations of the R/B, PCCV, containment internal structure coupled with the RCL, and a three dimensional FE model of the R/B complex basemat. The model provides a representation of the global dynamic properties of the R/B complex structures and ensures that their seismic responses are captured in the major modes of vibration. Two levels of stiffness and damping properties are assigned to the dynamic FE models of the R/B complex in order to capture the possible material property variations of structural members due to cracking. The model uses the complex formulation found in ACS SASSI (Reference 3.7-17) to simulate dissipation of energy due to material damping of structural members and soil. A set of validation analyses are performed to demonstrate the ability of the R/B complex lumped mass stick model to adequately represent dynamic properties of the structures. See Appendix 3H for a discussion of the lumped mass stick model methodology.~~

~~A set of site-independent SSI analyses are performed on the lumped mass stick model, and results are used to calculate base reactions that serve as input for evaluation of seismic stability of the R/B complex foundation. The model is also used in studies performed to determine the effect of embedment on seismic response and assess SSSI of the PS/B. The model enables incorporation of building seismic response effects due to frequency dependence of SSI impedance, subgrade layering and variation in water table elevation. Soil material damping values in the SSI analyses are based on those associated with the generic soil profiles discussed in Section 5.2 of Technical Report MUAP-10001 (Reference 3.7-47), and do not exceed 15%, in accordance with SRP 3.7.1.~~

~~The methodology for computing dynamic response of containment internal structures subjected to seismic loading while accounting for SSI effects are described in this section. The SSI analysis considers concrete cracking effects, including component stiffness and damping. The three-dimensional (3-D) linear elastic FE model of the RB complex includes the containment internal structures and soil foundations. The FE model stiffness and damping values are based on its stiffness before or after concrete cracking, as applicable. As an example, for seismic plus operating-thermal loading conditions, the concrete is expected to be mostly uncracked. For seismic plus accident-thermal loading conditions, the concrete is expected to be cracked in SC walls as well as the reinforced concrete slabs.~~

~~Perform two SSI analyses to bound the range of stress levels and associated cracking anticipated for the containment internal structures; one using the higher stiffness associated with seismic plus operating-thermal loading conditions, and one using the lower stiffness associated with seismic plus accident-thermal loading conditions.~~

~~The results of the SSI analysis are used to develop the following:~~

i. ~~ISRS for equipment and attachments~~

ii. ~~acceleration plots~~

iii. ~~translational ARS in the three orthogonal directions at the base of the containment internal structures. These results represent the envelope of responses obtained by conducting dynamic analyses of the linear elastic FE models with the two stiffness and damping levels described above.~~ DCD_03.07.02-35 MIC-03-03-00054

iv. ~~Compute response spectra acceleration as described in Subsection 3.7.2.~~

The seismic design of US-APWR standard plant is based on responses obtained from the site-independent SSI analysis of the R/B complex structures and the SSSI analyses for the R/B complex with the Turbine Building (T/B).

These two sets of analyses are performed for the R/B complex to obtain seismic responses for design. These seismic responses are the In Structure Response Spectra (ISRS) at various locations in the R/B complex, seismic loads indicated by maximum nodal accelerations, resulting floor shear loads and displacements of various locations relative to center of the basemat.

The Figures in Section 1.2 show the US-APWR Power Block which includes the R/B complex and the T/B. Figure 3.7.2-17 presents an elevation view of the foundation layout between the R/B complex and the T/B. The T/B is located on the south side of the R/B complex and the clear distance between two foundations is approximately 20 ft. The distance between the two foundations and the comparable stiffness and masses of the two buildings necessitate the investigation of effects of the presence of the T/B on the seismic response of the R/B complex. The effects on the response of the R/B complex structures due to the much smaller and lighter buildings and foundations are considered negligible. Therefore, the SSSI analysis is performed on the combined models of the R/B complex and T/B to investigate the dynamic coupling effects.

The foundation is modeled in SASSI (Reference 3.7-17) as embedded SSI and SSSI models of the soil structural system. Solid elements connecting free field soils and the basement structural elements are used in the models to directly include the near field backfills. In addition, backfills at the top of the ESWPC and in the space between the basements of the R/B complex and T/B are also modeled in the SSI and SSSI models as solid elements. Elastic properties defined by strain compatible S-wave and P-wave velocities and damping values are assigned to the near field backfill elements that are representative of strain compatible properties of typical backfill materials. Two layers of soil are modeled as solid elements below the T/B foundation in order to match the bottom of the R/B complex foundation.

Both the SSI and the SSSI analysis consider models with two levels of structural stiffness (cracked anduncracked).See Section 3.7.2.3.5 for discussion of the cracked and uncracked modeling approach.

The SSI analysis is performed for the six generic layered soil profiles developed in Section 3.7.1.3: 270-200, 270-500, 560-500, 900-100, 900-200 and 2032-100 and the SSSI analysis is performed for four selected generic layered soil profiles: 270-200, 560-500, 900-100 and 900-200. The 2032-100 soil profile was not included since it is a hard rock soil profile and the SSSI effects are minimal compared to the softer soil profiles. The 270-500 soil profile was not included because responses are on the same order as the

ones from 270-200 soil profile and due to higher radiation damping, generally lower. Therefore soil profiles 270-500 and 2032-100 are not analyzed for the SSSI effects. MIC-03-03-00054

A total of twelve cases combining two stiffness levels and six soil profiles are performed in the SSI analysis. A total of eight cases combining two stiffness levels with the selected four soil profiles are analyzed to consider SSSI effects in the seismic design of the R/B complex.

3.7.2.4.1 Dynamic Soil Properties

Section 3.7.1.3 describes the development of six generic soil profiles.The site models in the SASSI analyses use infinite horizontal layers (referred to as fixed layers whose depths vary with the soil profiles) to represent the approximately 1000 feet of the top soils. An additional 10 layers, referred to as variable layer, represents a half space of visco-elastic medium. For the same soil profile, the total thickness of variable depth layer varies with the frequency analyzed and is determined as $1.5V_S/f$, where V_S is shear wave velocity of the half space and f, in Hz, is the frequency of analysis.

Two sets of granular backfill properties are used to develop the elastic properties and damping values for the backfill elements. In order to cover a wide range of soil-structure frequencies, a backfill with relatively soft properties, represented by lower V_S values, is used in conjunction with the softer soil cases (270-200, 270-500 and 560-500). A backfill with relatively stiff properties is used in conjunction with the harder rock cases (900-100, 900-200, and 2032-100).Additional detail regarding the soil profiles is provided in MUAP-10006 (Reference 3.7-48).

3.7.2.4.2 Input Motions- Within Motions

Section 3.7.1.1 provides a set of three acceleration time histories (H1, H2, and V). In the SSI and SSSI analyses, the H1, H2, and V acceleration time histories are used to derive the input control motions in Standard Plant North-South (NS), East-West (EW) and Vertical direction, respectively.

The CSDRS is developed as a free field outcrop motions at the bottom level of the R/B complex foundation basemat. For the SSI and SSSI analyses of the embedded model, "within" (inlayer) motions at the base of the foundation are needed as input control motions for each of the soil profiles that are analyzed. Site response analyses are performed, using outcrop time histories H1, H2 and V as input, to develop the within motions for SSI and SSSI analyses.

The within motion at foundation level, at depth of 42.25 ft, is obtained in terms of acceleration time histories and 5% damped acceleration response spectrum for each analysis. The resulting time histories of within motions are used as input control motions for the SSI and SSSI analyses performed for corresponding soil profile and earthquake excitation for each corresponding direction. A total of six sets of within motions are generated for the six soil profiles, one set for each profile. Each set of within motion includes two horizontal and one vertical motion.

3.7.2.4.3 Structure-Soil-Structure Interaction (SSSI Model)

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SSSI analysis is performed to determine the influence of adjacent or nearby structures on the seismic Category I structures. The T/B is the only large structure near the R/B complex. To perform site-independent SSSI analyses, the dynamic FE model of the R/B complex used for SSI analyses is combined with the T/B FE model to create a single dynamic FE model. The relative locations of the two structures within the SSSI model are shown in Figure 3.7.2-17. Backfill is modeled in the surrounding areas of each building and in the gap between the two buildings below plant grade.

The T/B FE model accounts for two steel structures, namely the T/B and the Electrical Room. The two structures are supported by a common basemat. The steel columns and beams are modeled using BEAM elements. The foundation walls and base slabs are modeled with SHELL elements. The filled concrete parts are modeled with SOLID elements. The T/B foundation basemat has a large rectangular portion around the center that is filled by the Pedestal foundation, which is also modeled with SHELL elements.

In order to create an integrated R/B complex-T/B SSSI model, the foundation part of the T/B FE model is meshed so that the excavated soil layer interfaces for the R/B complex and T/B have the same nodal Z-coordinates, i.e., the T/B FE model basement and excavated volume meshes match the top five embedded layers of the R/B complex FE model. The bottom foundation level of the T/B is set at elevation -26.33 ft. Compared to the actual foundation bottom elevation of -24.58 ft the geometry adjustments have negligible effects on the T/B dynamic behavior.

Two layers of solid elements are added in the model below the T/B foundation to simulate the free field soils in order to create an excavated volume for the SSSI model with a level bottom.The corresponding free field soil strain compatible properties are assigned to the two layers of soil elements.

The excavated volume vertical meshes of the T/B portion match the R/B portion. In the T/B portion, the excavated volume has a nominal horizontal mesh size of 13 ft. The purpose of SSSI analysis is to investigate the effect of the presence of the T/B on the R/B complex. The larger horizontal mesh size in the T/B portion is acceptable.

As stated previously, SSSI analyses are performed for soil profiles 270-200, 560-500, 900-200, and 900-100. The results indicate that for the same soil profiles, the presence of the T/B has negligible effects on the seismic response of the R/B complex. Generally, for the locations in R/B complex remote from the T/B, SSSI effect tends to slightly reduce the amplitude of the response to input from NS direction which is parallel to the direction that is defined by the two foundations. The responses to the EW direction input are practically the same for the SSI and SSSI analyses. This observation also applies to the vertical response. See MUAP-10006 (Reference 3.7-48) for additional discussion about the SSSI analysis.

Based on the findings, it can be concluded that the R/B complex will not be affected by SSSI effects from the Access Building or Tank House. Unlike the T/B, with size and weight comparable to the R/B complex, these two buildings are too small and light to have any significant effect on the response of the much heavier and larger R/B complex.

The SSSI effects on ISRS are conservatively considered in the standard design by enveloping all twenty cases, i.e., the twelve cases for SSI and eight cases for SSSI. MIC-03-03-00054

3.7.2.4.4 Summary of the Site Independent SSI Analysis of US-APWR Standard Plant

The seismic analyses of the R/B complex structures considers the following effects:

- Concrete cracking and associated stiffness variation through the consideration of two bounding stiffness levels for the structures;
- Flexibility of the foundation and basement by using FE models;
- Layering of the subgrade by using layered generic soil profiles;
- Embedment by directly analyzing the structures as embedded structures;
- SSSI effects by performing the seismic coupling analysis of the structure soil structure system.

The analyses are performed using SASSI. Therefore, the frequency dependent impedance of foundation soils is considered as well. The CSDRS is identified as an outcrop motion in the free field at the R/B complex foundation level. The corresponding within (inlayer) motions at the foundation level are used as input control motion for the analyses. A set of three statistically independent artificial time histories representing the input ground motion for the three orthogonal directions is used to derive the within motions.

The results of the SSI and SSSI and analyses are used for development of the following seismic basis parameters for the structural design of the R/B complex:

- ISRS that are the input for design and seismic evaluation of SSCs and equipment in the R/B complex. The ISRS are developed by enveloping and broadening the results of the SSI and SSSI analyses.
- Maximum nodal accelerations and average floor accelerations. The information is used to develop seismic loads for equivalent-static analysis and design of the structures and then provide seismic demands for design of the R/B complex structures. The results from R/B complex SSI analyses are considered for this purpose.
- Maximum relative displacements that are used as input for evaluation of the adequacy of the gaps between the CIS and PCCV, PCCV and R/B, as well as the R/B complex and the T/B. The results from R/B complex SSI analyses are considered for this purpose.
- The R/B complex SSI analyses also provide artificial time histories of the seismic response of the R/B complex structure at each nodal point. These are used as input for the evaluation of overturning and bearing pressure of the R/B complex.

These results are presented in Part 03 of MUAP-10006 (Reference 3.7-48). MIC-03-03-00054

The SSI and SSSI analyses are performed for the generic soil profiles that consider full saturated soil conditions. MUAP-11007 (Reference 3.7-52) presents the evaluation of the significance of water table effects for seismic standard plant design basis.

3.7.2.4.5 Requirements for Site-Specific SSI Analysis of US-APWR Standard Plant

The COL Applicant referencing the US-APWR standard design is required to perform a site-specific SSI analysis for the R/B complex ~~PCCV containment internal structure, and PS/B model,~~ utilizing a SASSI program such as ~~the program~~ ACS SASSI ~~SSI~~ (Reference 3.7-17) which contains time history input incoherence function capability. The SSI analysis using SASSI is required in order to confirm that site-specific effects are enveloped by the standard design. ~~After the SASSI analysis is first performed for a specific unit, subsequent COLAs for other units may be able to forego SASSI analyses if the FIRS and GMRS derived for those subsequent units are much smaller than the US-APWR standard plant CSDRS, and if the subsequent unit can also provide justification through comparison of site-specific geological and seismological characteristics.~~ MIC-03-03-00054 MIC-03-03-00066 DCD_03.07.02-107

SSI effects are also considered by the COL Applicant in site-specific seismic design of any seismic category I and II structures that are not included in the US-APWR standard plant. ~~Consideration of structure-to-structure interaction is discussed in Subsection 3.7.2.8.~~ The site-specific SSI analysis is performed for buildings and structures including, but not limited to, to the following: MIC-03-03-00054

- Seismic category I ESWPT
- Seismic category I PSFSV
- Seismic category I UHSRS

It is the responsibility of the COL Applicant to address the potential SSSI effect of the R/B complex and T/B on the site specific seismic category I structures. MIC-03-03-00054

The site-specific seismic response analysis of the R/B complex ~~PCCV building structure~~ addresses factors that affect the response of the combined soil-structure dynamic system that include, but are not limited to, the following:

- Properties and layering of the soil, including fill concrete and backfill modeled depending on its horizontal extent
- Depth of the water table, including seasonal variations when appropriate MIC-03-03-00054
- Basemat embedment
- Flexibility of the basemat
- Presence of nearby structures

Up-to-date modeling techniques capable of capturing the various site-specific SSI effects are used for the analysis. The computer program SASSI is used for the site-specific SSI analysis, because it is based on the use of the FE technique and sub-structuring method with frequency-dependent impedance functions to model the interaction of the embedded flexible basemat with the surrounding soil.

The input used for the site-specific analysis must be derived from geotechnical and seismological investigations of the site. ~~The input control motion that is derived from the site-specific GMRS, is applied in the SASSI analysis as within motion at the bottom of the basemat.~~The input control motion derived from the site-specific FIRS is applied in the SASSI analyses at the bottom-of-foundation control point~~location as motion within a soil column that includes the embedment materials and their strain-compatible properties~~. Site-specific SSI analyses account for the uncertainties and variations of the subgrade properties by using at least three sets of site profiles that represent the best estimate, lower bound, and upper bound (BE, LB, and UB~~ for equations~~, respectively) soil and rock properties. If sufficient and adequate soil investigation data are available, the LB and UB ~~values of the initial (small strain) ~~soil properties are established to cover the mean plus or minus one standard deviation for every layer. In accordance with the specific guidelines for SSI analysis contained in Section II.4 of SRP 3.7.2 (Reference 3.7-16), the LB and UB values for initial soil shear modulii (G_S) are established as follows: DCD_03.07.02-106 MIC-03-03-00054 MIC-03-03-00054 MIC-03-03-00054

$$G_s^{(LB)} = \frac{G_s^{(BE)}}{(1+C_v)} \quad \text{and} \quad G_s^{(UB)} = G_s^{(BE)} \quad (1+C_v)$$

For well investigated sites, the C_V should be no less than 0.5. For sites that are not well investigated, the C_V for shear modulus shall be at least 1.0.

The site-specific SSI analysis must use stiffness and damping properties of the subgrade materials that are compatible with the strains generated by the site-specific design earthquake (SSE or/and OBE). However, soil material damping shall not exceed 15% as stipulated in SRP 3.7.1 (Reference 3.7-10). The COL Applicant is to evaluate the strain-dependent variation of the material dynamic properties for site materials. If the strains in the subgrade media are less than 2%, the strain-compatible properties can be obtained from equivalent linear site-response analyses using soil degradation curves. Degradation curves that are published in literature can be used after demonstrating their applicability for the specific site conditions. The strain-compatible soil profiles for the site-specific verification SSI analyses of the major seismic category I structures can be obtained from the results of the site response analyses that are performed to calculate site-amplification factors for the development of GMRS, as described in Subsection 3.7.1.1. MIC-03-03-00054

~~The depth of the water table must be considered when developing the P-wave velocities of the submerged subgrade materials. Significant variations in the water table elevation and significant variations of the subgrade properties in the horizontal direction are addressed by using additional sets of site profiles.Variations in the water table elevations and variations in the subgrade properties of the horizontal layers are presented in Subsection 3.7.2.4.~~ DCD_03.07.02-108 (15) MIC-03-03-00054

To assure the proper comparability, the site-specific verification SSI analyses must use the same verified and validated ~~lumped mass stick~~ models of the ~~building super-structure~~R/B complex as those used for the US-APWR standard plant design. ~~FE analyses are employed to evaluate the flexibility of the basemat and the embedded portion of the building. The floor slabs located at and above the ground surface are assumed absolutely rigid. In order to verify the converted structural model, a site-specific SSI analysis is performed with hard rock site profile that simulates fixed base conditions. The results of the SSI analysis with hard rock site profile are to match closely with the results from the analysis of fixedUS-APWR Standard Plant analysis of a fixed base stick model. In accordance with requirements of Section 1.2 of RG 1.61 (Reference 3.7-15), the lower OBE damping values in Table 3.7.3-1(b) are assigned to the structural model as complex damping.~~ DCD_03.07.02-35 MIC-03-03-00054

The ~~results for 5% damping~~ ISRS at major floor and equipment locations and soil pressures on the basement exterior walls that are obtained from all considered soil cases are enveloped and broadened in the site-~~specific~~independent analysis The plots, tables, and digitized data are then documented for review and comparison with the corresponding results from site-~~independent~~specific analyses. The COL Applicant is to verify that the results of the site-specific SSI analysis for the broadened ISRS ~~and basement walls lateral soil pressures~~ are enveloped by the US-APWR standard design. This is accomplished by comparing site specific ISRS results for all locations provided in Appendix 3B of MUAP-10006 (Reference 3.7-48) and ensuring the site-independent results in MUAP-10006 bound the site-specific results. DCD_03.07.02-35 MIC-03-03-00054 MIC-03-03-00054

~~The site-specific analyses use input soil properties derived from geotechnical investigations of the site that are compatible to the strains generated in the subgrade by the input design earthquake. The uncertainties and variations of the subgrade properties are considered using the methodology previously described for the development of the strain-compatible site profiles for the site-specific SSI analysis of the major seismic category I structures. The control motions are developed from site-specific FIRS that are described in Subsection 3.7.1.1 and applied to the models at the bottom of the basemat.~~ DCD_03.07.02-35 MIC-03-03-00054

~~In accordance with Section 1.2 of RG 1.61 (Reference 3.7-15), the lower OBE damping values in Table 3.7.3-1(b) are assigned to the structural model used for development of ISRS if the site-specific SSE is not large enough to use the damping values and Table 3.7.3-1(a), and OBE design loads. ISRS do not need to be generated for seismic category II buildings and structures which do not contain or support safety-related SSCs, such as the T/B and A/B.~~

Simplified SSI modeling approaches, such as a lumped parameter model, can be employed for the site-specific seismic response analyses of seismic category I and II buildings and structures that are not part of the US-APWR standard design if it is demonstrated that for the specific site conditions the following applies:

- The basemats are much stiffer than the supporting subgrade
- The SSI impedance functions remain relatively constant in the range of frequencies important for the design

- The consideration of basemat embedment yields conservative results

In accordance with SRP 3.7.2 (Reference 3.7-16), Section II.4, fixed base response analysis can be performed if the basemats are supported by subgrades having a shear wave velocity of 8,000 ft/s or higher, under the entire surface of the foundation.

3.7.2.5 Development of Floor Response Spectra

~~ISRS for the PS/Bs and RCL-R/B-containment internal structure are developed from the results of the site-independent seismic analyses of the seismic models described in Subsection 3.7.2.3 by applying methods described in Subsection 3.7.2.1, and capturing SSI effects as described in Subsection 3.7.2.4, using generic soil profiles described in Subsection 3.7.1.3. The statistically independent time histories developed from the CSDRS as described in Subsection 3.7.1.1 serve as input control motion in the analysis. Note that the dynamic properties of the stick model portions of the R/B complex seismic model presented in Technical Report MUAP-10001 (Reference 3.7-47) are modified to account for the effects of cracking for accuracy in the seismic design and development of the ISRS. The ISRS are derived from the calculated responses at locations and elevations where the majority of seismic category I and II SSCs are located.~~ MIC-03-03-00054 DCD_03.07.02-35 DCD_03.07.02-35

~~In developing the ISRS, the effects of floor slab system out-of-plane flexibility are considered by investigating floor slab systems (using local FE models or other means of analysis), independently from the overall lumped mass stick model in order to determine their natural frequencies. Depending on the results, the floor slab systems may then be analyzed as simple single DOF vertical oscillators to determine maximum accelerations (ZPA values) to be used for development of the ISRS for the respective floor locations. The concrete cracking of slabs is considered in the development of the single DOF models, in accordance with ACI 349-01 (Reference 3.7-31). If the results of the independent modal analyses indicate that higher modes of vibration have to be considered, the floor systems may also be analyzed as subsystems, as described further in Subsection 3.7.3.1. The local analyses of floor slab systems with respect to out-of-plane flexibility and effects on the ISRS are addressed in Technical Reports MUAP-10001 and MUAP-10006 (References 3.7-47 and 3.7-48).~~ DCD_03.07.02-35 DCD_03.07.02-35

~~The SSE ISRS for seismic category I buildings and structures of the US-APWR standard plant are developed directly from the results of the site-independent seismic analysis. As previously explained in Subsection 3.7.1.1, since the OBE ground motion is limited to a maximum of 1/3 times the CSDRS, explicit design and analysis for OBE is not required, as permitted by 10 CFR 50 Appendix S (Reference 3.7-7). Therefore, separate OBE ISRS are not developed for design and analysis of US-APWR standard plant systems and subsystems.~~

The SASSI analyses provide results for the response of the R/B complex due to the three directional design input ground motion for both the cracked and uncracked R/B complex models for each of the generic soil profiles. ISRS are generated for various areas of the R/B complex in accordance with RG 1.122 (Reference 3.7-26) to serve as the seismic design basis for the design of pipe and equipment. The ISRS may be developed from the SASSI ARS data for any node location or damping values, or for variable damping where permitted by ASME Code Case N411-1, as discussed in RG 1.61 (Reference 3.7-15). At selected node locations, ARS in the three orthogonal directions are calculated for each of

the three orthogonal directions of the input ground motion from time histories generated by SASSI. The ARS are calculated at 301 frequency points equally distributed on the logarithmic scale at the range of frequency from 0.1 Hz to 100 Hz. The ARS for particular damping value obtained for the three directions of the input ground motion are then combined using the Square Root Sum of the Squares (SRSS) method as follows: MIC-03-03-00054

$$ARS_X = \sqrt{ARS_{XX}^2 + ARS_{YX}^2 + ARS_{ZX}^2}$$

$$ARS_Y = \sqrt{ARS_{XY}^2 + ARS_{YY}^2 + ARS_{ZY}^2}$$

$$ARS_Z = \sqrt{ARS_{XZ}^2 + ARS_{YZ}^2 + ARS_{ZZ}^2}$$

where:

- $ARS_{(m)(n)}$ are the SASSI ARS results for the response in "n" direction due to earthquake in "m" direction;
- ARS_X, ARS_Y, and ARS_Z are the combined ARS of the structural response in NS (x), EW (y), and vertical (z) direction, respectively.

Once the results of each of the generic soil cases are combined through SRSS at the nodes, the results are grouped for the nodes within the footprint/support of the equipment or floor areas for which the ISRS is developed.

The grouped nodal results are then enveloped for each of the soil cases and both structural stiffness levels. Enveloping the responses at the grouped nodes is to provide an ISRS for the equipment design and qualification that considers the potential non-uniform input at their support locations including the rocking and torsional effects. The spectra from each analysis (SSI and SSSI) are then enveloped. The resulting spectra are broadened by 15% in spectral frequency to account for uncertainties in the analysis parameters.

Further, when the ISRS are used for equipment qualification,the valleys between adjacent peaks in the enveloped ISRS are filled to capture potential frequency shifts within the range of the SSI and SSSI responses obtained from the generic soil profiles. To fill in the valleys in the ISRS, the lower peak is extended diagonally until it intersects with the side slope of the adjacent higher peak.

To generate additional ISRS at other damping values as necessary for design of SSCs, the same process described above is repeated.

In the case where seismic qualification by testing is performed in accordance with IEEE Std 344-2004 (Reference 3.7-13), test response spectra which replicate the OBE response spectra are not required since the OBE condition is no longer used as a design basis. The US-APWR program for seismic and dynamic qualification of mechanical and electrical equipment is discussed in Section 3.10.

~~The ISRS are developed for damping values equal to 0.5%, 2%, 3%, 4%, 5%, 7%, 10%, 20% of critical damping and for variable damping where permitted by ASME Code Case N411-1, as discussed in RG 1.61 (Reference 3.7-15). The ISRS envelope the spectra obtained from the site-independent analyses for all generic subgrade conditions described in Subsection 3.7.1.3. ISRS developed from the site-independent seismic analyses of the R/B complex and PS/Bs are used for design. ISRS developed at 5% critical damping, which are presented in Technical Report MUAP-10006 (Reference 3.7-48) and referenced in Appendix 3I are used to validate the standard plant ISRS by comparison to site-specific ISRS that are also developed at 5% critical damping. The process for developing enveloped ISRS is described in detail in Section 3.5 of Technical Report MUAP-10006 and is summarized as follows: :~~ MIC-03-03-00054 DCD_03.07.02-35

- ~~The response spectra are generated for the three components of earthquake by SRSS, following the general guidance of RG 1.122 (Reference 3.7-26) for frequencies up to 100 Hz.~~
- ~~The maximum spectral acceleration at each frequency obtained from the seismic analysis of any general subgrade conditions is selected for the envelope.~~
- ~~The enveloped ISRS are smoothed and broadened by +/-15%. The valleys in the enveloped ISRS are filled when necessary to capture potential shifts in the seismic response caused by soil properties that are different from, but bounded by, the generic soil conditions of the standard plant. Alternatively in some locations, the peak shifting method described in Subsection 3.7.3.1.6 can be used instead of the broadened response spectra method.~~ DCD_03.07.02-108 (11)
- ~~The broadened response spectra method discussed in Subsection 3.7.3.1 is used or alternatively in some locations, the peak shifting method described in Subsection 3.7.3.1 can be used.~~ DCD_03.07.02-108 (11)

~~ISRS are not required for~~No safety-related systems and components are present in non-seismic category I building structures, such as the AC/B, A/B and T/B~~, since no safety-relatedsystems and components are present in non-seismic category I buildings and structures~~. The design, installation, and mounting of non safety-related systems and components in these buildings are based on the applicable site-specific building codes and standards. DCD_03.07.02-35 DCD_03.07.02-35

3.7.2.6 Three Components of Earthquake Motion

As previously discussed in Subsection 3.7.1.1, the seismic analyses of the major seismic category I structures are based on one set of three mutually orthogonal artificial time histories, with each of the three directional components being statistically independent of the other two. The acceleration time histories of the horizontal H1 and H2 components of the earthquake are applied in N-S direction and E-W directions respectively. The acceleration time history V is applied in the vertical direction.

The three components of the earthquake are applied on the seismic model separately in ACS SASSI (Reference 3.7-17) for obtaining the maximum accelerations of the response in the three orthogonal directions. The maximum responses of interest of SSCs obtained

from the responses of each of the three components of motion are then combined using SRSS~~ or the Newmark 100%-40%-40% method~~ in accordance with RG 1.92, Rev.2 (Reference 3.7-27). The combined maximum accelerations, obtained through the process described previously in Subsection 3.7.2, are then used as basis for development of the SSE loads used for the design of structural members, components and connections of US-APWR standard plant. These SSE design loads are applied as static loads on the detailed FE model in conjunction with other design loads and load combinations. MIC-03-03-00054

The development of the ISRS uses the SRSS method to combine the responses from the three components of the earthquake motion.

Although the above approach has been used for seismic analysis of the major seismic category I structures, seismic responses of other seismic systems and subsystems due to the three components of earthquake motion can be combined using any one of the following methods in accordance with RG 1.92, Rev.2 (Reference 3.7-27):

i. The peak responses due to the three earthquake components from the response spectra and equivalent static analyses are combined using the SRSS method.

ii. The peak responses due to the three earthquake components are combined directly, using the Newmark combination method that assumes that when the peak response from one component occurs, the responses from the other two components are 40% of the peak (100%-40%-40% method). Combinations of seismic responses from the three earthquake components, together with variations in sign (plus or minus) are considered.

iii. The time-history of the responses from the three earthquake components that are applied simultaneously can be combined algebraically at each time step to obtain the combined response time-history. The design seismic loads are selected from the maximum values or the most critical combination of values extracted from the time history results representing the responses directly related to the design of the particular ~~member~~element considering sign reversals, such as the relevant internal forces or stresses in the ~~member~~element.~~ Due to the uncertainties introduced by phasing effects, the design does not use time history results for other responses, such as accelerations or displacements at points in time that are indirectly related to the basic design inputs.~~ DCD_03.07.02-96

3.7.2.7 Combination of Modal Responses

As previously discussed, the seismic responses of the seismic category I building models are obtained using three-dimensional SSI models with the program ACS SASSI (Reference 3.7-17). ACS SASSI utilizes time history analysis in the frequency domain in which the equations of motion are solved using a global complex matrix that is assembled from the complex matrices for the soil and structural elements. Therefore modal combination is not utilized.

When the modal superposition time history analyses or response spectra analyses are used for seismic design of other seismic category I and seismic category II systems and subsystems, ~~all necessary modes are included in order to capture a minimum of 90% of the cumulative mass of the building or structure being analyzed~~it may not be practical to DCD_03.07.02-101

capture higher frequency modes that are not excited by the input motion. In modal superposition, only modes with frequencies less than the frequencies defining the cutoff or ZPA response participate in the modal solution. The modal contribution of the residual rigid response for modes with frequencies greater than the cutoff or ZPA frequency is accounted for by using the missing mass method. ~~As permitted by RG 1.92, Rev.2 (Reference 3.7-27)~~As permitted in Section 1.4.1 of RG 1.92 (Reference 3.7-27), the missing mass contribution, scaled to the instantaneous input acceleration, is treated as an additional mode in the algebraic summation of modal responses at each time step. The missing mass contribution is considered for all DOF. When using the Lindley-Yow method in response spectra analyses, the missing mass may be captured using the Static ZPA method as described in Section 1.4.2 of RG 1.92, Rev. 2 (Reference 3.7-27). DCD_03.07.02-101 DCD_03.07.02-101

When the response spectra method of analysis is used (see Subsection 3.7.3.1 for a discussion of response spectra methods of analysis), modal responses have been combined by one of the RG 1.92, Rev.2 (Reference 3.7-27), methods, or by the 10% grouping method described below. In some applications, the more conservative modal combination methods contained in Rev.1 of RG 1.92 (Reference 3.7-28) are also used, as permitted in Revision 2 of RG 1.92 (Reference 3.7-27).

For the grouping method, the total unidirectional seismic response for subsystems is obtained by combining the individual modal responses using the SRSS method for frequencies spaced more than 10%.

For subsystems having modes with closely spaced frequencies, this method is modified to include the possible effect of these modes. The groups of closely spaced modes are chosen so that the differences between the frequencies of the first mode and the last mode in the group do not exceed 10% of the lower frequency.

The combined total response for systems having such closely spaced modal frequencies is obtained by adding to the SRSS of all modes the product of the responses of the modes in each group of closely spaced modes.

This can be represented mathematically as follows:

$$R^2 = \sum_{k=1}^{N} R_k{}^2 + \sum_{q=1}^{P} \sum_{l=i}^{j} \sum_{m=i}^{j} \left| R_{lq} \cdot R_{mq} \right| \qquad l \neq m$$

where

R = total unidirectional response

R_k = the peak value of the response due to the k^{th} mode

R_{lq}, R_{mq} = are the modal responses, R_l and R_m within the q^{th} group

N = total number of modes considered

P = number of groups of closely spaced modes

i = lowest modal number associated with group j of closely spaced modes

j = highest modal number associated with group j of closely spaced modes

Alternatively, a more conservative ten percent grouping method can be used in the seismic response spectra analyses. The groups of closely spaced modes are chosen so that the difference between two frequencies (the first and last mode in a group) is no greater than 10%. Therefore,

$$R^2 = \sum_{k=1}^{N} R_k^{\,2} + 2\sum \left| R_i R_j \right| \qquad i \neq j$$

The second summation is to be done on all i and j modes whose frequencies are closely spaced to each other.

All terms for the modal combination remain the same as defined above.

The 10% grouping method is more conservative than the grouping method because the same mode can appear in more than one group. The 10% grouping method is used for piping as described in Subsection 3.12.3.2.4.

For the seismic response spectra analysis, the ZPA cut-off frequency is 50 Hz. High frequency or rigid modes must be considered using the static ZPA method, the left-out force method as described in Subsection 3.7.2.7 below, or the Kennedy Missing Mass method contained in Revision 2 of RG 1.92 (Reference 3.7-27).

3.7.2.7.1 Left-Out-Force Method (or Missing Mass Correction for High Frequency Modes)

The left-out-force method is based on the Left-Out-Force Theorem. This theorem states that for every time history load, there is a frequency, f_r, called the “rigid mode cutoff frequency” above which the response in modes with natural frequencies above f_r will very closely resemble the applied load at each instant of time. These modes are called “rigid modes.” The formulation follows and is based on the method used in the computer program PIPESTRESS (Reference 3.7-29). The left-out-force method is not used for seismic analysis of the major seismic category I structures; however, it may be used for other seismic category I and II systems and subsystems.

The left-out-force vector for time history analyses, { *Fr* }, is calculated based on lower modes:

$$\{ Fr \} = [\, 1 - \sum M\, e_j\, e_j^{T} \,]\, f(t)$$

where

f (t) = the applied load vector

M = the mass matrix

e_j = the eigenvector

Note that ∑ only represents the flexible modes, not including the rigid modes.

In the response spectra analysis, the total inertia force contribution of higher modes can be interpreted as:

$$\{ Fr \} = A_m [M] [\{ r \} - \sum P_j e_j]$$

where

A_m = the maximum spectral acceleration beyond the flexible modes

[M] = the mass matrix

{ r }= the influence vector or displacement vector due to unit displacement

P_j = participation factor, where

$$P_j = e_j^T [M] \{ r \}, \{ Fr \} = A_m [M] \{ r \} [1 - \sum M e_j e_j^T]$$

In the response spectra analysis, the low frequency modes are combined by one of the modal combination methods in accordance with RG 1.92, Rev.2 (Reference 3.7-27) as discussed above. For each support level, there is a pseudo-load vector or left-out-force vector in the X, Y, and Z directions.

These left-out-force vectors are used to generate left-out-force solutions which are multiplied by a scalar amplitude equal to a magnification factor specified by the user. As an alternative the acceleration associated with a cutoff frequency can be used instead of the ZPA provided the number of modes chosen is such that the results of the analysis are within 10 percent of the results of an analysis that considers the additional number of modes. This factor is usually the ZPA of the response spectra for the corresponding direction. The resultant low frequency responses are combined by the SRSS with the high frequency responses (rigid modes results). DCD_03.07.02-108 (3)

3.7.2.8 Interaction of Non-Seismic Category I Structures with Seismic Category I Structures

The locations of all major buildings within the power block are shown on the general arrangement drawings in Section 1.2.

Seismic category II structures have been analyzed for the same seismic loads and using the same seismic analysis methods described for seismic category I SSCs in Subsection 3.7.2.1 to verify that they will not collapse or adversely interfere with the standard plant seismic category I ~~SSCs~~R/B complex or adversely affect the MCR occupants. Seismic category II is defined in Section 3.2. By definition, seismic category II structures are designed to retain their position to the extent necessary to assure that they will not impact the function or integrity of seismic category I SSCs. MIC-03-03-00054

NS structures have been located such that, in case of their collapse or failure, they do not have the potential to impact seismic category I SSCs, either directly or indirectly.

~~NS structures that are not located beyond the range of impact are isolated by heavy concrete walls from seismic category I SSCs.~~ DCD_03.07.02-97

~~With respect to the coupling of the dynamic responses of adjacent structures through the soil, the phenomenon of structure-to-structure interaction is neglected in the SSI analyses for the standard plant discussed in Subsection 3.7.2.4. Instead, the variations of site properties considered by the four general subgrade conditions are deemed sufficient to address the uncertainties related to possible structure-to-structure interaction effects on the overall seismic response results.The same methodology used to evaluate structure-to-structure interaction between seismic Category I structures and non-seismic Category I structures is used to evaluate structure-to-structure interactions between seismic Category I structures. It is the responsibility of the COL Applicant to further address structure-to-structure interaction if the specific site conditions can be important for the seismic response of particular US-APWR seismic category I structures, or may result in exceedance of assumed pressure distributions used for the US-APWR standard plant design.~~ DCD_03.07.02-35 MIC-03-03-00066 MIC-03-03-00054

Maximum lateral earth pressure due to the backfill, surcharge due to live load or adjacent basemat bearing pressures, groundwater, and other such static-load effects on below-grade exterior walls are discussed in Section 3.8. The design of below grade exterior walls for US-APWR seismic category I structures takes into account any dynamic increases of these loads due to a seismic event. This is accomplished through the use of conservative maximum static and dynamic lateral pressure distribution profiles developed using analysis methods provided in Section 3.5.3 of ASCE 4-98 (Reference 3.7-9) and as discussed in Subsection 3.8.4. DCD_03.07.02-98

The COL Applicant is to assure that the design or location of any site-specific seismic category I SSCs, for example pipe tunnels or duct banks, will not expose those SSCs to possible impact due to the failure or collapse of non-seismic category I structures, or with any other SSCs that could potentially impact, such as heavy haul route loads, transmission towers, non safety-related storage tanks, etc. Alternately, site-specific seismic category I SSCs ~~are~~may be designed for impact loads due to postulated failure of the non-seismic category I SSCs. MIC-03-03-00054

Following is a discussion of major structures in the power block area with respect to potential interaction with seismic category I structures.

3.7.2.8.1 AC/B

The AC/B is ~~structurally~~ designed as a NS structure on reinforced concrete foundation located ~~at~~approximately 16 inches from the west side of the A/B (seismic category II). ~~The AC/B is not located adjacent to any seismic category I SSCs.~~ If the AC/B were to fail or collapse, it could impact the A/B which is a seismic category II structure located on the R/B complex common basemat. The AC/B is smaller, shorter, and much less massive than the reinforced concrete A/B. In the unlikely event of impact, there would not be sufficient kinetic energy transfer to cause the A/B to displace beyond acceptable limits. Specifically, MIC-03-03-00054

the A/B would not displace enough to impact the R/B~~,~~ or PS/Bs~~, or any other seismic category I SSCs~~. MIC-03-03-00054

The design philosophy of the AC/B is stated as follows.

- The seismic design is in accordance with the International Building Code (Reference 3.7-30) with an Importance Factor of 1.0.
- The structure is designed in accordance with applicable building codes.

3.7.2.8.2 T/B

The T/B is structurally designed as seismic category II, such that its integrity will not be impacted by a design basis seismic event; that is the T/B will not fail or collapse due to seismic loading. The T/B is located on the south sides of the R/B complex and is separated from these structures by approximately 20 feet (see Figures in Section 1.2 for details). This is sufficient distance to preclude interaction due to seismic motion of either structure. SSSI interaction is discussed in Section 3.7.2.4 and sliding interaction is discussed in Section 3.8.5. ~~and the PS/Bs, and is separated from these structures with an expansion joint at all above-grade interface locations. The expansion joints are sized prevent contact between buildings, even if the maximum translational and rotational displacements due to a seismic loading (and other loading) were to occur. The minimum sizes of expansion joints must be obtained by considering, at all potential contact locations, the absolute summation of the T/B deflection and the adjacent structures' deflection (R/B, PS/Bs, and ESWPT) obtained from the response spectra or time history analysis results for those structures. The nominal horizontal clearance between the T/B structure above grade to adjacent structures is 4 inches.~~ MIC-03-03-00054

The T/B is a reinforced concrete structure below grade and a braced steel frame structure above grade. The design philosophy of the T/B is stated as follows.

- The reinforced concrete structure is designed in accordance with the ACI 349-06 code (Reference 3.7-31), and the braced steel frame structure is designed in accordance with the AISC N690 code (Reference 3.7-32). MIC-03-03-00066
- The design of the T/B is based on a static ~~analysis utilizing a three-dimensional FE model, and a seismic dynamic analysis using a three-dimensional lumped mass model.~~and dynamic analyses utilizing three dimensional FE models. DCD_03.07.02-35
- The T/B is designed and analyzed as a seismic category I structure. This is described in MUAP-11002 (Reference 3.7-61). MIC-03-03-00054

~~3.7.2.8.3 ESWPT~~

~~The ESWPT passes underneath the T/B at the north end of the T/B. The ESWPT must be physically separated from the basemats of the PS/Bs and the T/B to assure that there will not be contact due to seismic or other loading. The ESWPT will not interact seismically or structurally with the R/B due to separation from the R/B and T/B. Where the ESPWT~~

~~passes underneath the T/B, the ESWPT is separated on its top and two sides from the T/B basemat elements with a compressible filler material and/or air gap.~~ MIC-03-03-00054

~~The ESWPT is entirely constructed of reinforced concrete. The design philosophy of the ESWPT is stated as follows.~~

- ~~The ESWPT reinforced concrete structure is designed in accordance with the ACI 349-06 code (Reference 3.7-31).~~ MIC-03-03-00066
- ~~The compressible filler is required to be designed (location and thickness) such that it can compress under seismic and other loads, and such that the bearing loads imposed by compression of the filler material under seismic or other displacement are structurally acceptable.~~
- ~~The SSE load condition is the same as for the R/B complex.~~ MIC-03-03-00066
- ~~To determine seismic loads and displacements, and to generate ISRS for design of essential service water piping, a two-dimensional SASSI analysis is performed that accounts for soil-structure and structure-structure interaction, if necessary~~. DCD_03.07.02-35
- ~~The SASSI analysis is required to be documented and comply with the same general requirements described for the R/B-PCCV-containment internal structure SASSI analysis with the exception that no stick model is required. Instead, plateSASSI analysis. Shell elements are to be directly included to represent the tunnel in the SASSI model. However, if the T/B is also included in the model for considering the effects of structure-structure interaction, the stick model of the T/B as described in Subsection 3.7.2.8.2 is used.~~ DCD_03.07.02-35 MIC-03-03-00066 DCD_03.07.02-35

3.7.2.8.3 A/B

The A/B contains the US-APWR standard plant radioactive waste processing facility. This facility is designated as Classification RW-IIa in accordance with RG 1.143 (Reference 3.7-19). However, the A/B is designed as seismic category II. The seismic, wind, tornado, hurricane, and flood design requirements for seismic category II are more stringent than those of Classification RW-IIa as outlined in RG 1.143 (Reference 3.7-19). The A/B is located on a common basemat with the R/B, PCCV, CIS, East and West PS/B, and ESWPC. The A/B is situated on the west side of the R/B, and has the west PS/B on its south side and the AC/B on its west side.~~The A/B is located on the west side of the R/B, and has one PS/B on its south side and the AC/B on its west side. The A/B is separated from these structures with expansion joint(s) sufficiently sized to prevent contact between buildings even if the maximum translational and rotational displacements due to a seismic loading (and other loading) were to occur. The minimum sizes of expansion joints to prevent interaction is determined by considering, at all potential interaction locations, the absolute summation of the A/B deflection and the adjacent structures' deflection (R/B, PS/B, and AC/B) obtained from the response spectra or time history analysis results for those structures, except for the AC/B, the deflection results are determined through the applicable code method.~~ DCD_02-03 S01 MIC-03-03-00054

The majority of the A/B is a reinforced concrete structure with one floor level below grade and three stories above grade. The design philosophy of the A/B is stated as follows.

- The reinforced concrete structure is designed in accordance with the ACI 349-06 code (Reference 3.7-31), and the steel beams supporting some floor slabs are designed in accordance with the AISC N690 code (Reference 3.7-32). MIC-03-03-00066

- ~~The design of the A/B is based on a static analysis utilizing a three-dimensional FE model, and a seismic dynamic analysis using a three-dimensional lumped mass model. An FE model is used in determination of the maximum accelerations and displacements. A lump mass stick model is used for seismic stability evaluation.~~The A/B is designed as a seismic category I structure and analyzed as part of the R/B complex. DCD_03.07.02-35 DCD_02-03 S01 MIC-03-03-00054

3.7.2.8.4 R/B and PCCV

The R/B and PCCV are seismic category I structures ~~and the seismic modeling is described in MUAP-10001 (Reference 3.7-47). The R/B borders the A/B, the two PS/Bs (discussed below), and the T/B on its south side.~~within the R/B complex. The modeling of the R/B complex is described in Technical Report MUAP-10006 (Reference 3.7-48). The R/B rests on a common basemat with and envelopes the PCCV up to the R/B roof, which varies in elevation as shown on the general arrangement drawings in Section 1.2. However, to preclude seismic and structural interaction above the common basemat, the R/B is separated from the PCCV with a 4 in. minimum ~~expansion joint~~gap at all above-basemat locations. The ~~expansion joint~~gap has been sized to prevent contact between the R/B and PCCV super-structures even if the maximum translational and rotational displacements due to a seismic loading (and other loading) were to occur. The ~~expansion joint~~gap size has been determined by considering, at all potential interaction locations, the absolute summation of the deflection associated with each super-structure, obtained from the time history analysis results for those structures. DCD_03.07.02-35 MIC-03-03-00054 MIC-03-03-00054 MIC-03-03-00054

~~3.7.2.8.5 PS/Bs~~

MIC-03-03-00054

~~The US-APWR standard plant PS/Bs are seismic category I structures and their seismic modeling and analyses are described in Technical Reports MUAP-10001 and MUAP-10006 (References 3.7-47 and 3.7-48, respectively). The west PS/B borders the A/B, R/B, and T/B. The east PS/B borders the R/B and the T/B. Each PS/B rests on its own basemat. Each PS/B is required to be designed with an expansion joint along its interface with the R/B to assure that no contact will occur between the buildings under a seismic or any other design basis loading. The expansion joint is sized to prevent contact between the structures even if the maximum translational and rotational displacements due to a seismic loading (and other design basis loading) were to occur. The expansion joints are to be determined by considering, at all potential interaction locations, the absolute summation of the deflection associated with each super-structure, obtained from the time history analysis results for those structures.~~

~~The plan dimension of each PS/B is nominally 111'-6" x 66'-0" between centerlines of exterior walls. Each PS/B is a reinforced concrete structure with one floor level under~~

~~ground and the main floor level above ground. The design philosophy of the PS/Bs is stated as follows.~~ MIC-03-03-00054

- ~~The east and west PS/Bs are nearly identical structurally, and one bounding analysis is performed to represent both.~~
- ~~Reinforced concrete structure of the PS/Bs is designed by ACI 349-06 code (Reference 3.7-31).~~ MIC-03-03-00066
- ~~The SSE load condition is the same as for the R/B.~~
- ~~The design of the PS/Bs is based on a static analysis utilizing a three-dimensional FE model, and a seismic dynamic analysis using a three-dimensional FE model in SASSI (Reference 3.7-17).~~

3.7.2.9 Effects of Parameter Variations on Floor Response Spectra

~~ISRS are generated for all US-APWR seismic category I structures.~~ MIC-03-03-00054

To account for variations in the structural frequencies due to the uncertainties in parameters, such as material and mass properties of the structures, damping values, soil properties, SSI analysis techniques, and the seismic modeling methods, the ISRS are developed from six SSI soil profiles representing a range of soft soil to hard rock conditions and two structural stiffnesses representing cracked and uncracked conditions values. These 12 cases and 8 additional cases from SSSI analysis are enveloped and then broadened by ±15% as described in Section 3.7.2.5. Developing enveloping ISRS using this range of parameters and the CSDRS as an input motion creates a design envelop that will encompass most variations in site-specific conditions.~~computed ISRS are smoothed by filling in valleys between peaks as described in Subsection 3.7.2.5, and the peaks associated with the structural frequencies are broadened by ±15% in accordance with RG 1.122 (Reference 3.7-26).~~ MIC-03-03-00054

~~The SSI analyses of standard plant seismic category I buildings described in Subsection 3.7.2.4 include generic supporting media with frequency-dependent properties that are intended to bound the varying subgrade conditions and to account for the variation in SSI analysis techniques and seismic modeling methods. The generic supporting media are described in Subsection 3.7.1.3 and capture a broad range of subgrade conditions ranging from relatively soft soil to hard rock. This wide range of supporting media conditions captures SSI and other seismic response effects, including the resulting variances in ISRS. Further, valleys between the peaks of the standard plant design ISRS are filled in, if necessary to account for variations of site-specific soil properties within the range of supporting media conditions considered in the standard plant design.~~

~~The effects of potential concrete cracking on the structural stiffness of reinforced concrete structures are are considered in the development of local vibration modes as described in Subsection 3.7.2.5.~~ DCD_03.07.02-35

3.7.2.10 Use of Constant Vertical Static Factors

The plant design does not utilize constant vertical static factors in the seismic design. The vertical component of the seismic motion is obtained using one of the analysis methods described in Subsection 3.7.2.1. The vertical component is combined with the horizontal components of the seismic motion as described in Subsection 3.7.2.6.

3.7.2.11 Method Used to Account for Torsional Effects

~~The seismic analyses of seismic category I buildings and structures incorporate the torsional DOF in the mathematical models, as discussed in Subsection 3.7.2.3.~~ DCD_03.07.02-35

Inertial torsional effects are inherently considered in the seismic analysis using a 3D FE model.The site-independent SSI analyses are performed using FE models described in Section 3.7.2.3 that represent the general layout of the building and explicity account for eccentricities between the center of mass and center of rigidities. MIC-03-03-00066

The structural members of category I and II buildings are designed for two types of torsional effects: (1) torsional responses captured in the seismic response analysis; and (2) accidental torsion. The accidental torsion considers torsional effects that are not captured in the seismic response analyses such as torsion that is due to incoherency (spatial variation) of the input ground motion, non vertically propagating incident waves, and/or accidental eccentricities. The accidental torsional effect is included in accordance with SRP 3.7.2 Section II (Reference 3.7-16) in the design of all seismic category I and II structures by use of the following process:

- ~~The horizontal mass properties, center of rigidity, and the corresponding nodal accelerations, are computed in order to determine the inertial torsional moments. These computations are performed separately for each floor elevation of the building lumped mass stick models that are used for seismic analysis, which are described in Subsection 3.7.2.3.~~ DCD_03.07.02-35

- The accidental torsional moments are computed by determining an additional building torsion equal to story shear force with a moment arm of +/- 5% of the plan dimension of the floor perpendicular to the direction of the applied motion. This computation is performed for both horizontal directions.

- The accidental torsional moments ~~due to eccentricities of the masses at each floor elevation~~ are assumed to act in the same direction on each structure unless otherwise demonstrated in the seismic analysis. Both positive and negative accidental torsional moments are considered in the design of building structures in order to capture worst case effects. DCD_03.07.02-35

- The accidental torsional moment is combined with the inertial torsional moment. This is computed conservatively so that the combined torsional moment is additive for each floor elevation. The combined torsional moment is distributed to the resisting structural elements in proportion to their relative stiffnesses.

~~The methods and approaches used to capture torsional effects in seismic category I buildings are described further in Subsection 3.7.2.3.~~ DCD_03.07.02-35

3.7.2.12 Comparison of Responses

The ~~major seismic category I structures are~~R/B complex is analyzed using time history analysis methods. MIC-03-03-00054

As described in Subsection 3.7.1.1, the time history analyses are based on design ground motion time histories which have been ~~synthesized~~developed from seed recorded time histories and meet the requirements of “Acceptance Criteria, Design of Time History Option 1: Single Set of Time Histories, Approach ~~2~~1”, NUREG-0800, SRP 3.7.1, Section II (Reference 3.7-10). Since only a time history analysis method is used, comparison of the responses between the response spectrum method and a time history analysis method, as per SRP Section 3.7.2.II.12 (Reference 3.7-16), is not applicable. MIC-03-03-00054 MIC-03-03-00054

3.7.2.13 Methods for Seismic Analysis of Dams

The US-APWR standard plant design does not include dams. It is the responsibility of the COL Applicant to perform any site-specific seismic analysis for dams that may be required.

3.7.2.14 Determination of Dynamic Stability of Seismic Category I Structures

~~Based on NUREG-0800, SRP 3.8.5 (Reference 3.7-34), for all structures, for load combinations involving SSE loads, a minimum factor of safety of 1.1 against overturning and sliding under the worst condition of loading is provided. If an OBE value is chosen to be greater than 1/3 of the site-specific SSE for the design of site-specific seismic category I structures, then load combinations involving the site-specific OBE must have a minimum factor of safety of 1.5 against overturning and sliding.~~Dynamic stability of the R/B complex is determined in Section 3.8.5. The dynamic FE model described in Section 3.7.2.3 is used to calculate overturning, flotation and dynamic bearing pressure. The R/B complex and T/B will slide during a large earthquake. A non-linear analysis utilizing five separate acceleration time histories and the dynamic FE model is used for the sliding analysis described in Section 3.8.5.5. MIC-03-03-00054

The US-APWR standard plant design is based on the assumption, as discussed in Chapter 2, that there is no potential for liquefaction of the supporting media. In order to verify the dynamic stability of US-APWR standard plant and site-specific seismic category I structures, site-specific investigations are performed of the supporting media as described in Subsection 2.5.4.8 to verify that there is no potential for liquefaction. The site-specific factor of safety against liquefaction is determined to confirm the dynamic stability of seismic category I structures for the US-APWR standard design with respect to liquefaction.

3.7.2.15 Analysis Procedure for Damping

The analysis procedure of damping in the various elements of the soil-structure system model has been discussed in Subsections 3.7.1.2, 3.7.2.3, and 3.7.2.4.

3.7.3 Seismic Subsystem Analysis

This section addresses seismic analysis of civil structure-related seismic category I subsystems, which are analyzed in accordance with NUREG-0800, SRP 3.7.3

(Reference 3.7-35). The civil structure-related subsystems are accounted for in the global seismic models of the seismic category I building structures described in Subsection 3.7.2.3 by considering the mass and mass distribution of the subsystems in the models. However, seismic analysis of the subsystems are generally performed separately because the subsystems do not contribute to the building stiffness and because the seismic responses of the buildings (ISRS as discussed in Subsection 3.7.2.5) serve as the seismic design input motion for the subsystems. SSCs that are seismically analyzed as civil structure-related subsystems include:

- Structures such as miscellaneous steel platforms, stairs, and walkways.
- Structures such as reinforced masonry block walls and enclosures.
- HVAC ducts and duct supports. The design of HVAC ducts and duct supports is addressed further in Appendix 3A.
- Conduits and conduit supports. The design of conduits and conduit supports is addressed further in Appendix 3F.
- Cable trays and tray supports. The seismic qualification of cable trays and tray supports is addressed in Appendix 3G.
- Pipe racks and pipe support framing. These structures may also be analyzed as part of mechanical piping subsystems as discussed in Section 3.12.
- Pipe whip restraints. See Section 3.6 and Appendix 3B for a discussion of the design of pipe whip restraints for dynamic loads due to pipe rupture and Appendix 3E for discussion of high energy piping design.
- Equipment cabinet structural framing and/or mounting.

In addition to the above, civil structure-related subsystems also include those seismic category I and II SSCs such as pipe tunnels, conduit tunnels, dams, dikes, aboveground tanks, and the like, which are exterior to the R/B, PCCV, PS/Bs, and the ESWPT.

Each non-category I system and component is designed to be isolated from any seismic category I systems and components by either a constraint or barrier, or is remotely located with regard to the seismic category I systems and components. If it is not feasible or practical to isolate the seismic category I systems and components, adjacent non-category I systems and components are analyzed for the same seismic input motion that is applicable to the seismic category I systems and components. In this case, the analysis demonstrates position retention of the non-category I subsystems and components, with no adverse interaction effects on seismic category I SSCs. For non-category I systems and components attached to seismic category I systems and components, the dynamic effects of the non-category I subsystems and components are simulated in the modeling of the seismic category I systems and components. The attached non-category I systems and components, up to the first anchor beyond the interface, are designed in such a manner that during an earthquake of SSE intensity, the structural integrity and safety functions of the seismic category I systems and components are not jeopardized.

Seismic and dynamic qualification of mechanical and electrical equipment and subsystems performed by testing is discussed in Section 3.10 and Appendix 3D. Mechanical subsystems include mechanical equipment, piping, vessels, tanks, heat exchangers, valves, and instrumentation tubing and tubing supports. The seismic analysis of mechanical subsystems is addressed in Sections 3.9 and 3.12. The RCL analysis is discussed in Appendix 3C.

A list of seismic category I mechanical and fluid systems, components, and equipment is given in Table 3.2-2. Seismic analysis of civil structural items related to those subsystems is discussed in this subsection.

3.7.3.1 Seismic Analysis Methods

Modal response spectra analysis, time history analysis, or equivalent static load analysis methods may be used for seismic analysis of seismic category I subsystems. 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 (References 3.7-10 and 3.7-16).

Time history analysis of seismic systems is discussed in Subsection 3.7.2. The time-history seismic analysis of a subsystem can be performed by simultaneously applying the displacements and rotations at the interface point(s) between the subsystem and the system. These displacements and rotations are the results obtained from a model of a larger subsystem or a system that includes a simplified representation of the subsystem.

The choice of applied seismic analysis method depends on the desired level of precision and the level of complexity of the particular subsystem being designed. The equivalent static load method of analysis is predominantly used for civil structure-related seismic subsystems and is generally the preferred method because it is relatively simple and at least as conservative as the other more detailed methods. For example, the equivalent static load analysis method is generally used for miscellaneous steel platforms, stairs, and walkways, reinforced masonry block walls and enclosures, HVAC ducts and duct supports, electrical tray and tray supports, and conduits and conduit supports.

~~The time history or response spectra generated at the support point of the subsystem are utilized as the input motion for performing the seismic dynamic analysis of the subsystem. However, where these data are not readily available, the data generated for a distance away from the structural support point may be used. To account for the structural linkage (i.e., intervening structural element) between these two locations, the additional amplification of the response due to the presence of the intervening structural element can be calculated and the remote input motion can be transformed. For cases where the intervening structure is rigid (i.e., frequency > 50 Hz), the transformation can be achieved by adding the effect due to the rigid body motion of the intervening structure to the existing input motion at the remote location. The new translational time history at the interface location is generated by algebraic summation of the translational acceleration time history at the reference location and the time-history contribution arising from the rocking and torsional effects of the intervening structural element. The new translational response spectra are obtained by absolute sum of the translational response spectra at the reference location and the contributions arising from the rocking and torsional effects of the intervening structural element. For places where the intervening structural element~~ DCD_03.07.03-10

~~is judged to be flexible, the new ISRS are generated by incorporating the flexibility of the intervening structural element. Or alternatively, the seismic dynamic analysis of the subsystem shall be expanded to include the flexibility of the intervening structural element.~~ DCD_03.07.03-10

The time history or response spectra generated at the support locations of the subsystem are utilized as the input motion for performing the seismic dynamic analysis of the subsystem. However, where these input motions are not readily available, the input motions generated at the closest distances away from the structural support location can be adapted for use. The structural linkage (i.e., intervening structural element) between these two locations, and the additional amplification of the response due to the presence of the intervening structural element are considered in the analysis. For cases where the intervening structure is rigid (i.e., frequency > 50 Hz), the transformation effect due to the rigid body motion of the intervening structure can be taken into account by linear interpolation of the ISRS at the reference locations adjacent to the structural supported locations of the subsystem. Alternatively, the effect can be represented by adding a rigid link in the subsystem model from the reference location associated with the input motion to the support of subsystem location.

For places where the intervening structural element is flexible (i.e., frequency < 50 Hz), the seismic dynamic analysis of the subsystem model can be expanded to include the mass and stiffness of the flexible intervening structural element to analyze the subsystem response. Alternatively, the subsystem seismic input amplified time history and, if necessary, additional ISRS at the subsystem support locations can be generated by using a detailed de-coupled model of the flexible intervening structure provided the applicable de-coupling criteria of SRP 3.7.2 Acceptance Criteria 3B (Reference 3.7-35) or Section 4153.2 of NOG1-2004 (Reference 3.7-22) for cranes are met for the subsystem. When time histories of in-structure motions from dynamic analysis of the supporting soil-structure system are used, frequency content of the time histories is varied to be consistent with the broadening of ISRS. An acceptable method to vary the frequency content of the in-structure accelerations time history for the best estimate soil properties is by expanding and shrinking the time history within 1/(1 ± 0.15) so as to change the frequency content within ± 15%.

Torsional effects due to the significant effect of eccentric masses connected to a subsystem are included in the subsystem analysis. For rigid components (i.e., those with natural frequencies greater than the ZPA cutoff frequency of 50 Hz), the lumped mass is modeled at the center of gravity of the component with a rigid link to the appropriate point in the subsystem. For flexible components having a frequency less than the ZPA, the subsystem model is expanded to include an appropriate model of the component.

Regardless of the method chosen, to avoid resonance, the fundamental frequencies of components and equipment are preferably selected to be less than one half or more than twice the dominant frequencies of the support structure. If this is not practical, equipment and components with fundamental frequencies within this range are designed for any associated resonance effects in conjunction with all other applicable loads.

The equivalent static load method of analysis and the various modal response spectra analysis methods are described in the following subsections.

3.7.3.1.1 Equivalent Static Load Method of Analysis

The equivalent static load method involves the use of equivalent horizontal and vertical static forces applied at the center of gravity of various masses. The equivalent force at a mass location is computed as the product of the mass and the seismic acceleration value applicable to that mass location. Loads, stresses, or deflections obtained using the equivalent static load methods are adjusted to account for the relative motion between points of support when significant.

3.7.3.1.2 Single DOF, Single Mode Dominant or Rigid Structures and Components

For rigid structures and components, single DOF structures and components, or for cases where the response is such that the response of the system is single mode dominant, the following procedures may be used:

- For rigid SSCs (fundamental frequency greater than 50 Hz), an equivalent seismic load is defined for the direction of excitation as the product of the component mass and the ZPA value obtained from the applicable ISRS.

- A rigid component (fundamental frequency greater than 50 Hz), whose support can be adequately represented by a flexible spring, can be modeled as a single DOF model in the direction of excitation (horizontal or vertical directions). The equivalent static seismic load for the direction of excitation is defined as the product of the component mass and the seismic acceleration value corresponding to the natural frequency of the supported component from the applicable ISRS. If the frequency of the supported component is not determined, the peak acceleration from the applicable ISRS ~~times a factor of 1.5~~ of the supported component is used. Supported components~~s~~ which have been determined to have natural frequencies less than the frequency corresponding to the peak floor acceleration (i.e., whose natural frequencies are to the left of spectra peak on an acceleration versus the frequency spectra plot) also utilize the peak acceleration ~~times a factor of 1.5~~. DCD_03.07.03-6 DCD_03.07.03-6 DCD_03.07.03-6

- If the structure, equipment, or component has a distributed mass whose dynamic response is single mode dominant, the equivalent static seismic load for the direction of excitation is defined as the product of the component mass and the seismic acceleration value at the component natural frequency from the applicable ISRS times a factor of 1.5, with exceptions noted as follows. A factor of less than 1.5 may be used if justified, such as using a factor of 1.0 when the component natural frequency is in the rigid range (greater than 50 Hz), such that no dynamic amplification will occur. A factor of 1.0 is used for structures or equipment that can be represented as simply supported, fixed-simply supported, or fixed-fixed beams as discussed in References 3.7-36 and 3.7-37. In accordance with ASCE 4-98, Subsection 3.2.5.2 (Reference 3.7-9), for cantilever beams with uniform mass distribution, the equivalent-static-load base shear is determined using the peak acceleration, and the base moment is determined using the peak acceleration times a factor of 1.1. If the frequency of a structure, equipment, or component is not determined, the peak acceleration from the

applicable ISRS times a factor of 1.5 is used, unless a lower factor is applicable as discussed herein or otherwise justified. Any structures, equipment, or components which have been determined to have natural frequencies less than the frequency corresponding to the peak floor acceleration (i.e., whose natural frequencies are to the left of spectra peak on an acceleration versus the frequency spectra plot) also utilize the peak acceleration times a factor of 1.5 unless a lower factor is applicable as discussed herein or as otherwise justified.

3.7.3.1.3 Multiple DOF Response

This procedure applies to piping, instrumentation tubing, conduit, cable trays, HVAC, and other structural subsystems consisting of multiple spans. The equivalent static load method of analysis can be used for the design of piping systems, and the instrumentation and supports that have significant responses at several vibrational frequencies. In this case, a static load factor of 1.5 is applied to the peak accelerations of the applicable ISRS, unless a lower value is justified. For runs with axial supports, the acceleration value of the mass of piping in its axial direction may be limited to 1.0 times its calculated spectral acceleration value. The spectral acceleration value is based on the frequency of the piping system along the axial direction. The relative motion between support points is also considered.

3.7.3.1.4 Modal Response Spectra Analysis

The methods of modal response spectra analysis that have been utilized for the design of seismic category I and II SSCs are the envelope broadened response spectra method, the peak shifting method, the uniform support motion method and the independent support motion method, described in the following subsections.

3.7.3.1.5 Envelope Broadened Response Spectra Method

The envelope broadened response spectra method is based on the utilization of the ISRS that are developed for the US-APWR seismic category I structures and buildings. The envelope broadened response spectra method is discussed in Subsection 3.7.2.5.

3.7.3.1.6 Seismic Response Spectra Peak Shifting

The peak shifting method may be used in place of the broadened spectra method. It determines the natural frequencies $(f_e)_n$ of the system to be qualified in the broadened range of the maximum spectra acceleration peak. If no equipment or piping system natural frequencies exists in the ±15% interval associated with the maximum spectra acceleration peak, then the interval associated with the next highest spectra acceleration peak is selected and used in the following procedure.

Consider all *N* natural frequencies in the interval:

$$f_j - 0.15f_j \leq (f_e)_n \leq f_j + 0.15f_j$$

where

f_j = the frequency of maximum acceleration in the envelope spectra

n = 1 to N

The system is evaluated by performing N+3 separate analyses using the envelope un-broadened ISRS and the envelope un-broadened spectra modified by shifting the frequencies associated with each of the spectral values by a factor of +0.15; -0.15; and

$$\frac{(f_e)_n - f_j}{f_j}$$

where

n = 1 to N

The results of these separate seismic analyses are then enveloped to obtain the final result desired (e.g., stress, support loads, acceleration) at any given point in the system. If three different ISRS curves are used to define the response in the two horizontal and the vertical directions, then the shifting of the spectral values, as defined above, may be applied independently to these three response spectra curves.

3.7.3.1.7 Multiple Support Response Spectra Input Methods

The uniform support motion method and the independent support motion methods use multiple-input response spectra which account for the phasing and interdependence characteristics of the various support points. These methods are based on the guidelines provided by the "Pressure Vessel Research Committee Technical Committee on Piping Systems" (Reference 3.7-38) and have been most often applied to plant piping subsystems but are also applicable to other subsystems with multiple support points.

~~For select equipment (e.g., RCS components), the time history approach using a coupled model with supported structures is applied.~~ DCD_03.12-26 S01

3.7.3.1.7.1 Uniform Support Motion Method

For analyzing plant SSCs supported at multiple locations within a single structure, a uniform response spectrum is defined that envelopes all of the individual response spectra at the various support locations. The uniform response spectrum is applied at all support locations to calculate the maximum inertial responses of the plant SSCs. This is referred to as the uniform support motion method. Modal combinations for this method including missing mass computations must be performed in accordance with RG 1.92, Rev. 2 (Reference 3.7-27). The analysis of seismic anchor motions (i.e., maximum relative support displacement), is performed as a static analysis with all dynamic supports active and the results of this analysis are combined with the piping system seismic inertia analysis results by absolute summation. The seismic response spectrum, which envelopes the supports, is used in place of the spectra at each support in the envelope uniform response spectra. The contribution from the seismic anchor motion of the support points is assumed to be in phase and is added algebraically as follows:

$$q_i = d_j \Sigma P_{ij} \cancel{d_{ij}}$$ DCD_03.07.03-7

where

q_i = combined displacement response in the normal coordinate for mode i

d_j = maximum value of d_{ij}

~~d_{ij} = displacement spectral value for mode i associated with support j~~ MIC-03-03-00066

P_{ij} = participation factor for mode i associated with support j

Σ = summation for support points from j = 1 to N

N = total number of support points

The enveloped response spectra are developed as the seismic input in three perpendicular directions of the coordinate system to include the spectra at all floor elevations of the attachment points and the piping module or equipment, if applicable. The mode shapes and frequencies below the cut-off frequency are calculated in the response spectra analysis. The modal participation factors in each direction of the earthquake motion and the spectral accelerations for each significant mode are calculated. Based on the calculated mode shapes, participation factors, and spectral accelerations of individual modes, the modal inertia response forces, moments, displacements, and accelerations are calculated. For a given direction, these modal inertia responses are combined based on the consideration of closely spaced modes and high frequency modes to obtain the resultant forces, moments, displacements, accelerations, and support loads. The total seismic responses are combined by the SRSS method for all three earthquake directions.

3.7.3.1.7.2 Independent Support Motion Method

When there is more than one supporting structure, the independent support motion method for seismic response spectra may be used.

Each support group is considered to be in a random-phase relationship to the other support groups. The responses caused by each support group are combined by the absolute sum method. The analysis of piping systems for multiply supported piping with independent inputs will be consistent with the recommendation provided in Section 2.4 of NUREG-1061, Volume 4 (Reference 3.7-46), which describes independent support motion (ISM) methodology, sequence of combination, and high frequency modes. If the ISM method is utilized, the criteria presented in NUREG-1061 related to the ISM method are required to be followed according to SRP subsection 3.7.2.II, item 9 (Reference 3.7-16) as provided under SRP Acceptance Criteria. The displacement response in the modal coordinate becomes:

$$q_i = \Sigma P_{ij} d_{ij}$$

A support group is defined by supports that have the same time-history input. This usually means all supports located on the same floor (or portions of a floor) of a structure.

3.7.3.1.7.3 Analysis of Seismic Subsystems versus Qualification by Testing

For the purpose of seismic and dynamic qualification of civil structure-related SSCs by *analysis* using the methods described above in this section, the rigid range is defined as having a natural frequency greater than 50 Hz. This is consistent with the CSDRS defined in Subsection 3.7.1.1. However, for the purpose of *testing* equipment that is not sensitive to response levels caused by high frequency ground motions, rigid is defined as equipment with a natural frequency greater than 33 Hz. If the equipment to be tested is sensitive to the response caused by high frequency ground motions, then rigid is defined as equipment having a natural frequency greater than 50 Hz. This approach is further clarified in the following paragraphs.

Historically, there have been occurrences of ground motions which have caused an exceedance of a plant's design spectra in the high frequency range, where high frequency is defined as 10 Hz or greater. Based on this nuclear plant operating experience, the high frequency response motion exceedances were found to be non-damaging to passive civil structure-related components such as those addressed in the section above, which are typically qualified by analysis. However, nuclear industry experience has found that certain SSCs, in particular components such as relays and other electrical and instrumentation and control devices whose output signals could be affected by high frequency excitation, are potentially sensitive to high frequency motion and can be damaged by high frequency exceedances of the design spectra. A test program is established to identify, evaluate, and qualify or eliminate such SSCs that are potentially sensitive to high frequency exceedances. The US-APWR seismic and dynamic equipment qualification test program for active components including valves, piping, and other plant SSCs is in accordance with IEEE Std 344-2004 (Reference 3.7-13) and is addressed in Section 3.10.

3.7.3.2 Procedures Used for Analytical Modeling

Seismic subsystems are defined as those systems that are not analyzed in conjunction with basemats and subgrade, as previously discussed in Subsection 3.7.2. The procedures used for analytical modeling of subsystems include the use of mathematical computer models comprised of nodes and elements used to represent connections and members. Depending on the complexity of the subsystem, the models may be lumped mass stick models or FE models. The models contain sufficient detail and DOFs to represent the structural and seismic response of the subsystem, and are incorporated into the overall building model when required by the coupling criteria discussed in Subsection 3.7.2.3.4. Depending on the complexity of the seismic subsystem, structure, or component being analyzed, detailed member design may be performed by hand calculations using the results of the overall building structural and seismic analyses. Alternatively, the computer model may be sufficiently detailed to be used for the design calculation of the individual members. In all cases, the computer programs used for analytical modeling of subsystems are verified and validated in accordance with ANSI/ASME NQA-1-2004 (Reference 3.7-23) requirements.

3.7.3.3 Analysis Procedure for Damping

Energy dissipation within a structural system is represented by equivalent viscous dampers in the mathematical model. The damping coefficients used are based on the

material, load conditions, and type of construction used in the structural system. The SSE damping values to be used in the dynamic analysis for various seismic category I and II subsystems and their related supports are shown in Table 3.7.3-1(a). The damping values are based on RG 1.61 (Reference 3.7-15). The damping value of conduit, empty cable trays, and their related supports is similar to that of a bolted structure, namely 7% of critical. The damping value of filled cable trays and supports increases with increased cable fill and level of seismic excitation. The use of higher damping values for cable trays with flexible support systems (e.g., rod-hung trapeze systems, strut-hung trapeze systems, and strut-type cantilever and braced cantilever support systems) is permissible, subject to obtaining NRC review for acceptance on a case-by-case basis.

For subsystems that are composed of different material types, the composite modal damping approach with either the weighted mass or stiffness method is used to determine the composite modal damping value. Alternately, the minimum damping value may be used for these systems.

Piping systems are analyzed for SSE using 4% damping. Alternatively, frequency-dependent damping values may be utilized as noted and described in Tables 3.7.3-1(a) and 3.7.3-1(b). The seismic analysis of piping and other mechanical subsystems is addressed in further detail in Sections 3.9 and 3.12.

For subsystems analyzed with the time history direct integration method, Rayleigh damping is used. The Rayleigh damping matrix of the system [C] proportional to the stiffness matrix [K] and mass matrix [M] is obtained as $[C] = \alpha [M] + \beta [K]$. In order to model the dissipation of energy in the dynamic system in a conservative manner, the values of the coefficients α and β are adjusted to assure that the damping of the system in a selected range of dominant frequencies remains below the target values of critical damping ratios ξ_i. The selected damping ratio is in accordance with the requirements of RG 1.61. The dominant frequency range is selected considering the natural frequencies of the system being analyzed and the frequency content of the input seismic excitation.

3.7.3.4 Three Components of Earthquake Motion

For seismic category I subsystems, the three components of earthquake motion are considered in the same manner as described in Subsection 3.7.2.6.

Two horizontal components and one vertical component of seismic response spectra are employed as input to a modal response spectra analysis. The spectra are associated with the SSE. In the response spectra and equivalent static analyses, the effects of the three components of earthquake motion are combined using one of the following methods:

- The peak responses due to the three earthquake components from the response spectra analyses are combined using the SRSS method.

- The peak responses due to the three earthquake components are combined directly, using the assumption that when the peak response from one component occurs, the responses from the other two components are 40% of the peak (100%-40%-40% method). Combinations of seismic responses from the three earthquake components, together with variations in sign (plus or minus), are considered. This method is not used for piping systems.

3.7.3.5 Combination of Modal Responses

Where seismic subsystems are analyzed by the equivalent static load method of analysis, a combination of modal responses is not applicable. For this method of analysis, static load factors are applied to acceleration values, which are taken from the appropriate ISRS discussed in Subsection 3.7.2.5. The static load factors are chosen using the guideline of Reference 3.7-9 to be sufficiently conservative to capture multi-modal response effects.

For the response spectra method of analysis, the combination of modal responses is performed in the same manner as described in Subsection 3.7.2.7.

3.7.3.6 Use of Constant Vertical Static Factors

As discussed in Subsection 3.7.2.10, the plant design does not utilize constant vertical static factors in the seismic design.

3.7.3.7 Buried Seismic Category I Piping, Conduits, and Tunnels

Buried seismic category I piping, conduits, and tunnels are not present in the US-APWR standard plant design. Physical space is reserved and planned to provide a site-specific seismic category I ESWPT ~~passing underneath the north end of the T/B which is also a structure designed as site-specific~~which connects the east and west ends of the ESWPC to the site specific UHS structures. A representative anticipated configuration of the ESWPT is shown on the general arrangement drawings in Section 1.2. MIC-03-03-00054

The ESWPT provides access for in-service inspection. It will support safety-related piping, conduit, and equipment. Backfill material is present on the sides and likely the top of the ESWPT, and engineered structural or concrete backfill may be utilized beneath the tunnel as well. To design and qualify the site-specific safety-related SSCs mounted or housed within the tunnel, the following requirements apply to the site-specific design of the ESWPT as described in Subsection 3.7.2.8:

- ISRS are required. To generate the ISRS on the tunnel walls, basemat and roof, a SASSI program (Reference 3.7-17) SSI analysis is required if soil supported. The SASSI analysis is required to be documented and comply with the same general requirements described for the R/B-PCCV-containment internal structure SASSI analysis with the exception that no stick model is required. Instead, plate elements are to be directly included to represent the tunnel in the SASSI model.

3.7.3.8 Methods for Seismic Analysis of Category I Concrete Dams

The US-APWR standard plant design does not include dams. It is the responsibility of the COL Applicant to perform any site-specific seismic analysis for dams that may be required.

3.7.3.9 Methods for Seismic Analysis of Aboveground Tanks

It is the responsibility of the COL Applicant to design seismic category I below- or above-ground liquid-retaining metal tanks such that they are enclosed by a ~~tornado-~~ DCD_02-03 S01

~~missile~~tornado/hurricane missile protecting concrete vault or wall, in order to confine the emergency gas turbine fuel supply. DCD_02-03 S01

The other seismic category I liquid-retaining vessels utilized in the design are reinforced concrete vessels whose walls and floors form part of the building structural framework, including the following:

- Spent fuel pit, located in the R/B with top of vessel at level 4F
- Refueling cavity, located in PCCV with top of vessel at level 4F
- Fuel transfer canal, which connects the spent fuel pit and refueling cavity
- Cask washdown pit located in the R/B with top of vessel at level 4F
- Cask loading pit and fuel inspection pit located in the R/B and connected to the spent fuel pit with a canal, with tops of vessels at level 4F
- New fuel storage pit located in the R/B with top of vessel at level 4F
- Refueling water storage pit, located in PCCV below level 2F

Hydrodynamic loads on these liquid-retaining vessels are determined using methods that conform to the provisions of Subsection II.14 of SRP 3.7.3 (Reference 3.7-35) and guidance of ASCE 4-98, Subsection 3.5.4 (Reference 3.7-9). The horizontal response analysis considers both the impulsive mode (in which a portion of the water moves in unison with the tank wall) and the horizontal convective mode (water motion associated with wave oscillation). The seismic analysis of convective hydrodynamic effects also considers the maximum wave oscillation with respect to the potential of creating flooding, which is discussed in Section 3.4.

3.7.4 Seismic Instrumentation

The proposed seismic instrumentation program for the US-APWR is in accordance with NUREG-0800, SRP 3.7.4 (Reference 3.7-39) and all aspects of 10 CFR 50, Appendix S (Reference 3.7-7), which requires that “suitable instrumentation must be provided so that the seismic response of nuclear power plant features important to safety can be evaluated promptly after an earthquake.” Appendix S of 10 CFR 50 (Reference 3.7-7) also requires a shutdown of the plant if vibratory ground motion exceeding that of the OBE ground motion occurs, or significant plant damage occurs.

3.7.4.1 Comparison with Regulatory Guide 1.12

The proposed seismic instrumentation program is generally in accordance with RG 1.12 and RG 1.166 (References 3.7-40, 3.7-41), and consistent with the methodology used for seismic analysis that is discussed in Subsection 3.7.2. The seismic design of US-APWR standard plant is based on site-independent seismic response analysis of basemats resting on generic supporting media that are subjected to the CSDRS input control motion. The site-independent OBE is defined as 1/3 of the CSDRS presented in Subsection 3.7.1.1. Verification of the site-independent standard design is performed

during seismic analyses that consider site-specific conditions, such as soil layering, basemat embedment, water table depth etc. The FIRS, which are developed consistent with the site-specific GMRS define the site-specific control design motion.

The criteria that define the vibratory motion that requires the shutdown of the US-APWR plant are based on the site-specific OBE. The 5% damping FIRS associated with the site-specific OBE must be enveloped by 1/3 of the 5% damping CSDRS. The conditions that require a shutdown of the US-APWR plant are defined by the site-specific OBE at the free-field instrumentation located at grade in the plant yard, unless otherwise justified by the COL Applicant. Unless site-specific OBE is set at 1/3 of the site-specific SSE or lower, these spectra shall be obtained from analysis using as input the site-specific OBE ground motion and properties of the supporting media that are strain-compatible to the site-specific OBE ground motion. When the site-specific OBE is equal or lower than 1/3 of site-specific SSE, the spectra scaled from the 5% damping site-specific SSE response spectra may be used directly for OBE exceedance checks. An OBE exceedance check is performed in accordance with Section 4 of RG 1.166 (Reference 3.7-41) using both a response spectrum check and a cumulative absolute velocity (CAV) check. The comparison evaluation is to be performed within 4 hours of the earthquake using data obtained from the three components of the earthquake motion as defined by the three orthogonal axes of the standard plant (two horizontal and one vertical) on the uncorrected earthquake records. The evaluation is also to include a check on the operability of the seismic instrumentation as mandated by Section 4.3 of RG 1.166 (Reference 3.7-41).

The locations of seismic monitors for the US-APWR standard plant are provided in Subsection 3.7.4.2. The COL Applicant shall provide free-field seismic instrumentation in the vicinity of the power block area at surface grade, which shall be used for shutdown determination, unless otherwise justified. Any such justification shall be based on conditions and requirements specific to the site, and shall include justification for evaluation of OBE exceedance using only measurements from instrumentation installed on the buildings and the structures of the US-APWR standard plant.

The calculation of the CAV is performed in the manner provided in Electric Power Research Institute (EPRI) Report TR-100082 (Reference 3.7-42). As stated in RG 1.166 (Reference 3.7-41), the range of the spectral velocity limit should be 1.0 to 2.0 Hz which is different than that recommended by EPRI. In accordance with RG 1.166 (Reference 3.7-41), for each component of the free-field ground motion, the CAV should be calculated as follows: (1) the absolute acceleration (g units) time-history is divided into 1-second intervals, (2) each 1-second interval that has at least 1 exceedance of 0.025 g is integrated over time, (3) all the integrated values are summed together to arrive at the CAV. The approaches in EPRI Report NP-5930 (Reference 3.7-43) and EPRI Report TR-100082 (Reference 3.7-42) provide additional guidance on determining the CAV.

The site-specific OBE is exceeded and plant shutdown is required in accordance with the criteria of RG 1.166 (Reference 3.7-41), if the first of the following three conditions in combination with either the second or third conditions are met:

1. Any calculation of CAV described above yields a value that is greater than 0.16 g-second.

2. 5% damping ARS generated by free-field ground motion ARS are higher than 0.2 g at frequencies between 2 and 10 Hz, or higher than the site-specific OBE ARS between 2 and 10 Hz, whichever is greater.

3. 5% damping velocity response spectra generated by free-field ground motion are higher than 6 in./sec at frequencies between 1 and 2 Hz, or higher than the site-specific OBE velocity response spectra between 1 and 2 Hz, whichever is greater.

If free-field instrumentation is not used, the criteria of RG 1.166 Appendix A are used for OBE exceedance checks, it is assumed that the checks of CAV and free-field ground spectra are exceeded, and shutdown of the plant is required if the 5% damping spectra are exceeded at any of the in-structure instrumentation.

Additionally, low-level seismic effects would be included in the design of certain equipment potentially sensitive to a number of such events, based on a percentage of the responses calculated for the SSE.

3.7.4.2 Location and Description of Instrumentation

Consistent with the guidance of RG 1.12 (Reference 3.7-40), the seismic instrumentation for the US-APWR standard plant is solid-state multi-channel digital instrumentation with computerized recording and playback capability that allows the processing of data at the plant site within 4 hours of a seismic or other dynamic event.

The US-APWR triaxial time-history accelerograph consists of a centralized digital time history analyzer/recorder with multi-channel capability, which is located in a panel in a room adjacent to the plant MCR, and triaxial acceleration sensors that are provided at the following plant locations:

- On the PCCV basemat, located in the R/B on the B1F level at elevation -23 ft, 4 in.

- On level 2F of PCCV at elevation 25 ft, 3 in., located in the southwest quadrant outside the steam generator and reactor coolant compartment.

- On level 4F of PCCV operating deck slab at elevation 76 ft, 5 in., located in the southwest quadrant outside the steam generator and reactor coolant compartment underneath the access stairs adjacent to the west PCCV buttress.

- On the basemat of the east PS/B on the B1F level at elevation -23 ft, 4 in., in the non-safety related turbine generator anteroom.

- On level 1F of the east PS/B at elevation 3 ft, 7 in., in the non-safety related turbine generator control room.

- Unless otherwise justified by the COL Applicant based on site-specific conditions, at a surface grade location in the vicinity of the power block area, sufficiently far away from structures in order to appropriately measure free-field ground motion.

The locations listed above correlate to structural elements in the structures which have been modeled as mass points in the dynamic analysis so that the measured motion can be directly compared to the design spectra. The instrumentation mounted at the locations listed above is not mounted on equipment, piping, supports, or secondary structural frame members. These locations have been reviewed in accordance with RG 8.8 (Reference 3.7-44) and determined to be consistent with maintaining dose rates as low as practical and maintaining occupational radiation exposures as low as is reasonably achievable for access and maintenance of the instrumentation.

A time-history analyzer/recorder is provided which has the capability to provide pre-event recording time of 3 seconds minimum and post-event recording time of 5 seconds minimum, and to record at least 25 minutes of sensed motion. The recorder portion of the time-history analyzer is to have the capability of a sample rate of at least 200 samples per second in each of the three orthogonal directions of the plant, a bandwidth of 0.20 Hz to 100 Hz, and a dynamic range of 1,000:1 zero to peak. The triaxial acceleration sensors are to have the same dynamic range as the time-history analyzer recorder and a frequency range of 0.20 Hz to 100 Hz. The triggers of the tri-axial acceleration sensor units are to be capable of being set within the range of 0.001g to 0.02g. Batteries are provided with enough capacity for a minimum of 25 minutes of system operation at any time over a 24-hour period, without recharging, in combination with a battery charger whose line power is connected to an uninterruptible power supply.

The seismic instrumentation serves no safety-related function and, therefore, has no nuclear safety design requirements. However, its design and location are in accordance with RG 1.12 (Reference 3.7-40), which requires that the seismic instrumentation:

- will not be affected by the failure of adjacent SSCs during an earthquake;
- will operate during all modes of plant operation, including periods of plant shutdown; and
- is protected as much as practical against accidental impacts.

As required by RG 1.12 (Reference 3.7-40), the seismic instrumentation is rigidly mounted and oriented so that the horizontal components are parallel to the horizontal axes of the standard plant used in the seismic analyses. These features of the seismic monitoring instrumentation are obtained by qualifying the equipment to IEEE Std 344-2004 (Reference 3.7-13); the seismic qualification program is discussed in Section 3.10.

3.7.4.3 Control Room Operator Notification

The US-APWR standard plant is designed such that triggering of the instrumentation described above is annunciated in the MCR of the plant. For sites which will have more than one US-APWR unit, only one unit is required to have seismic instrumentation, provided that the anticipated seismic response at each of the units is considered essentially the same and provided that annunciation is provided at all unit MCRs. The COL Applicant is to determine from the site-specific geological and seismological conditions if multiple US-APWR units at a site will have essentially the same seismic

response, and based on that determination, choose if more than one unit is provided with seismic instrumentation at a multiple-unit site.

3.7.4.4 Comparison with Regulatory Guide 1.166

As previously discussed in Subsection 3.7.4.1, the seismic instrumentation and OBE exceedance checks meet the intent of RG 1.166 (Reference 3.7-41). In the case that the COL Applicant provides acceptable justification for not utilizing free-field instrumentation, the OBE exceedance checks can be performed using only uncorrected earthquake data for the three orthogonal plant directions (two horizontal and one vertical) obtained from seismic instrumentation installed at five plant locations (two basemat locations and three upper level locations as described in Subsection 3.7.4.2). It should be noted that the use of five instrument locations is more conservative than the interim OBE exceedance guidelines given in Appendix A of RG 1.166 (Reference 3.7-41), which allow basemat-level only instrumental checks.

The seismic instrumentation program must be in accordance with the guidelines of RG 1.166 (Reference 3.7-41) and EPRI NP-6695 (Reference 3.7-45) which are summarized as follows:

- Assure that a file containing information on all seismic instrumentation is maintained at the plant in accordance with regulatory position C1.1 of RG 1.166 (Reference 3.7-41).
- Implement planning for post-earthquake walkdown inspections by pre-selecting equipment and structures for inspections and pre-determining the content of the baseline inspections.
- Implement guidelines for actions to be performed immediately after an earthquake, including a check of the neutron flux monitoring sensors as part of the specific MCR board checks.
- Assure proper evaluation of ground motion records.
- Assure that after an earthquake at the plant site, an operability check is performed on the seismic instrumentation.
- If a shutdown is required, assure that the pre-shutdown inspections, including a check of the containment isolation system, are performed.

3.7.4.5 Instrument Surveillance (Including calibration and testing)

The seismic instrumentation is in accordance with the type and location requirements discussed in Subsection 3.7.4.2 and RG 1.12 (Reference 3.7-40). The instrumentation requires minimal maintenance and in-service inspection, as well as minimal time and numbers of personnel to conduct installation and maintenance. The seismic monitoring instrumentation is configured such that testing or maintenance can be performed on a single channel without affecting the functioning of other channels.

A seismic monitoring system preoperational test is outlined in Chapter 14.

As required by RG 1.12 (Reference 3.7-40), instrumentation systems are to be given channel checks every 2 weeks for the first 3 months of service after startup. Failures of devices normally occur during initial operation. After the initial 3-month period and 3 consecutive successful checks, monthly channel checks are sufficient. The monthly channel check is to include checking the batteries. The channel functional test should be performed every 6 months. Channel calibration should be performed during each refueling outage at a minimum.

3.7.4.6 Program Implementation

The COL Applicant is to identify the implementation milestone for the seismic instrumentation implementation program based on the discussion in Subsections 3.7.4.1 through 3.7.4.5.

3.7.5 Combined License Information

COL 3.7(1) *The COL Applicant is to confirm that the site-specific PGA at the basemat level control point of the CSDRS is less than or equal to 0.3 g.* MIC-03-03-00041

COL 3.7(2) *The COL Applicant is to perform an analysis of the US-APWR standard plant seismic category I design to verify that the site-specific FIRS at the basemat level control point of the CSDRS are enveloped by the site-independent CSDRS.* MIC-03-03-00041

COL 3.7(3) *It is the responsibility of the COL Applicant to develop analytical models appropriate for the seismic analysis of buildings and structures that are designed on a site-specific basis including, but not limited to, the following:* MIC-03-03-00041

- *PSFSVs (seismic category I)*
- *ESWPT (seismic category I)*
- *UHSRS (seismic category I)*

COL 3.7(4) *To prevent non-conservative results, the COL Applicant is to review the resulting level of seismic response and determine appropriate damping values for the site-specific calculations of ISRS that serve as input for the seismic analysis of seismic category I and seismic category II subsystems.* MIC-03-03-00041

COL 3.7(5) *The COL Applicant is to assure that the horizontal FIRS defining the site-specific SSE ground motion at the bottom of seismic category I or II basemats envelope the minimum response spectra required by 10 CFR 50, Appendix S, and the site-specific response spectra obtained from the response analysis.* MIC-03-03-00041

COL_3.7(6) *The COL Applicant is to develop site-specific GMRS and FIRS ~~by an analysis methodology, which accounts for the upward propagation of the GMRS~~. The FIRS are compared to the CSDRS to assure that the US-APWR standard plant seismic design is valid for a particular site. If the FIRS are not enveloped by the CSDRS, the US-APWR standard plant seismic design is modified as part of the COLA in order to validate the US-APWR for installation at that site.* MIC-03-03-00041 MIC-03-03-00066

COL_3.7(7) *The COL Applicant is to determine the allowable static and dynamic bearing capacities based on site conditions, including the properties of fill concrete placed to provide a level surface for the bottom of foundation elevations, and to evaluate the bearing loads to these capacities.* MIC-03-03-00041

COL_3.7(8) *The COL Applicant is to evaluate the strain-dependent variation of the material dynamic properties for site materials.* MIC-03-03-00041

COL_3.7(9) *The COL Applicant is to assure that the design or location of any site-specific ~~seismic category I~~safety-related SSCs, for example pipe tunnels or duct banks, will not expose those SSCs to possible impact due to the failure or collapse of non-seismic category I structures, or with any other SSCs that could potentially impact, such as heavy haul route loads, transmission towers, non safety-related storage tanks, etc.* MIC-03-03-00041 DCD_03.07.02-88

COL_3.7(10) *It is the responsibility of the COL Applicant to ~~further address structure-to-structure interaction if the specific site conditions can be important for the seismic response of particular US-APWR seismic category I structures, or may result in exceedance of assumed pressure distributions used for the US-APWR standard plant design.~~address the potential SSSI effect of the R/B complex and T/B on the site specific seismic category I structures.* MIC-03-03-00041 MIC-03-03-00054

COL_3.7(11) *~~Deleted~~It is the responsibility of the COL Applicant to confirm the masses and frequencies of the PCCV polar crane and fuel handling crane and to determine if coupled site-specific analyses are required.* DCD_03.07.02-102 MIC-03-03-00041

COL_3.7(12) *It is the responsibility of the COL Applicant to design seismic category I below- or above-ground liquid-retaining metal tanks such that they are enclosed by a ~~tornado missile~~tornado/hurricane missile protecting concrete vault or wall, in order to confine the emergency gas turbine fuel supply.* MIC-03-03-00041 DCD_02-03 S01

COL_3.7(13) *The COL Applicant is to set the value of the OBE that serves as the basis for defining the criteria for shutdown of the plant, according to the site specific conditions.* MIC-03-03-00041

COL_3.7(14) *The COL Applicant is to determine from the site-specific geological and seismological conditions if multiple US-APWR units at a site will have essentially the same seismic response, and based on that determination, choose if more than one unit is provided with seismic instrumentation at a multiple-unit site.* MIC-03-03-00041

COL_3.7(15) *Deleted* MIC-03-03-00041

COL 3.7(16) *The COL Applicant shall provide free-field seismic instrumentation in the vicinity of the power block area at surface grade which shall be used for shutdown determination, unless otherwise justified. Any such justification shall be based on conditions and requirements specific to the site, and shall include justification for evaluation of OBE exceedance using only measurements from instrumentation installed on the buildings and the structures of the US-APWR standard plant.* MIC-03-03-00041

COL 3.7(17) *Deleted* MIC-03-03-00041

COL 3.7(18) *Deleted* MIC-03-03-00041

COL 3.7(19) *The COL Applicant is to identify the implementation milestone for the seismic instrumentation implementation program based on the discussion in Subsections 3.7.4.1 through 3.7.4.5.* MIC-03-03-00041

COL 3.7(20) *The COL Applicant is to validate the site-independent seismic design of the standard plant for site-specific conditions, including geological, seismological, and geophysical characteristics, and to develop the site-specific GMRS.* MIC-03-03-00041

COL 3.7(21) *The COL Applicant is responsible for the seismic design of those seismic category I and seismic category II SSCs that are not part of the US-APWR standard plant using site-specific SSE design ground motion.* MIC-03-03-00041 MIC-03-03-00066

COL 3.7(22) *The COL Applicant is required to perform site-specific seismic analyses, including SSI analysis which may consider seismic wave transmission incoherence ~~and analysis of the CAV~~ of the seismic input motion, in order to determine if high-frequency exceedances of the CSDRS could be transmitted to SSCs in the plant superstructure with potentially damaging effects.* MIC-03-03-00041 MIC-03-03-00066

COL 3.7(23) *The COL Applicant is to verify that the results of the site-specific SSI analysis for the broadened ISRS ~~and basement walls lateral soil pressures~~ are enveloped by the US-APWR standard design.* MIC-03-03-00041 MIC-03-03-00054

COL 3.7(24) *The COL Applicant is to verify that the site-specific ratios V/A and AD/V^2 (A, V, D, are PGA, ground velocity, and ground displacement, respectively) are consistent with characteristic values for the magnitude and distance of the appropriate controlling events defining the site-specific uniform hazard response spectra.* MIC-03-03-00041

COL 3.7(25) *The COL Applicant referencing the US-APWR standard design is required to perform a site-specific SSI analysis for the R/B complex ~~PCCV containment internal structure, and PS/B model~~, utilizing ~~the program~~a SASSI program such as ACS SASSI (Reference 3.7-17) which contains time history input incoherence function capability. The SSI analysis using SASSI is required in order to confirm that site-specific effects are enveloped by the standard design. ~~After the SASSI analysis is first performed for a specific unit, subsequent COLAs for other units may be able to forego SASSI analyses if the FIRS and GMRS derived for those subsequent units are much smaller than the US-APWR standard plant CSDRS, and if the subsequent unit can also provide justification through comparison of site-specific geological and seismological characteristics.~~* MIC-03-03-00041 MIC-03-03-00054 DCD_03.07.02-107

COL 3.7(26) *SSI effects are also considered by the COL Applicant in site-specific seismic design of any seismic category I and II structures that are not included in the US-APWR standard plant. ~~Consideration of structure-to-structure interaction is discussed in Subsection 3.7.2.8.~~ The site-specific SSI analysis is performed for buildings and structures including, but not limited to, to the following:* MIC-03-03-00041 MIC-03-03-00054

- *Seismic category I ESWPT*
- *Seismic category I PSFSV*
- *Seismic category I UHSRS*

COL 3.7(27) *It is the responsibility of the COL Applicant to perform any site-specific seismic analysis for dams that may be required.* MIC-03-03-00041

COL 3.7(28) *Deleted.~~The overall basemat dimensions, basemat embedment depths, and maximum height of the US-APWR R/B, PCCV, and containment internal structure on their common basemat are given in Table 3.7.1-3 and as updated by the COL Applicant to include site-specific seismic category I structures.~~* MIC-03-03-00041 MIC-03-03-00054

COL 3.7(29) *Table 3.7.2-1, as updated by the COL Applicant to include site-specific seismic category I structures, presents a summary of dynamic analysis and combination techniques including types of models and computer programs used, seismic analysis methods, and method of combination for the three directional components for the seismic analysis of the US-APWR standard plant seismic category I buildings and structures.* MIC-03-03-00041

COL 3.7(30) *The COL Applicant is to provide site-specific design ground motion time histories and durations of motion.*

3.7.6 References

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Table 3.7.1-1 CSDRS Horizontal Acceleration Values and Control Points

Control Point (Hz)	Acceleration (g)
~~0.5~~2% Damping	
A (50)	0.3
B (12)	~~1.49~~1.06
C (2.5)	~~1.79~~1.28
D (0.25)	~~0.22~~0.17
E (0.1)	~~0.035~~0.028
~~2~~3% Damping	
A (50)	0.3
B (12)	~~1.06~~0.92
C (2.5)	~~1.28~~1.10
D (0.25)	~~0.17~~0.154
E (0.1)	~~0.028~~0.0251
5% Damping	
A (50)	0.3
B (12)	0.78
C (2.5)	0.94
D (0.25)	0.14
E (0.1)	0.0226
7% Damping	
A (50)	0.3
B (12)	0.68
C (2.5)	0.82
D (0.25)	0.13
E (0.1)	0.021
10% Damping	
A (50)	0.3
B (12)	0.57
C (2.5)	0.68
D (0.25)	0.12
E (0.1)	0.019

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Notes:

1. 0.3 g PGA
2. Based on RG 1.60, Rev. 1 (Reference 3.7-6) amplification factors
3. For Control Points D & E, acceleration is computed as follows:

Acceleration = $(\varpi^2 D / 386.4 \text{ in/sec}^2) \times F_A \times 0.3$

ϖ = 2π x frequency (rad/sec)

D = Displacement (in)

F_A = Amplification Factor from Regulatory Guide 1.60

Table 3.7.1-2 CSDRS Vertical Acceleration Values and Control Points

MIC-03-03-00054

Control Point (Hz)	Acceleration (g)
~~0.5~~2% Damping	
A (50)	0.3
B (12)	~~1.49~~1.06
C (3.5)	~~1.70~~1.22
D (0.25)	~~0.15~~0.12
E (0.1)	~~0.024~~0.018
~~2~~3% Damping	
A (50)	0.3
B (12)	~~1.06~~0.92
C (3.5)	~~1.22~~1.05
D (0.25)	~~0.12~~0.106
E (0.1)	~~0.018~~0.0164
5% Damping	
A (50)	0.3
B (12)	0.78
C (3.5)	0.89
D (0.25)	0.094
E (0.1)	0.015
7% Damping	
A (50)	0.3
B (12)	0.68
C (3.5)	0.78
D (0.25)	0.086
E (0.1)	0.014
10% Damping	
A (50)	0.3
B (12)	0.57
C (3.5)	0.65
D (0.25)	0.08
E (0.1)	0.012

Notes:

1. 0.3 g PGA
2. Based on RG 1.60, Rev. 1 (Reference 3.7-6) amplification factors
3. For Control Points D & E, acceleration is computed as follows:

Acceleration = $(\varpi^2 D / 386.4 \text{ in/sec}^2) \times F_A \times 0.3$

ϖ = 2π x frequency (rad/sec)

D = Displacement (in)

F_A = Amplification Factor from Regulatory Guide 1.60

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~~Table 3.7.1-3 Major Dimensions of Seismic Category I Structures(1)~~

~~Structure~~	~~Basemat Embedment Depth Below Grade (ft)~~	~~Basemat Width and Length (ft)~~	~~Max. Structure Height~~
~~R/B~~	~~38' 10"~~	~~210' x 309'(3)~~	~~190' - 9"~~
~~PCCV~~	~~See note 2.~~	~~See note 2.~~	~~268' - 3"~~
~~Containment Internal Structure~~	~~See note 2.~~	~~See note 2.~~	~~175' 9" (top of pressurizer compartment)~~
~~PS/B~~	~~38' 10"~~	~~66' 0" x 111' 6"(3)~~	~~87' 4"~~

~~Notes:~~

1. ~~The dimensions shown are approximate and are based on the general arrangement drawings in Section 1.2.~~
2. ~~The R/B, PCCV, and containment internal structure rest on a common basemat as shown on the general arrangement drawings in Section 1.2.~~
3. ~~Width and length are the distances between column lines of exterior walls.~~

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Table 3.7.1-3 Summary of SRP 3.7 Option 1, Approach 1 Requirements Compliance

Requirement		H2(90)	H1(180)	V(UP)
Time Histories Requirements				
Total duration in seconds (if ≥ 20 seconds OK)		22.08	22.08	22.08
Rise time in seconds: Arias intensity 5% (if 1 second or longer OK)		2.815	3.031	1.337
Strong motion duration in seconds: Arias intensity between 5% and 75% (minimum 6 seconds and satisfying NUREG/CR-6728 criteria)(1)		9.543	7.868	10.35
Decay time in seconds: Arias intensity between 75% and 100% (if 5 seconds or longer OK)		9.722	11.181	10.393
Statistical independence (if absolute value ≤ 0.16 OK)		-0.0179	-0.0179	
		-0.0552		-0.0552
			-0.0696	-0.0696
V/A (if 7.51 ≤ V/A ≤ 66.40 OK)(2)		53.179	66.355	42.661
AD/V^2 (if $1.86 \le AD/V^2 \le 16.79$ OK)(2)		4.306	2.997	5.766
Response Spectra Requirements				
SRP 3.7.1 Option 1, Approach 1				
Number of points with acceleration ratio < 1 (if ≤ 5 OK)	2%	2	5	5
	3%	0	0	1
	5%	0	0	0
	7%	0	0	0
	10%	4	0	2
Number of points with acceleration ratio < 0.9 (if 0 OK)	All	0	0	0
Power Spectral Density Function Requirements				
Number of points below 80% of target between 0.3 and 50 hz (if 0 OK)		0	0	0

(1) Refer to Table 3.7.1-4.

(2) Refer to Table 3.7.1-5.

~~Table 3.7.1-4 Comparison of 5% Damping ARS of Synthesized Time History CSDRS~~ MIC-03-03-00054

~~Time History~~	~~Frequency Range~~		~~0.1 – 1 Hz~~	~~1 – 10 Hz~~	~~10 – 100 Hz~~	~~0.1 – 100 Hz~~
	~~No. Freq. Data Points~~		~~100~~	~~100~~	~~100~~	~~300~~
~~Horizontal H1~~	~~ARS/CSDRS ratio~~	~~Min.~~	~~0.9420.94~~	~~0.9370.94~~	~~0.9200.92~~	~~0.9200.92~~
		~~Max~~	~~1.2511.25~~	~~1.2621.26~~	~~1.2181.22~~	~~1.2621.26~~
	~~Max. No. of Data Point Non-Exceedances Within Any One Particular Frequency Window~~ ~~(1)~~		~~1~~	~~4~~	~~7~~	~~7~~
~~Horizontal H2~~	~~ARS/CSDRS ratio~~	~~Min.~~	~~0.8980.90~~	~~0.94~~	~~0.9680.97~~	~~0.8980.90~~
		~~Max~~	~~1.2921.29~~	~~1.14231.14~~	~~1.1261.13~~	~~1.2921.29~~
	~~Max. No. of Data Point Non-Exceedances Within Any One Particular Frequency Window~~ ~~(1)~~		~~7~~	~~6~~	~~6~~	~~7~~
~~Vertical V~~	~~ARS/CSDRS ratio~~	~~Min.~~	~~0.9420.94~~	~~0.9310.93~~	~~0.9660.97~~	~~0.9310.93~~
		~~Max~~	~~1.2061.21~~	~~1.2121.21~~	~~1.1821.18~~	~~1.2121.21~~
	~~Max. No. of Data Point Non-Exceedances Within Any One Particular Frequency Window~~ ~~(1)~~		~~6~~	~~3~~	~~6~~	~~6~~

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~~Note:~~

1. ~~Maximum number of frequency data points in any one particularsingle sequence (frequency window) for which the acceleration values of the time histories ARS are below those of the CSDRS.~~ DCD_03.07.02-35

Table 3.7.1-4 Magnitudes and Distance Bins and Strong Motion Duration Criteria (NUREG/CR-6728, Table 3-2, Reference 3.7-14)

M	R (Km)	Duration	
		Rock	Soil
6.5 (6 – 7)	10–50	**3.1**–7.0	3.6–8.2
	50–100	5.1–11.6	5.7–12.8
	100–200	8.1–18.3	8.7–19.5
7.5 (7+)	10–50	6.6–14.0	7.2–16.1
	50–100	8.7–19.5	12.2–27.5
	100–200	11.7–26.3	16.2–**36.5**

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~~Table 3.7.1-5 Duration of Motion of US-APWR Time Histories with Respect to Arias Intensity~~

	~~Arias Intensity~~		~~Duration (seconds)~~
	~~Time for 5% (seconds)~~	~~Time for 75% (seconds)~~	
~~H1~~	~~3.94~~	~~11.46~~	~~7.52~~
~~H2~~	~~4.635~~	~~11.78~~	~~7.145~~
~~V~~	~~2.08~~	~~10.84~~	~~8.77~~

Table 3.7.1-5 CEUS V/A & AD/V2 Mean Ratios ± One Standard Deviation

Distance Bin	M	V / A (cm/sec/g), σ_{ln} (1)	AD / V^2, σ (1)	V/A / $e^{(\sigma_{ln})}$ (in/sec/g) (2)	V/A * $e^{(\sigma_{ln})}$ (in/sec/g) (2)	AD/V^2/ $e^{(\sigma_{ln})}$	AD/V^2 * $e^{(\sigma_{ln})}$
10-50, Rock	6.32	31.75, 0.51	6.58, 0.70	**7.51**	20.82	3.27	13.25
10-50, Soil	6.41	51.74, 0.35	3.49, 0.47	14.35	28.91	2.18	5.58
50-100, Rock	6.38	32.59, 0.33	4.66, 0.52	9.22	17.85	2.77	7.84
50-100, Soil	6.57	56.04, 0.36	3.01, 0.48	15.39	31.62	**1.86**	4.86
10-50, Rock	7.38	58.24, 0.72	7.78, 0.63	11.16	47.11	4.14	14.61
10-50, Soil	7.47	128.74, 0.27	3.57, 0.35	38.69	**66.4**	2.52	5.07
50-100, Rock	7.49	50.29, 0.56	10.60, 0.46	11.31	34.66	6.69	**16.79**

(1) See NUREG/CR-6728, Table 3-6, Reference 3.7-14.

(2) Units are changed to facilitate comparison to time history results.

Table 3.7.1-6 Generic Soil Profile Categories

Category (Initial V_S [in top 30m])	Depth to Rock* (ft) for each Category (ft)
270 m/s	200 500
560 m/s	~~100~~ ~~200~~ 500
900 m/s	100 200
2,032 m/s	100

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* For soil and soft rock profiles 270 m/sec and 560 m/sec, underlying baserock conditions reflect soft rock with a shear wave velocity of 1 km/sec. For firm rock profiles 900 m/sec and 2,032 m/sec, underlying baserock conditions reflect hard rock with a shear wave velocity of 2.83 km/sec.

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~~Table 3.7.1-7 Spectra Matching Requirements for Converted Time Histories~~

~~SRP 3.7.1 Criteria for 5% Critical Damping:~~	~~H1 (180)~~ SSE_x	~~H2 (090)~~ SSE_y	~~V (up)~~ SSE_z
~~Average of converted/target acceleration ratios for all frequency points (if > 1.0 then acceptance criteria are met)~~	~~1.0705~~	~~1.0402~~	~~1.0509~~
~~Rise time duration~~			
~~Arias Intensity 5%~~	~~3.94~~	~~4.635~~	~~2.08~~
~~Required for Magnitude 6.5 earthquake~~ (1) (2)	~~1~~	~~1~~	~~1~~
~~Strong motion time duration~~			
~~Arias Intensity 75% - 5%~~	~~7.52~~	~~7.145~~	~~8.77~~
~~Required for Magnitude 6.5 earthquake~~ (1) (2)	~~7~~	~~7~~	~~7~~
~~Decay time duration~~			
~~Arias Intensity 100% - 75%~~	~~10.63~~	~~10.31~~	~~11.24~~
~~Required for Magnitude 6.5 earthquake~~ (1) (2)	~~5~~	~~5~~	~~5~~
~~Statistical Independence~~			
~~Correlation coefficient X and Y (if absolute value < 0.16 then acceptance criteria are met)~~	~~0.0892~~	~~0.0892~~	
~~Correlation coefficient X and Z (if absolute value < 0.16 then acceptance criteria are met)~~	~~-0.0654~~		~~-0.0654~~
~~Correlation coefficient Y and Z (if absolute value < 0.16 then acceptance criteria are met)~~		~~-0.0836~~	~~-0.0836~~
~~*SRP 3.7.1 Option 1, Approach 2*~~			
~~Number of points with acceleration ratio > 1.30 (if = 0 then acceptance criteria are met)~~	~~0~~	~~0~~	~~0~~
~~Number of points with acceleration ratio < 0.90 (if = 0 then acceptance criteria are met)~~	~~0~~	~~0~~	~~0~~
~~Number of windows wider than 9 points below the target spectra (if = 0 then acceptance criteria are met)~~	~~0~~	~~0~~	~~0~~

~~Notes:~~

~~(1) The seed recorded time history earthquake for the US-APWR Standard Plant CSDRS has a Magnitude of 6.5 (References 3.7-55 and 3.7-56).~~

~~(2) See Table 2.3-1 of ASCE 4-98 (Reference 3.7-9) for guidance on length of time appropriate for rise, strong motion, and decay.~~

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Table 3.7.1-7 Magnitudes, Distances, and Median Peak Accelerations

Profile	Magnitude	Distance(km)	PGAH(g)	PGAV(g)
270-200	7.5	68.0	0.268	0.117
270-500	7.5	62.0	0.232	0.124
560-500	7.5	59.5	0.259	0.130
900-100	7.5	68.0	0.198	0.078
900-200	7.5	65.0	0.204	0.087
2032-100	7.5	52.0	0.193	0.089

Table 3.7.2-1 Summary of Dynamic Analysis and Combination Techniques

Summary of Dynamic Analyses & Combination Techniques					
Model	**Analysis Method**	**Program**	**Three Components Combination (for purposes of dynamic analysis)**	**Modal Combination**	
Three-dimensional R/B complex ~~PCCV containment internal structure~~ SSI Model (1)	Time History Analysis in Frequency Domain using sub-structuring technique	ACS SASSI	SRSS	N/A	DCD_03.07.02-35 MIC-03-03-00054
Three-dimensional ~~RCL~~ R/B complex ~~PCCV containment internal structure~~ FE Model (2)	1g Static Analysis & Time History Analysis in ~~Frequency~~Time Domain	ANSYS	N/A(2)	N/A	DCD_03.07.02-108 (8) MIC-03-03-00054 DCD_03.07.02-108 (8)
Three-dimensional ~~PS~~T/B SSI Model(3)	Time History Analysis in Frequency Domain using sub-structuring technique	ACS SASSI	SRSS	N/A	DCD_03.07.02-35 MIC-03-03-00054
Three-dimensional ~~PS~~T/B FE models(2)	1g Static Analysis & Time History Analysis in ~~Frequency~~Time Domain	ANSYS	N/A(2)	N/A	DCD_03.07.02-108 (8) MIC-03-03-00054 DCD_03.07.02-108 (8)

Notes:

1. The three-dimensional ~~RCL~~ R/B complex ~~PCCV containment internal structure~~ SSI model is addressed in Technical Report~~s MUAP-10001 and~~ MUAP-10006 (Reference~~s 3.7-47 and~~ 3.7-48). MIC-03-03-00054
2. The FE models for the ~~RCL~~ R/B complex is ~~PCCV containment internal structure on their common basemat and the PS/Bs are~~ used ~~only~~ for validation of the dynamic FE seismic models and for static analysis for design of structural members and components as addressed in Section 3.8. DCD_03.07.02-35
3. The three-dimensional ~~PS~~T/B model is addressed in Technical Report~~s MUAP-10001 and MUAP-10006 (References 3.7-47 and 3.7-48)~~ MUAP-11002 (Reference 3.7-61). MIC-03-03-00054

Table 3.7.2-2 Concrete Material Constants

	Modulus of Elasticity (Young's Modulus) E_c (ksi)	Shear Modulus G_c (ksi)	Poisson's Ratio $\nu_{\cancel{c}}$	Remark
PCCV	4,769	2,040	0.17	f'_c = 7,000 psi
R/B	~~3,605~~4,031	~~1,540~~1,723	0.17	f'_c = 5~~4~~,000 psi
Containment Internal Structure	3,605	1,540	0.17	f'_c = 4,000 psi

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~~Table 3.7.2-3~~ ~~Deleted~~~~Material Properties of Models Used for Seismic Response Analyses (Sheet 1 of 2)~~

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~~Stiffness Level~~	~~Model~~	~~Structural Component~~	~~Stiffness~~	~~Damping~~	~~Design Basis~~		
					~~SSE Load~~	~~ISRS~~	~~Max. Displ.~~
~~Full (Uncracked) Stiffness~~	~~R/B Complex~~	~~SC modules (containment internal structure)~~	~~See Table 3.8.3-4~~		~~X~~	~~X~~	~~X~~
		~~Pre-stressed (PCCV)~~	~~100%~~	~~3%~~	~~X~~	~~X~~	~~X~~
		~~Reinforced Concrete (R/B)~~	~~100%~~	~~4%~~	~~N/C~~	~~X~~	~~X~~
		~~Composite (fuel handling srea~~	~~100%~~	~~4% conc. 3% steel~~	~~N/C~~	~~X~~	~~X~~
		~~Steel~~	~~100%~~	~~3%~~	~~X~~	~~N/A~~	~~N/A~~
		~~RCL~~	~~100%~~	~~3%~~			
		~~Massive Concrete~~	~~100%~~	~~4%~~	~~X~~	~~X~~	~~X~~
	~~PS/B~~	~~Reinforced Concrete~~	~~100%~~	~~4%~~	~~N/C~~	~~X~~	~~X~~
		~~Steel~~	~~100%~~	~~3%~~	~~X~~	~~N/A~~	~~N/A~~

~~Table 3.7.2-3 DeletedMaterial Properties of Models Used for Seismic Response Analyses (Sheet 2 of 2)~~

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~~Stiffness Level~~	~~Model~~	~~Structural Component~~	~~Stiffness~~	~~Damping~~	~~Design Basis~~		
					~~SSE Load~~	~~ISRS~~	~~Max. Displ.~~
~~Reduced (Cracked) Stiffness~~	~~R/B Complex~~	~~SC module (containment internal structure)~~	~~See Table 3.8.3-4~~		~~X~~	~~X~~	~~X~~
		~~Pre-stressed (PCCV)~~	~~50%~~	~~5%~~	~~X~~	~~X~~	~~X~~
		~~Reinforced Concrete (R/B)~~	~~50%~~	~~7%~~	~~X~~	~~X~~	~~X~~
		~~Composite (fuel handling area~~	~~50% concrete~~	~~7% conc. 4% steel~~	~~X~~	~~X~~	~~X~~
		~~Steel~~	~~100%~~	~~4%~~	~~X~~	~~N/A~~	~~N/A~~
		~~RCL~~	~~100%~~	~~3%~~	~~N/A~~	~~X~~	~~X~~
		~~Massive Concrete~~	~~100%~~	~~4%~~	~~X~~	~~X~~	~~X~~
	~~PS/B~~	~~Reinforced Concrete~~	~~50%~~	~~7%~~	~~X~~	~~X~~	~~X~~
		~~Steel~~	~~100%~~	~~4%~~	~~X~~	~~N/A~~	~~N/A~~

~~Note:~~

~~(1) Consistent with industry practice, no reduction in axial stiffness is required for the cracked condition~~

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Table 3.7.2-3 Material Properties of Models Used for Seismic Response Analyses

Stiffness Level	Structural Component	Stiffness	Damping
Full (Uncracked) Stiffness	SC module (CIS)	Loading Condition A in Table 3.8.3-4	
	Pre-stressed (PCCV)	100%	3%
	Reinforced Concrete	100%	4%
	Composite (FH/A)	See note (1)	4% concrete 3% steel
	Steel	100%	3%
	RCL	100%	3%
	Massive concrete	100%	4%
Reduced (Cracked) Stiffness	SC module (CIS)	Loading Condition B in Table 3.8.3-4	
	Pre-stressed (PCCV)	50%	5%
	Reinforced Concrete	50%	7%
	Composite (FH/A)	See note (2)	7% concrete 4% steel
	Steel	100%	4%
	RCL	100%	3%
	Massive concrete	100%	4%

(1) See equations in Section 02.4.1.1.6 of MUAP-10006

(2) See equations in Section 02.4.1.1.6 of MUAP-10006, use E = 50% E_c

Table 3.7.3-1(a) SSE Damping Values

Welded and friction-bolted steel structures and equipment (%)	4
Bearing bolted structures and equipment (%)	7
Prestressed concrete structures (%)	5
Reinforced concrete structures (%)	7(4)
Steel-Concrete Modules (%)	5(4)
Piping systems(1)	4
Full cable trays & related supports (%)	10(2)
Empty cable trays and related supports (%)	7
Full Conduits & related supports (%)	7
Empty conduits & related supports (%)	5
HVAC pocket lock ductwork (%)	10
HVAC companion angle ductwork (%)	7
HVAC welded ductwork (%)	4
Cabinets and panels for electrical equipment (%)	3
Equipment such as welded instrument racks and tanks (impulsive mode) (%)	3(3)
Motors, fans, housings, pressure vessels, heat exchangers, pumps, valve bodies (%)	3

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Table 3.7.3-1(b) OBE Damping Values

Welded and friction-bolted steel structures and equipment (%)	3
Bearing bolted structures and equipment (%)	5
Prestressed concrete structures (%)	3
Reinforced concrete structures (%)	4
Steel Concrete Modules (%)	4
Piping systems(1)	3
Full cable trays & related supports (%)	7(2)
Empty cable trays and related supports (%)	5
Full conduits & related supports (%)	5
Empty conduits & related supports (%)	3
HVAC pocket lock ductwork (%)	7
HVAC companion angle ductwork (%)	5
HVAC welded ductwork (%)	3
Cabinets and panels for electrical equipment (%)	2
Equipment such as welded instrument racks and tanks (impulsive mode)(%)	2(3)
Motors, fans, housings, pressure vessels, heat exchangers, pumps, valve bodies (%)	2

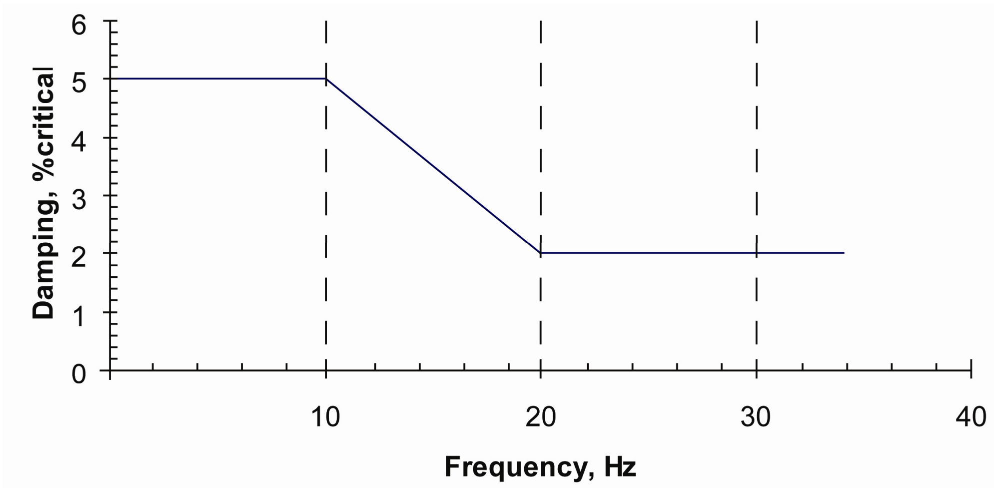
Notes for Tables 3.7.3-1(a) and 3.7.3-1(b):

1. As an alternative for response spectrum analyses using an envelope of the SSE or OBE response spectra at all support points (uniform support motion), frequency-dependent damping values shown in the graph below may be used, subject to the following restrictions:

   - Frequency-dependent damping should be used completely and consistently, if at all. Damping values for equipment other than piping are to be consistent with the values in the above table and RG 1.61 (Reference 3.7-15).

   - Use of the specified damping values is limited only to response spectral analyses. Acceptance of the use of the specified damping values with other types of dynamic analyses (e.g., time-history analyses or independent support motion method) requires further justification.

- When used for reconciliation or support optimization of existing designs, the effects of increased motion on existing clearances and online mounted equipment should be checked.
- Frequency-dependent damping is not appropriate for analyzing the dynamic response of piping systems using supports designed to dissipate energy by yielding.
- Frequency-dependent damping is not applicable to piping in which stress corrosion cracking has occurred, unless a case-specific evaluation is provided and reviewed, and found acceptable by the NRC staff.



2. The use of higher damping values for cable trays with flexible support systems (e.g., rod-hung trapeze systems, strut-hung trapeze systems, and strut-type cantilever and braced cantilever support systems) is permissible, subject to obtaining NRC review for acceptance on a case-by-case basis.
3. Use 0.5% damping for sloshing mode for tanks
4. Refer to Table 3.8.3-4 for appropriate damping values of the containment internal structure DCD_03.07.02-35

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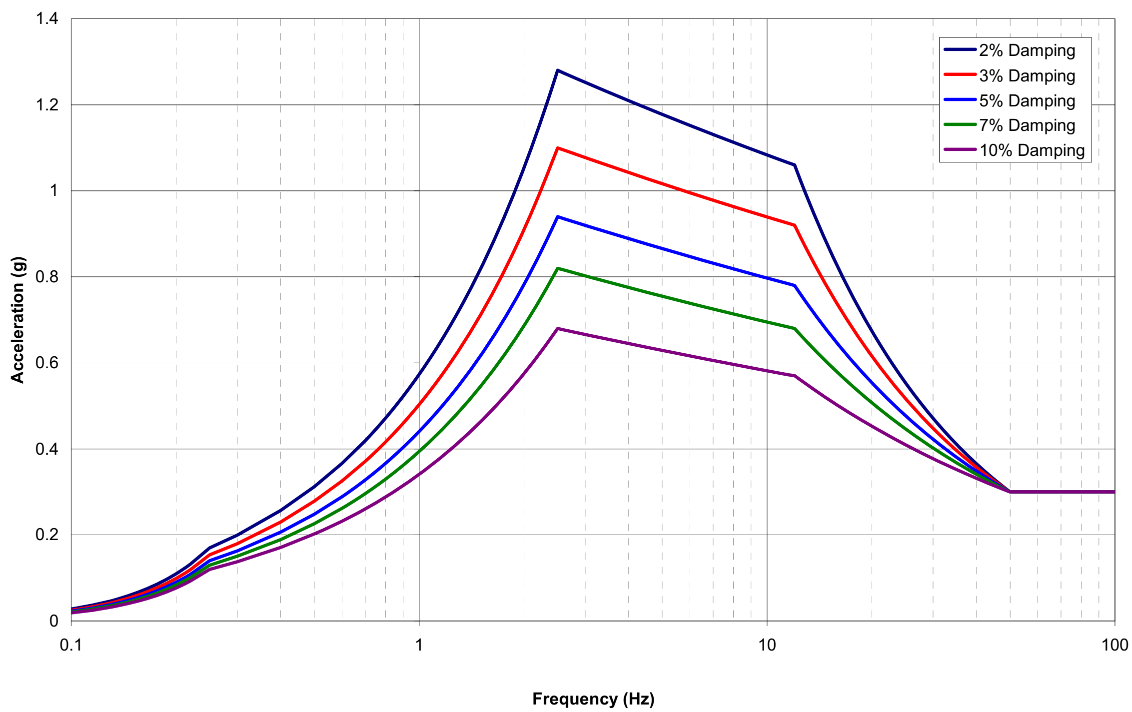

Figure 3.7.1-1 US–APWR Horizontal CSDRS

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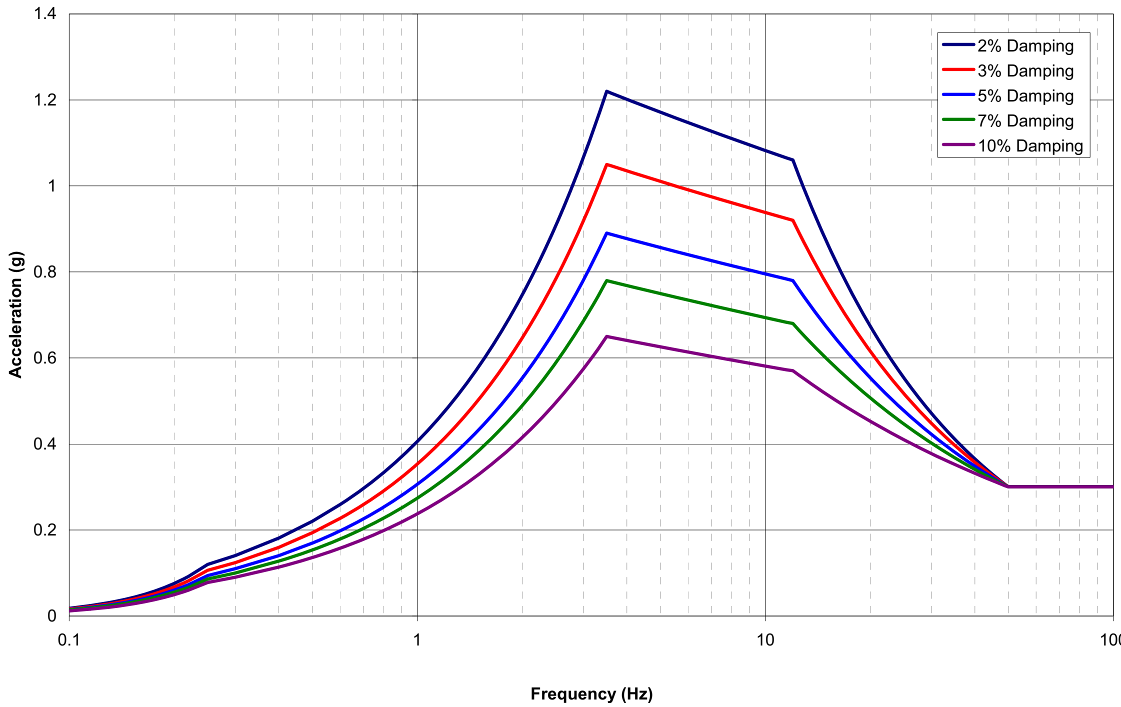

Figure 3.7.1-2 US–APWR Vertical CSDRS

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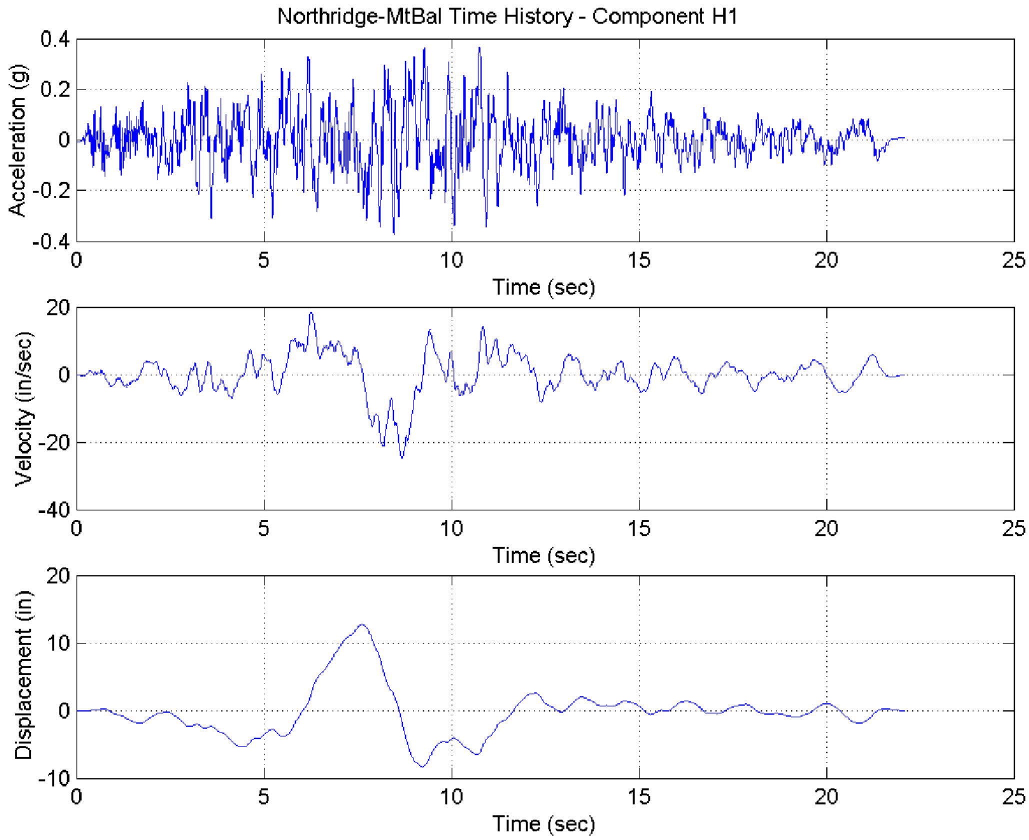

Figure 3.7.1-3 Acceleration, Velocity, and Displacement Time History for Component H1 [NS]

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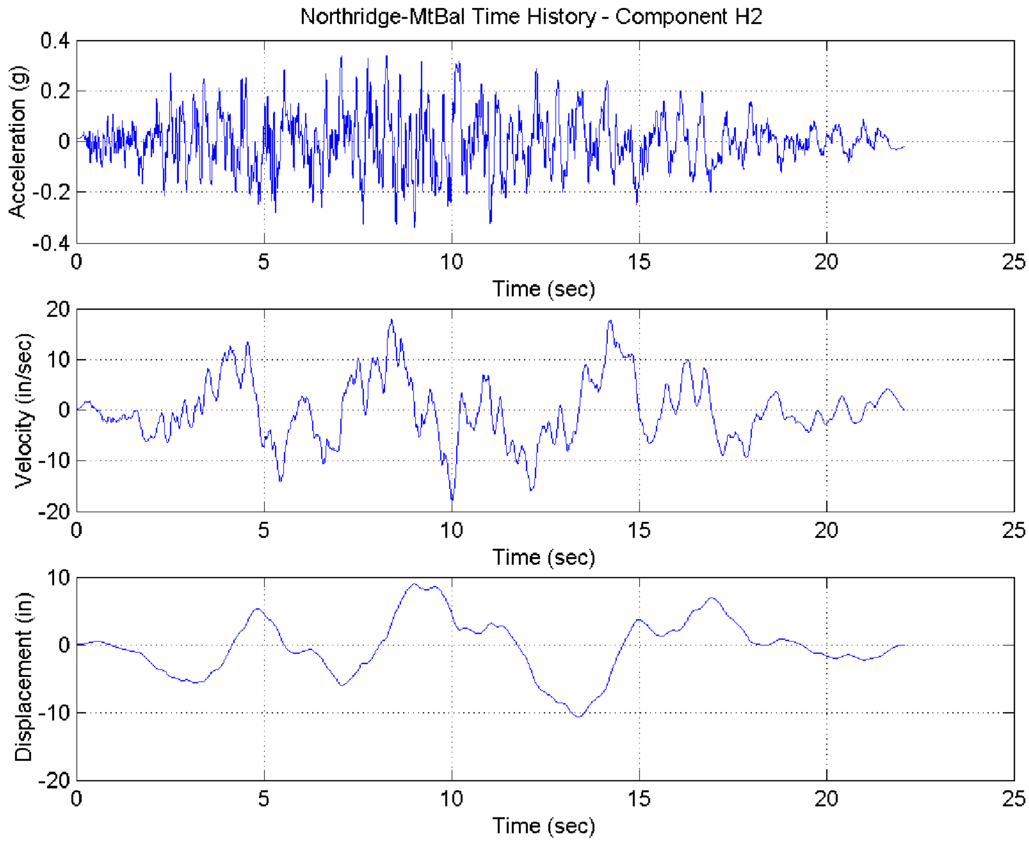

Figure 3.7.1-4 Acceleration, Velocity, and Displacement Time History for Component H2 [EW]

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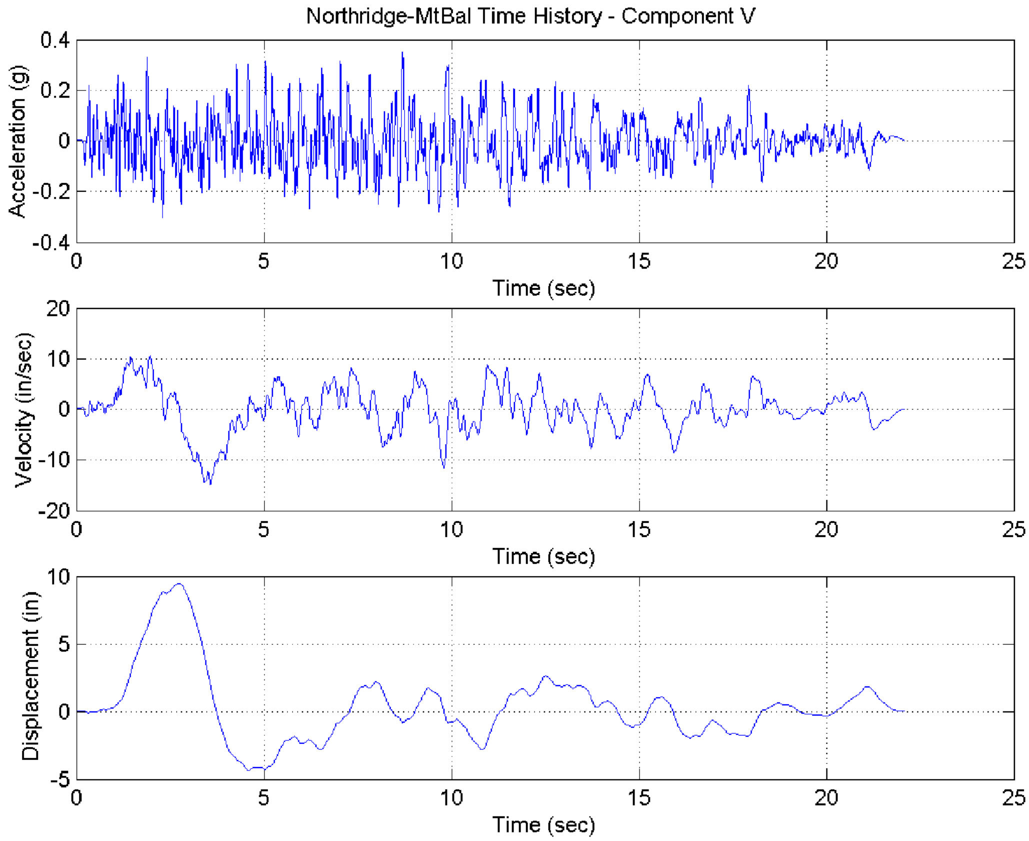

Figure 3.7.1-5 Acceleration, Velocity, and Displacement Time History for Component V

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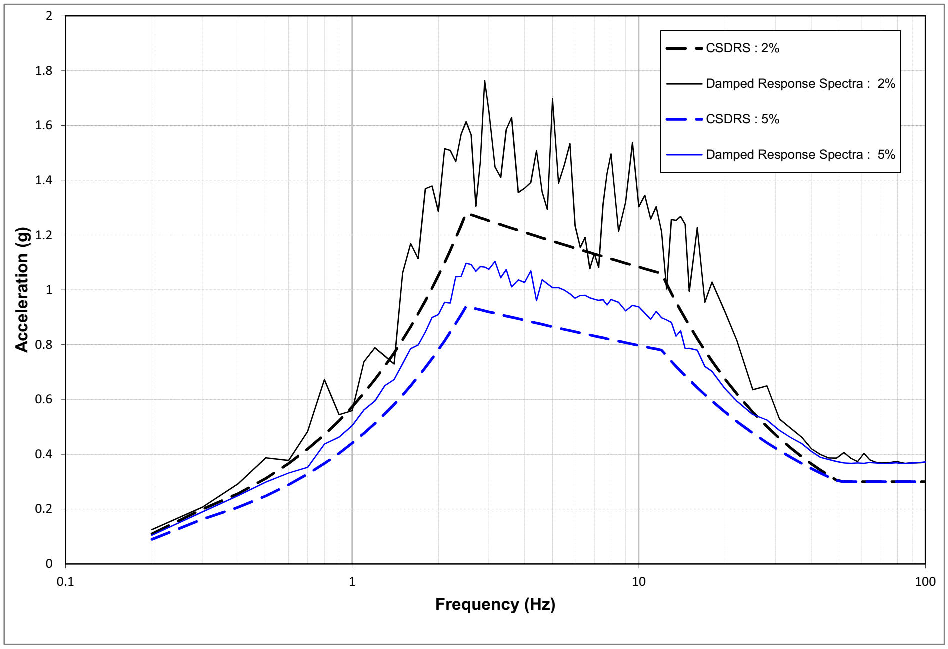

Figure 3.7.1-6a ~~5% Damped Time History Response Spectra (H1)~~Damped Response Spectra Plots for Northridge Mount Baldy Component H1 (180) [NS]

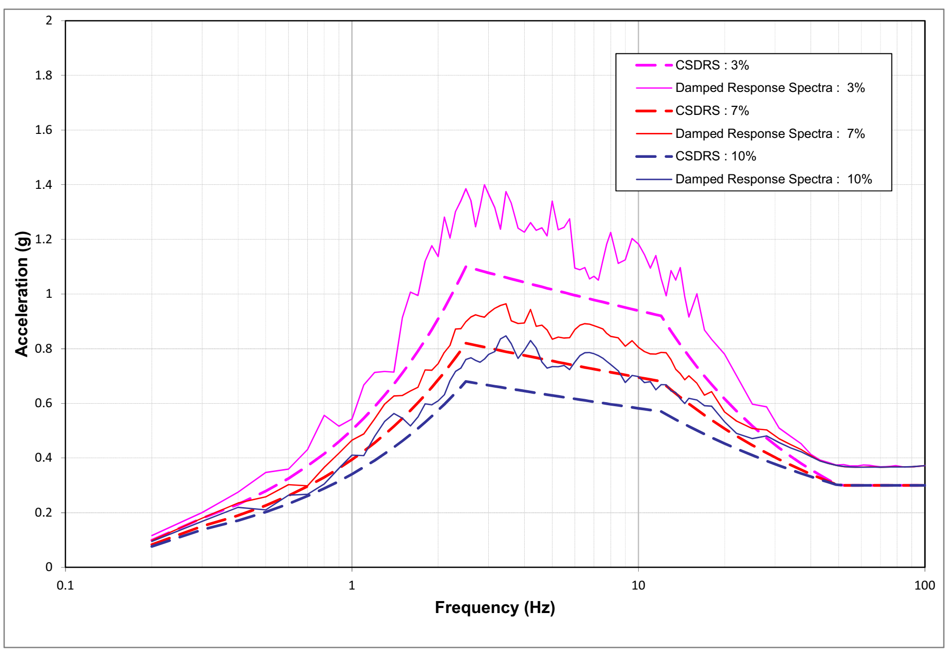

Figure 3.7.1-6b Damped Response Spectra Plots for Northridge Mount Baldy Component H1 (180) [NS]

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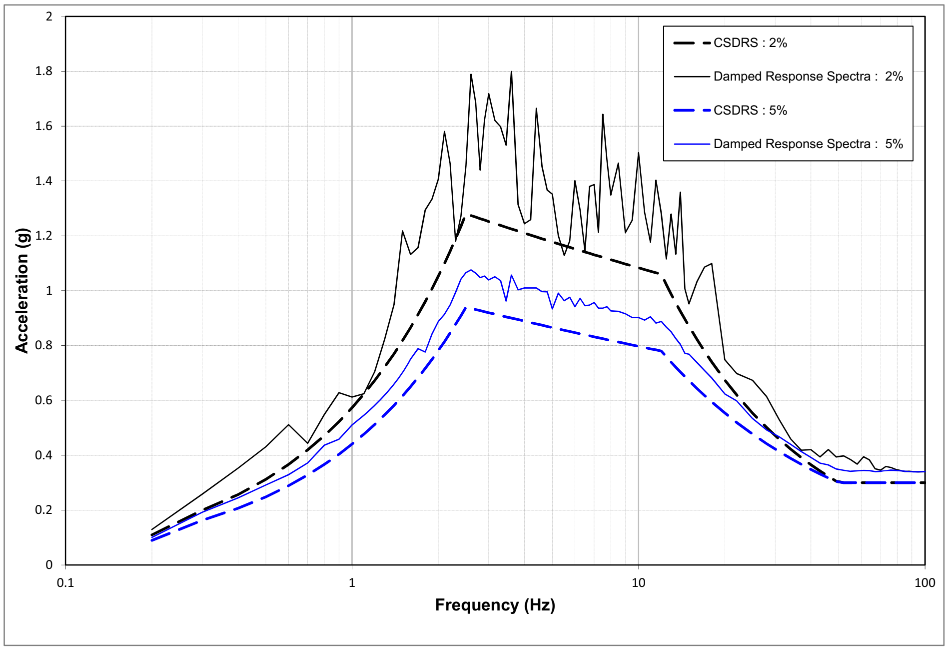

Figure 3.7.1-7a ~~5% Damped Time History Response Spectra (H2)~~Damped Response Spectra Plots for Northridge Mount Baldy Component H2 (090) [EW]

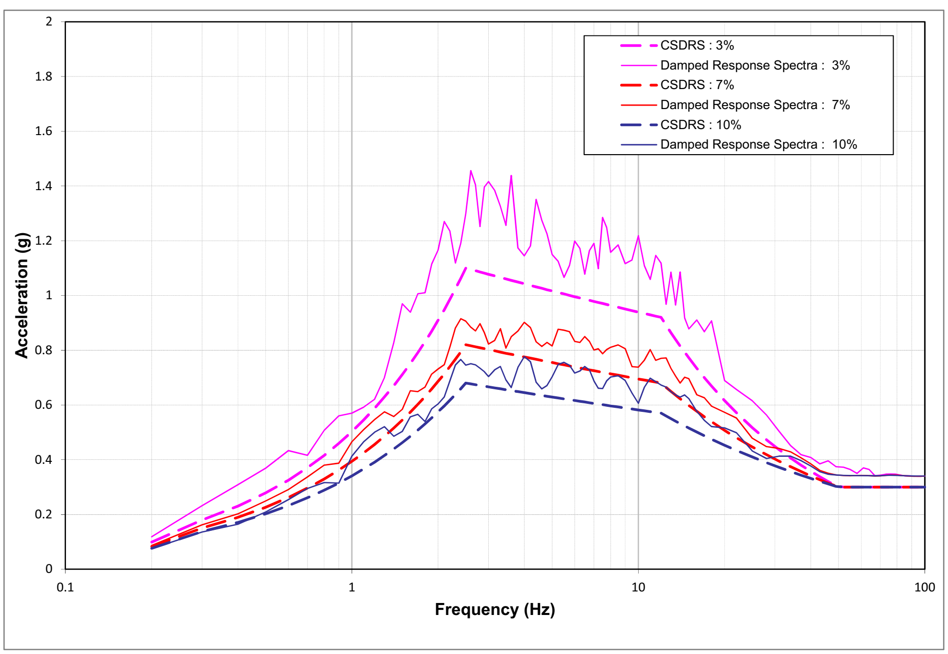

Figure 3.7.1-7b Damped Response Spectra Plots for Northridge Mount Baldy Component H2 (090) [EW]

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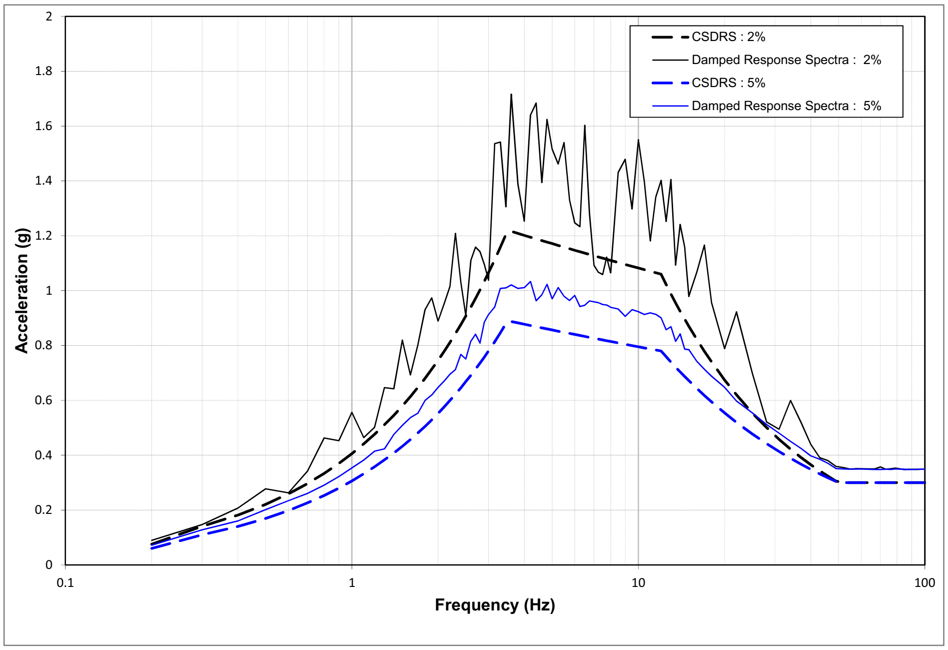

Figure 3.7.1-8a ~~5% Damped Time History Response Spectra (V)~~Damped Response Spectra Plots for Mount Baldy Component V (UP)

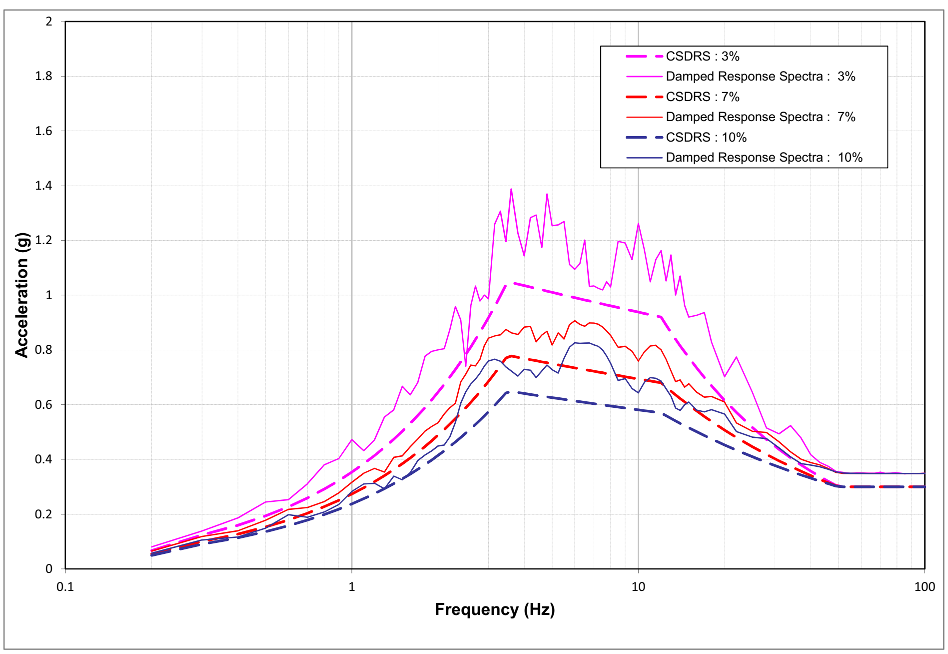

Figure 3.7.1-8b Damped Response Spectra Plots for Mount Baldy Component V (UP)

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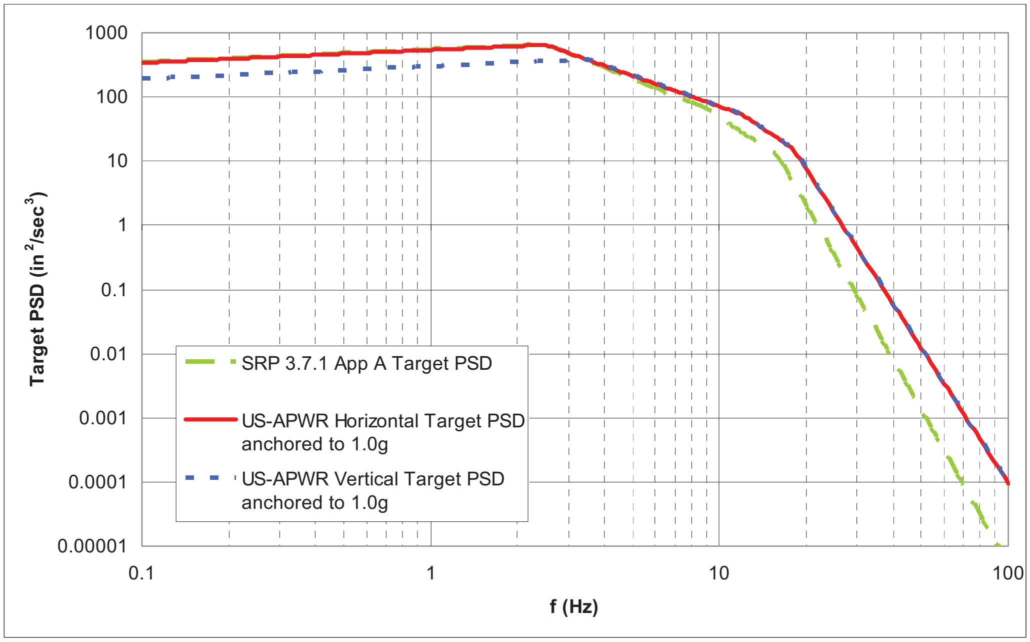

The final horizontal target PSD anchored to 1.0g is as follows:

$$S_{0H}(f) = \begin{cases} 650(f/2.5)^{0.2} & \text{for } f < 2.5\text{Hz} \\ 650(2.5/f)^{1.6} & \text{for } 2.5\text{Hz} \le f < 12\text{Hz} \\ 52.9(12.0/f)^{3} & \text{for } 12\text{Hz} \le f < 18\text{Hz} \\ 15.7(18/f)^{7} & \text{for } 18\text{Hz} \le f \end{cases} \qquad (\text{in}^2/\text{sec}^3)$$

The final vertical target PSD anchored to 1.0g is as follows:

$$S_{0V}(f) = \begin{cases} 380(f/3.5)^{0.2} & \text{for } f < 3.5\text{Hz} \\ 380(3.5/f)^{1.6} & \text{for } 3.5\text{Hz} \le f < 12\text{Hz} \\ 52.9(12.0/f)^{3} & \text{for } 12\text{Hz} \le f < 18\text{Hz} \\ 15.7(18/f)^{7} & \text{for } 18\text{Hz} \le f \end{cases} \qquad (\text{in}^2/\text{sec}^3)$$

Figure 3.7.1-9 US-APWR Final Horizontal and Vertical Target PSDs

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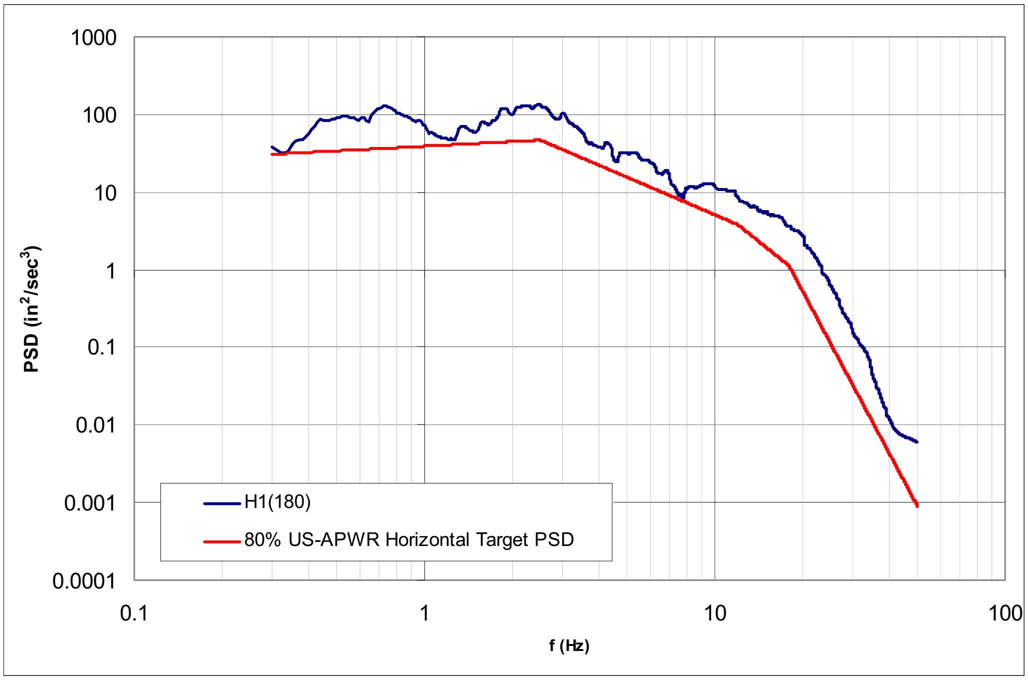

Figure 3.7.1-10 Smoothed Power Spectral Density Plots for Component H1 (180)

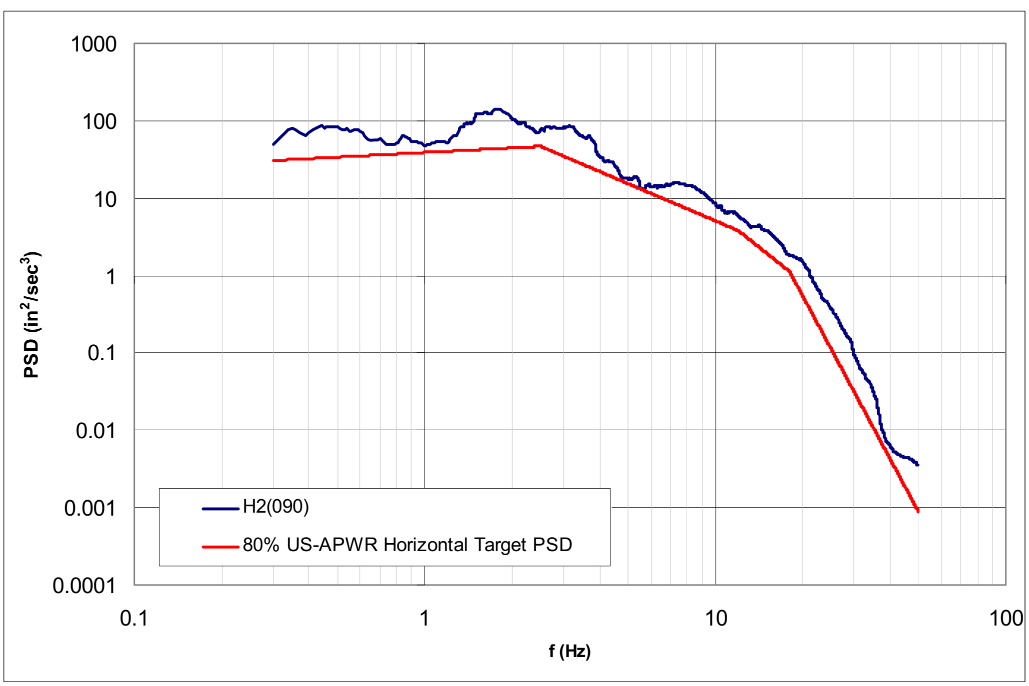

Figure 3.7.1-11 Smoothed Power Spectral Density Plots for Component H2 (090)

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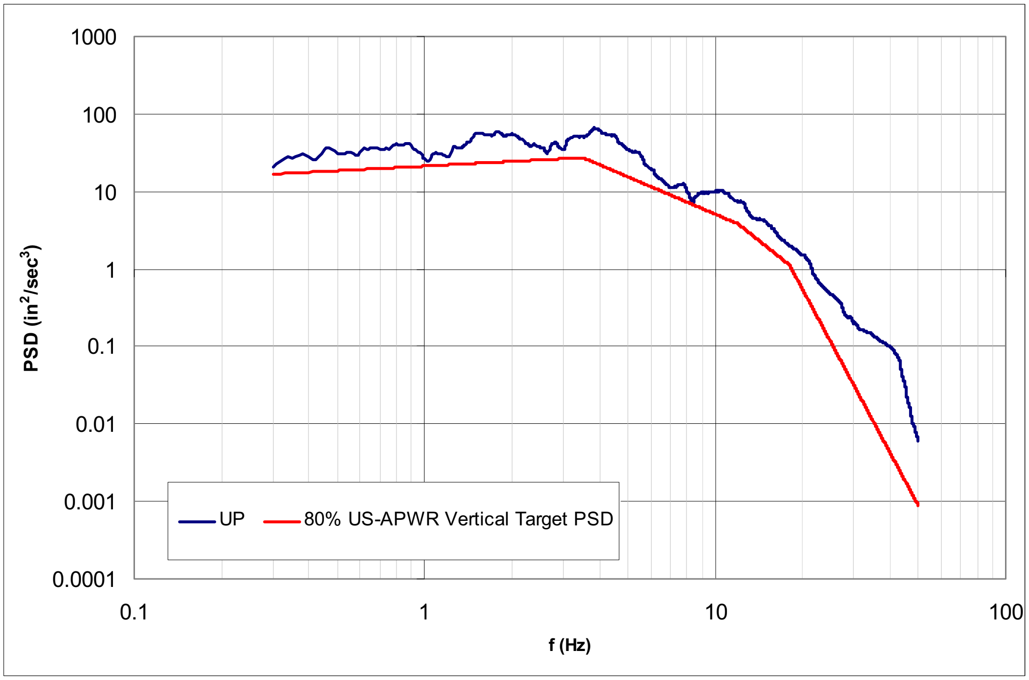

Figure 3.7.1-12 Smoothed Power Spectral Density Plots for Component V (UP)

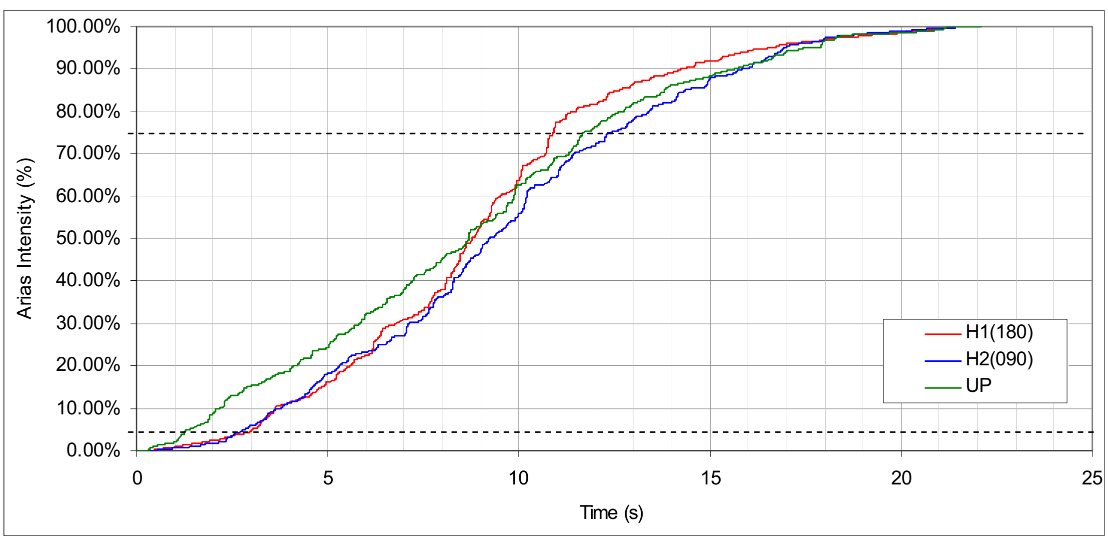

Figure 3.7.1-13 Arias Intensities of the Northridge – Mount Baldy Artificial Time History Components Showing 5%-75% Duration

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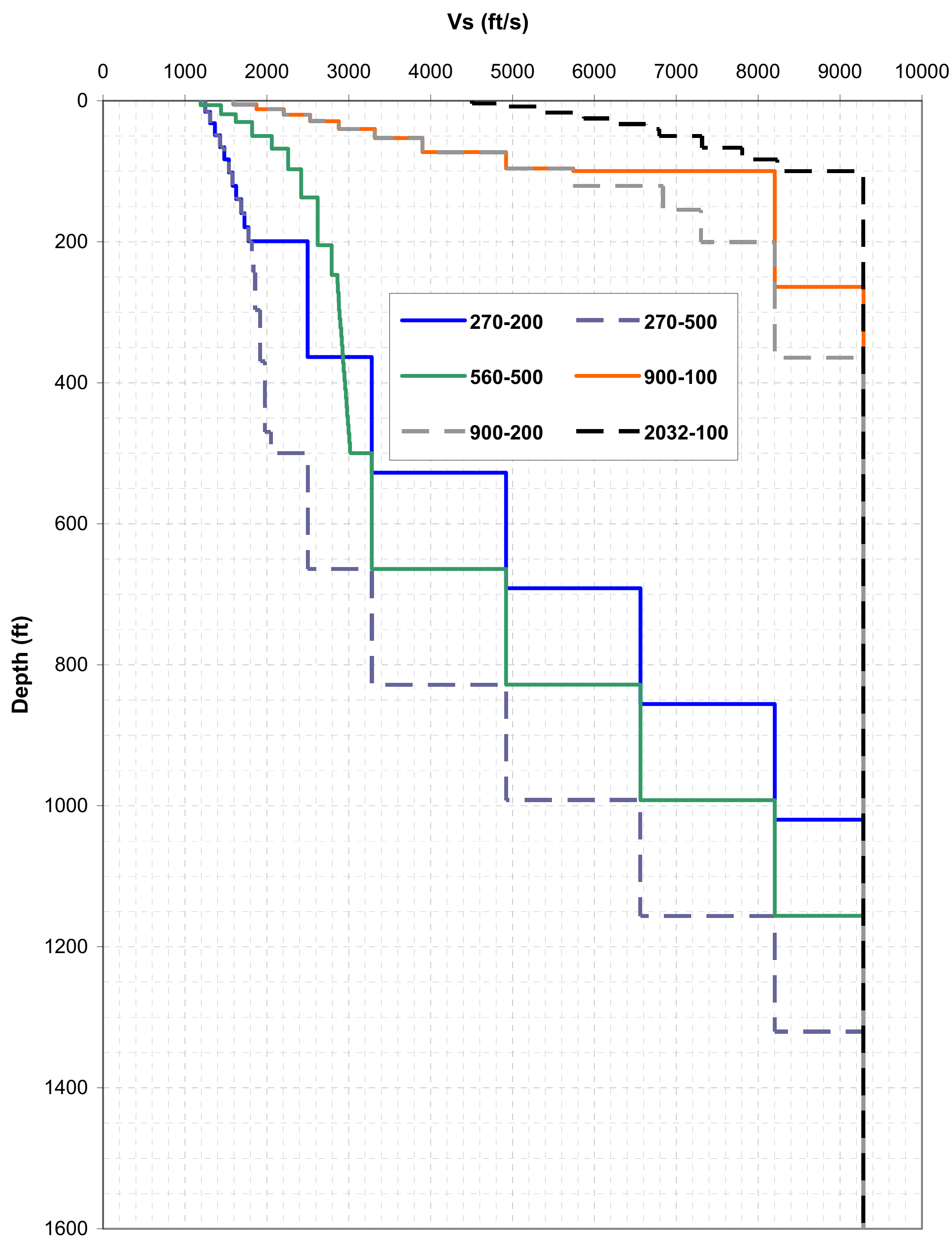

Figure 3.7.1-14 Six Generic Soil Profiles, Shear Wave Velocity $V_{(s)}$

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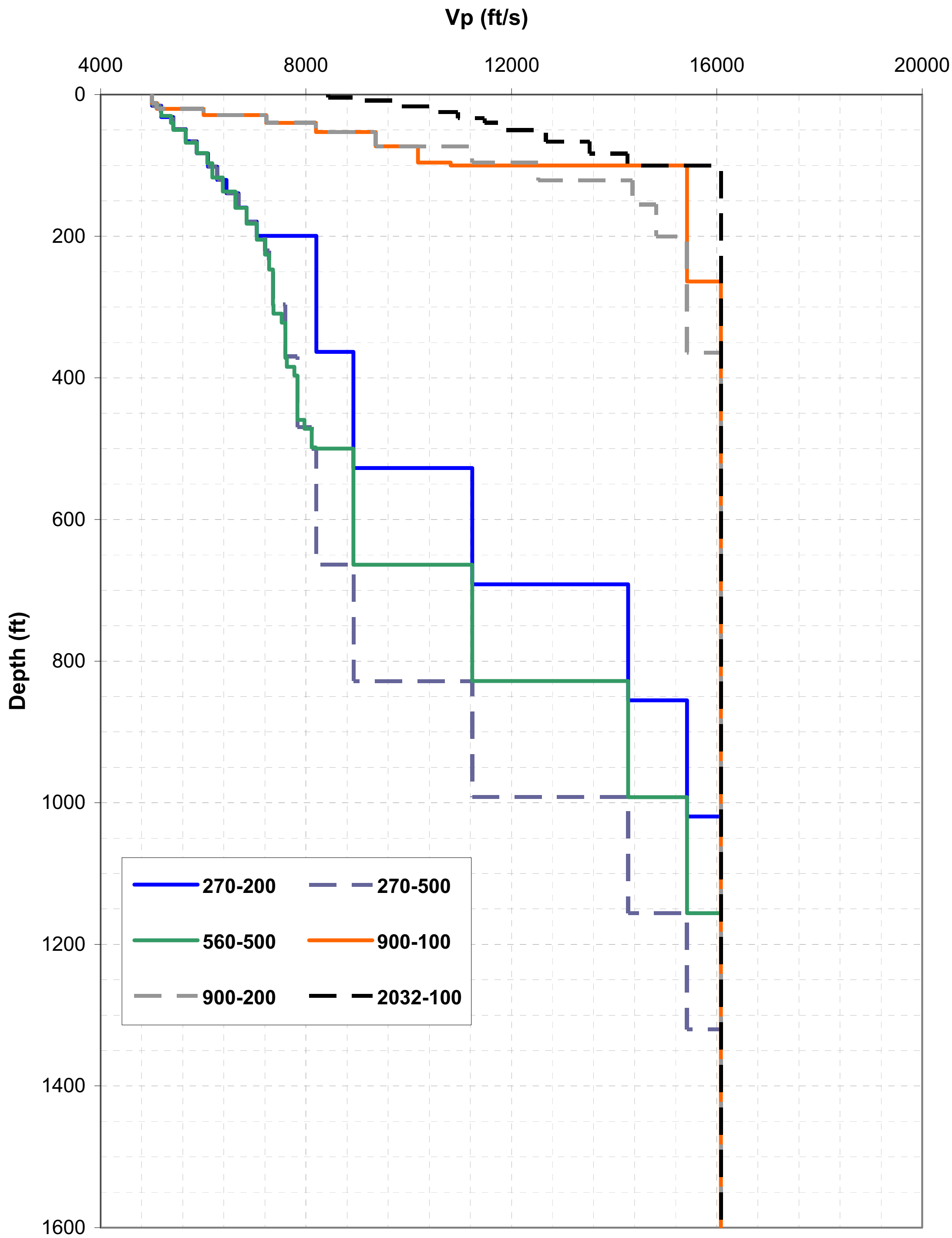

Figure 3.7.1-15 Six Generic Soil Profiles, Compression Wave Velocity (V_p)

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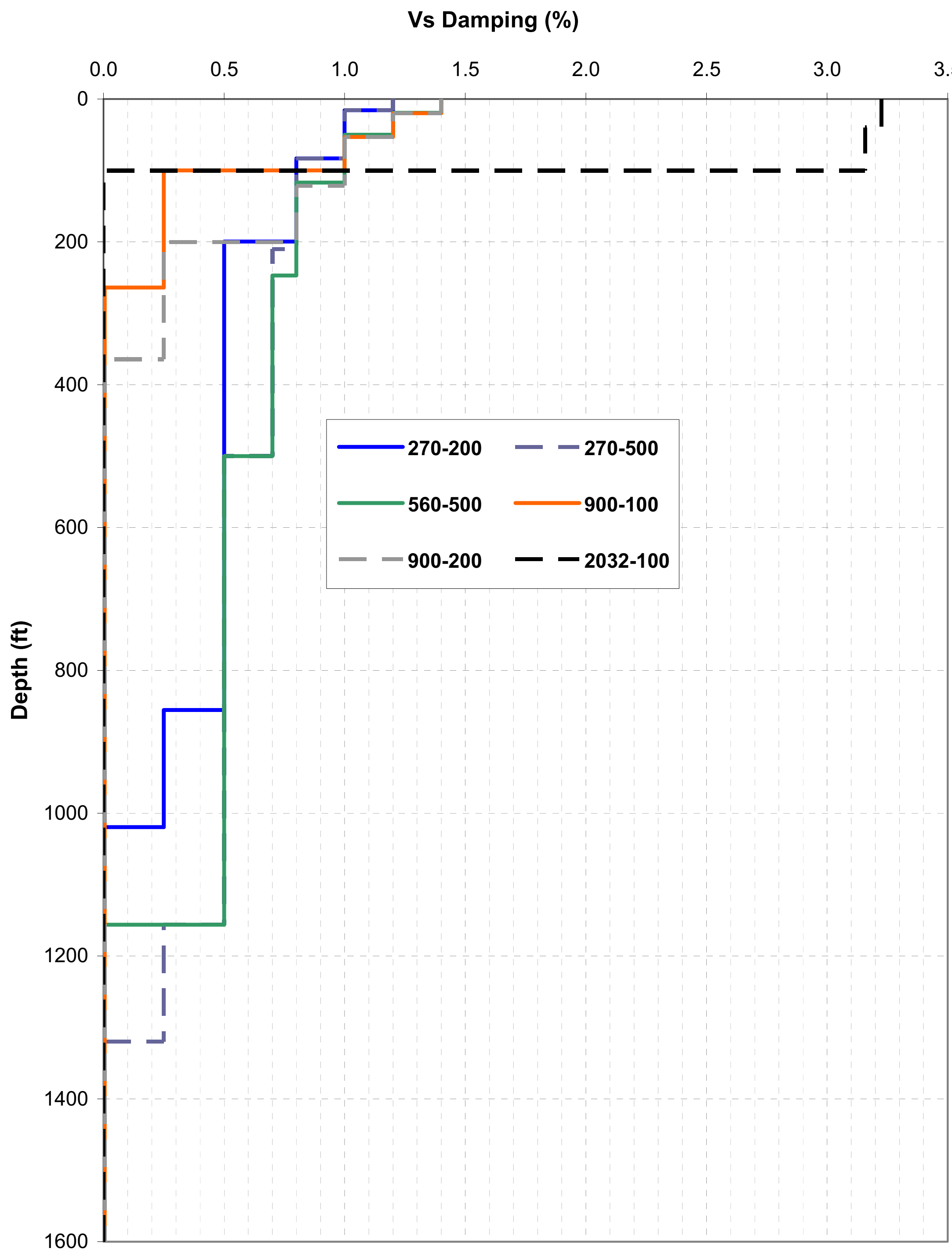

Figure 3.7.1-16 Six Generic Soil Profiles, Shear Wave Velocity Damping (%)

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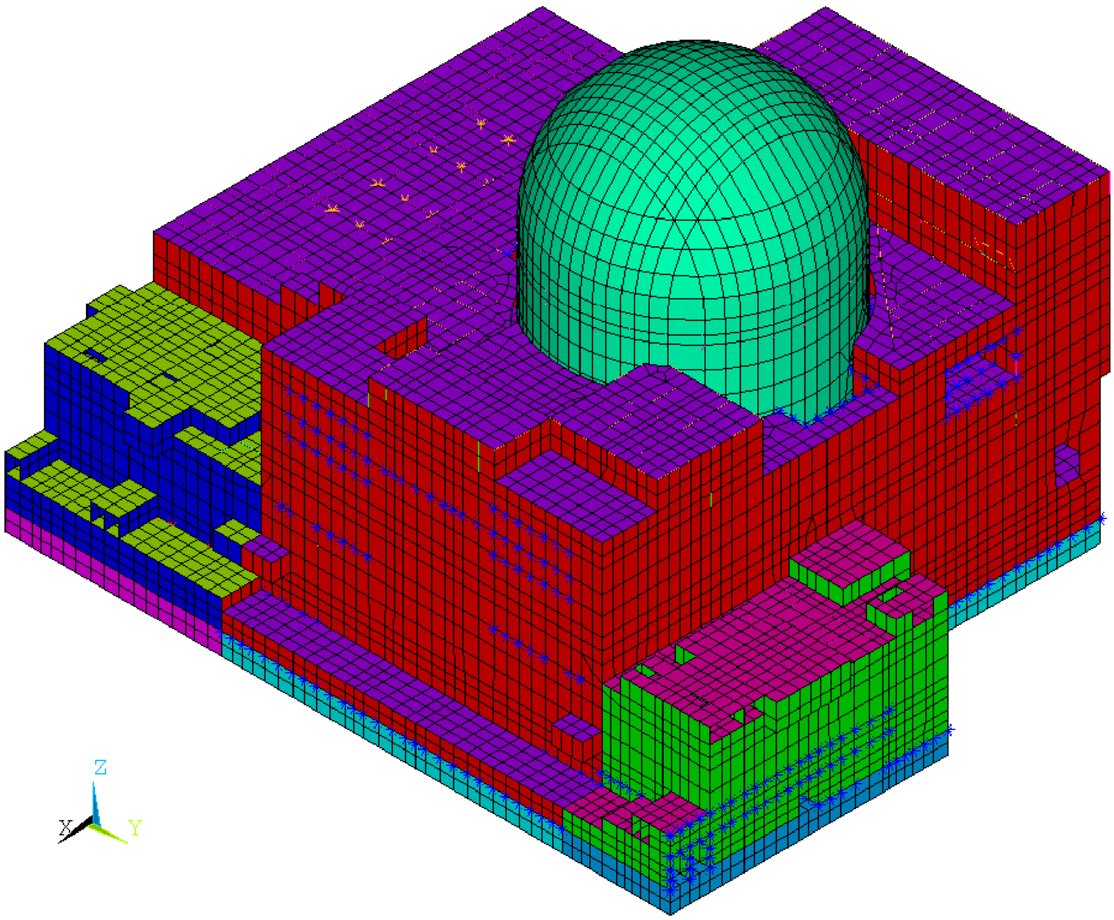

Figure 3.7.2-1 ~~FE Model of PCCVOverview of the PCCV Detailed~~Integrated R/B Complex Dynamic FE Model

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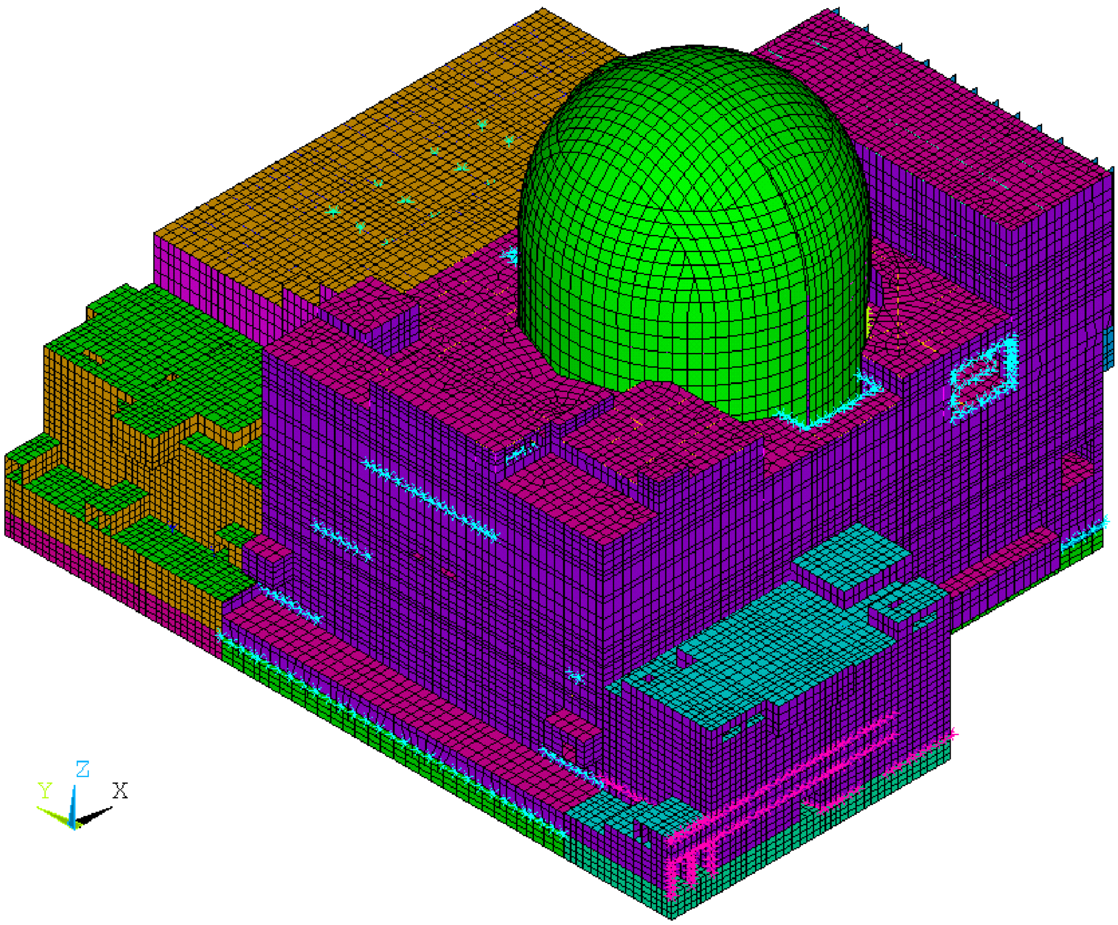

Figure 3.7.2-2 ~~Overview of the Containment Internal Structure Detailed FE Model (includes the Containment Internal Structure and the Reactor Coolant Loop Models)~~Integrated R/B Complex Detailed FE Model

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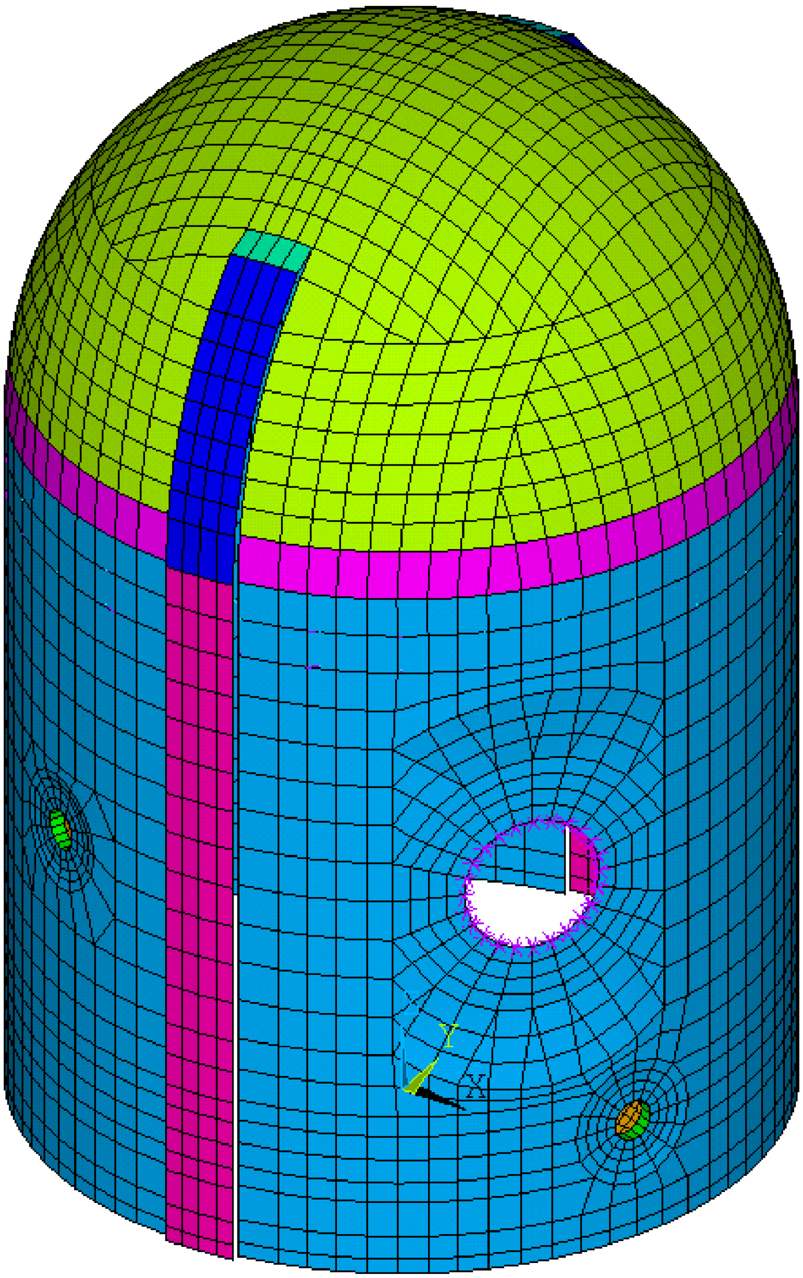

Figure 3.7.2-3 ~~FE Model of R/B on Common BasematOverview of the R/B Detailed FE~~PCCV Detailed Model

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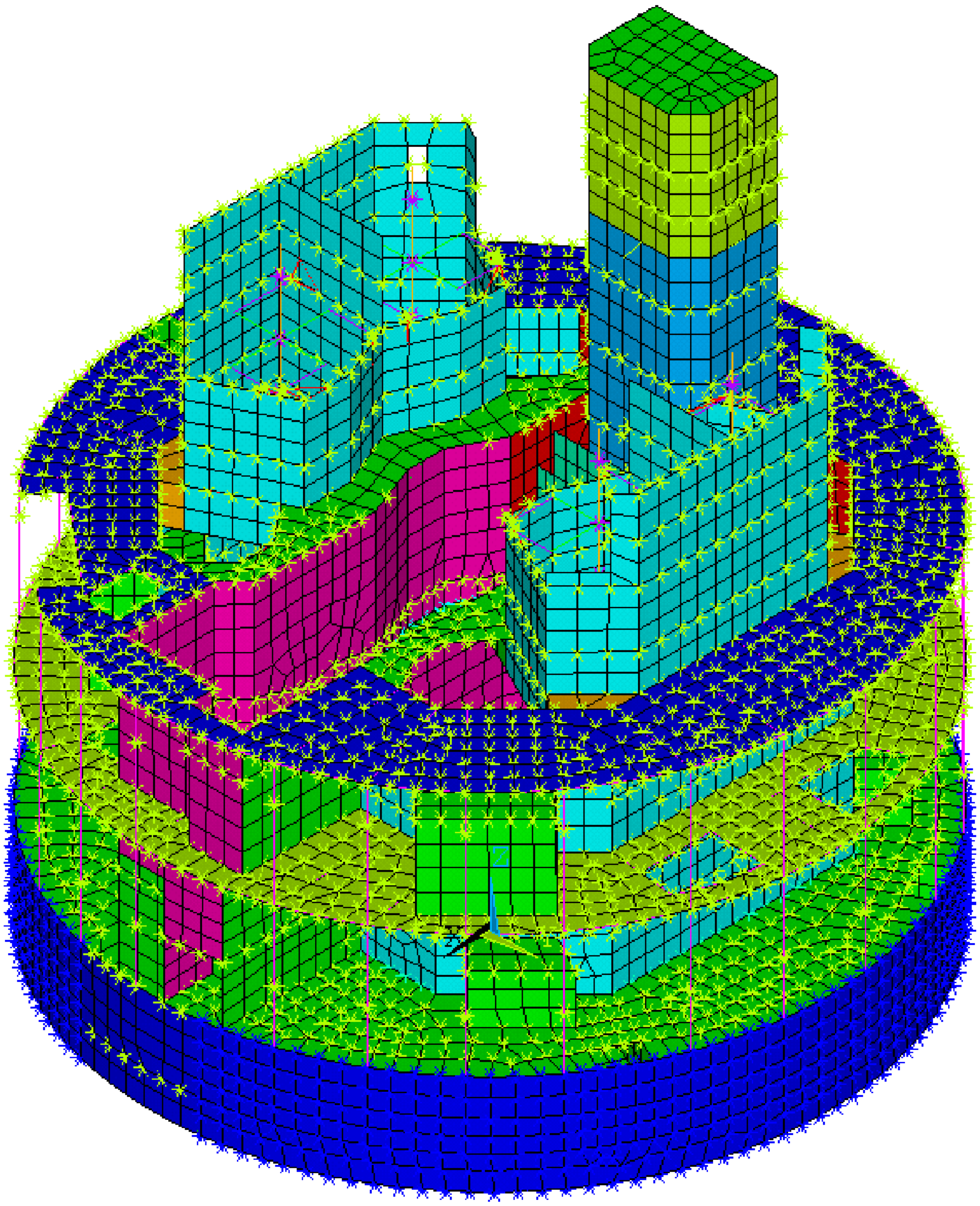

Figure 3.7.2-4 ~~FE Model of R/B on Common Basemat~~~~Integrated R/B Complex Dynamic FE Model~~CIS Detailed Model (includes the CIS and the RCL models)

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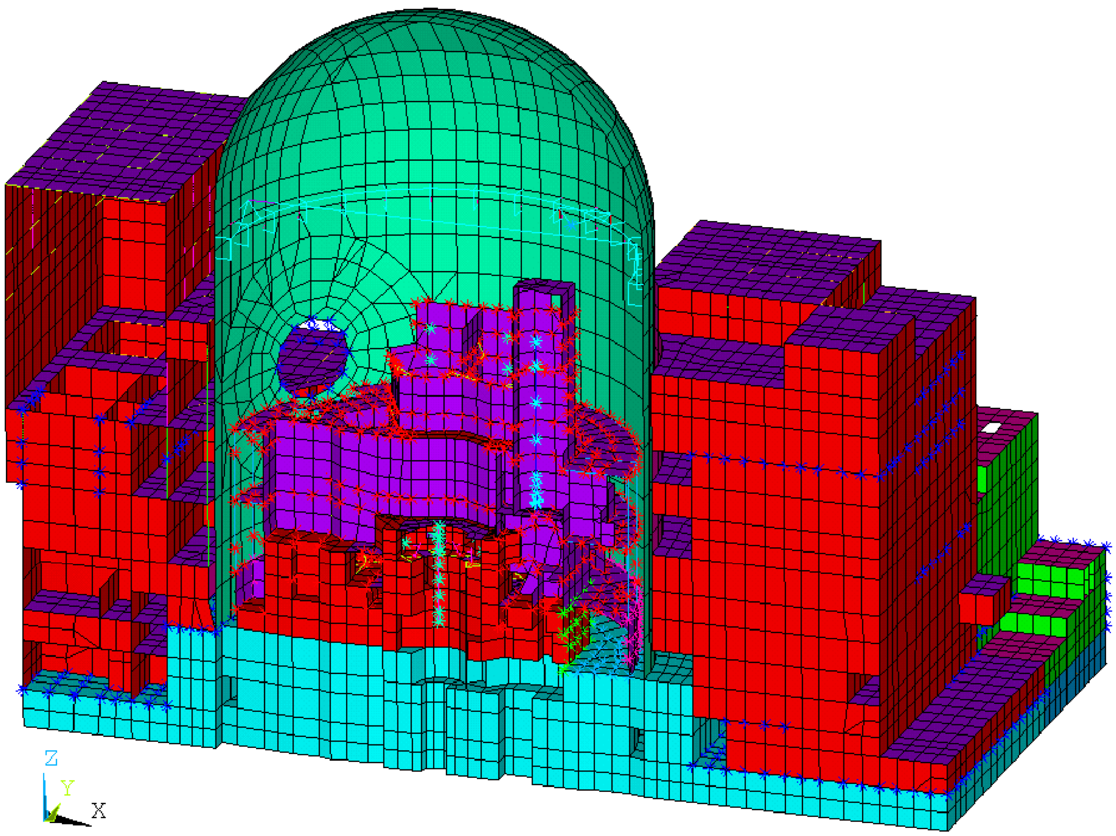

Figure 3.7.2-5 ~~Method to Compute Weight and Inertia Moment of Lumped Masses~~~~Dynamic R/B Model~~Section View of Dynamic R/B Complex FE Model Looking East

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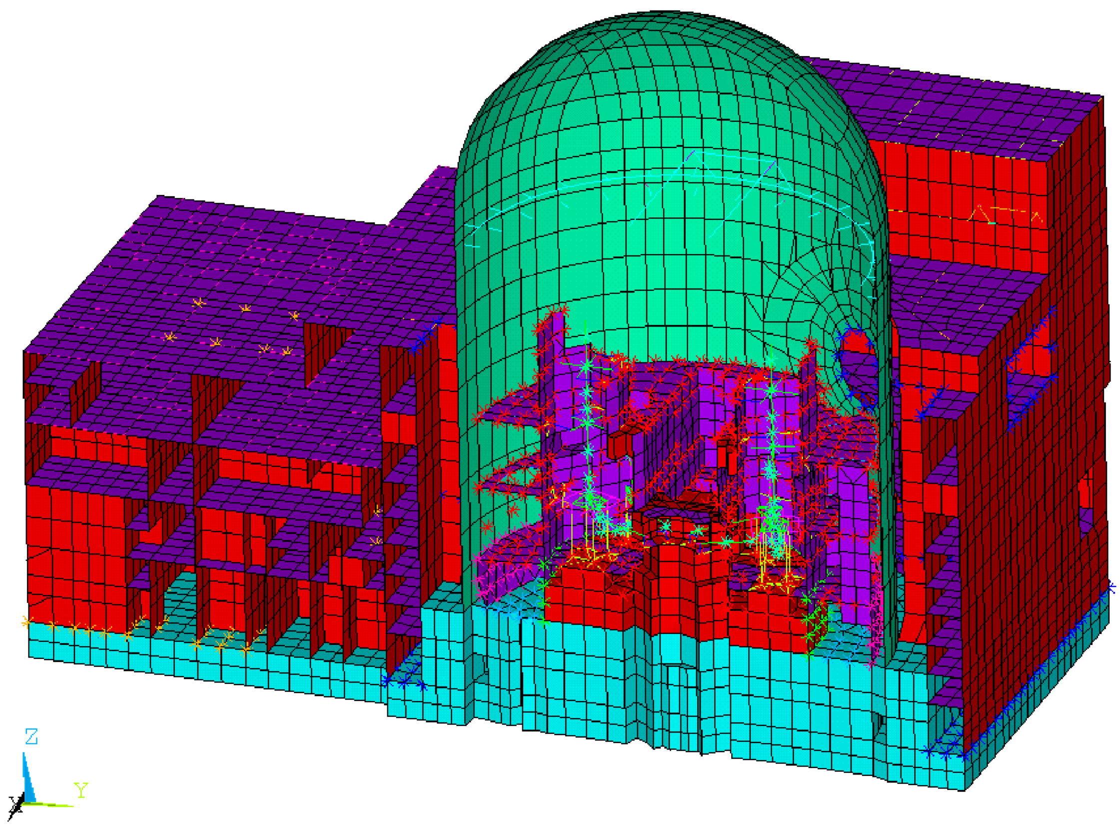

Figure 3.7.2-6 ~~Method to Compute Section Area, Inertia Moment and Axial Area by Hand Calculations~~~~Section View of Dynamic R/B Model Looking East~~Section View of Dynamic R/B Complex FE Model Looking North

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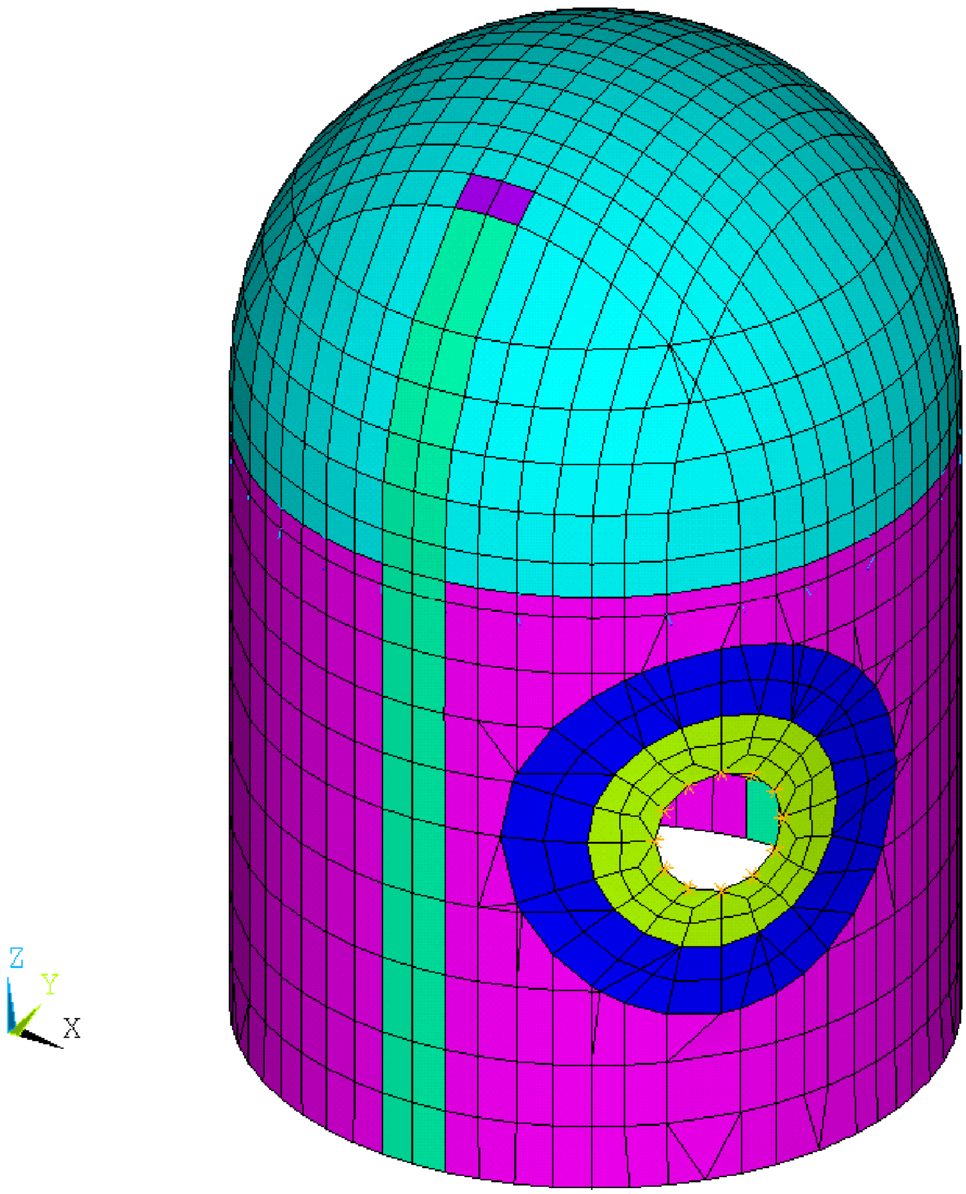

Figure 3.7.2-7 ~~Method to Calculate Equivalent Shear AreaSection View of Dynamic R/B Model Looking South~~PCCV Dynamic Model

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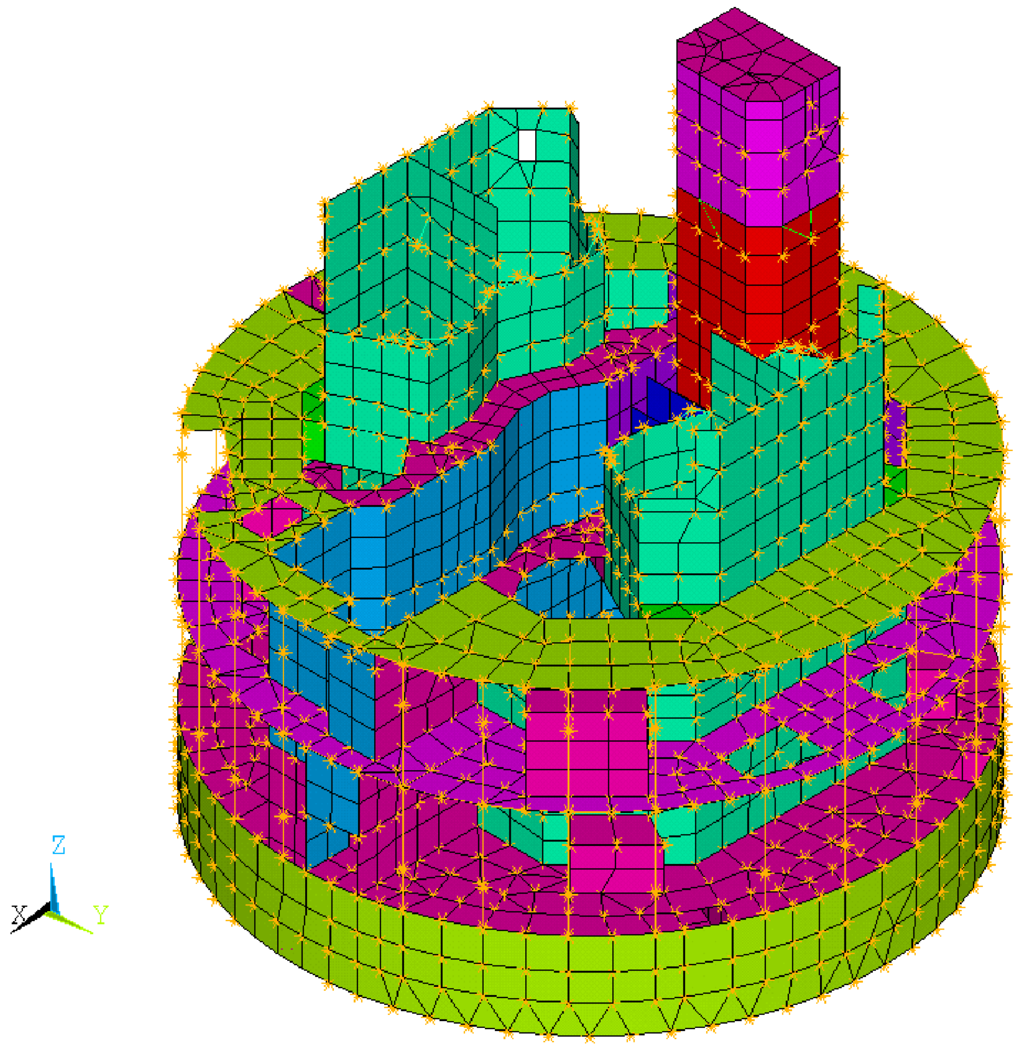

Figure 3.7.2-8 ~~Method to Evaluate Vertical Stiffness of PCCV DomeDynamic PCCV Model~~CIS Dynamic Model – Shell Elements (Excluding RCL)

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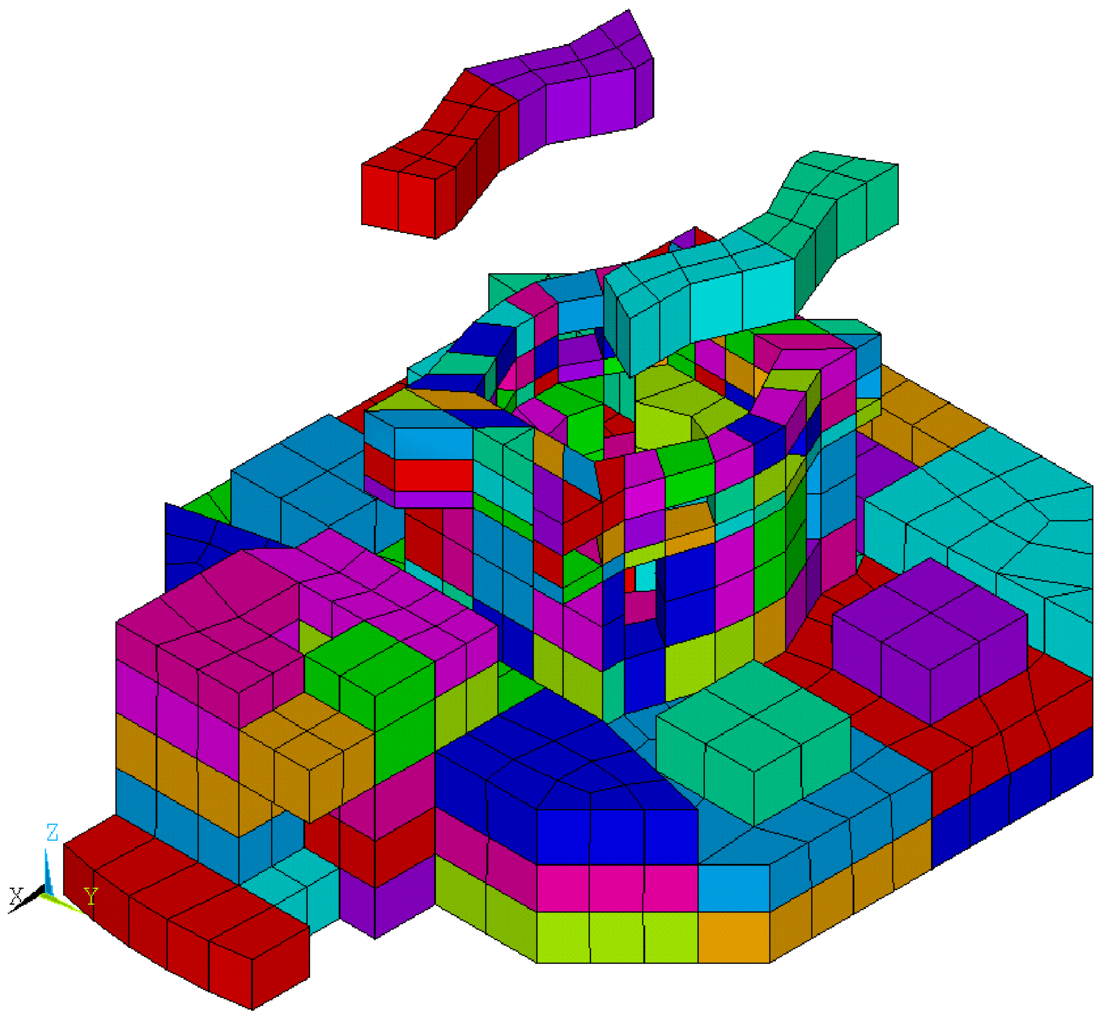

Figure 3.7.2-9 ~~Fixed-Base FE Model of R/B~~CIS Dynamic ~~CIS~~ Model - Solid Elements

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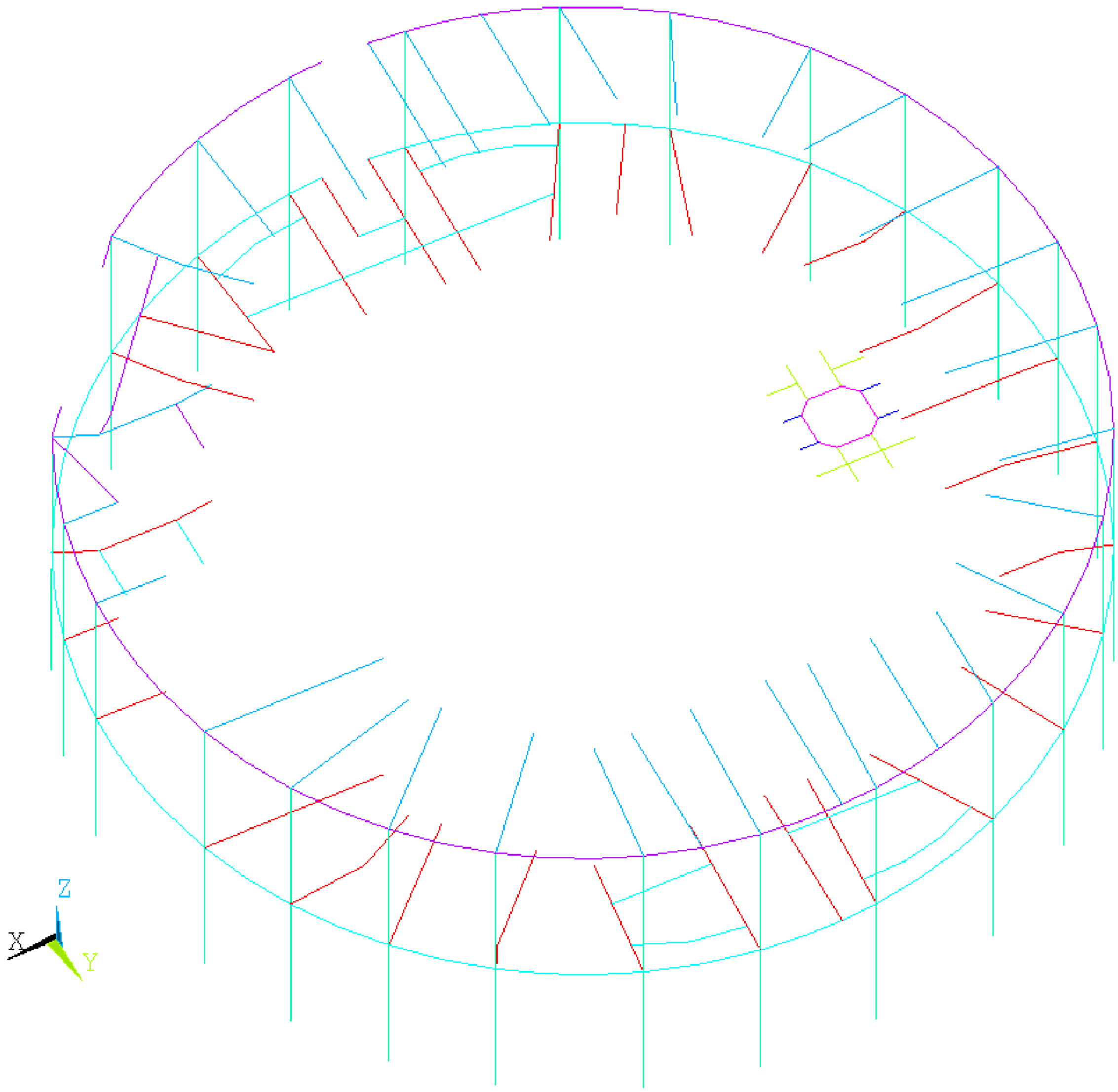

Figure 3.7.2-10 ~~Fixed-Base FE Model of Containment Internal StructureDynamic CIS Model – Shell and RCL Elements~~CIS Dynamic Model – Beam Elements (Excluding RCL)

MIC-03-03-00054

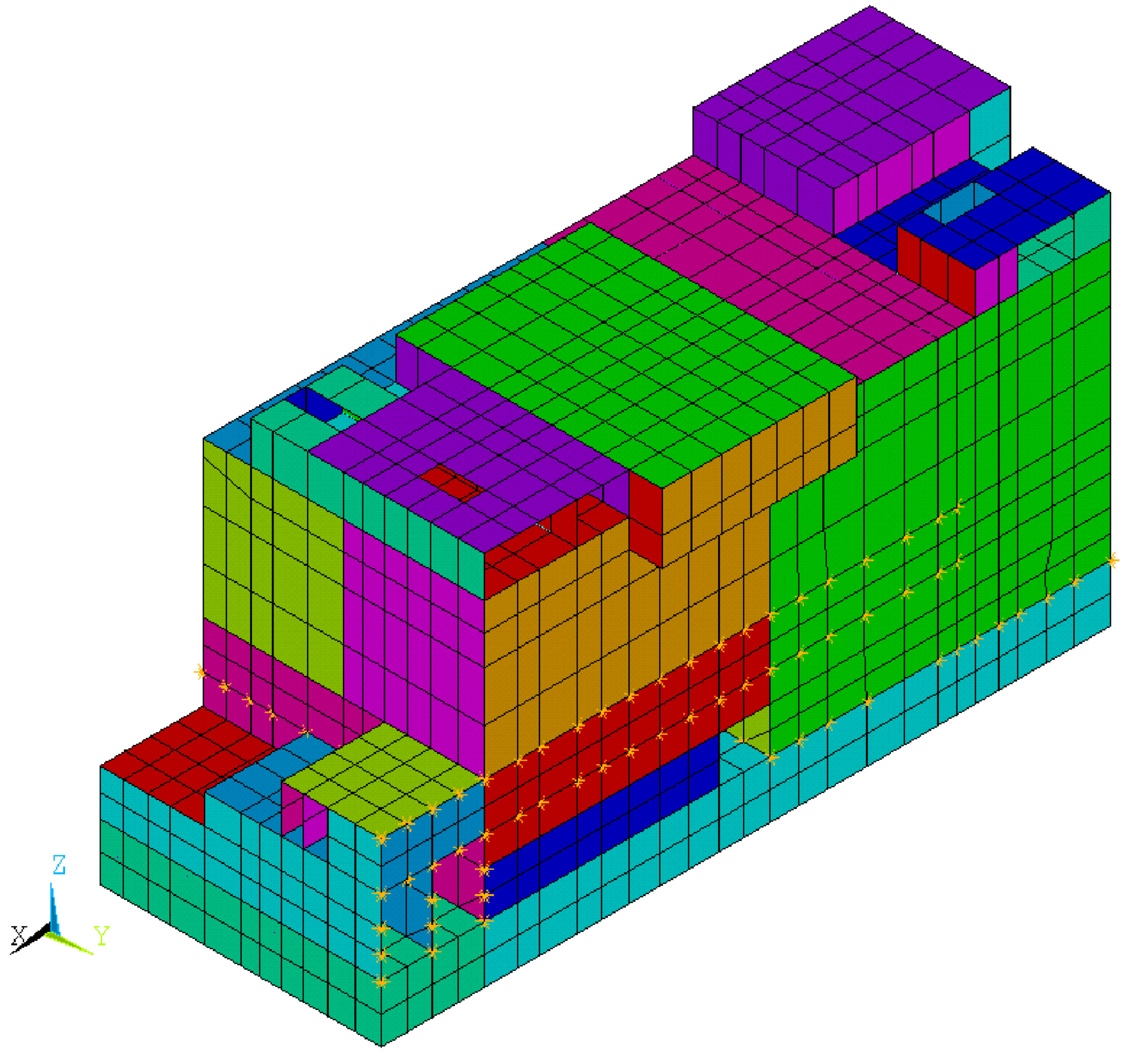

Figure 3.7.2-11 ~~Deleted~~East PS/B Dynamic Model with ESWPC

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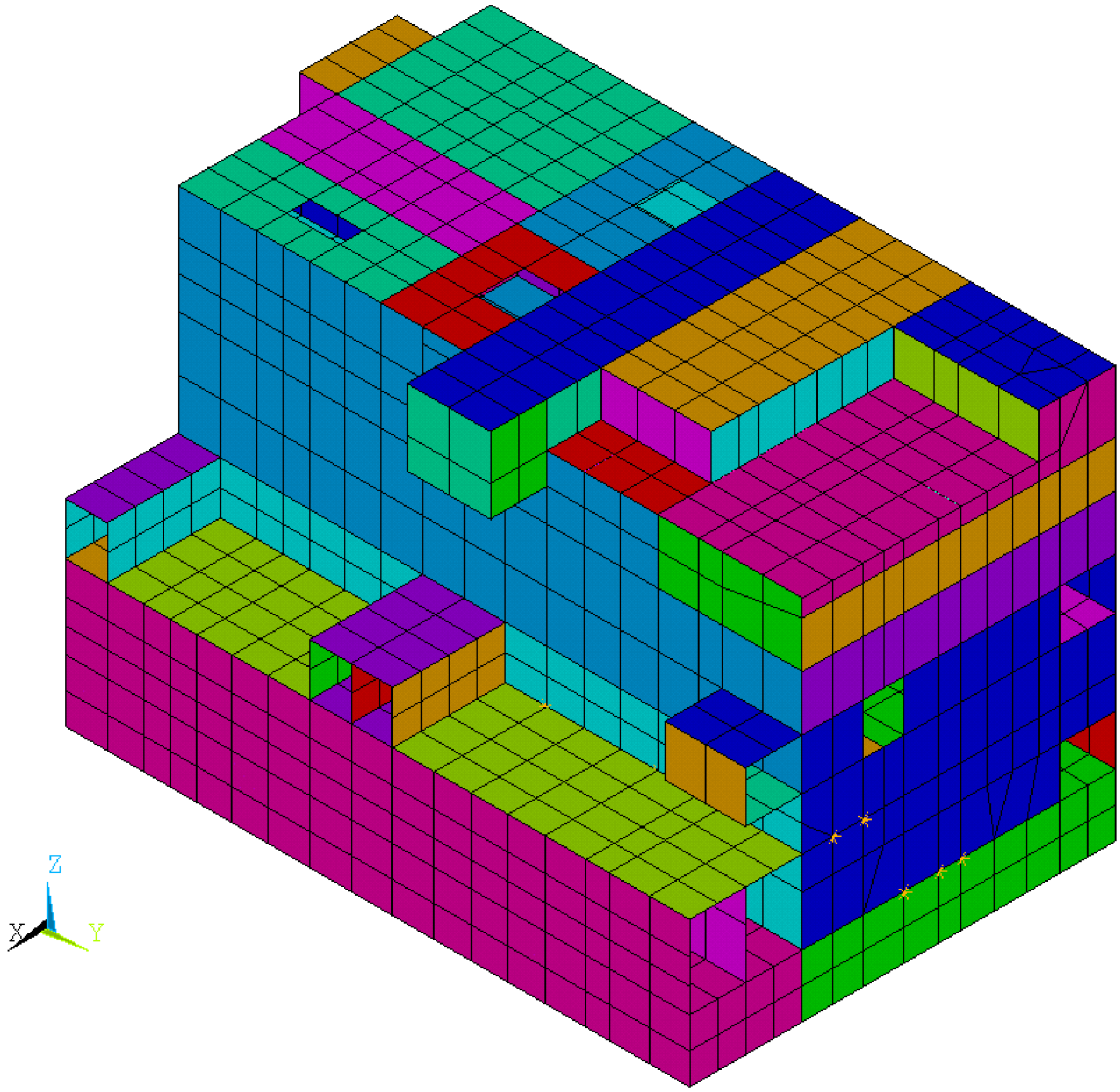

Figure 3.7.2-12 ~~Deleted~~West PS/B Dynamic Model with ESWPC

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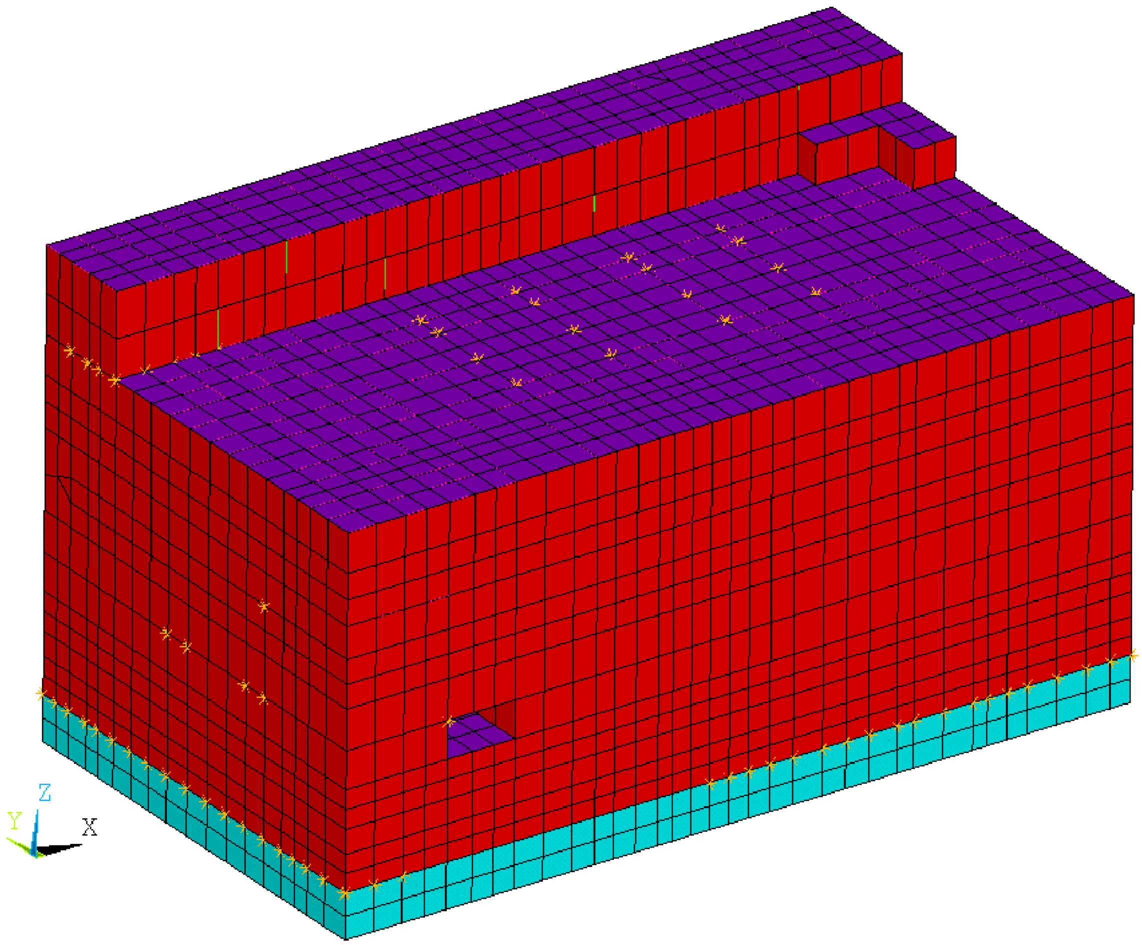

Figure 3.7.2-13 A/B Dynamic Model

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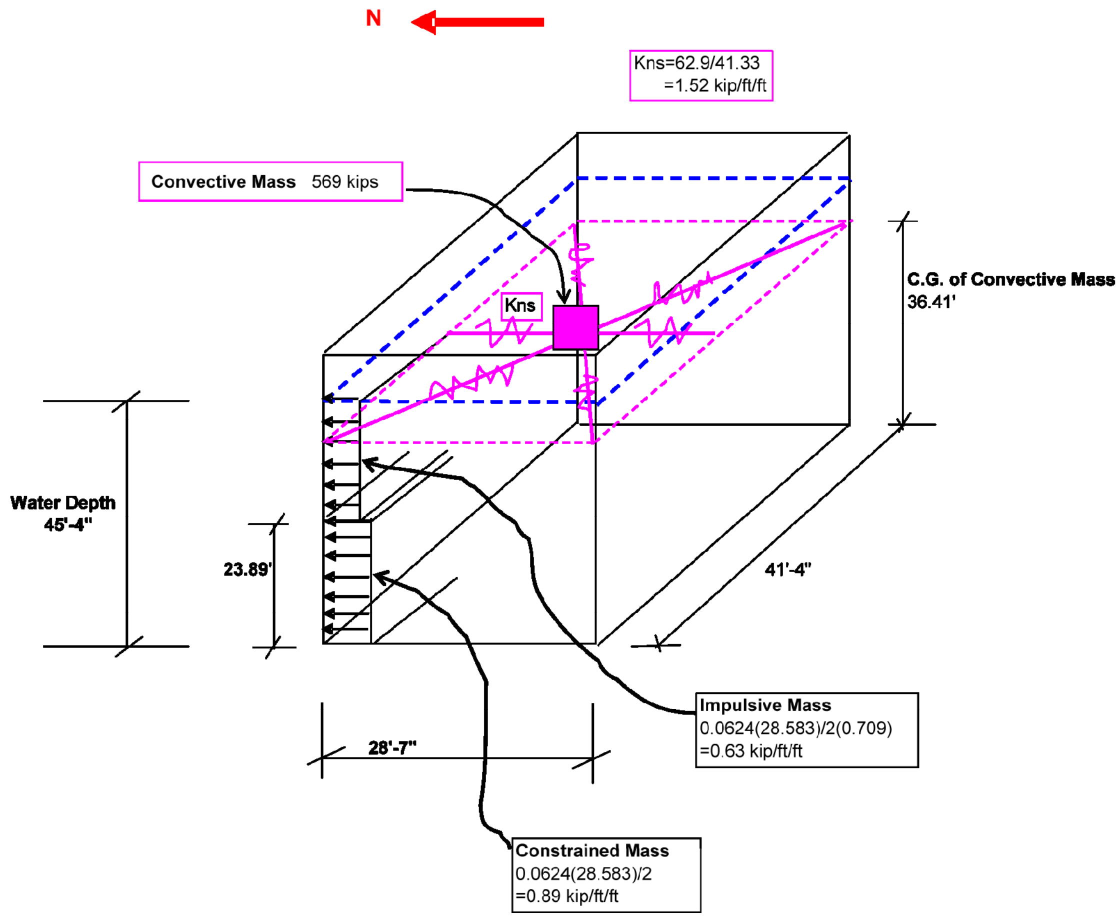

Figure 3.7.2-14 Example of Hydrodynamic Masses (Example shown for SFP)

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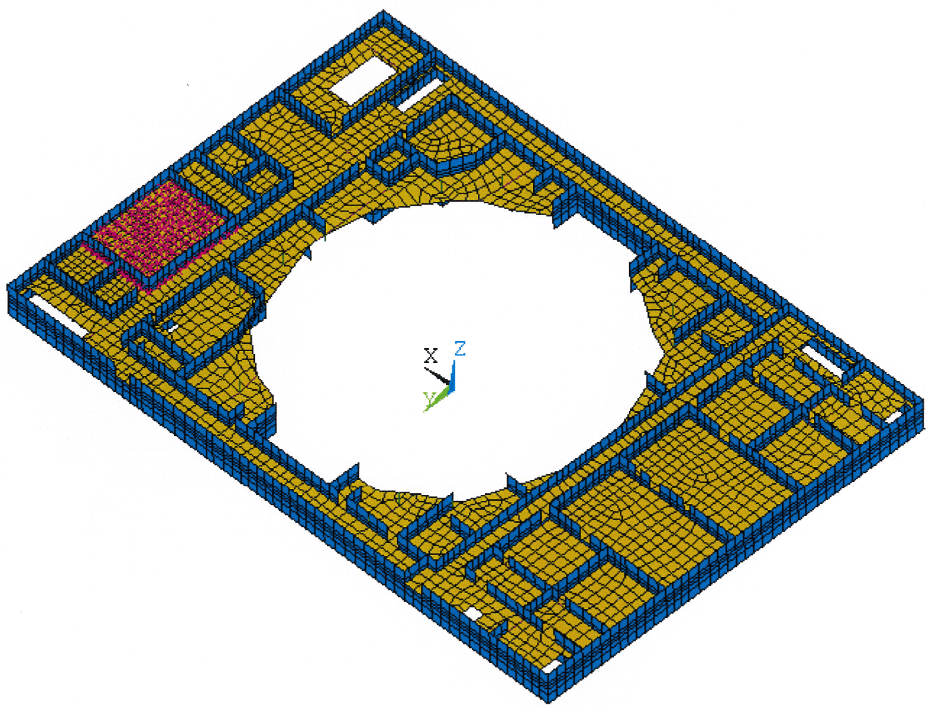

Figure 3.7.2-15 Extracted Detailed FE Model of Floor Slabs

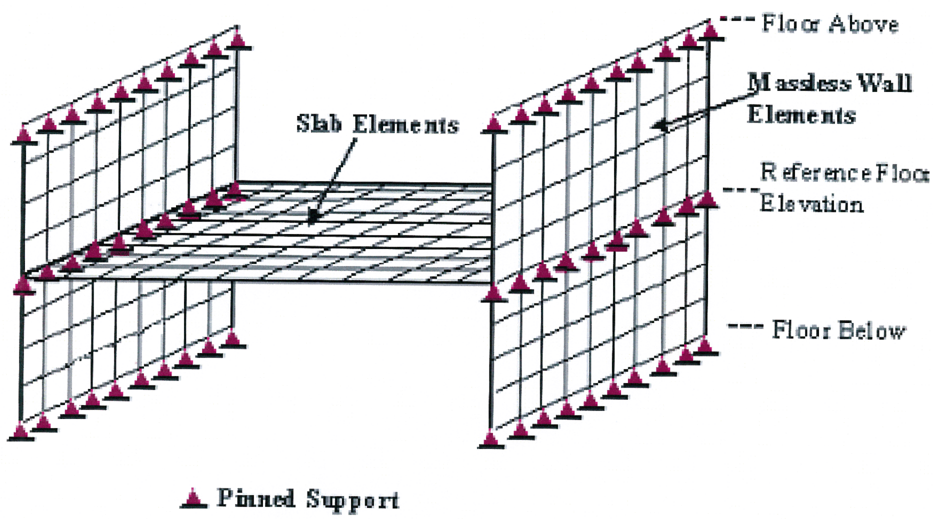

Figure 3.7.2-16 Floor Slab Model Boundary Conditions

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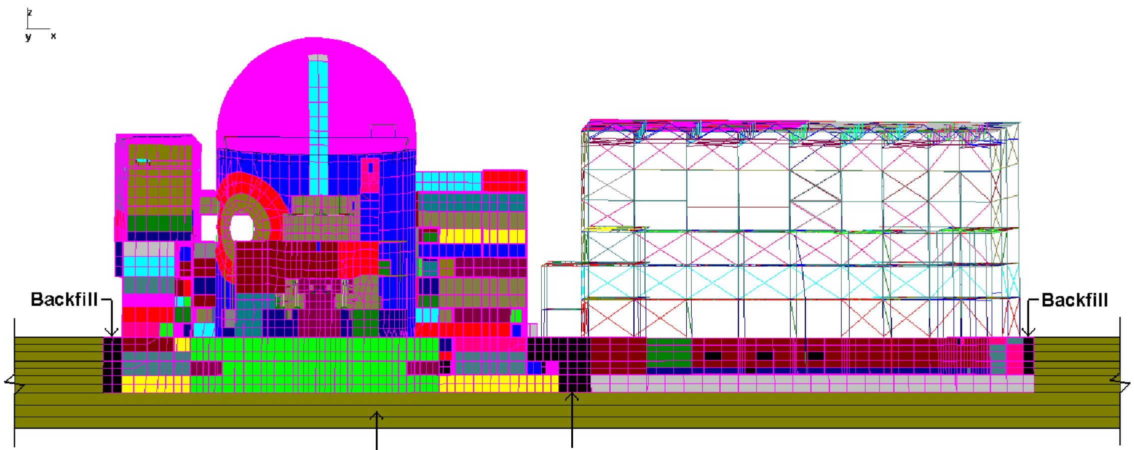

Figure 3.7.2-17 SSSI Model with Backfill and Free Field Soils (Looking East)

specific material selection for construction of the PCCV meet the requirements specified in ASME Code, Section III (Reference 3.8-2), Article CC-2000, and supplementary requirements of RG 1.136 (Reference 3.8-3) as well as SRP 3.8.1 (Reference 3.8-7).

Quality control programs are in accordance with applicable portions of Articles CC-4000 and CC-5000 of the ASME Code, Section III (Reference 3.8-2). Additional quality assurance requirements are also implemented as provided by RG 1.136 (Reference 3.8-3). Chapter 17 provides additional discussion of the QAP.

The information listed below is specifically for the PCCV and does not preclude the selection of site-specific material provided that they are rectified with the standard design and meet the ASME Code, Section III (Reference 3.8-2), SRP 3.8.1 (Reference 3.8-7), and RG 1.136 (Reference 3.8-3) requirements.

Concrete

The concrete constituents and concrete mix design comply with the requirements of Article CC-2200 of the ASME Code, Section III (Reference 3.8-2).

Cement used in the concrete conforms to the requirements of ASTM C 150, Specification for Portland Cement, Type I, Type II, Type IV, Type V, or ASTM C 595, Specification for Blended Hydraulic Cements, Type IP, Type IP (MS), or Type (MH).

Aggregates used in the concrete conform to the requirements of ASTM C 33, Specification for Concrete Aggregates (Reference 3.8-44).

Mixing water used in the concrete conforms to the requirements of Subarticle CC-2223 of the ASME Code, Section III (Reference 3.8-2).

Admixtures include air-entraining admixtures, chemical admixtures, and mineral admixtures. The admixtures, except mineral admixtures, are stored in a liquid state. Air-entraining admixtures conform to the requirements of ASTM C 260, Air-Entraining Admixtures for Concrete (Reference 3.8-48).

Mineral admixtures conform to the requirements of ASTM C 618, Fly Ash and Raw or Calcined Natural Pozzolans for Use in Portland Cement Concrete (Reference 3.8-49). Chemical admixtures conform to the requirements of ASTM C 494, Chemical Admixtures for Concrete (Reference 3.8-50).

Compressive Strength

The concrete design compressive strength for the PCCV is f'_c = 7,000 psi

The concrete design compressive strength for the basemat is f'_c = 5~~4~~,000 psi MIC-03-06-00073

As previously discussed in Subsection 3.8.1.5, concrete is not allowed to rely on tensile strength to resist flexural and membrane tension except where permitted in ASME Code, Section III (Reference 3.8-2) allowable shear provisions. The concrete creep for the 60 year design life is 400μ in/in; for purposes of design, it is considered that 2/3 of this occurs in the first year after completion of prestressing. The concrete shrinkage for the 60

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 which has a compressive strength of 5000 psi is utilized in standard plant seismic category I structures other than PCCV, upper part of the tendon gallery in the basemat and the containment internal structure (CIS). 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 349-06 (Reference 3.8-8), ACI 304R (Reference 3.8-~~38~~39), 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). MIC-03-06-00073 MIC-03-03-00066

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~~ACI 349-06 (Reference 3.8-8). MIC-03-03-00066

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

Table 3.8.1-1 US-APWR PCCV Basic Design Specification

	US-APWR	Remarks
Design Condition		
Design Pressure (P_d)	68 psig	
Test Pressure (P_t)	78.2 psig	
Design External Pressure (P)	-3.9 psig	
Design Accident Temperature	300°F	
Dimension		
Inner Diameter	149 ft - 2 in.	
Inner Height	226 ft - 5 in.	
Wall Thickness (Cylinder)	4 ft - 4 in.	
Wall Thickness (Dome)	3 ft - 8 in.	
Liner Thickness	0.25 in.	
Large Opening		
Equipment Hatch	ID 27 ft - 11 in.	One Set
Personnel Air Lock	ID 8 ft - 6 3/8 in.	Two Sets
Free Volume	2.80 x 10^6 ft^3	
Design Leakage Rate	0.1% mass/24 hours	
Design Life	60 years	
Material		
Concrete Design Strength	7000 psi	PCCV
	5~~4~~000 psi	Basemat ~~& Modules~~
	4000 psi	Modules
Reinforcement	ASTM A615 Gr. 60 or ASTM A706 Gr. 60	
Liner Plate	ASTM A516 Gr. 60	
Tendon Specification		
PS System	VSL (or BBR)	
Tendon Capacity	13 MN Class	
Strands	ASTM A416 Grade 1860 #15 (Lower Relaxation)	
Number of Strands per Tendon	49	
Number of Cylinder Hoop Tendons	94	1 ft – 6 in. Pitch
Number of Cyl. Dome Tendons	18	2.5° Radial Pitch
Number of Inverted U-shape Tendons	90	2° Radial Pitch

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Table 3.8.5-2 Concrete Properties

Part		Compressive Strength f'c	Modulus of Elasticity Ec(1)	Poisson's Ratio υ	Thermal Expansion Coefficient α	Unit Weight γ
PCCV		7,000 psi	4,769 ksi	0.17	0.99×10^{-5}/°C	150lb/ft^3
Containment Internal Structure		4,000 psi	3,605 ksi	0.17	0.99×10^{-5}/°C	150lb/ft^3
R/B		54,000 psi	3,605 ksi	0.17	0.99×10^{-5}/°C	150lb/ft^3
Basemat	Peripheral	54,000 psi	3,605 ksi	0.17	0.99×10^{-5}/°C	150lb/ft^3
	Upper part of Tendon Gallery	7,000 psi	4,769 ksi	0.17	0.99×10^{-5}/°C 5.5×10^{-6}/°F	150lb/ft^3

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NOTE :

1. 1. Ec=57,000(Fc)$^{1/2}$ psi (ACI 349-06, 8.5.1)

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Table 3D-2 US-APWR Environmental Qualification Equipment List (Sheet 25 of 67)

Item Num	EquipmentTag	Description	Location		Purpose	Operational Duration	Environmental Conditions	Radiation Condition	Influence of Submergence for Total Integrated Dose	Qualification Process	Seismic Category	Comments
			Building	Zone	RT, ESF, PAM, Pressure Boundary (PB), Other(1)		Harsh or Mild	Harsh or Mild	Yes/No	E=Electrical M=Mechanical	I, II, Non	
37	MC-D	D-Class 1E 6.9kV Switchgear	R/B	4	ESF	2wks	Mild	Mild	No (1)	E	I	
38	LC-D	D-Class 1E 480V Load Center	R/B	4	ESF	2wks	Mild	Mild	No (1)	E	I	
39	MCC-D	D-Class 1E Motor Control Center	R/B	4	ESF	2wks	Mild	Mild	No (1)	E	I	
40	MCC-D1	D1-Class 1E Motor Control Center	R/B	4	ESF	2wks	Mild	Mild	No (1)	E	I	
41	RIO-D	D-Safety Remote I/O Cabinet	R/B	4	ESF, PAM	2wks	Mild	Mild	No (1)	E	I	
42	PBH-D	D-Pressurizer Heater Distribution Panel	R/B	4	ESF	2wks	Mild	Mild	No (1)	E	I	
43	RPTS-D	D-RCP Trip Switchgear	R/B	4	Other	2wks	Mild	Mild	No (1)	E	I	
44	RSC	Remote Shutdown Console	R/B	2	RT, ESF, PAM	2wks	Mild	Mild	No (1)	E	I	
45	MRTP-1	MCR/RSR Transfer Panel (1)	R/B	14	RT, ESF, PAM	2wks	Mild	Mild	No (1)	E	I	
46	MRTP-2	MCR/RSR Transfer Panel (2)	R/B	2	RT, ESF, PAM	2wks	Mild	Mild	No (1)	E	I	
47	RTBC-1	Reactor Trip Breaker Cabinet (1)	R/B	4	RT	5min	Mild	Mild	No (1)	E	I	
48	RTBC-2	Reactor Trip Breaker Cabinet (2)	R/B	4	RT	5min	Mild	Mild	No (1)	E	I	
49	BCP-A	A-Class 1E Battery Charger	PS/B	4	RT, ESF	2wks	Mild	Mild	No (1)	E	I	
50A	DCC-A	A-Class 1E DC Switchboard	PS/B	4	RT, ESF	2wks	Mild	Mild	No (1)	E	I	
50B	DCC-A1	A1-Class 1E DC Switchboard	PS/B	4	RT, ESF	2wks	Mild	Mild	No (1)	E	I	
51	DDP-A	A-Reactor Building DC Distribution Panel	R/B	4	RT, ESF	2wks	Mild	Mild	No (1)	E	I	
52	SDC-A	A-Solenoid Distribution Panel	R/B	4	ESF	2wks	Mild	Mild	No (1)	E	I	
53	IBC-A	A-Class 1E UPS Unit	R/B	4	RT, ESF, PAM	2wks	Mild	Mild	No (1)	E	I	
54	IBB-A	A-Class 1EI&C Power Transformer	R/B	4	RT, ESF, PAM	2wks	Mild	Mild	No (1)	E	I	
55	IBD-A	A-Class 1E AC120V Panelboard	R/B	4	RT, ESF, PAM	2wks	Mild	Mild	No (1)	E	I	
56	MVIA1	A-MOV Inverter1	~~R/B~~PS/B	4	ESF, PB	2wks	Mild	Mild	No (1)	E	I	
57	MVIA2	A-MOV Inverter2	~~R/B~~PS/B	4	ESF, PB	2wks	Mild	Mild	No (1)	E	I	
58	MVCA1	A-MOV Motor Control Center1	~~R/B~~PS/B	4	ESF, PB	2wks	Mild	Mild	No (1)	E	I	
59	MVCA2	A-MOV Motor Control Center2	~~R/B~~PS/B	4	ESF, PB	2wks	Mild	Mild	No (1)	E	I	
60	BCP-B	B-Class 1E Battery Charger	PS/B	4	RT, ESF	2wks	Mild	Mild	No (1)	E	I	

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Table 3D-2 US-APWR Environmental Qualification Equipment List (Sheet 26 of 67)

Item Num	EquipmentTag	Description	Location		Purpose	Operational Duration	Environmental Conditions	Radiation Condition	Influence of Submergence for Total Integrated Dose	Qualification Process	Seismic Category	Comments
			Building	Zone	RT, ESF, PAM, Pressure Boundary (PB), Other(1)		Harsh or Mild	Harsh or Mild	Yes/No	E=Electrical M=Mechanical	I, II, Non	
61	DCC-B	B-Class 1E DC Switchboard	PS/B	4	RT, ESF	2wks	Mild	Mild	No (1)	E	I	
62	DDP-B	B-Reactor Building DC Distribution Panel	R/B	4	RT, ESF	2wks	Mild	Mild	No (1)	E	I	
63	SDC-B	B-Solenoid Distribution Panel	R/B	4	ESF	2wks	Mild	Mild	No (1)	E	I	
64	IBC-B	B-Class 1E UPS Unit	R/B	4	RT, ESF, PAM	2wks	Mild	Mild	No (1)	E	I	
65	IBB-B	B-Class 1E I&C Power Transformer	R/B	4	RT, ESF, PAM	2wks	Mild	Mild	No (1)	E	I	
66	IBD-B	B-Class 1E AC120V Panelboard	R/B	4	RT, ESF, PAM	2wks	Mild	Mild	No (1)	E	I	
67	MVIB	B-MOV Inverter	~~R/B~~PS/B	4	ESF, PB	2wks	Mild	Mild	No (1)	E	I	
68	MVCB	B-MOV Motor Control Center	~~R/B~~PS/B	4	ESF, PB	2wks	Mild	Mild	No (1)	E	I	
69	BCP-C	C-Class 1E Battery Charger	PS/B	4	RT, ESF	2wks	Mild	Mild	No (1)	E	I	
70	DCC-C	C-Class 1E DC Switchboard	PS/B	4	RT, ESF	2wks	Mild	Mild	No (1)	E	I	
71	DDP-C	C-Reactor Building DC Distribution Panel	R/B	4	RT, ESF	2wks	Mild	Mild	No (1)	E	I	
72	SDC-C	C-Solenoid Distribution Panel	R/B	4	ESF	2wks	Mild	Mild	No (1)	E	I	
73	IBC-C	C-Class 1E UPS Unit	R/B	4	RT, ESF, PAM	2wks	Mild	Mild	No (1)	E	I	
74	IBB-C	C-I&C Power Transformer	R/B	4	RT, ESF, PAM	2wks	Mild	Mild	No (1)	E	I	
75	IBD-C	C-Class 1E AC120V Panelboard	R/B	4	RT, ESF, PAM	2wks	Mild	Mild	No (1)	E	I	
76	MVIC	C-MOV Inverter	~~R/B~~PS/B	4	ESF, PB	2wks	Mild	Mild	No (1)	E	I	
77	MVCC	C-MOV Motor Control Center	~~R/B~~PS/B	4	ESF, PB	2wks	Mild	Mild	No (1)	E	I	
78	BCP-D	D-Class 1E Battery Charger	PS/B	4	RT, ESF	2wks	Mild	Mild	No (1)	E	I	
79A	DCC-D	D-Class 1E DC Switchboard	PS/B	4	RT, ESF	2wks	Mild	Mild	No (1)	E	I	
79B	DCC-D1	D1-Class 1E DC Switchboard	PS/B	4	RT, ESF	2wks	Mild	Mild	No (1)	E	I	
80	DDP-D	D-Reactor Building DC Distribution Panel	R/B	4	RT, ESF	2wks	Mild	Mild	No (1)	E	I	
81	SDC-D	D-Solenoid Distribution Panel	R/B	4	ESF	2wks	Mild	Mild	No (1)	E	I	
82	IBC-D	D-Class 1E UPS Unit	R/B	4	RT, ESF, PAM	2wks	Mild	Mild	No (1)	E	I	
83	IBB-D	D-Class 1EI&C Power Transformer	R/B	4	RT, ESF, PAM	2wks	Mild	Mild	No (1)	E	I	
84	IBD-D	D-Class 1E AC120V Panelboard	R/B	4	RT, ESF, PAM	2wks	Mild	Mild	No (1)	E	I	

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Table 3D-2 US-APWR Environmental Qualification Equipment List (Sheet 27 of 67)

Item Num	EquipmentTag	Description	Location		Purpose	Operational Duration	Environmental Conditions	Radiation Condition	Influence of Submergence for Total Integrated Dose	Qualification Process	Seismic Category	Comments
			Building	Zone	RT, ESF, PAM, Pressure Boundary (PB), Other(1)		Harsh or Mild	Harsh or Mild	Yes/No	E=Electrical M=Mechanical	I, II, Non	
85	MVID1	D-MOV Inverter1	~~R/B~~PS/B	4	ESF, PB	2wks	Mild	Mild	No (1)	E	I	
86	MVID2	D-MOV Inverter2	~~R/B~~PS/B	4	ESF, PB	2wks	Mild	Mild	No (1)	E	I	
87	MVCD1	D-MOV Motor Control Center1	~~R/B~~PS/B	4	ESF, PB	2wks	Mild	Mild	No (1)	E	I	
88	MVCD2	D-MOV Motor Control Center2	~~R/B~~PS/B	4	ESF, PB	2wks	Mild	Mild	No (1)	E	I	
89	VCC-A	A-Ventilation Chiller Control Cabinet	PS/B	9	ESF	2wks	Mild	Mild	No (1)	E	I	
90	VCC-B	B-Ventilation Chiller Control Cabinet	PS/B	9	ESF	2wks	Mild	Mild	No (1)	E	I	
91	VCC-C	C-Ventilation Chiller Control Cabinet	PS/B	9	ESF	2wks	Mild	Mild	No (1)	E	I	
92	VCC-D	D-Ventilation Chiller Control Cabinet	PS/B	9	ESF	2wks	Mild	Mild	No (1)	E	I	
93	BAT-A	A-Class 1E Battery	PS/B	5	RT, ESF	2hr	Mild	Mild	No (1)	E	I	
94	BAT-B	B-Class 1E Battery	PS/B	5	RT, ESF	2hr	Mild	Mild	No (1)	E	I	
95	BAT-C	C-Class 1E Battery	PS/B	5	RT, ESF	2hr	Mild	Mild	No (1)	E	I	
96	BAT-D	D-Class 1E Battery	PS/B	5	RT, ESF	2hr	Mild	Mild	No (1)	E	I	
97	EPBA	A-Class 1E Gas Turbine Generator Control Board	PS/B	11	ESF	2wks	Mild	Mild	No (1)	E	I	
98	EPBB	B-Class 1E Gas Turbine Generator Control Board	PS/B	11	ESF	2wks	Mild	Mild	No (1)	E	I	
99	EPBC	C-Class 1E Gas Turbine Generator Control Board	PS/B	11	ESF	2wks	Mild	Mild	No (1)	E	I	
100	EPBD	D-Class 1E Gas Turbine Generator Control Board	PS/B	11	ESF	2wks	Mild	Mild	No (1)	E	I	
101	TTSD-A	A-Turbine Trip Solenoid Distribution Panel	R/B	4	Other	2wks	Mild	Mild	No (1)	E	I	
102	TTSD-D	D-Turbine Trip Solenoid Distribution Panel	R/B	4	Other	2wks	Mild	Mild	No (1)	E	I	
103	SRPP-A	Source Range Neutron Flux Preamplifier Panel (Train A)	R/B	6	RT, Other	36hr	Mild	Harsh	No (1)	E	I	
104	SRPP-D	Source Range Neutron Flux Preamplifier Panel (Train D)	R/B	6	RT, Other	36hr	Mild	Harsh	No (1)	E	I	
105	WRPP-A	Wide Range Neutron Flux Preamplifier Panel (Train A)	R/B	6	PAM	4mos	Mild	Harsh	No (1)	E	I	
106	WRPP-D	Wide Range Neutron Flux Preamplifier Panel (Train D)	R/B	6	PAM	4mos	Mild	Harsh	No (1)	E	I	
Equipment (Reactor Coolant System)												

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Table 3D-2 US-APWR Environmental Qualification Equipment List (Sheet 31 of 67)

Item Num	EquipmentTag	Description	Location		Purpose	Operational Duration	Environmental Conditions	Radiation Condition	Influence of Submergence for Total Integrated Dose	Qualification Process	Seismic Category	Comments
			Building	Zone	RT, ESF, PAM, Pressure Boundary (PB), Other(1)		Harsh or Mild	Harsh or Mild	Yes/No	E=Electrical M=Mechanical	I, II, Non	
37	CVS-AOV-001C	Air Operated Valve	PCCV	1-4	PB	1yr	Harsh	Harsh	No (1)	M	I	
38	CVS-HCV-012	Air Operated Valve	PCCV	1-5	PB	1yr	Harsh	Harsh	No (1)	M	I	
39	CVS-HCV-100	Air Operated Valve	PCCV	1-5	PB	1yr	Harsh	Harsh	No (1)	M	I	
40	CVS-AOV-224	Air Operated Valve	PCCV	1-5	PB	1yr	Harsh	Harsh	No (1)	M	I	
41	~~CVS-LCV-031F~~Deleted	~~Level Control Valve~~	~~R/B~~	~~7~~	~~Other~~	~~2 wks~~	~~Mild~~	~~Harsh~~	~~No (1)~~	~~M~~	~~I~~	
42	~~CVS-LCV-031G~~Deleted	~~Level Control Valve~~	~~R/B~~	~~7~~	~~Other~~	~~2wks~~	~~Mild~~	~~Harsh~~	~~No (1)~~	~~M~~	~~I~~	
43	CVS-AOV-196A	Air Operated Valve	PCCV	1-4	PB	1yr	Harsh	Harsh	No (1)	M	I	
44	CVS-AOV-196B	Air Operated Valve	PCCV	1-4	PB	1yr	Harsh	Harsh	No (1)	M	I	
45	CVS-AOV-196C	Air Operated Valve	PCCV	1-4	PB	1yr	Harsh	Harsh	No (1)	M	I	
46	CVS-AOV-196D	Air Operated Valve	PCCV	1-4	PB	1yr	Harsh	Harsh	No (1)	M	I	
Equipment (Safety Injection System)												
1	SIS-MPP-001A	A-Safety Injection Pump	R/B	6	ESF	1yr	Mild	Harsh	No (1)	M	I	
2	SIS-MPP-001B	B-Safety Injection Pump	R/B	6	ESF	1yr	Mild	Harsh	No (1)	M	I	
3	SIS-MPP-001C	C-Safety Injection Pump	R/B	6	ESF	1yr	Mild	Harsh	No (1)	M	I	
4	SIS-MPP-001D	D-Safety Injection Pump	R/B	6	ESF	1yr	Mild	Harsh	No (1)	M	I	
5	~~SIS-MTK-001A~~Deleted	~~A-Accumulator~~	~~PCCV~~	~~1-5~~	~~ESF~~	~~1yr~~	~~Harsh~~	~~Harsh~~	~~No (1)~~	~~M~~	~~I~~	
6	~~SIS-MTK-001B~~Deleted	~~B-Accumulator~~	~~PCCV~~	~~1-5~~	~~ESF~~	~~1yr~~	~~Harsh~~	~~Harsh~~	~~No (1)~~	~~M~~	~~I~~	
7	~~SIS-MTK-001C~~Deleted	~~C-Accumulator~~	~~PCCV~~	~~1-5~~	~~ESF~~	~~1yr~~	~~Harsh~~	~~Harsh~~	~~No (1)~~	~~M~~	~~I~~	
8	~~SIS-MTK-001D~~Deleted	~~D-Accumulator~~	~~PCCV~~	~~1-5~~	~~ESF~~	~~1yr~~	~~Harsh~~	~~Harsh~~	~~No (1)~~	~~M~~	I	
9	SIS-MOV-001A	Motor Operated Valve	R/B	6	ESF	1yr	Harsh	Harsh	No (1)	M	I	
10	SIS-MOV-001B	Motor Operated Valve	R/B	6	ESF	1yr	Harsh	Harsh	No (1)	M	I	
11	SIS-MOV-009A	Motor Operated Valve	R/B	6	ESF	1yr	Mild	Harsh	No (1)	M	I	

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Table 3D-2 US-APWR Environmental Qualification Equipment List (Sheet 36 of 68)

Item Num	EquipmentTag	Description	Location		Purpose	Operational Duration	Environmental Conditions	Radiation Condition	Influence of Submergence for Total Integrated Dose	Qualification Process	Seismic Category	Comments
			Building	Zone	RT, ESF, PAM, Pressure Boundary (PB), Other(1)		Harsh or Mild	Harsh or Mild	Yes/No	E=Electrical M=Mechanical	I, II, Non	
3	EFS-MPP-001C	C-Emergency Feedwater Pump	R/B	8	ESF	2wks	Mild	Harsh	No (1)	M	I	
4	EFS-MPP-001D	D-Emergency Feedwater Pump	R/B	8	ESF	2wks	Mild	Harsh	No (1)	M	I	
5	~~EFS-MPT-001A~~Deleted	~~A-Emergency Feedwater Pit~~	~~R/B~~	~~14~~	~~ESF~~	~~2wks~~	~~Mild~~	~~Mild~~	~~No (1)~~	~~M~~	~~I~~	
6	~~EFS-MPT-001B~~Deleted	~~B- Emergency Feedwater Pit~~	~~R/B~~	~~14~~	~~ESF~~	~~2wks~~	~~Mild~~	~~Mild~~	~~No (1)~~	~~M~~	~~I~~	
7	EFS-MOV-006A	Motor Operated Valve	R/B	14	ESF	2wks	Mild	Mild	No(1)	M	I	
8	EFS-MOV-006B	Motor Operated Valve	R/B	14	ESF	2wks	Mild	Mild	No(1)	M	I	
9	EFS-MOV-006C	Motor Operated Valve	R/B	14	ESF	2wks	Mild	Mild	No(1)	M	I	
10	EFS-MOV-006D	Motor Operated Valve	R/B	14	ESF	2wks	Mild	Mild	No(1)	M	I	
11	EFS-MOV-014A	Motor Operated Valve	R/B	8	ESF	2wks	Mild	Harsh	No (1)	M	I	
12	EFS-MOV-014B	Motor Operated Valve	R/B	8	ESF	2wks	Mild	Harsh	No (1)	M	I	
13	EFS-MOV-014C	Motor Operated Valve	R/B	8	ESF	2wks	Mild	Harsh	No (1)	M	I	
14	EFS-MOV-014D	Motor Operated Valve	R/B	8	ESF	2wks	Mild	Harsh	No (1)	M	I	
15	EFS-MOV-017A	A-Emergency Feedwater Control Valve	R/B	10	ESF	2wks	Harsh	Harsh	No (1)	M	I	
16	EFS-MOV-017B	B-Emergency Feedwater Control Valve	R/B	10	ESF	2wks	Harsh	Harsh	No (1)	M	I	
17	EFS-MOV-017C	C-Emergency Feedwater Control Valve	R/B	10	ESF	2wks	Harsh	Harsh	No (1)	M	I	
18	EFS-MOV-017D	D-Emergency Feedwater Control Valve	R/B	10	ESF	2wks	Harsh	Harsh	No (1)	M	I	
19	EFS-MOV-019A	A-Emergency Feedwater Isolation Valve	R/B	10	ESF	2wks	Harsh	Harsh	No (1)	M	I	
20	EFS-MOV-019B	B-Emergency Feedwater Isolation Valve	R/B	10	ESF	2wks	Harsh	Harsh	No (1)	M	I	
21	EFS-MOV-019C	C-Emergency Feedwater Isolation Valve	R/B	10	ESF	2wks	Harsh	Harsh	No (1)	M	I	
22	EFS-MOV-019D	D-Emergency Feedwater Isolation Valve	R/B	10	ESF	2wks	Harsh	Harsh	No (1)	M	I	
23	EFS-MOV-101A	A-Emergency Feedwater Pump A-Main Steam Line Steam Isolation Valve	R/B	10	ESF	2wks	Harsh	Harsh	No (1)	M	I	
24	EFS-MOV-101B	A-Emergency Feedwater Pump B-Main Steam Line Steam Isolation Valve	R/B	10	ESF	2wks	Harsh	Harsh	No (1)	M	I	

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APPENDIX 3H

MODEL PROPERTIES FOR LUMPED MASS STICK MODELS OF R/B-PCCV-CONTAINMENT INTERNAL STRUCTURE ON A COMMON BASEMAT

CONTENTS

~~TABLES~~

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~~FIGURES~~

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~~ACRONYMS AND ABBREVIATIONS~~

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~~ASCE~~	~~American Society of Civil Engineers~~
~~DOF~~	~~degree of freedom~~
~~DCD~~	~~design control document~~
~~FE~~	~~finite element~~
~~FH/A~~	~~fuel handling area~~
~~PCCV~~	~~prestressed concrete containment vessel~~
~~PS/B~~	~~power source building~~
~~R/B~~	~~reactor building~~
~~RCL~~	~~reactor coolant loop~~
~~RCP~~	~~reactor coolant pump~~
~~RV~~	~~reactor vessel~~
~~SG~~	~~steam generator~~
~~SSI~~	~~soil-structure interaction~~

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3H Model Properties for Lumped Mass Stick Models of R/B-PCCV-Containment Internal Structure on a Common Basemat

3H.1 Introduction

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~~Refer to MUAP-10001, "Seismic Design Bases of the US-APWR Standard Plant" (Reference 3H-4) for the model properties for lumped mass stick model of reactor building (R/B) – prestressed concrete containment vessel (PCCV) – Containment Internal structure on a common basemat.This Appendix discusses the general approach taken and the general properties of the reactor building (R/B), prestressed concrete containment vessel (PCCV), and containment internal structure lumped mass stick models. Technical Report MUAP-11006 (Reference 3H-7) describes further details on the development and validation of the ACS SASSI (Reference 3H-1) lumped mass stick model of the R/B complex. The results of the soil-structure interaction (SSI) analyses performed on this model provide base reactions that serve for evaluation of seismic stability of the R/B complex foundation. The lumped mass stick model is also used in embedment effects and structure-soil-structure-interaction SSSI studies as representation of the overall dynamic properties of the R/B complex.~~Lumped mass stick models are no longer used in the seismic analysis of the R/B complex. Refer to MUAP-10006, "Soil-Structure Interaction Analyses and Results for the US-APWR Standard Plant," (Reference 3H-1) for discussion of the Finite Element model.

3H.2 ~~Deleted~~~~Model Properties~~

~~The methodology used to develop the lumped mass stick model is provided in the following subsections. The procedures used for development of analytical models for seismic analysis are consistent with the procedures and guidelines of SRP 3.7.2, Section II.3 (Reference 3H-3). Structural element mass and stiffness characteristics, as well as load and tributary masses, and damping characteristics, are incorporated into the models.~~

~~The mass inertia and stiffness properties of the basemat and basement exterior walls of the R/B, and the entire power source buildings (PS/Bs), are characterized through the use of finite element (FE) models. Lumped mass stick models, interconnected by stiffness elements, are used to discretize the mass inertia and stiffness properties of the remaining portions of the R/B, the PCCV, and containment internal structure. The mass is lumped in selected nodes in a way that provides an adequate representation of the mass distribution considering the high-stress concentration points of the system. In general, lumped mass inertia is assigned at the selected locations in all six degrees of freedom (DOF) corresponding to translations along three orthogonal axes, and rotations about these axes. Figures 3H-1 and 3H-2 present the lumped mass stick model of the R/B, PCCV, and the containment internal structure.~~

~~The lumped mass stick models for the United States-Advanced Pressurized Water Reactor (US-APWR) R/B, PCCV, and containment internal structure and their basemat consider the eccentricities between the center of rigidity and the center of mass of structures. The models represent the actual locations of masses and centers of rigidity, thus, accounting for the torsional effects caused by the eccentricity. The modeling approach also accounts for the differences between the vertical and horizontal centers of rigidity by using two stick elements to model the stiffness of the structural members at~~

~~each story. A truss element located at the vertical center of rigidity represents the vertical stiffness of the floor, and a beam element located at the shear center of rigidity represents the shear and bending stiffness of the floor. Both stick elements are rigidly connected to the common center of mass at each major floor elevation. This modeling approach helps eliminate errors in computation of seismic responses that are due to rocking SSI effects caused by an inaccurate representation of the vertical center of rigidity.~~ DCD_03.07.02-35 MIC-03-03-00054

~~The structural elements of the R/B, which includes the fuel handling area (FH/A), are concentrated and reduced to one set of stick models below the operating floor level. The part of the R/B above the operation floor level, except for the FH/A, is represented by several stick models that are interconnected by horizontal rigid links representing the floor diaphragm. The rigid links restrain only the in-plane translational displacements of the floor without affecting the deformations in the other DOF. The containment internal structure and the PCCV are modeled separately, and they are rigidly connected to the R/B stick model at the surface of the common basemat. The R/B, PCCV, and containment internal structure are all structurally separated from each other above their common basemat by expansion joints. Detailed descriptions of the R/B, PCCV, and containment internal structure and their common basemat are presented in Section 3.8, where the structural design of those buildings and structures is addressed.~~

3H.2.1 ~~Subsystem Coupling Requirements~~

~~For purposes of modeling the R/B-PCCV-containment internal structure on their common basemat, large seismic subsystems contained within these structures are evaluated against the mass and frequency ratio criteria given in SRP 3.7.2, Section II.3(B) (Reference 3H-3), as follows:~~

i. ~~If R_m < 0.01, decoupling can be done for any R_f~~

ii. ~~If 0.01 ≤ R_m ≤ 0.1, decoupling can be done if 0.8 ≥ R_f ≥ 1.25~~

iii. ~~If R_m > 0.1, a subsystem model should be included in the primary system model.~~

~~where,~~

~~*Rm* = (total mass of supported subsystem)/(total mass of supporting system)~~

~~*Rf* = (fundamental frequency of supported system)/(dominant frequency of support motion)~~

~~If these criteria require the subsystem to be coupled with the primary seismic model, both the stiffness and the mass of the subsystem are included in the overall model to assure the accuracy of the calculated frequencies. This is the approach used for including the reactor coolant loop (RCL) seismic subsystem in the coupled RCL-R/BPCCV-containment internal structure lumped mass stick model discussed in Technical Report MUAP-11006 (Reference 3H-7).~~

~~When it has been determined through investigation of the above criteria that a subsystem is not required to be coupled with the primary seismic model, then the subsystem is~~

~~assumed absolutely rigid and only its mass is included at appropriate node points of the global seismic model. The PCCV polar crane and fuel handling crane are incorporated into the overall lumped mass stick model in this manner. In addition, the requirements of NOG-1 (Reference 3H-8) for the design of cranes may require that the crane design analysis be performed by coupling the crane model with the overall building model. If required, the site-specific seismic analysis of the US-APWR standard plant must be performed on models that incorporate the PCCV polar crane and the fuel handling crane, as appropriate.~~

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3H.2.2 ~~Section and Material Properties for Lumped Mass Stick Models~~

~~The values of the modulus of elasticity and Poisson's ratio (ν) for concrete and steel used in the lumped mass stick models are discussed below. The values are for materials at or near ambient temperatures.~~

~~(a) Concrete~~

~~The concrete modulus of elasticity E_c, and shear modulus G_c corresponding to the compressive strengths of normal weight concrete used in the R/B, PCCV, and containment internal structure and their common basemat are summarized in Table 3H-1 and are computed as follows:~~

$\cancel{E_c \text{ (ksi)} = 57{,}000 \sqrt{f'_c}}$

$\cancel{G \text{ (ksi)} = E_c / 2 (1 + \nu_c)}$

~~where~~

$\cancel{f'_c}$ ~~= specified 28-day compressive strength of concrete (psi)~~

$\cancel{\nu_c = 0.17}$ ~~(Poisson's ratio for concrete)~~

~~(b) Steel~~

~~The properties of ferritic structural steel and non-prestressed reinforcement, E_s and ν_s are as follows:~~

$\cancel{E_s = 29{,}000 \text{ ksi and } \nu_s = 0.3}$

3H.2.3 ~~Masses~~

~~The inertial properties include all tributary mass expected to be present at the time of the earthquake. This mass includes the effects of dead load, stationary equipment, piping, and the appropriate part of the live and snow load. The mass properties of the stick model consist of the total weight W, the weight moment of inertia (J_{xx}, J_{yy}, J_{zz}), and the center of mass. They are in principle evaluated by hand calculation as described below.~~

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3H.2.3.1 ~~Mass Points and Associated Weights (*W*)~~

~~The mass points are, in principle, established at the major floor levels and/or major equipment locations represented by nodes in the lumped mass stick model. Figure 3H-5 depicts how the mass moments of inertia and weights associated with the lumped masses are computed.~~

~~In addition to the structural mass, mass equivalent to 60 psf miscellaneous dead load is considered to represent miscellaneous weights such as minor equipment, piping, and raceways. The mass equivalent to 25% of the floor design live load and 75% of the roof design snow load is also included. The mass of major equipment is considered to be distributed over a representative floor area or included as concentrated lumped masses at the equipment locations.~~

~~Vertical amplification effects on the masses of floor slab systems due to out-of-plane flexibility are addressed as part of Technical Reports MUAP-11006 and MUAP-10006 (References 3H-7 and 3H-5).~~

3H.2.3.2 ~~Mass Moment of Inertia (J_{xx}, J_{yy}, J_{zz})~~

~~Mass moments of inertia are considered at all of the mass points for all three rotational DOF.~~

3H.2.4 ~~Shear Stiffness~~

~~The effect of in-plane shear deformation is included in the model. For the lumped mass stick model, effective shear area is computed from the sum of the component shear areas of the individual walls parallel to the direction of the applied force. The stiffness of the stick model is evaluated by hand calculation as shown in Figure 3H-6, which summarizes the methods used to compute the section areas, geometrical moments of inertia, and axial areas for the PCCV and R/B structures, under the following assumptions:~~

- ~~Walls continuously built up from the basemat, whose thickness is more than 40 in. are treated as seismic walls.~~
- ~~Openings with area more than 2,880 in^2 (80 in. x 36 in.) are considered in evaluating the stiffness of walls.~~

~~Generally, in accordance with American Society of Civil Engineers (ASCE) 4 (Reference 3H-4) Subsection C3.1.8.3, if the shear wall has no flange elements at its ends, the shear area is equal to the total web area divided by 1.2. If flanges are present, the shear area is equal to the total web area. The effective flange width of each perpendicular wall may be calculated using the following reduction due to shear lag effects:~~

$$\cancel{W_e = \frac{H}{3} \le \frac{W}{2}} \qquad \text{(Eq. 3.H.2.1)}$$

~~where~~

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~~W_e = effective flange width on each side of the wall~~

~~H = height of the wall~~

~~W = actual width of the flange on each side of the wall~~

~~The above guidance on shape factors and reductions to flange width to account for shear lag effects is adjusted where necessary for some shear walls based on detailed local analysis in order to assure that the overall lumped mass stick model contains accurate shear stiffness values. For example, for the containment internal structure the effective shear area of the stick elements is calculated by a FE model as follows:~~

i. ~~A FE model of the containment internal structure above the upper level of the basemat, considering the walls, columns and floor slabs, is developed using brick, shell and beam elements.~~

ii. ~~By fixing the containment internal structure where it intersects with the upper level of the basemat, a set of vertically distributed horizontal loads, which is established considering the earthquake excitation, is applied at each main floor level and the resulting horizontal displacements are evaluated at the top level of each floor. To determine which portion of the resulting displacement at each floor is attributable to shear stiffness and which portion is related to bending stiffness, another analytical model in which the vertical DOF is constrained is also prepared separately. The flexibility coefficients for the equivalent beam are evaluated from the results of these analyses.~~

iii. ~~The equivalent stiffness properties (the equivalent shear stiffness, bending stiffness, etc) are evaluated from (ii) above.~~

3H.2.4.1 ~~Effective Shear Area (A_x, A_y)~~

~~Two effective shear areas A_x and A_y are calculated for each floor by considering the seismic walls that are parallel to each two horizontal earthquake directions. For the walls having openings, an equivalent shear area (A_e) is considered that is calculated using the equal shear deformation methodology depicted in Figure 3H-7. From the requirement that the shear deformation δ_s of the wall with openings be equal to the shear deformation δ_s of a wall without openings with height equal the story height (H), the effective cross section area of the wall A_e is obtained from the following equation:~~

$$\cancel{A_e = \frac{H}{\sum_{i=1}^{n} \frac{\kappa_i h_i}{A_i}}} \qquad \text{(Eq. 3H.2.2)}$$

~~where~~

~~A_i = the shear area~~

~~h_i = height of wall segment~~ DCD_03.07.02-35 MIC-03-03-00054

~~κ_i = a shape factor~~

3H.2.4.2 ~~Bending Moment of Inertia (I_{yy}, I_{xx})~~

~~Bending moment of inertia of the shear walls are calculated around the horizontal axes established at the centroid of the floor shear walls area. The effective flange width of each perpendicular wall is calculated using the following reduction due to shear lag effects:~~

$$W_e = \frac{H}{3} \le \frac{W}{2} \qquad \text{(Eq. 3H.2.3)}$$

~~where~~

~~W_e = effective flange width on each side of the wall~~

~~H = height of the wall~~

~~W = actual width of the flange on each side of the wall~~

~~Equivalent moments of inertia (I_e) are calculated for the walls with openings using the equal bending rotation methodology as shown in Figure 3H-7. The bending rotation θ of the wall with openings loaded with unit bending moment M is calculated as follows:~~

$$\theta = \int_0^{h_1} \frac{M}{EI_1} dh + \dots + \int_{h_1 + \dots + h_{n-1}}^{h_1 + \dots + h_{n-1} + h_n} \frac{M}{EI_n} dh \qquad \text{(Eq. 3H.2.4)}$$

~~where I_i and h_i are the moment of inertia and height of wall segment as shown in Figure 3H-7. The bending rotation θ' of equivalent wall without openings with height equal the story height (H) and cross section with equivalent moment of inertia I_e is:~~

$$\theta' = \int_0^{H} \frac{M}{EI_e} dh = \frac{MH}{EI_e} \qquad \text{(Eq. 3.H.2.5)}$$

~~From the requirement that the bending rotation of the two walls to be equals ($\theta = \theta'$) the effective moment of inertia is obtained as follows:~~

$$I_e = \frac{H}{\sum_{i=1}^{n} \frac{h_i}{I_i}} \qquad \text{(Eq. 3.H.2.6)}$$

3H.2.4.3 ~~Torsional Stiffness~~

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~~The stick elements modeling the stiffness of the floor shear walls and columns are to be located at the center of rigidity of the floor, and an appropriate torsional stiffness must be assigned if the center of mass is not coincident with the center of rigidity. The torsional rigidity,~~ ~~K_p~~~~, can be computed from the following equations:~~

$$\cancel{K_p = \sum_{i=1}^{N}(K_{yi}\bar{X}_i^2 + K_{xi}\bar{Y}_i^2) - X_{cr}^2\sum_{i=1}^{N}K_{yi} - Y_{cr}^2\sum_{i=1}^{N}K_{xi}} \qquad \text{~~(Eq. 3.H.2.7)~~}$$

~~where~~

~~$\bar{X}_i$, $\bar{Y}_i$ = coordinates of i^{th} wall or column elements~~

~~K_{xi}, K_{yi} = stiffness of i^{th} wall or column effective for shear, assuming rigid connection to the floor, in x and y directions, respectively~~

~~X_{cr}, Y_{cr} = coordinates of center of rigidity~~

$$\cancel{X_{cr} = \frac{\sum_{i=1}^{N}(\bar{X}_i K_{yi})}{\sum_{i=1}^{N}(K_{yi})},\ Y_{cr} = \frac{\sum_{i=1}^{N}(\bar{Y}_i K_{yi})}{\sum_{i=1}^{N}(K_{xi})}} \qquad \text{~~(Eq. 3.H.2.8)~~}$$

~~Alternatively, the torsional stiffness of the floor can be obtained from the FE analysis of the floor model. Multiple stick elements may be used to represent the stiffness of the floor by locating the stick stiffness elements at the centers of rigidity of the respective groups of structural members (shear walls or beams). The multiple floor stick stiffness elements must be properly interconnected among each other and with the node(s) where the mass inertia of the floor is lumped.~~

3H.2.4.3.1 ~~Torsional Constant (I_{zz})~~

~~Torsional constant of seismic shear walls is calculated around the vertical axis that goes through the center of the floor shear rigidity.~~

3H.2.4.4 ~~Axial Stiffness~~

~~Axial stiffness is calculated considering all walls that contribute to shear stiffness and from vertically acting supports such as columns.~~

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3H.2.4.4.1 ~~Vertical Axial Area (A_a)~~

~~Vertical axial area of each element of the lumped mass stick model is equal to the summation of the effective shear areas for the two horizontal directions and axial area contributed by other vertically acting supports such as columns. However, overlapping areas such as the corner areas of box walls are subtracted from the summation. The axial stiffness of the dome of PCCV is evaluated as follows:~~

i. ~~A FE model of PCCV as shown in Figure 3H-8 is developed.~~

ii. ~~The static analysis is performed by applying to the FE model vertical loads with magnitude equivalent to the weight at the corresponding mass point weights at locations which correspond to the locations of the mass points in the stick model.~~

iii. ~~Vertical displacements are calculated from the static FE analysis and used to evaluate the effective vertical axial area as follows:~~

$$K_i = \frac{A_i E}{L_i} = \frac{\sum W_i}{\Delta_i - \Delta_{i-1}} \qquad \text{(Eq. 3.H.2.9)}$$

~~where~~

~~K_i = effective vertical axial stiffness of the element i~~

~~A_i = effective vertical axial area of the element i~~

~~L_i = length of the element i~~

~~W_i = weight of the mass i~~

~~Δ_i = vertical displacement of node i~~

~~E = Young's modulus of the concrete material~~

3H.3 ~~Lumped Mass Stick Models of Major Equipment and Piping~~

~~Since the RCL spans several locations of the building and is characterized by several significant frequencies and participating masses, a lumped parameter model connected at appropriate locations of the containment internal structure is used to represent the stiffness and mass inertia properties of the major piping and equipment, such as the reactor vessel (RV), steam generators (SGs), and main coolant piping. Based on the decoupling criteria of SRP 3.7.2 (Reference 3H-3), with the exception of the RCL, the subsystems and components inside the containment and in the R/B are included in the coupled model by lumped masses placed at appropriate node locations.~~

~~The RCL lumped mass stick model represents the dynamic properties of RV, SG, RCP, main coolant piping, and component supports, as applicable, for each of the four loops. The RCL piping and support system is modeled as a three-dimensional model~~

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~~representing the stiffness of components, pipes, and supports as beam elements and springs and the mass inertia with mass lumped at selected node locations.~~

~~The RCL of the US-APWR has four loops, which are modeled as a combination of RV, SG, RCP and main coolant piping. These combined system models include both the translational and rotational stiffness, mass characteristics of the RCL piping and components, and the stiffness of supports. The RCL lumped mass stick model considers the stiffness and mass of the auxiliary line piping that affect the dynamic response of the RCL system. Refer to Section 5.3.2 of Technical Report MUAP-10001 (Reference 3H-6) for the mass, material properties and stiffness characteristics of the individual components of the RCL such as the RV, SG, RCP and main coolant piping.~~

~~The RV support system consists of eight steel support pads which are integrated with the inlet and outlet nozzle forgings. The support pads are placed on brackets, which are supported by an embedded steel structure on the primary shield wall. The supports allow radial thermal growth of the RCS and RV. Details of the RV supports and their relationship with the primary shield wall are presented in Section 3.8 of the US-APWR design control document (DCD).~~

~~The SG support system consists of an upper shell support structure, an intermediate shell support structure and a lower support structure. The upper and intermediate shell supports are lateral restraints (snubbers) attached to structural steel brackets, while the lower support structure is constructed entirely of structural steel and provides both vertical and lateral support. Four pinned-end columns to the slab support the vertical loads of the SG.~~

~~The RCP support system consists of a lateral support structure, and three pinned-end structural columns to the slab. Both support structures are designed considering thermal expansion of connected piping.~~

~~Figure 3H-3 presents the analytical model of the entire RCL including the individual components of the four loops and the support representation. Figure 3H-4 and Table 3H-3 identify the connectivity between the RCL nodes and the building nodes. Typically, the fixed end nodes of the RCL support springs are attached to the nodes associated with the internal concrete by rigid beam elements. For example, the nodes which represent the ends of the RV support springs at the RV inlet and outlet nozzle are all connected by rigid beams to IC03 of the containment internal structure at that elevation. The stiffness characteristics of the supports are provided in Table 3H-2.~~

3H.4 ~~Validation of Lumped Mass Stick Models~~

3H.4.1 ~~PCCV Model Validation Results~~

3H.4.1.1 ~~1g Static Analysis~~

~~A 1g static analysis in the vertical (Z) direction with full constraints at the bottom of the containment structure is performed on both PCCV models to verify their overall static weight. The difference in weight between the 3-D FE model and the lumped mass stick model is found to be within 1.2% and is therefore found to be acceptable.~~

~~A 1g static analysis in the horizontal (X) direction with full constraints at the bottom of the PCCV structure is also performed on both models. The displacement results from both static analyses are used to compare the structure stiffness of the two models in all directions. The displacement distributions along the height of the PCCV indicate that the stick model stiffness is comparable to that of the 3-D FE model.~~ DCD_03.07.02-35 MIC-03-03-00054

3H.4.1.2 ~~Modal Analysis~~

~~A modal analysis using ANSYS (Reference 3H-2) is performed on the 3-D FE model and a harmonic analysis is performed on the lumped mass stick model under a fixed base support condition. The comparison of the results indicate that the stick model captures the structure response to dynamic loading in all directions properly. Refer to Technical Report MUAP-11006 (Reference 3H-7) for further details.~~

3H.4.1.3 ~~Mode-Superposition Transient Dynamic Analysis~~

~~Refer to Technical Report MUAP-11006 (Reference 3H-7) for the discussion and results of the mode-superposition transient dynamic analysis performed.~~

3H.4.2 ~~R/B and FH/A Model Validation Results~~

3H.4.2.1 ~~1g Static Analysis~~

~~A 1g static analysis in the vertical direction (Z) with full constraints at the bottom of the R/B structure is performed on both R/B models (lumped mass stick model and 3-D FE model, shown in Figure 3H-9) in ANSYS to verify their overall static weight. The difference in weight between the 3-D FE model and the lumped mass stick model is found to be within 4.67%. The weight of the lumped mass stick model is slightly higher than the weight of the 3-D FE model due to node RE00 of the Stick Model incorporating additional weight from the basemat.~~

~~A 1g static analysis is performed on each R/B model in both horizontal (X and Y) directions with full constraints at the bottom of each model. The displacement results from both static analyses are used to compare the structure stiffness of the two models in all directions. The displacement distributions along the height of the R/B at each side of the structure indicate that the displacements in NS direction are consistent in shape for all but the NE corner of the FH/A located above R/B operational floor elevation. The comparison of the shapes of the deformations in EW direction calculated from the ANSYS analyses of both models indicates that they are consistent for all areas of the building but the FH/A portion. For the FH/A, the stiffness of the crane steel support and concrete exterior walls in the SASSI stick model are lumped together in a single stick element, so the local effects are not explicitly captured when modeling the overall lateral stiffness of the FH/A.~~

3H.4.2.2 ~~Modal Analysis~~

~~A modal analysis using ANSYS (Reference 3H-2) is performed on the 3-D FE model and a harmonic analysis is performed on the lumped mass stick model under a fixed base support condition. The comparison of the results indicate that the stick model captures~~

~~the structure response to dynamic loading in all directions properly. Refer to Technical Report MUAP-11006 (Reference 3H-7) for further details.~~ DCD_03.07.02-35 MIC-03-03-00054

3H.4.2.3 ~~Mode-Superposition Transient Dynamic Analysis~~

~~Refer to Technical Report MUAP-11006 (Reference 3H-7) for the discussion and results of the mode-superposition transient dynamic analysis performed.~~

3H.4.3 ~~Containment Internal Structure Model Validation Results~~

3H.4.3.1 ~~1g Static Analysis~~

~~A 1g static analysis in the vertical (Z) direction with full constraints at the bottom of the containment structure is performed on both containment internal structure models (lumped mass stick model and 3-D FE model, shown in Figure 3H-10) to verify their overall static weight. The difference in weight between the 3-D FE model and the lumped mass stick model is found to be within 1.01% and is therefore found to be acceptable.~~

~~A 1g static analysis in the horizontal (X and Y) directions with full constraints at the bottom of the containment internal structure is also performed on both models. The displacement results from both static analyses are used to compare the structure stiffness of the two models in all directions. The displacement distributions along the height of the PCCV indicate that the stick model stiffness is comparable to that of the 3-D FE model.~~

3H.4.3.2 ~~Modal Analysis~~

~~A modal analysis using ANSYS (Reference 3H-2) is performed on the 3-D FE model and a harmonic analysis is performed on the lumped mass stick model under a fixed base support condition. The comparison of the results indicate that the stick model captures the structure response to dynamic loading in all directions properly. Refer to Technical Report MUAP-11006 (Reference 3H-7) for further details.~~

3H.4.3.3 ~~Mode-Superposition Transient Dynamic Analysis~~

~~Refer to Technical Report MUAP-11006 (Reference 3H-7) for the discussion and results of the mode-superposition transient dynamic analysis performed.~~

3H.5 References

3H-1 ~~Deleted.ACS SASSI: Version 2.3.0 Including Option A, An Advanced Computational Software for 3D Dynamic Analysis including Soil-Structure Interaction, Users Manuals, Revision 3.0, Ghiocel Predictive Technologies, Inc., December 30, 2010.~~

3H-2 ~~Deleted.ANSYS, Advanced Analysis Techniques Guide, Release 11.0, ANSYS, Inc., 2007.~~

3H-3 ~~Deleted.Seismic System Analysis, Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants. NUREG-0800, SRP 3.7.2, Rev. 3, United States Nuclear Regulatory Commission, March 2007.~~ DCD_03.07.02-35 MIC-03-03-00054

3H-4 ~~Seismic Design Bases of the US-APWR Standard Plant, MUAP-10001, Rev. 2, Mitsubishi Heavy Industries, January 2011.Seismic Analysis of Safety-Related Nuclear Structures and Commentary, American Society of Civil Engineers, ASCE 4-98, Reston, VA, 2000.~~

~~3H-5~~ Soil-Structure Interaction Analyses and Results for the US-APWR Standard Plant, MUAP-10006, Rev. ~~1~~3, Mitsubishi Heavy Industries, Ltd., ~~January 2011~~November 2012.

~~3H-6~~ ~~Seismic Design Bases of the US-APWR Standard Plant, MUAP-10001, Rev. 3, Mitsubishi Heavy Industries, Ltd., June 2011.~~

~~3H-7~~ ~~Lumped Mass Stick Model of US-APWR Reactor Building Complex, MUAP-11006, Revision 0, Mitsubishi Heavy Industries, Ltd., June 2011.~~

~~3H-8~~ ~~Rules for Construction of Overhead and Gantry Cranes, (Top Running Bridge, Multiple Girder), ASME NOG-1-2004.~~

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~~**Table 3H-1 Concrete Material Constants**~~

	~~Modulus of Elasticity (Young's Modulus) E_c (ksi)~~	~~Shear Modulus G_c (ksi)~~	~~Poisson's Ratio ν_c~~	~~Remark~~
~~**PCCV**~~	~~4,769~~	~~2,040~~	~~0.17~~	~~f'_c = 7,000 psi~~
~~**R/B**~~	~~3,605~~	~~1,540~~	~~0.17~~	~~f'_c = 4,000 psi~~
~~**Containment Internal Structure**~~	~~3,605~~	~~1,540~~	~~0.17~~	~~f'_c = 4,000 psi~~

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~~Table 3H-2 Support Stiffness of RV, SG and RCP~~

	~~Support Name~~	~~Location~~					~~Support~~	~~Stiffness $\times 10^3$ (kip/ft)~~
		~~x (ft)~~		~~y (ft)~~		~~z (ft)~~		
		~~A&D Loop~~	~~B&C Loop~~	~~A&B Loop~~	~~C&D Loop~~			
~~RV~~	~~Inlet Nozzle Support~~	~~-6.93~~	~~13.60~~	~~4.25~~	~~-4.25~~	~~40.39~~	~~Krh~~	~~1,002~~
							~~Krv~~	~~1,086~~
	~~Outlet Nozzle Support~~	~~-0.92~~	~~7.59~~	~~10.27~~	~~-10.27~~		~~Krh~~	~~1,002~~
							~~Krv~~	~~1,086~~
~~SG~~	~~Upper Shell Support~~	~~-10.62~~	~~17.29~~	~~33.69~~	~~-33.69~~	~~96.58~~	~~Kux~~	~~76.93~~
							~~Kuy~~	~~78.07~~
	~~Intermediate Shell Support~~					~~75.42~~	~~Kmx~~	~~83.39~~
							~~Kmy~~	~~80.46~~
	~~Lower Support~~	~~-13.80~~	~~20.47~~	~~28.40~~	~~-~~	~~45.64~~	~~Kℓ5y~~	~~33.43~~
		~~-15.91~~	~~22.55~~	~~36.87~~	~~-~~		~~Kℓ6y~~	~~69.38~~
		~~-7.44~~	~~14.11~~	~~30.98~~	~~-30.98~~		~~Kℓ7y~~	~~484.0~~
		~~-5.33~~	~~12.00~~	~~30.51~~	~~-30.51~~		~~Kℓ8y~~	~~675.9~~
		~~-~~	~~22.58~~	~~-~~	~~-36.87~~		~~Kℓ6x~~	~~555.6~~
		~~-~~	~~14.11~~	~~-~~	~~-38.98~~		~~Kℓ7x~~	~~745.8~~
~~RCP~~	~~Lower Support~~	~~-33.13~~	~~39.80~~	~~28.66~~	~~-28.66~~	~~42.61~~	~~Kpt~~	~~221.8~~

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~~Table 3H-3 Node Connectivity Between RCL and Containment Internal Structure~~

~~No.~~	~~Item~~	~~Containment Internal Structure Location~~		~~RCL Node~~		~~RCL Location~~				
		~~El. (ft)~~	~~Node~~			~~EL. (ft)~~	~~x (ft)~~		~~y (ft)~~	
							~~A & D~~	~~B & C~~	~~A & B~~	~~C & D~~
~~1~~	~~Base Floor of SG, RCP Columns~~	~~25.25~~	~~IC02~~	~~PartD~~	~~S(A/B/C/D)C1~~	~~25.68~~	~~-13.80~~	~~20.47~~	~~28.40~~	~~-28.40~~
					~~S(A/B/C/D)C2~~		~~-15.91~~	~~22.58~~	~~36.87~~	~~-36.87~~
					~~S(A/B/C/D)C3~~		~~-7.44~~	~~14.11~~	~~38.98~~	~~-38.98~~
					~~S(A/B/C/D)C4~~		~~-5.32~~	~~12.00~~	~~30.51~~	~~-30.51~~
				~~PartE~~	~~P(A/B/C/D)S1~~	~~25.68~~	~~-33.13~~	~~39.80~~	~~28.66~~	~~-28.66~~
					~~P(A/B/C/D)S2~~		~~-24.50~~	~~31.17~~	~~24.94~~	~~-24.94~~
					~~P(A/B/C/D)S3~~		~~-29.67~~	~~36.33~~	~~20.09~~	~~-20.09~~
	~~Rigid connection between IC02 and base plates of SG or RCP Columns.~~									
~~2~~	~~RV Support~~	~~35.88~~	~~IC03~~	~~PartF~~	~~RV(A/B/C/D)S1~~	~~40.39~~	~~-5.68~~	~~12.35~~	~~7.28~~	~~-7.28~~
					~~RV(A/B/C/D)S2~~	~~37.11~~	~~-6.93~~	~~13.60~~	~~4.25~~	~~-4.25~~
					~~RV(A/B/C/D)S3~~	~~40.39~~	~~2.11~~	~~11.52~~	~~11.52~~	~~-11.52~~
					~~RV(A/B/C/D)S4~~	~~37.11~~	~~-0.92~~	~~10.26~~	~~10.27~~	~~-10.27~~
	~~Rigid connection between IC03 and RV.~~									
~~3~~	~~SG (RCP) Lower Support~~	~~45.67~~	~~IC14~~	~~PartC~~	~~S(A/C)L1~~	~~45.64~~	~~-16.83~~	~~23.49~~	~~27.14~~	~~-27.14~~
					~~S(A/C)L2~~		~~-18.95~~	~~25.61~~	~~35.61~~	~~-35.61~~
				~~PartG~~	~~P(A/B/C/D)S4~~	~~42.61~~	~~-35.10~~	~~41.77~~	~~26.04~~	~~-26.04~~
	~~Rigid connection between IC14 and SG or RCP Lower Supports~~									
~~4~~	~~SG Intermediate Support~~	~~76.42~~	~~IC05~~	~~PartB~~	~~S(A/B/C/D)M1~~	~~75.42~~	~~-11.88~~	~~18.54~~	~~36.72~~	~~-36.72~~
					~~S(A/B/C/D)M2~~		~~-13.65~~	~~20.32~~	~~32.43~~	~~-32.43~~
	~~Rigid connection between IC05 and SG Intermediate Supports.~~									
~~5~~	~~SG Upper Support~~	~~97.00~~	~~IC61 & IC62~~	~~PartA~~	~~S(A/B/C/D)T1~~	~~96.58~~	~~-11.88~~	~~18.54~~	~~36.72~~	~~-36.72~~
					~~S(A/B/C/D)T2~~		~~-13.65~~	~~20.31~~	~~32.43~~	~~-32.43~~
	~~Rigid connection between IC61 and IC62 and SG Upper Supports.~~									
	~~SGs of Loop A and B connected to IC62, and SGs of Loop C and D connected to IC61~~									

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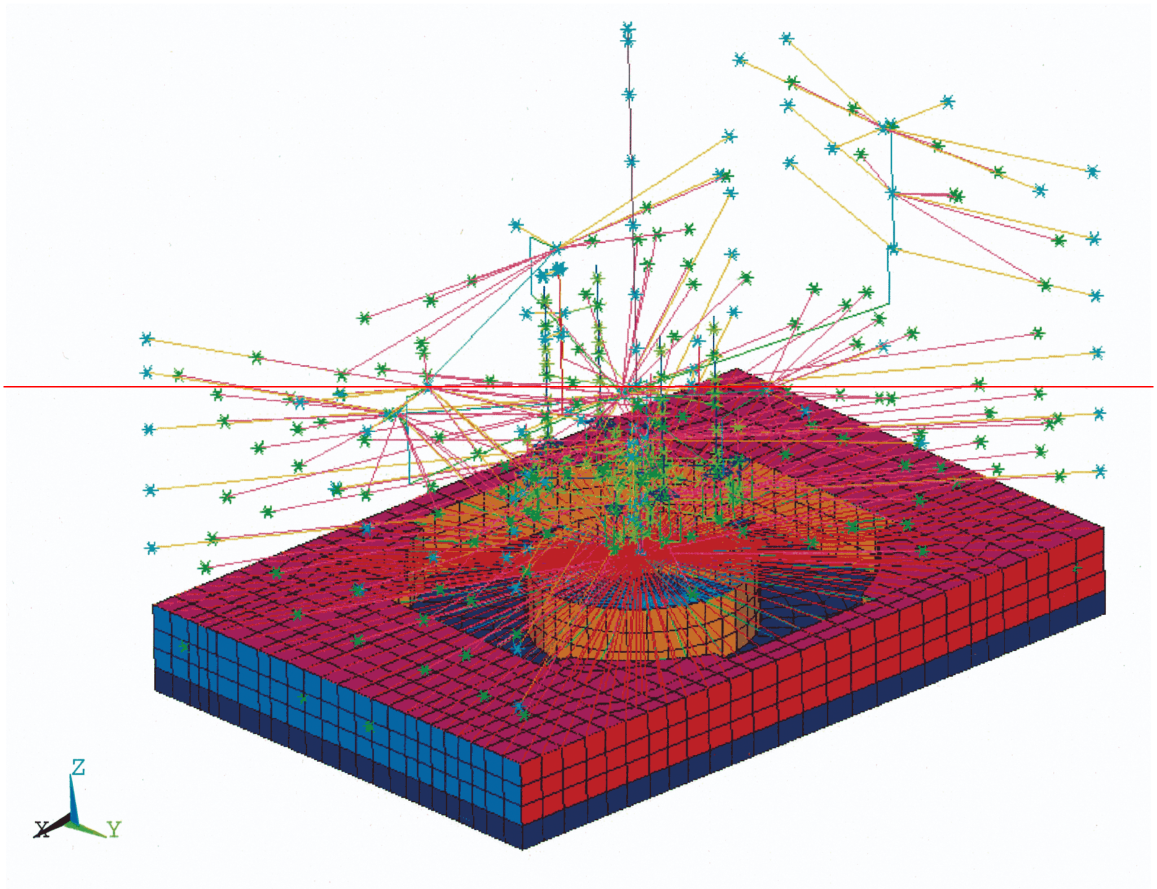

~~Figure 3H-1 Enhanced Lumped Mass-Stick Model of the R/B Complex~~

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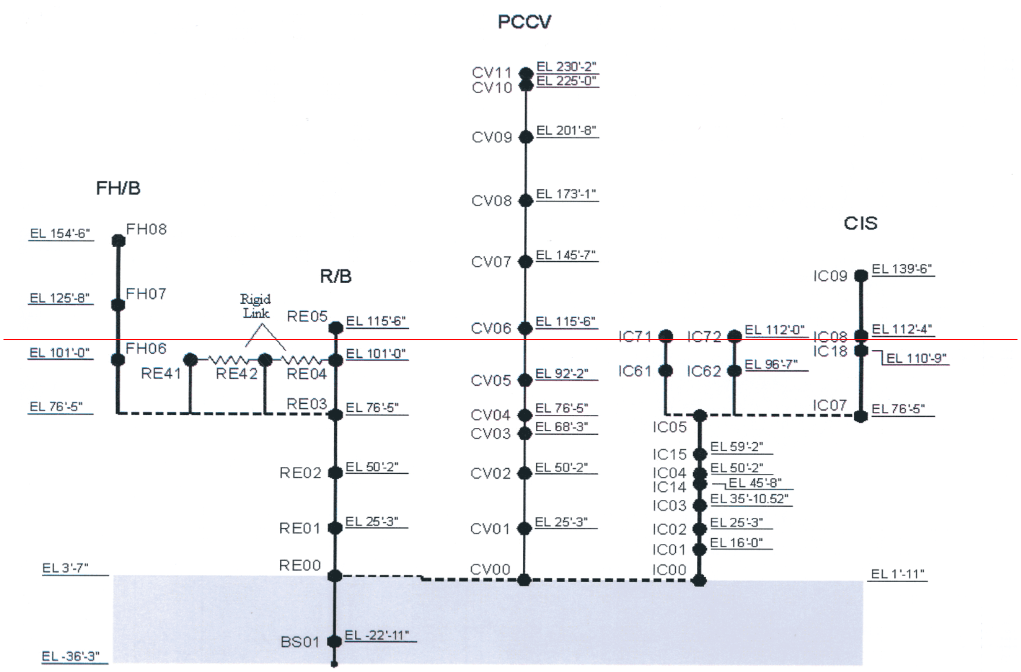

~~**Figure 3H-2 (Sheet 1 of 2) Lumped Mass Stick Model for Buildings (R/B, PCCV, Containment Internal Structure)**~~

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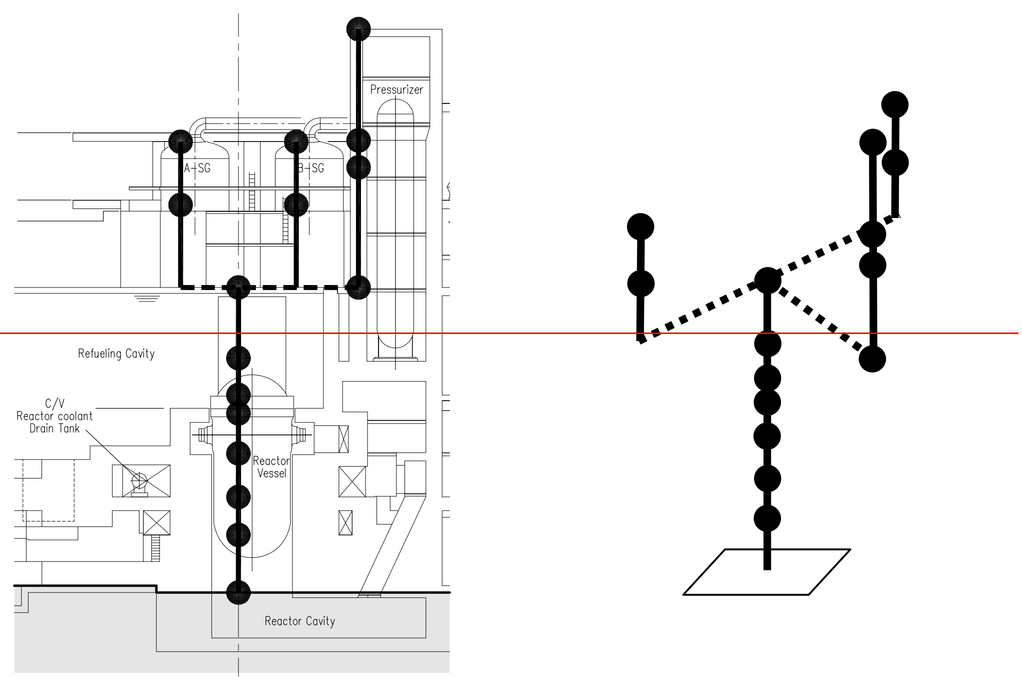

~~Note: Upper portion of this sheet shows the zoning above the operating floor at elevation 76' 5" and the lower portion shows conceptual stick model with respect to configuration of the containment internal structure~~

~~**Figure 3H-2 (Sheet 2 of 2) Lumped Mass Stick Model for Buildings (R/B, PCCV, Containment Internal Structure)**~~

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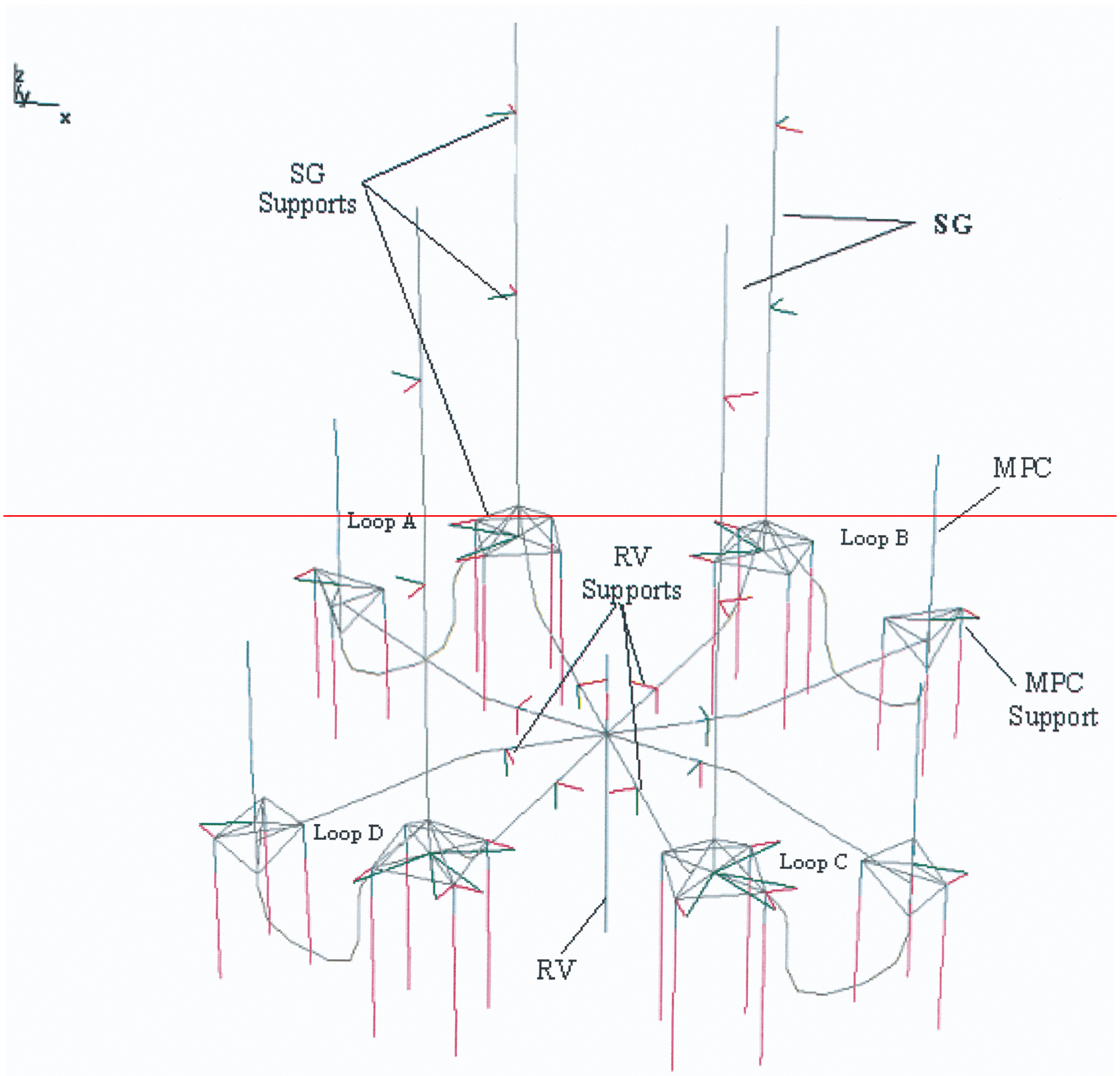

~~Figure 3H-3 Stick Mass Spring Model for RCL~~

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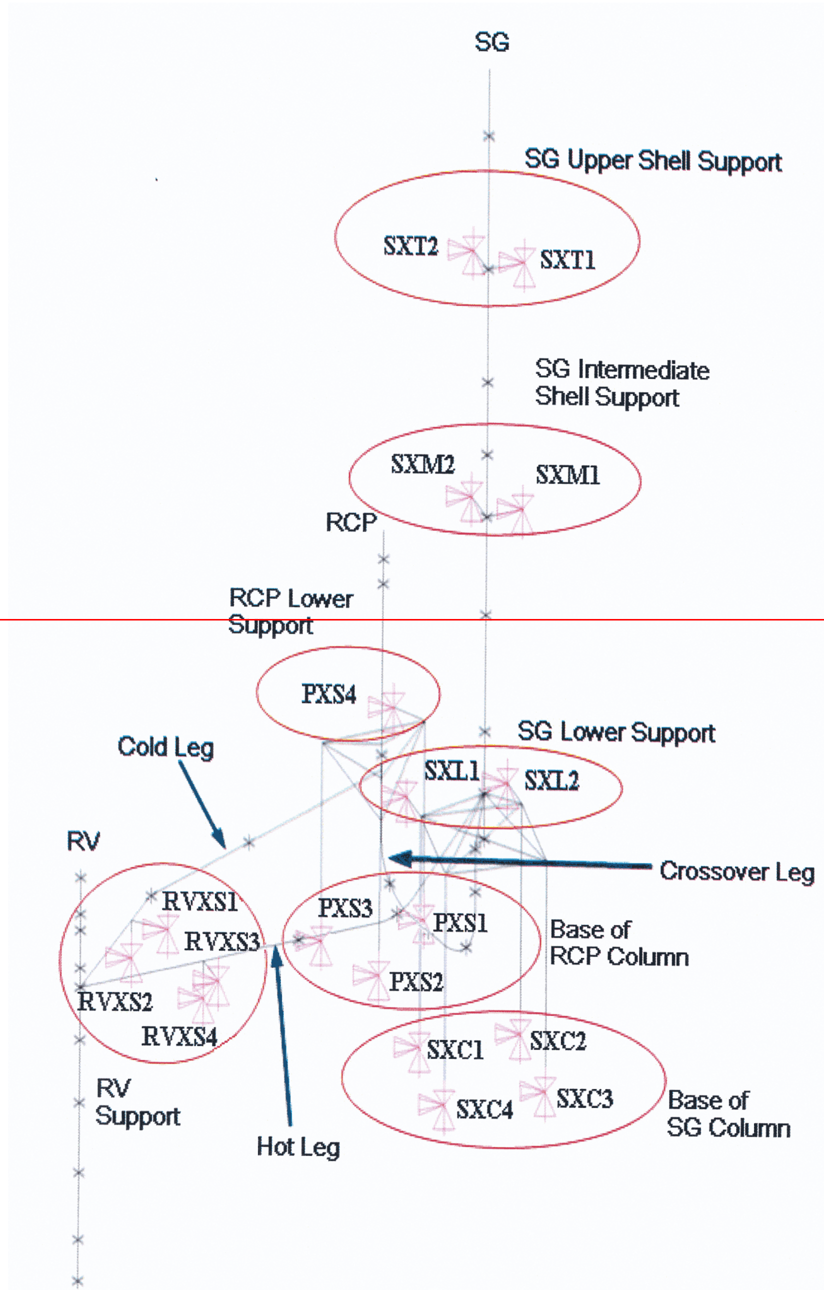

~~Note:~~

~~Connectivity of each support groups to building nodes are presented in Technical Report MUAP-11006. (Reference 3H-7)~~

~~**Figure 3H-4 Connectivity between RCL and Buildings**~~

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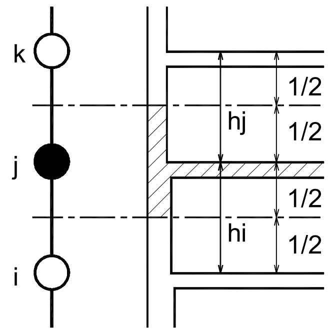

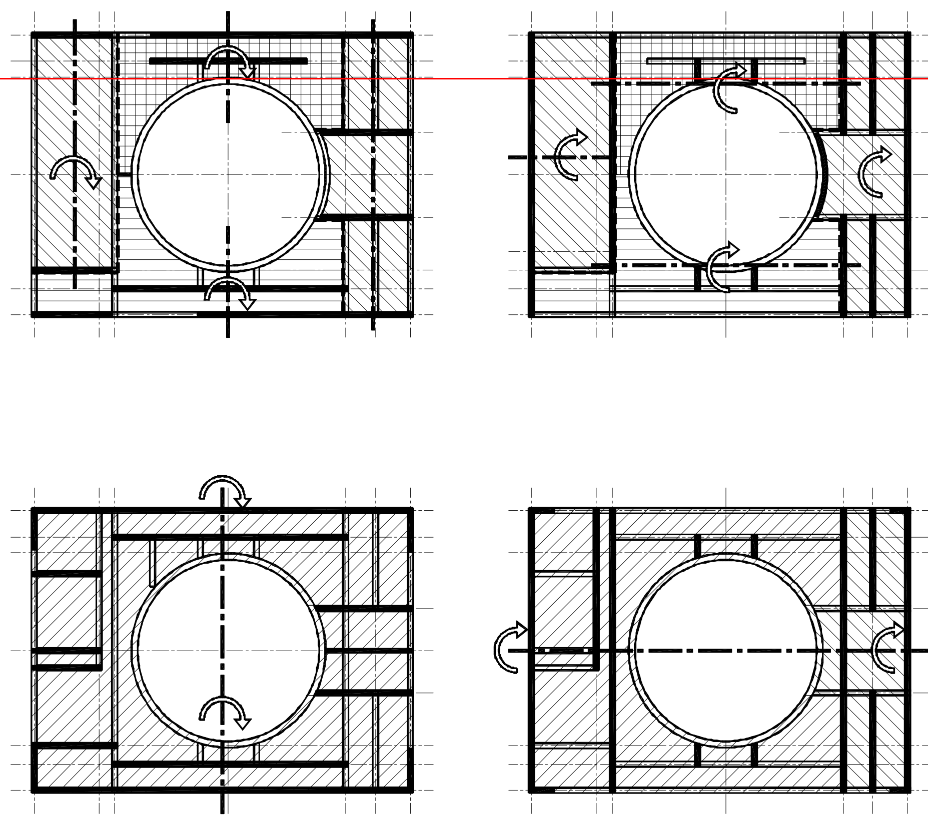

~~Figure 3H-5 Method to Compute Weight and Inertia Moment of Lumped Masses~~

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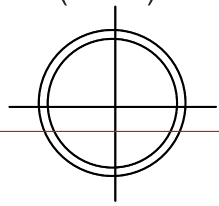
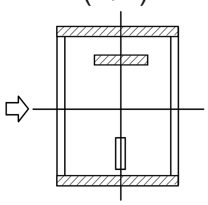
Configuration (Building)	Section Area	Geometrical Moment of Inertia	Axial Area
(PCCV) 	Half of the total area half the total	Calculated from the total area	Total area
(R/B) 	Area of the shear walls effective to the seismic force (shown in the figure)	Estimated for the shear walls effective to the seismic force (shown in the figure)	Total axial area of the shear walls and columns effective to the seismic force

~~**Figure 3H-6 Method to Compute Section Area; Inertia Moment and Axial Area by Hand Calculations**~~

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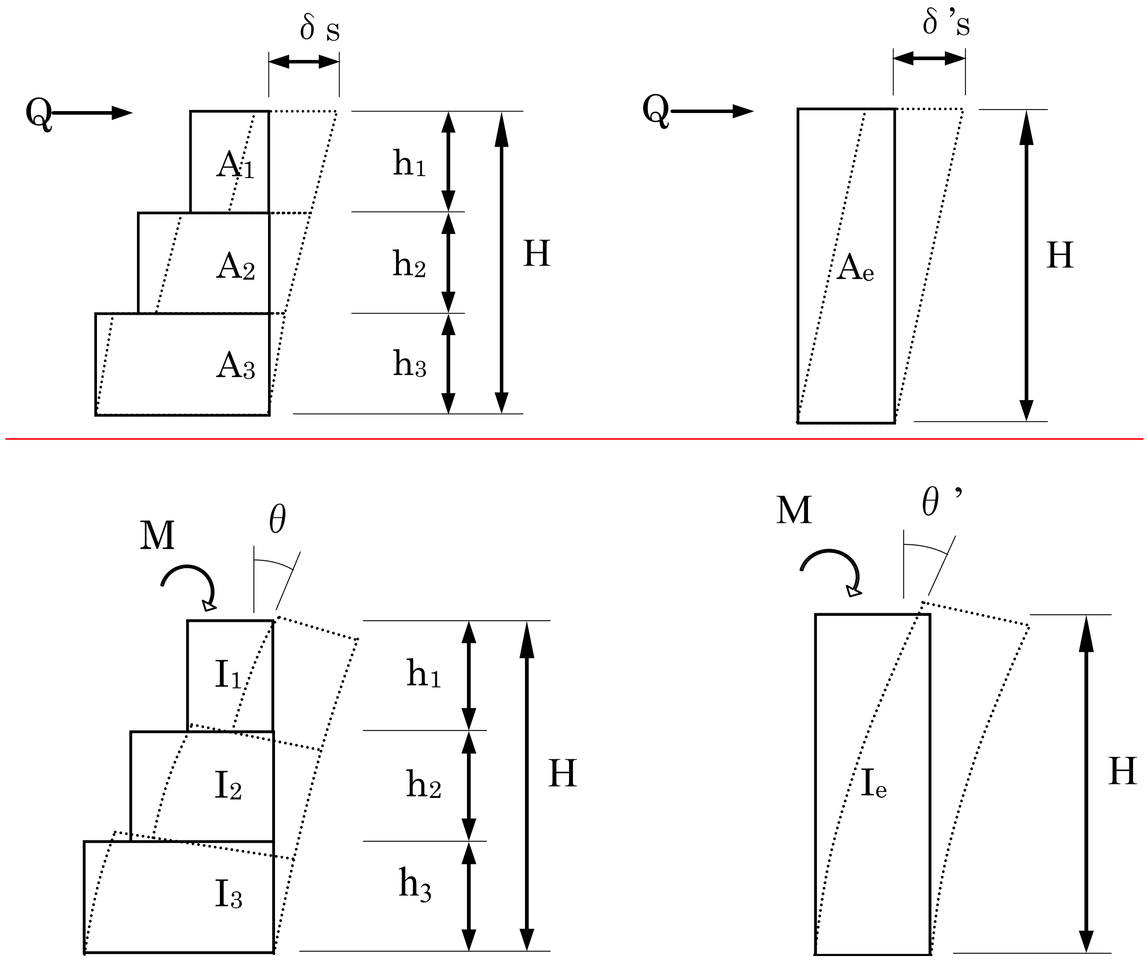

~~**Figure 3H-7 Method to Calculate Equivalent Shear Area**~~

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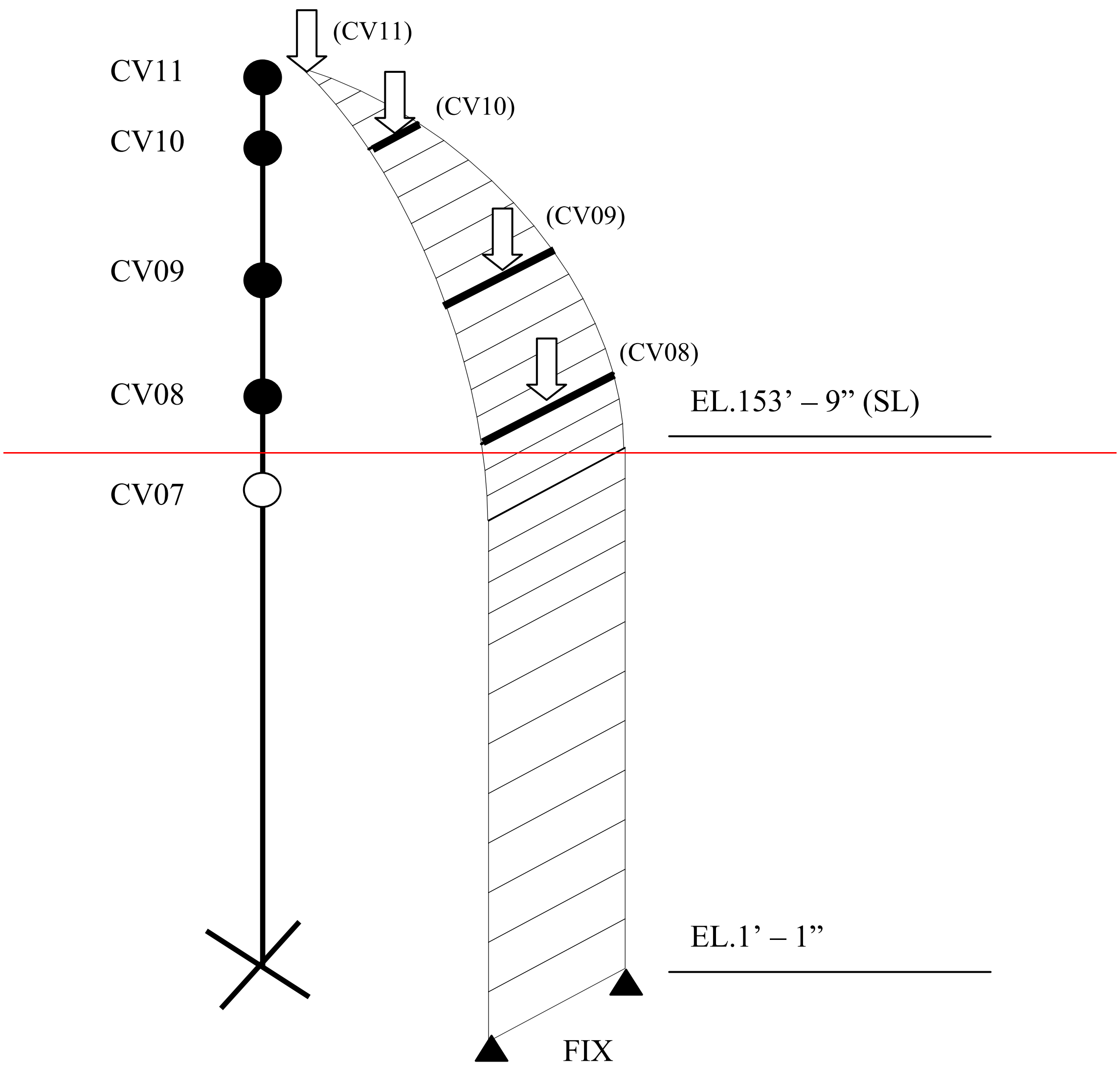

~~Figure 3H-8 Method to Evaluate Vertical Stiffness of PCCV Dome~~

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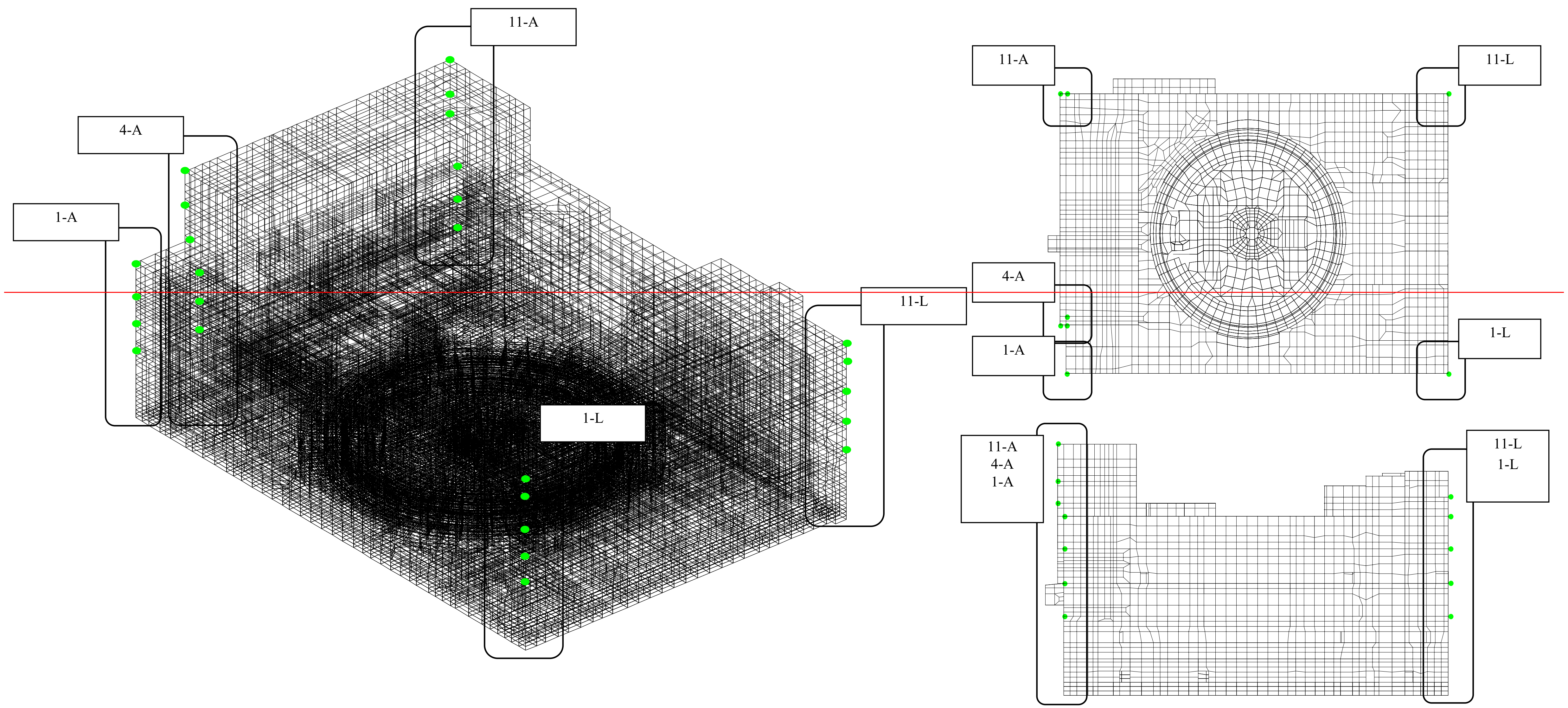

~~Figure 3H-9 Fixed-Base FE Model of R/B~~

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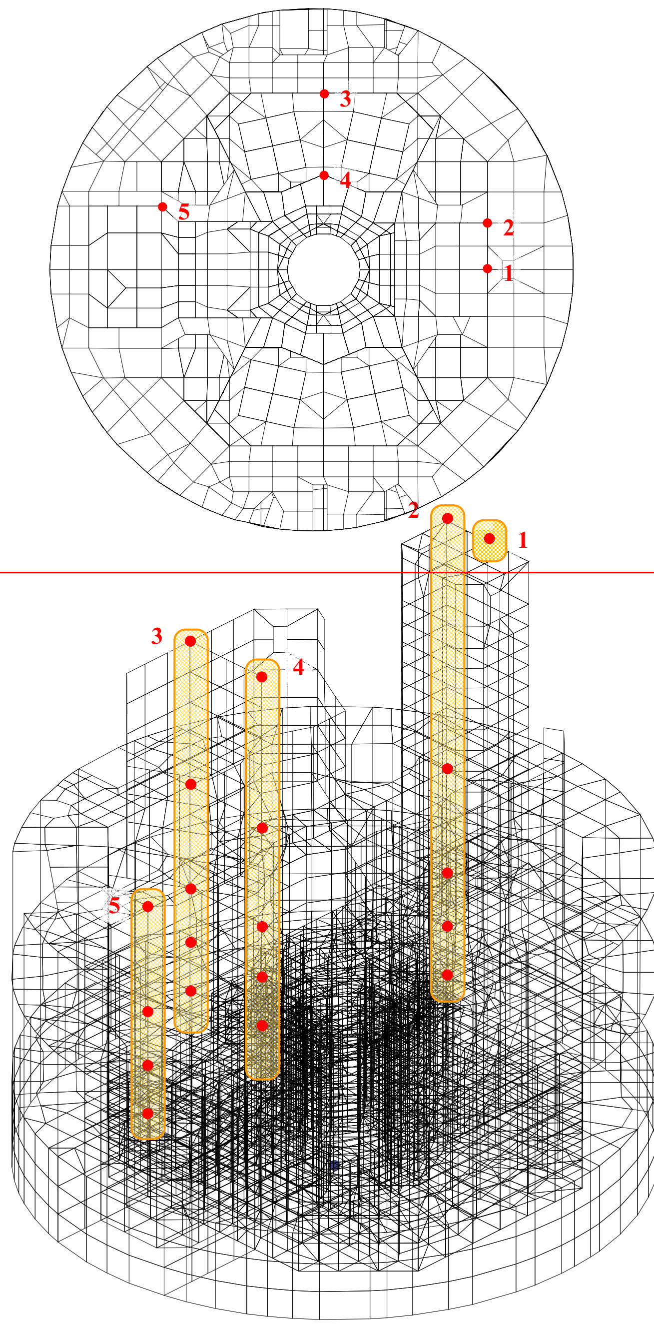

~~Figure 3H-10 Fixed-Base FE Model of Containment Internal Structure~~

CONTENTS

ACRONYMS AND ABBREVIATIONS

ISRS	in-structure response spectra

3I In-Structure Response Spectra

3I.1 Introduction

Refer to MUAP-10006, "Soil-Structure Interaction Analyses and Results for the US-APWR Standard Plant" (Reference 3I-~~3~~1) for the in-structure response spectra (ISRS) for various buildings and elevations of the US-APWR standard plant. MIC-03-03-00054

3I.2 References

3I-1 ~~Deleted.~~ MIC-03-03-00054

~~3I-2~~ ~~Deleted.~~

~~3I-3~~ Soil-Structure Interaction Analyses and Results for the US-APWR Standard Plant, MUAP-10006, Rev. ~~1~~3, Mitsubishi Heavy Industries, ~~January 2011~~November 2012.

ACRONYMS AND ABBREVIATIONS

R/B	reactor building
PS/B	power source building
ESWPC	essential service water pipe chase

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3J Reactor, Power Source, ~~and~~ Containment Internal and the Essential Service Water Pipe Chase Structural Design

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3J.1 Introduction

This appendix provides the structural drawings for the reactor building (R/B), containment internal structure, ~~and~~ the east and west power source buildings (PS/Bs) and the Essential Service Water Pipe Chase (ESWPC) for the US-APWR.

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Figure 3J-1 R/B Structural Drawings (Sheet 1 of 14)

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Figure 3J-1 R/B Structural Drawings (Sheet 2 of 14)

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Figure 3J-1 R/B Structural Drawings (Sheet 3 of 14)

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Figure 3J-1 R/B Structural Drawings (Sheet 4 of 14)

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Figure 3J-1 R/B Structural Drawings (Sheet 5 of 14)

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Figure 3J-1 R/B Structural Drawings (Sheet 6 of 14)

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Figure 3J-1 R/B Structural Drawings (Sheet 7 of 14)

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Figure 3J-1 R/B Structural Drawings (Sheet 8 of 14)

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Figure 3J-1 R/B Structural Drawings (Sheet 9 of 14)

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Figure 3J-1 R/B Structural Drawings (Sheet 10 of 14)

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Figure 3J-1 R/B Structural Drawings (Sheet 11 of 14)

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Figure 3J-1 R/B Structural Drawings (Sheet 12 of 14)

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Figure 3J-1 R/B Structural Drawings (Sheet 13 of 14)

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Figure 3J-1 R/B Structural Drawings (Sheet 14 of 14)

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Figure 3J-3 PS/B East Structural Drawings
(Sheet 1 of 3)

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Figure 3J-3 PS/B East Structural Drawings
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Figure 3J-3 PS/B East Structural Drawings
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Figure 3J-4 PS/B West Structural Drawings
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Figure 3J-4 PS/B West Structural Drawings
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Figure 3J-4 PS/B West Structural Drawings
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Security-Related Information – Withheld Under 10 CFR 2.390

Figure 3J-5 Essential Service Water Pipe Chase Structural Drawings (Sheet 1 of 3)

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Security-Related Information – Withheld Under 10 CFR 2.390

Figure 3J-5 Essential Service Water Pipe Chase Structural Drawings (Sheet 2 of 3)

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Security-Related Information – Withheld Under 10 CFR 2.390

Figure 3J-5 Essential Service Water Pipe Chase Structural Drawings (Sheet 3 of 3)