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U.S. Nuclear Regulatory Commission
Washington, DC20555-0001

Attention: Mr. Perry Buckberg

Docket No. 52-021
MHI Ref: UAP-HF-13240

Subject: Identification of Tier 2* Information for US-APWR Seismic Analyses – DCD Section 3.7 and Associated Technical Reports

With this letter, Mitsubishi Heavy Industries, Ltd. (MHI) is providing the identification of Tier 2* information within DCD Section 3.7. Enclosure 1 submits DCD Section 3.7 with the highlighted designation of Tier 2* in accordance with the Tier 2* Selection Criteria, submitted as Enclosure 2. In accordance with Appendix 1A of the DCD Revision 4, the highlighted text will be italicized and bracketed, with an asterisk after the closing bracket, when Section 3.7 Tier 2* information is incorporated into a future Update Tracking Report (UTR) to the Living DCD.

To assist the NRC in understanding MHI's basis for selection, right margin notations are provided which corresponds to selection criteria numbers identified on page 2 of Enclosure 2. In addition, Light blue highlighted text represents Tier 1 content that is therefore not applicable to Tier 2* designation.

MHI has also assessed the associated technical reports MUAP-10006, Revision 3, MUAP-11007, Revision 2, and MUAP-12002, Revision 1, and determined there is no Tier 2* information within these technical reports as explained in the following paragraphs.

MUAP-10006 Rev. 3 "Soil-Structure Interaction Analyses and Results for the US-APWR Standard Plant"

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This Technical Report presents the seismic Soil-Structure Interaction (SSI) and Structure-Soil-Structure Interaction (SSSI) analyses and results for the US-APWR Standard Plant. The report is divided in the following three parts:

Part 1: Describes the development of the design basis time histories and design basis soil profiles. These time histories are developed in accordance with SRP 3.7.1 for use in the standard plant analysis. COL applicants are required to develop site-specific ground motion time histories and durations of motion. Therefore, this material is tracked via a COL Item and is not considered Tier 2*. Part 1 also develops multiple generic soil profiles for use in the SSI and SSSI analyses. COL applicants are to provide site-specific soil data for use in site-specific seismic analyses which is also tracked via a COL Item. As such, this material is also not considered Tier 2*.

Part 2: Describes the development and validation of the design basis Dynamic FE model of the R/B complex. COL applicants are required to perform site-specific SSI analyses for the R/B complex to confirm that site-specific effects are enveloped by the standard design. As such, a site-dependent dynamic model would be developed to the same NRC regulations and guidance as the standard plant models for use in the analyses. Therefore, information in this part of the report is not considered Tier 2*.

Part 3: Presents the methodologies and results of the SSI and SSSI analyses. As stated before, COL applicants are required to perform site-specific SSI analyses for the R/B complex to confirm that site-specific effects are enveloped by the standard design. Again, all applicable NRC review plans and regulatory guides would apply to the analysis methodology and results processing. Therefore, the information presented in this part of the report is not considered Tier 2*.

MUAP-11007 Rev. 2 "Ground Water Effects on SSI"

This Technical Report presents a sensitivity study on the effects of groundwater level for the R/B complex seismic response. It also evaluates the significance of groundwater level for the MHI US-APWR standard seismic design. Since this report only presents a sensitivity study to confirm the inputs to the SSI analyses presented in MUAP-10006 Rev. 3, it was determined that there is no Tier 2* information present. As discussed for MUAP-10006 Rev. 3, the soil profiles presented are generic for the standard plant design and will be confirmed by the COL applicant via a COL Item in the DCD. It has been assessed that no Tier 2* designations are required in MUAP-11007 Rev. 2.

MUAP-12002 Rev. 1 "Sliding Evaluation and Results"

This Technical Report presents the methodology and results for the nonlinear sliding analyses of the R/B complex and T/B. The methodology presented is in lieu of a typical pseudo-static analysis to demonstrate a safety factor against sliding ≥ 1.1. DCD Section 3.8.5.5.2 states that unless the COL applicant can demonstrate by means of pseudo-static analysis that the factor of safety is met, they are to follow the methodology for determining the amount of sliding presented in MUAP-12002. Since the COL applicant is required to follow the methodology of 12002 via a COL Item, it is all ready tracked and therefore no information contained in MUAP-12002 is considered Tier 2*.

Please contact Mr. Joseph Tapia, General Manager of Licensing Department, Mitsubishi Nuclear Energy Systems, Inc. if the NRC has questions concerning any aspect of this letter. His contact information is provided below.

Sincerely,

Y. Ogata

Yoshiki Ogata,
Executive Vice President
Mitsubishi Nuclear Energy Systems, Inc.
On behalf of Mitsubishi Heavy Industries, LTD.

Enclosures:

1. DCD Section 3.7 Tier 2* Designations
2. Seismic Analyses and Structural Design Tier 2* Selection Criteria

CC: P. Buckberg
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<u>Contact Information</u>

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ENCLOSURE 1
DCD Section 3.7 Tier 2* Designations

INDEX:

Tier 2* identification is recommended

Tier 2* is not recommended, Tier 1 Material Content

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 define the magnitude of the design basis earthquake. Certified seismic design response spectra (CSDRS) 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 SSE design motion.

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).

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. The response spectra of site-specific SSEs are developed following the requirements of RG 1.208 (Reference 3.7-3). The COL Applicant is to develop site specific GMRS and FIRS. 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.

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 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.

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.

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 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 are the R/B, PCCV, containment internal structure, (CIS), east PS/B, west PS/B, and essential service water pipe chase (ESWPC) all on a common basemat. The common basemat also includes the seismic category II A/B. This combination of buildings on the common mat is defined as the R/B complex.

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 , a site-specific SSE can be used. The site-specific SSE is developed following the requirements of RG 1.208 (Reference 3.7-3).

CSDRS

The CSDRS are presented as the site-independent seismic design response spectra for an approved certified design of the US-APWR standard nuclear power plant. The CSDRS 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.

The site-independent CSDRS that are employed for the seismic category I design of the US-APWR standard plant are shown for 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. 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 where high-frequency exceedances of the CSDRS may occur at close distances (≤15 km) from larger magnitude (M ~ 5) earthquake sources. In these cases, the COL Applicant may consider the seismic wave transmission incoherence of the input ground motion when performing the site-specific SSI analyses.

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 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. 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).

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 FIRS 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 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.

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.

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.

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. Subsection 3.7.4 describes the criteria and the seismic instrumentation used to determine whether the OBE has been exceeded.

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 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). S.C. 7

Design Ground Motion Time History

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.

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 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.

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. S.C. 6

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 dynamic FE models used for frequency domain SSI analyses described in subsection 3.7.2.4 use linear hysteric damping to account for the dissipation of energy in the subgrade materials and structural members. The hysteretic damping is proportional to the displacements of the dynamic system and is independent of frequency. The shear wave and compression wave damping coefficients (β_S and β_P) define the hysteric damping in the flowing complex formulations of the material shear modulus G^* and constrain modulus M^*:

$$G^* = G \cdot \left(1 - 2 \cdot \beta_S + 2 \cdot i \cdot \beta_S \cdot \sqrt{1 - \beta_S^{\,2}}\right) \qquad M^* = M \cdot \left(1 - 2 \cdot \beta_P + 2 \cdot i \cdot \beta_P \cdot \sqrt{1 - \beta_P^{\,2}}\right)$$

where: G and M are linear elastic shear and constrain modulii of the material and

$i = \sqrt{-1}$ is the complex number.

The strain compatible damping values assigned to the subgrade materials in the six generic profiles presented in Subsection 3.7.1.2 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.

The same values of strain compatible shear wave damping are also used for 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.

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.

In the dynamic FE models used for frequency domain SSI analyses, the damping values listed in Table 3.7.2-3 are assigned both to the shear wave and compression wave damping coefficients representing the dissipation of energy in the different types of structural members. The damping values assigned to the structural model are consistent with the critical damping values specified in RG 1.61 (Reference 3.7-15) for elastic modal dynamic seismic analysis where energy dissipation is accounted for by frequency dependent viscous damping proportional to the velocity of the dynamic system. The implemented modeling approach results in the same amplitude of peak resonance responses for structures with viscous damping and hysteretic damping with less dissipation of energy occurring at other frequencies for the models with frequency independent hysteretic damping. S.C. 7

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.

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). CIS and PCCV stiffness and damping are based on loading conditions as described in Section 3.7.2.3.5.

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.

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.

The damping coefficient values in Table 3.7.2-3 are assigned to the models used for the response spectra analyses to quantify the dissipation energy associated with the two bounding levels of stiffness. Unlike in the frequency domain SSI analyses where different values can be assigned to different finite elements, in the response spectra analysis and modal superposition time history analysis only a single value of critical damping is used to represent the dissipation of energy in the whole dynamic system. S.C. 7

The damping values for response spectra and modal superposition time history analyses of systems that include two or more substructures, such as a concrete and steel composite structure, may also be obtained using the strain energy method. This is the same as the stiffness weighted composite modal damping method as provided in to SRP 3.7.2 (Reference 3.7-16). S.C. 7

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

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 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. S.C. 3

The minimum allowable static bearing capacity for the R/B complex is 15 ksf. The minimum allowable dynamic bearing capacity for the R/B complex is 35 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.

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.

The generic backfill properties used in the SSI and SSSI analyses for the standard plant are discussed in Subsection 3.7.2.4.

The site-specific SSI analyses will use 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. 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-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 damping of geological hard rock stratum. 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.

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 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).

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. The details of the seismic system analysis is provided in Technical Report MUAP-10006 (Reference 3.7-48).

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.

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 effects of structure-soil-structure interaction (SSSI) between the T/B and the R/B have been considered and shown to have a negligible effect on SSI response of the R/B complex and have been included in the development of the design ISRS. SSSI effects and how the effects were considered are discussed in more detail in Section 3.7.2.8.

The seismic responses of the major seismic category I and seismic category II structures are obtained from frequency domain time history analysis of seismic models considering a frequency-dependent SSI system. 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 dynamic analyses considered the torsional, rocking, and translational responses of the structures and their foundations. The effects are discussed in Sections 3.7.2.3.

The hydrodynamic effects considered are discussed in Section 3.7.2.3.

The standard design of R/B Complex superstructures is based on SSE load demands obtained from Response Spectra Analyses (RSA) of fixed base models of PCCV, CIS and R/B-A/B-PS/B's integrated shear wall structure. ISRS/ARS obtained from the results of site-independent SSI analyses serve as input for the RSA. Subsection 3.7.2.1 describes the methodology used for RSA. The methodology used for combination of modal responses is described in subsection 3.7.2.7. In Subsection 3.7.2.12, the responses obtained from the time history SSI analyses of R/B Complex dynamic model are compared with the responses obtained from RSA analyses of the major Category I structures to demonstrate the adequacy of the seismic design of the major structural members.

Subsection 3.7.3 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 addressed in the seismic system analyses, as explained further in Subsection 3.7.2.3.

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. S.C. 2

Seismic Response SSE Analysis Methodology

The seismic design of US-APWR standard plant is based on responses obtained from time history analyses performed using the SASSI computer program (Reference 3.7-17). The program uses the substructuring method to account for the interaction of the structure with the subgrade consisting of horizontally infinite layers overlaying a uniform half-space. For that purpose, the near field zone of the SSI system is partitioned in two substructures, the building superstructure and the basement minus the excavated soil. The dynamic properties are represented using the following complex frequency dependent stiffness matrix, $C(\omega)$, formulation:

$$C(\omega) = K - 2 \cdot i \cdot D \cdot K - \omega^2 M$$

where, ω is frequency of vibration $i = \sqrt{-1}$ is the complex number and K, M and D are stiffness, mass and linear hysteric damping matrices, respectively. The global complex stiffness matrix are assembled from the element complex stiffness matrices that are developed using FE technique.

The seismic response of the near field zone is obtained from the solution for the following complex matrix equation of motion in frequency domain:

$$\begin{bmatrix} C_{SS}(\omega) & C_{SI}(\omega) \\ C_{IS}(\omega) & \left(C_{II}(\omega) - C_{FF}(\omega) + X_{FF}(\omega)\right) \end{bmatrix} \cdot \begin{bmatrix} u_S(\omega) \\ u'_F(\omega) \end{bmatrix} = \begin{bmatrix} 0 \\ X_{FF}(\omega) \cdot u'_F(\omega) \end{bmatrix}$$

The subscripts S, I and F in the above matrix equations refer to the degrees of freedom associated with the building, basement and excavated soil. $u(\omega)$ are the vectors of complex nodal point displacements of the structure.

The vector $u'_F(\omega)$ of free field displacements at the interaction nodes is obtained from the solution of the site response problem. The impedance matrix $X_{FF}(\omega)$ representing the dynamic stiffness of the foundation at the interaction with the subgrade is calculated from the impedance analysis. Two methods are used for computation of the impedance matrix of embedded foundation SSI models:

a. The Direct Flexible Volume Method in which all the nodes of the excavated volume FE are specified as interaction nodes; and

b. The Modified Subtraction Method in which the solution is simplified by specifying the interaction nodes at the periphery of the excavated volume.

The seismic response analyses of R/B complex are performed on embedded models using the Modified Subtraction Method. The seismic response analysis for the Turbine Building (T/B) described in Subsection 3.7.2.8 is performed using the explicit Flexible Volume Method. The modified subtraction method is a simplified approach of representing the continuity between the free field zone and the near field zone. Based on comparisons with solutions obtained from the Flexible Volume Method, the results of the study demonstrate that the Modified Subtraction Method provides a realistic and reasonably accurate representation of the R/B Complex seismic response.

The structural analysis provides solution of the near field zone response by solving the equation of motion mentions above for selected frequencies of analysis. The solution is 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. Subsection 3.7.2.4 describes the development of within (inlayer) acceleration time histories used as input control motion in the SSI analyses of embedded foundations.

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.

The three components of the earthquake are applied to the SSI 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 the SSI analyses. The SSI analyses use within motions at the bottom of the R/B complex as control motions. Refer to Section 3.7.2.4 for details.

Cut-off Frequency of the Analyses

The cut-off frequency is the highest frequency used in the dynamic analysis of the soil structure system. It sets an upper limit on the number of frequencies to be analyzed, and controls the maximum allowable element size. 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 SSI 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

Table 3.7.2-4 presents a summary of the fixed base dynamic properties of R/B Complex Category I structures that are obtained from the ANSYS modal analysis. These are correlated with ACS SASSI analysis of R/B Complex Dynamic FE Model used for the site-independent SSI analyses presented in subsection 3.7.2. The natural frequencies, periods and effective masses of the dominant fixed base modes of vibration are provided for the R/B, the PCCV, the CIS and the east and west PS/Bs. Part 2 of Technical Report MUAP-10006 (Reference 3.7-48) provides comparisons and plots of the dominant mode shapes for each of the R/B Complex structures.

The seismic design of the US-APWR standard plant is based on responses obtained from a set of twelve (12) SSI analyses performed for the six generic site profiles of dynamic soil/rock properties presented in subsection 3.7.1.3 and the two levels of structural stiffness and damping properties (i.e. cracked and uncracked conditions) are described in subsection 3.7.2.3.

Amplitudes of the acceleration transfer functions are calculated from each SSI analysis for the response of the R/B Complex at the center of containment at the bottom of the foundation. The SSI responses for the generic site conditions are identified by comparing these transfer functions peak frequencies as described in Section 03.4.1.1 of Technical Report MUAP-10006 (Reference 3.7-48). These comparisons show that the site-independent SSI analyses provide a range of SSI responses of the R/B complex that envelope the possible responses of the building at a number of candidate sites.

The analyses of the generic soil profiles 270-200, 270-500, 560-500 provide seismic responses that are dominated by SSI effects and the dynamic characteristics of the subgrade. The responses obtained from the SSI analyses of these generic soil profiles define the standard design ISRS up to a frequency of approximately 5 Hz. The seismic responses obtained from the analyses performed for the rock profiles 900-200, 900-100 and 2032-100 are dominated by the dynamic properties of the structures. The responses for these generic rock sites define the design basis ISRS at higher frequencies.

Table 03.4.3-1 through Table 03.4.3-6 of Technical Report MUAP-10006 (Reference 3.7-48) provide weighted average floor accelerations for the R/B Complex structures that are calculated from the results of the site-independent SSI analyses. These weighted average accelerations are the envelope of the results obtained from the SSI analyses for the six generic site profiles of the model with full (uncracked concrete) stiffness properties and the model with reduced (cracked concrete) stiffness properties. Figure 03.4.3-1 through Figure 03.4.3-12 of Technical Report MUAP-10006 (Reference 3.7-48) show shear force diagrams calculated using the weighted average floor accelerations in the two horizontal directions. Table 03.4.3-7 of Technical Report MUAP-10006 (Reference 3.7-48) provides a comparison of the base reaction results calculated from the twelve different site-independent SSI analyses. The comparison shows that maximum base shears are from the SSI analysis of full stiffness model for hard rock site 2032-100. The maximum vertical base reaction is from the SSI analyses of reduced stiffness model for 900-200 generic rock profile.

The site independent SSI analyses of R/B Complex also provide results for the maximum displacements relative to the free field motion and the building basemat. These maximum relative displacements are calculated by following the methodology described in Section 03.3.8 of Technical Report MUAP-10006 (Reference 3.7-48). The maximum displacements due to three directions of the input motion are combined using the SRSS method. Tables 03.4.4-1 and 03.4.4-2 of Technical Report MUAP-10006 (Reference 3.7-48) present the envelope of the results from the site-independent SSI analyses for maximum displacements for different locations at the PCCV - R/B and PCCV - CIS interfaces. The adequacy of the 4 inch gaps between the PCCV - R/B and PCCV - CIS are evaluated based on the largest coupled seismic displacement conservatively calculated. The maximum relative seismic displacements of 3.2 and 1.6 inches are obtained for the gaps at the R/B-PCCV and PCCV-CIS interfaces, respectively. This results in clearances of 0.8 and 2.4 inches respectively. Therefore the gaps of 4 inches are adequate.

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 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 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.

The R/B complex Dynamic FE model consists of beam, shell, solid, and spring elements. S.C. 7 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) 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 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.

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)

3.7.2.3.2 R/B ComplexDynamic Finite Element 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

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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.

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.

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 Not Used

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 $R_m < 0.01$, decoupling can be done for any R_f
- If $0.01 \leq R_m$ and ≤ 0.1, decoupling can be done if $0.8 \geq R_f \geq 1.25$
- If $R_m > 0.1$, 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 integrating the RCL seismic subsystem with the R/B complex dynamic FE model discussed in Technical Report MUAP-10006 (Reference 3.7-48). 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.

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.

3.7.2.3.5 Section and Material Properties

The values of the modulus of elasticity and Poisson's ratio (ν) for concrete and steel used in the dynamic 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 3.7.2-2 and are computed as follows: S.C. 1, 7

$$E_c \text{ (psi)} = 57{,}000 \sqrt{f'_c}$$

$$G_c \text{ (psi)} = E_c / 2 (1 + \nu_c)$$

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: S.C. 1, 7

E_s = 29,000 ksi and ν_s = 0.3

Effects of Concrete Cracking on Reinforced Concrete Structures

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: S.C. 7

1. Full stiffness representing low stress levels corresponding to uncracked concrete properties where the stiffness of the members are represented 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.

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. S.C. 7

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 considered during seismic analysis. S.C. 7 Additional parametric studies were performed considering a variety of upper bound and lower bound CIS member stiffness values. These studies demonstrated that the dynamic response for the range of probable CIS stiffness is enveloped by the dynamic response of the CIS considering the stiffness values of Table 3.8.3-4. The CIS members are classified in six categories, two stiffness levels corresponding to: S.C. 7

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. S.C. 7

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: S.C. 7

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

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 Modeling of Mass

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.

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 rigidly attached to walls and slabs. The sloshing effects are not considered in the model since the effects are negligible.

3.7.2.3.7 Adjustment of Stiffness and Mass Properties

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.

Finite Element analyses 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 (y_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).

3.7.2.3.8 Stiffness of Steel Reinforced Concrete Beams and Columns

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.

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

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

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:

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

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. The effect of concrete cracking on composite members is a 50% reduction in stiffness as described in Section 3.7.2.3 and shown in Table 3.7.2-3.

3.7.2.3.9 Dynamic Properties of R/B Slabs and SC Modules

3.7.2.3.9.1 Dynamic Properties of R/B Slabs

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

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.

3.7.2.3.10 Validation of the Seismic Models

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.

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.

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

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.

3.7.2.4 Soil-Structure Interaction

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. Subsection 3.7.2.2 presents the results and describes the SSI responses captured by the site-independent SSI analyses.

The SSI 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.

A total of twelve cases combining two stiffness levels and six soil profiles are performed in the SSI analysis that as described in subsection 3.7.2.2, envelope the seismic responses of the US-APWR standard plant structures at wide range of candidate sites.

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.

The site-independent SSI analyses are performed on embedded models with near field soil solid elements connecting the FE model of the building basement with the free field zone. These near field solid elements represent the dynamic properties of the soil backfilled around the building basement after the construction of the plant. Table 03.3.1-10 through Table 03.3.1-15 of MUAP-10006 (Reference 3.7-48) present the properties assigned to the near field soil elements representative of strain compatible properties of typical granular backfill materials. In order to cover a wide range of soil-structure frequencies, a backfill with relatively soft properties is used in conjunction with the generic soil sites, 270-200, 270-500 and 560-500. A backfill with relatively stiff properties is used in conjunction with the generic rock sites, 900-100, 900-200, and 2032-100. Additional detail regarding the soil profiles is provided in MUAP-10006 (Reference 3.7-48).

The first six inches of natural soil located beneath the R/B and the T/B is required to satisfy the requirements for subgrade materials to achieve a kinetic friction coefficient of 0.5 or higher. If the kinetic friction coefficient is less than 0.5 then the natural soil shall be replaced by compacted granular backfill to provide a kinetic friction coefficient of 0.5 or higher.

3.7.2.4.2 Input Control Ground 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.

CSDRS and the CSDRS compatible time histories define the standard design ground motion as a free field outcrop motion at the bottom elevation of the R/B complex foundation basemat. A set of linear elastic site response analyses are performed on the generic strain compatible shear wave profiles presented in Figure 3.7.1-14 and compression wave profiles presented in Figure 3.7.1-15 to convert the outcrop motion time histories to within (inlayer) acceleration time histories used in the SSI analyses. Section 03.3.2 of MUAP-10006 (Reference 3.7-48) provides details about these site response analyses and the resulting within motions.

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)

See MUAP-10006 (Reference 3.7-48) for discussion about the SSSI analysis. The results also indicate that the presence of the R/B complex produces a noticeable effect on the seismic response of the T/B. SSSI effects tend to increase the seismic design forces in the NS direction, result in seismic design forces that are generally the same or slightly higher in the EW direction, and increase the seismic design forces in the vertical direction. See MUAP-11002 (Reference 3.7-61) for additional discussion of the SSSI analysis. Seismic Load demands for the structural design of the Seismic Category II T/B envelope both the SSI and the SSSI analyses results.

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.

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 as described in Subsection 3.7.2.8.

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 analyses are used for development of the following seismic basis parameters for the structural design of the R/B complex:

- The building analysis and design are conducted in accordance with the procedures and guidelines provided in Subsection 3.8.4.4.
- 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 relative displacements that are used as input for evaluation of the adequacy of the gaps between the CIS and PCCV, PCCV and R/B.
- The R/B complex SSI analyses also provide artificial time histories of the seismict 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).

The SSI 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 and Site-Specific Structures

The COL Applicant referencing the US-APWR standard design is required to perform a site-specific SSI analysis for the R/B complex utilizing a computer 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.

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. The site-specific SSI analysis is performed for buildings and structures including, but not limited to, to the following:

- 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.

The site-specific seismic response analysis of the R/B complex and the site-specific Category I structures 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
- Basemat embedment
- Flexibility of the basemat
- Presence of nearby structures

Appropriate modeling techniques capable of capturing the various site-specific SSI effects are used for the analysis. The dynamic properties are assigned to the models for site-specific Category I structures corresponding to the level of stresses generated.

The input used for the site-specific analysis must be derived from geotechnical and seismological investigations of the site. The input control motion derived from the site-specific FIRS is applied in the SASSI analyses at the bottom-of-foundation control point. 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, respectively) soil and rock properties. If sufficient and adequate soil investigation data are available, the LB and UB 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:

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

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.

To assure the proper comparability, the site-specific verification SSI analyses must use the same verified and validated models of the R/B complex as those used for the US-APWR standard plant design.

The 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-independent analysis. The plots, tables, and digitized data are then documented for review and comparison with the corresponding results from site-specific analyses. The COL Applicant is to verify that the results of the site-specific SSI analysis for the broadened ISRS 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.

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

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:

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

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

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

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 enveloped. In order to incorporate the effects of SSSI with the adjacent T/B in the R/B Complex design ISRS, the results obtained from the site-independent SSI of R/B Complex FE model are enveloped with the SSSI analyses of the combined model of R/B Complex and T/B presented in subsection 3.7.2.8. 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.

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. 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.

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 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. S.C. 7

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 element considering sign reversals, such as the relevant internal forces or stresses in the element.

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, it may not be practical to 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 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).

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. S.C. 7

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:

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

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. S.C. 7

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 $\sum$ 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).

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 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. S.C. 6

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. S.C. 6

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.

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 may be designed for impact loads due to postulated failure of the non-seismic category I SSCs.

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 designed as a NS structure on reinforced concrete foundation located approximately 16 inches from the west side of the A/B (seismic category II). 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, the A/B would not displace enough to impact the R/B or PS/Bs.

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.

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).
- The design of the T/B is based on static and dynamic analyses utilizing three dimensional FE models.
- Although the T/B is a seismic Category II structure, the T/B is designed and analyzed as a seismic category I structure. This is described in MUAP-11002 (Reference 3.7-61).

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, the criteria in Sections 5.1 and 5.2 of (Reference 3.7-19). However, the A/B is designed as seismic category II. The seismic, severe 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 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).
- The A/B is designed as a seismic category I structure and analyzed as part of the R/B complex.

3.7.2.8.4 R/B and PCCV

The R/B and PCCV are seismic category I structures 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 gap at all above-basemat locations. The 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 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.

3.7.2.9 Effects of Parameter Variations on Floor Response Spectra

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.

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

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.

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 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 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.
- 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.

3.7.2.12 Comparison of Responses

The R/B complex is analyzed using time history analysis methods.

As described in Subsection 3.7.1.1, the time history analyses are based on design ground motion time histories which have been 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 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.

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

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.

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 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 of the supported component is used. Supported 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 .
- 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 may be 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. 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.

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:

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

where

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

d_j = maximum value of d_{ij}

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. S.C. 7

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 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.

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 standard plant design.

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/hurricane missile protecting concrete vault or wall, in order to confine the emergency gas turbine fuel supply.

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.*

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.*

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:*

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

COL 3.7(4) *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.*

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.*

COL 3.7(6) *The COL Applicant is to develop site-specific GMRS and FIRS. 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.*

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.*

COL 3.7(8) *The COL Applicant is to evaluate the strain-dependent variation of the material dynamic properties for site materials.*

COL 3.7(9) *The COL Applicant is to assure that the design or location of any site-specific 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.*

COL 3.7(10) *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.*

COL 3.7(11) *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.*

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/hurricane missile protecting concrete vault or wall, in order to confine the emergency gas turbine fuel supply.*

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.*

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.*

COL 3.7(15) *Deleted*

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.*

COL 3.7(17) *Deleted*

COL 3.7(18) *Deleted*

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.*

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.*

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.*

COL 3.7(22) *The COL Applicant may consider the seismic wave transmission incoherence of the input ground motion when performing the site-specific SSI analyses.*

COL 3.7(23) *The COL Applicant is to verify that the results of the site-specific SSI analysis for the broadened ISRS are enveloped by the US-APWR standard design.*

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.*

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, utilizing 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.*

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. The site-specific SSI analysis is performed for buildings and structures including, but not limited to, to the following:*

- *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.*

COL 3.7(28) *Deleted.*

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.*

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)
2% Damping	
A (50)	0.3
B (12)	1.06
C (2.5)	1.28
D (0.25)	0.17
E (0.1)	0.028
3% Damping	
A (50)	0.3
B (12)	0.92
C (2.5)	1.10
D (0.25)	0.154
E (0.1)	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

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

Control Point (Hz)	Acceleration (g)
2% Damping	
A (50)	0.3
B (12)	1.06
C (3.5)	1.22
D (0.25)	0.12
E (0.1)	0.018
3% Damping	
A (50)	0.3
B (12)	0.92
C (3.5)	1.05
D (0.25)	0.106
E (0.1)	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 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 ≤ AD/V^2 ≤ 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 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**

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	500
900 m/s	100 200
2,032 m/s	100

* 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.

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 SSI Model (1)	Time History Analysis in Frequency Domain using sub-structuring technique	ACS SASSI	SRSS	N/A
Three-dimensional R/B complex FE Model (2)	1g Static Analysis & Time History Analysis in Time Domain	ANSYS	N/A(2)	N/A
Three-dimensional T/B SSI Model(3)	Time History Analysis in Frequency Domain using sub-structuring technique	ACS SASSI	SRSS	N/A
Three-dimensional T/B FE models(3)	1g Static Analysis & Time History Analysis in Time Domain	ANSYS	N/A	N/A

Notes:

1. The three-dimensional R/B complex SSI model is addressed in Technical Report MUAP-10006 (Reference 3.7-48).
2. The FE models for the R/B complex is used 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.
3. The three-dimensional T/B model is addressed in Technical Report MUAP-11002 (Reference 3.7-61).

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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 ν	Remark
PCCV	4,769	2,040	0.17	f'_c = 7,000 psi
R/B	4,031	1,723	0.17	f'_c = 5,000 psi
Containment Internal Structure	3,605	1,540	0.17	f'_c = 4,000 psi

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%

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(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 3.7.2-4 Fixed Base Dynamic Properties of US-APWR Category I Structures (Sheet 1 of 2)

Structure	Direction	Fixed Base Modal Properties: Frequency (Hz)	Fixed Base Modal Properties: Effective Mass (kip sec^2/ft)
PCCV	NS	4.3	2,042
		12.6	233.4
	EW	4.2	1,535
		12.4	423.9
	Vertical	12.1	2,797
		20.4	258.7
CIS	NS	7.6	80.37
		8.3	570.8
		12.0	338.6
		20.0	132.5
	EW	6.3	1,344
		12.4	423.9
		24.8	171.1
	Vertical	12.1	2,796
		15.4	187.0
		21.0	144.3
R/B	NS	4.9	2,996
		6.0	600.1
		9.5	661.7
		10.9	1,478
		13.8	718.6
	EW	6.1	4,070
		7.5	176.5
		10.4	718.6
		15.6	135.6
	Vertical	11.3	445.0
		12.1	2,797
		13.1	878.1
		16.5	134.7
		18.9	147.0

Table 3.7.2-4 Fixed Base Dynamic Properties of US-APWR Category I Structures (Sheet 2 of 2)

Structure	Direction	Fixed Base Modal Properties	
		Frequency (Hz)	Effective Mass (kip sec^2/ft)
East PS/B	NS	6.4	3,636
		10.9	1,478
		13.2	174.2
	EW	7.1	1,549
		14.3	460.7
		15.6	135.6
	Vertical	12.7	3,645
		16.5	134.7
		21.0	144.3
West PS/B	NS	8.8	302.7
		10.9	1,477
		18.7	112.4
	EW	7.1	1,549
		13.2	128.9
	Vertical	12.7	3,654
		15.1	123.7
		20.6	153.9

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

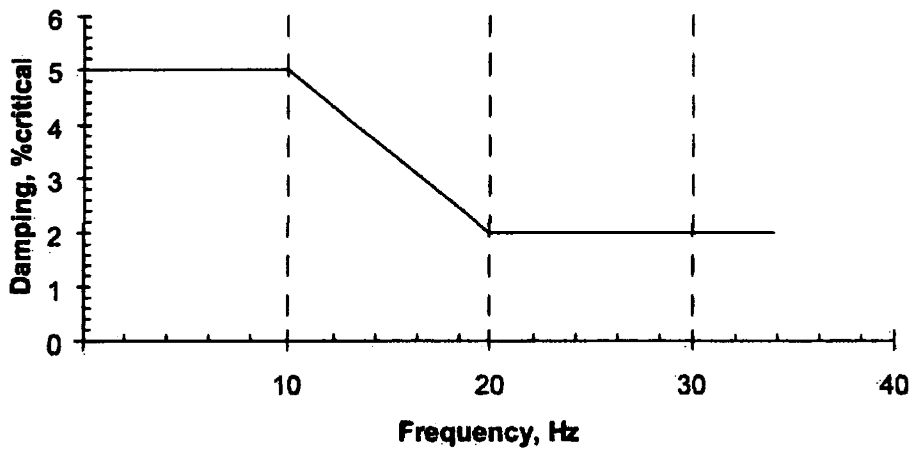
Notes for Tables 3.7.3-1(a) and 3.7.3-1(b):

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

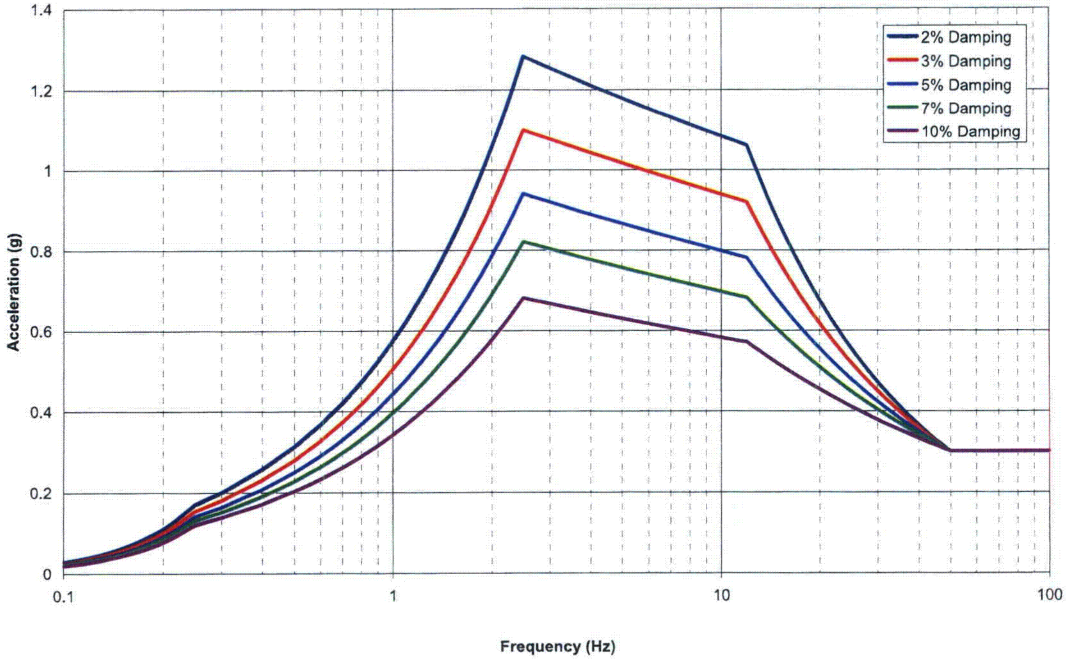


Figure 3.7.1-1 US–APWR Horizontal CSDRS

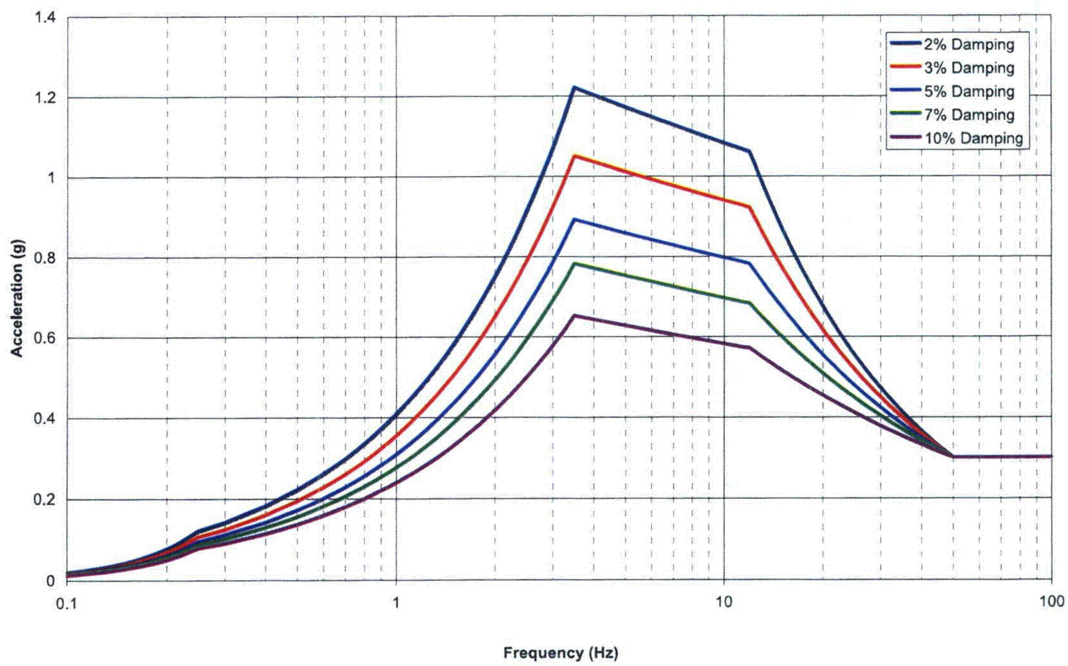


Figure 3.7.1-2 US–APWR Vertical CSDRS

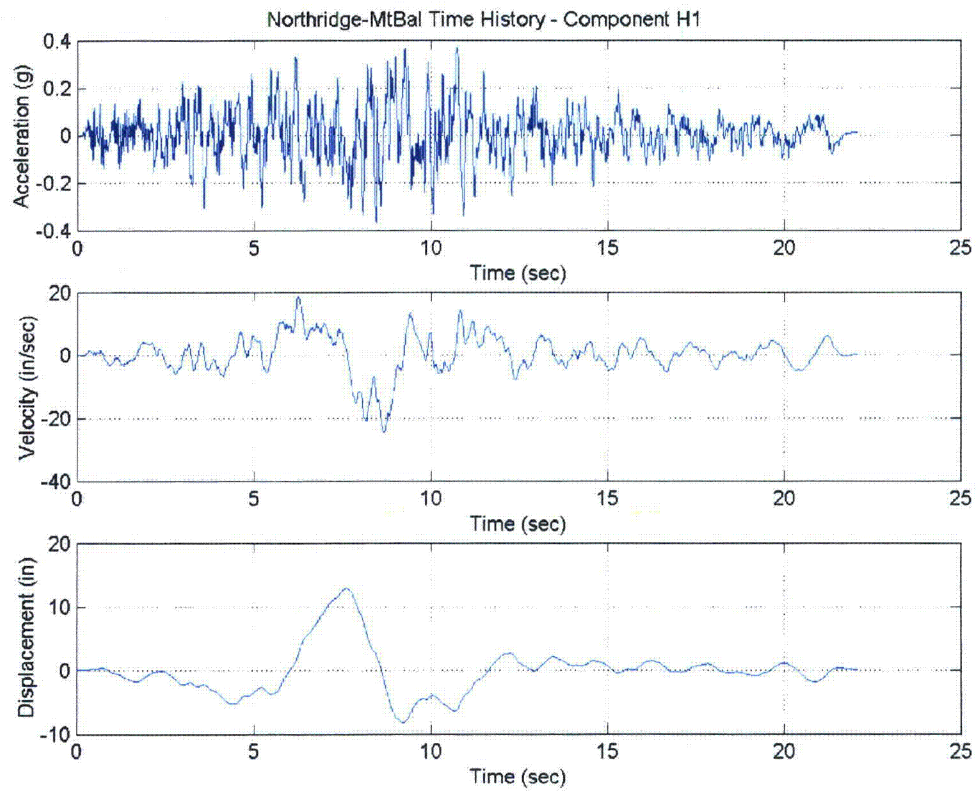


Figure 3.7.1-3 Acceleration, Velocity, and Displacement Time History for Component H1 [NS]

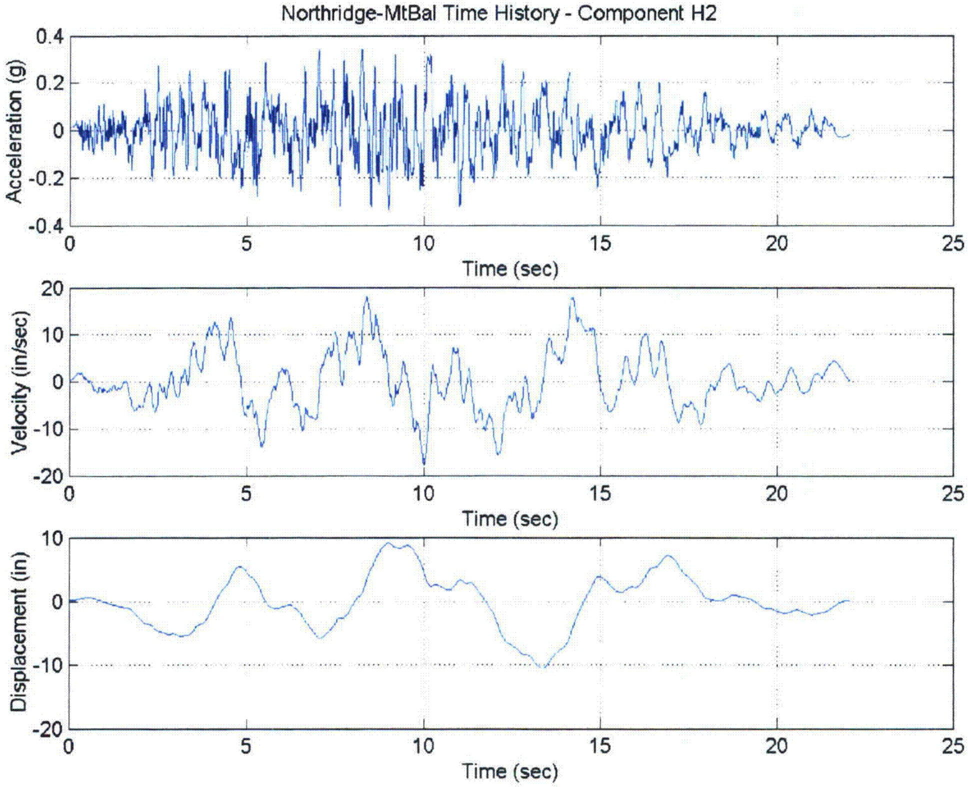


Figure 3.7.1-4 Acceleration, Velocity, and Displacement Time History for Component H2 [EW]

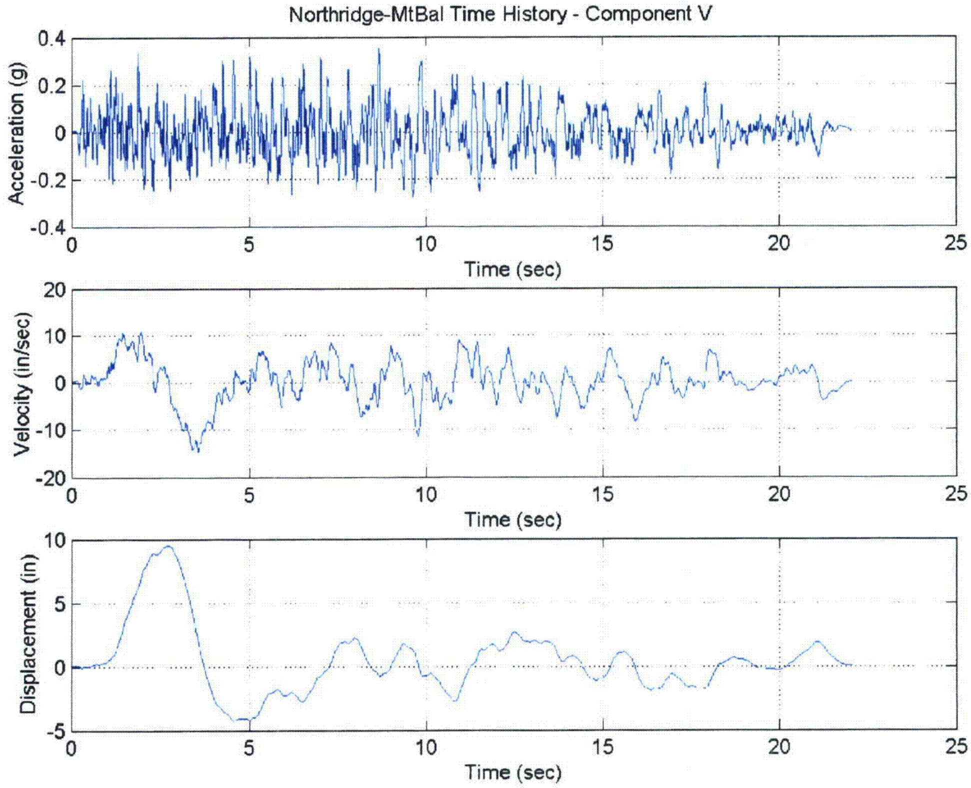


Figure 3.7.1-5 Acceleration, Velocity, and Displacement Time History for Component V

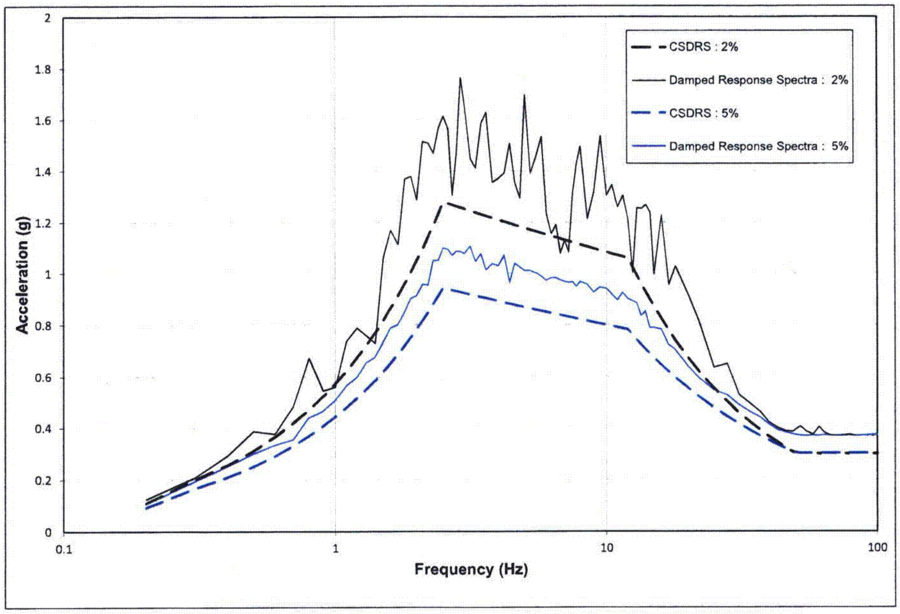


Figure 3.7.1-6a Damped Response Spectra Plots for Northridge Mount Baldy Component H1 (180) [NS]

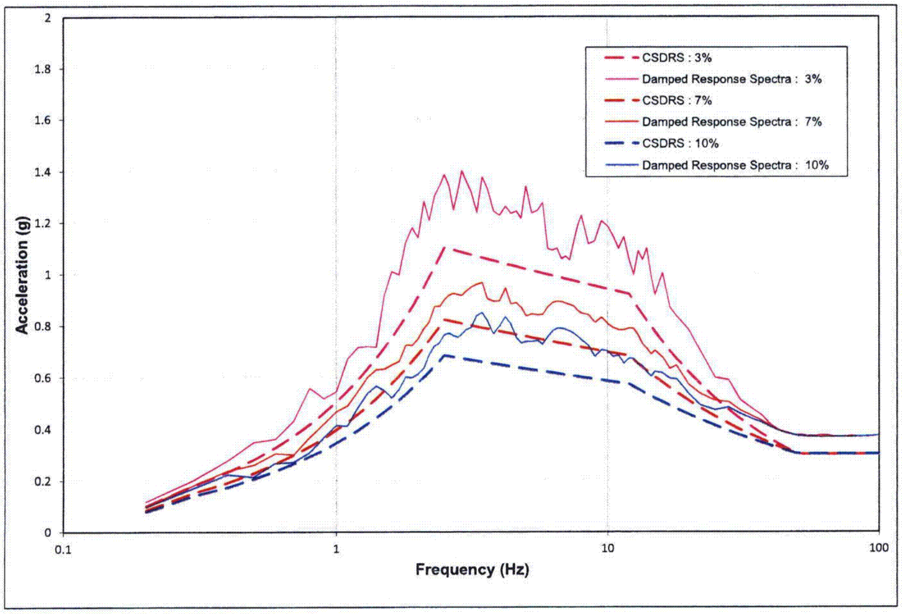


Figure 3.7.1-6b Damped Response Spectra Plots for Northridge Mount Baldy Component H1 (180) [NS]

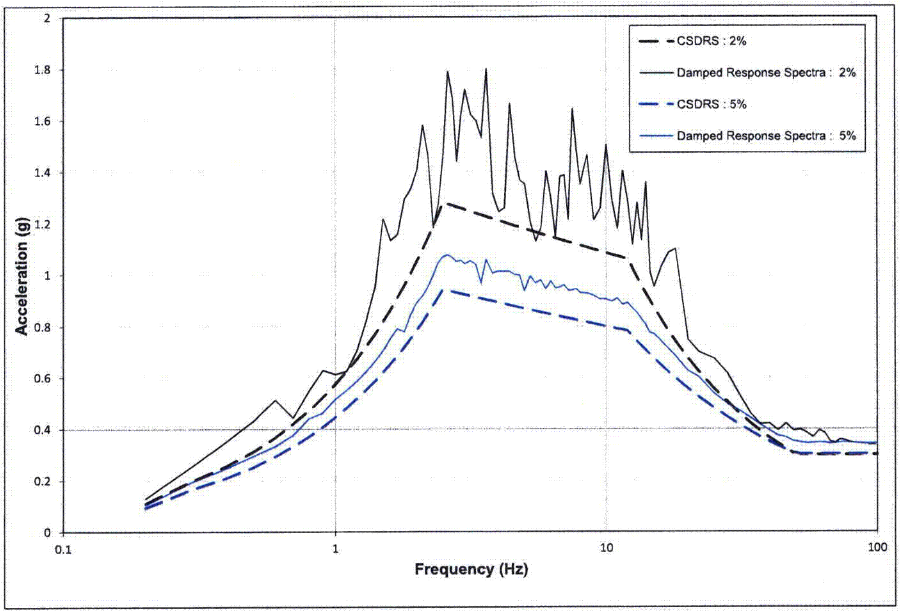


Figure 3.7.1-7a Damped Response Spectra Plots for Northridge Mount Baldy Component H2 (090) [EW]

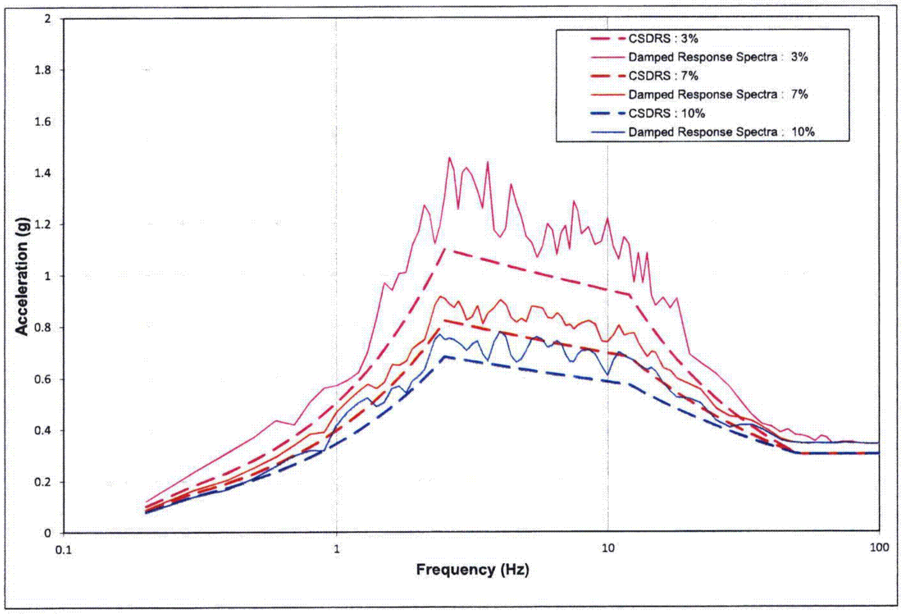


Figure 3.7.1-7b Damped Response Spectra Plots for Northridge Mount Baldy Component H2 (090) [EW]

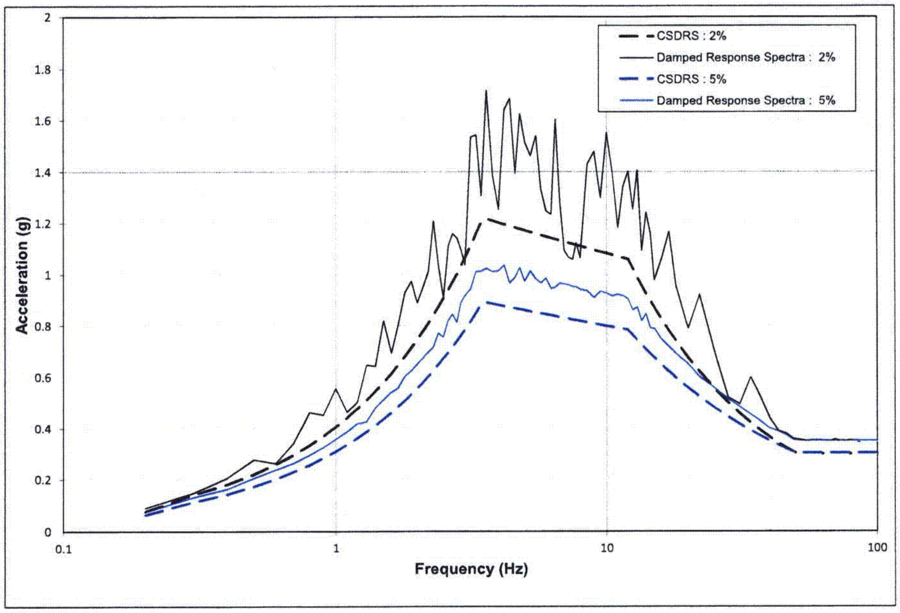


Figure 3.7.1-8a Damped Response Spectra Plots for Mount Baldy Component V (UP)

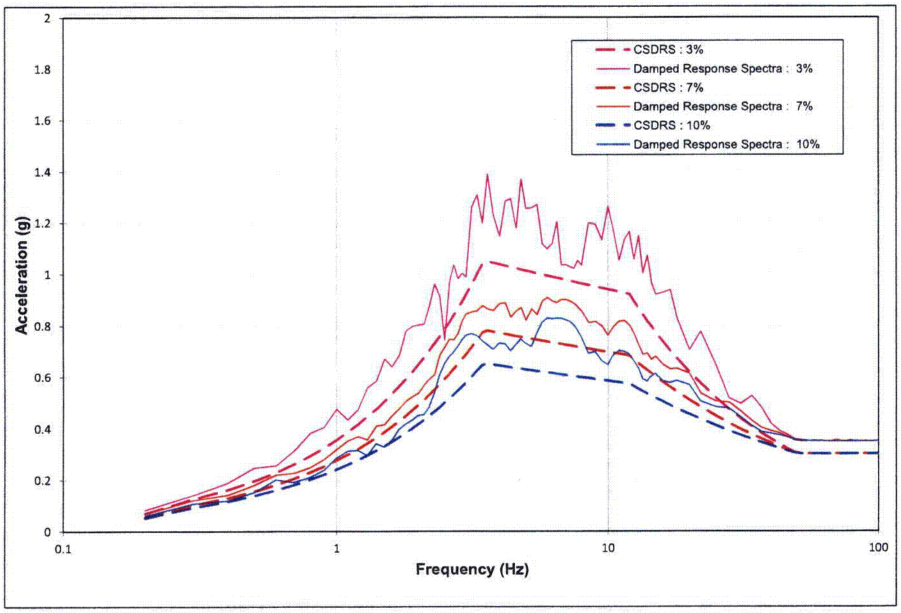
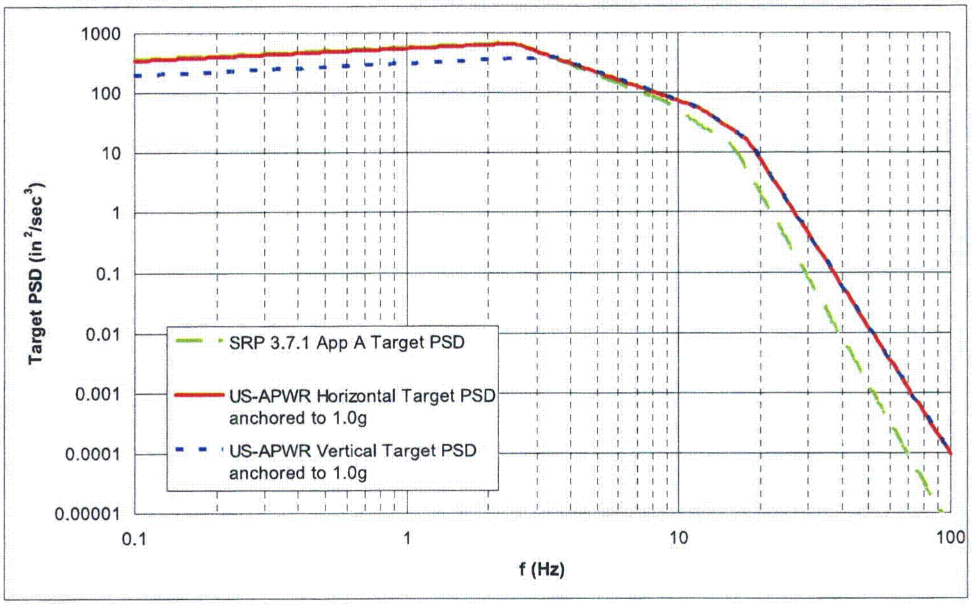


Figure 3.7.1-8b Damped Response Spectra Plots for Mount Baldy Component V (UP)



The final horizontal target PSD anchored to 1.0g is as follows:

$$S_{0H}(f) = \begin{cases} 650(f/2.5)^{0.2} & \text{for } f < 2.5\text{Hz} \\ 650(2.5/f)^{1.6} & \text{for } 2.5\text{Hz} \le f < 12\text{Hz} \\ 52.9(12.0/f)^{3} & \text{for } 12\text{Hz} \le f < 18\text{Hz} \\ 15.7(18/f)^{7} & \text{for } 18\text{Hz} \le f \end{cases} \quad (\text{in}^2/\text{sec}^3)$$

The final vertical target PSD anchored to 1.0g is as follows:

$$S_{0V}(f) = \begin{cases} 380(f/3.5)^{0.2} & \text{for } f < 3.5\text{Hz} \\ 380(3.5/f)^{1.6} & \text{for } 3.5\text{Hz} \le f < 12\text{Hz} \\ 52.9(12.0/f)^{3} & \text{for } 12\text{Hz} \le f < 18\text{Hz} \\ 15.7(18/f)^{7} & \text{for } 18\text{Hz} \le f \end{cases} \quad (\text{in}^2/\text{sec}^3)$$

Figure 3.7.1-9 US-APWR Final Horizontal and Vertical Target PSDs

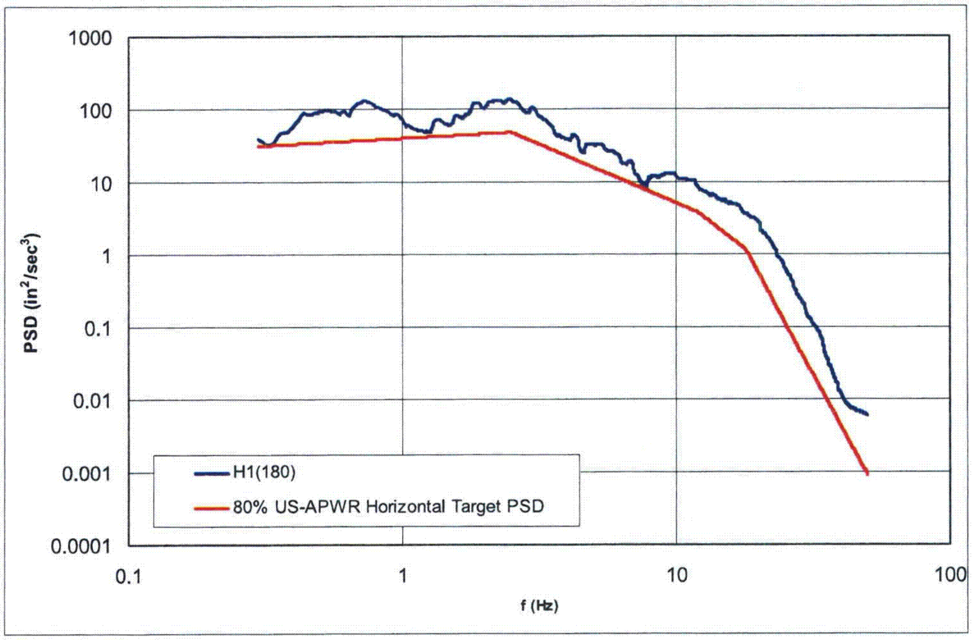


Figure 3.7.1-10 Smoothed Power Spectral Density Plots for Component H1 (180)

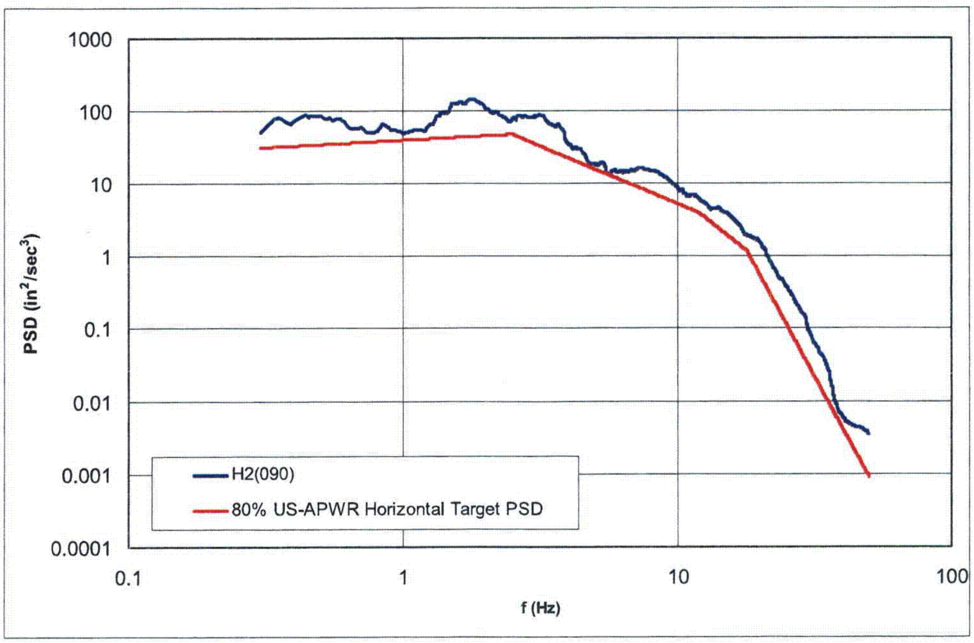


Figure 3.7.1-11 Smoothed Power Spectral Density Plots for Component H2 (090)

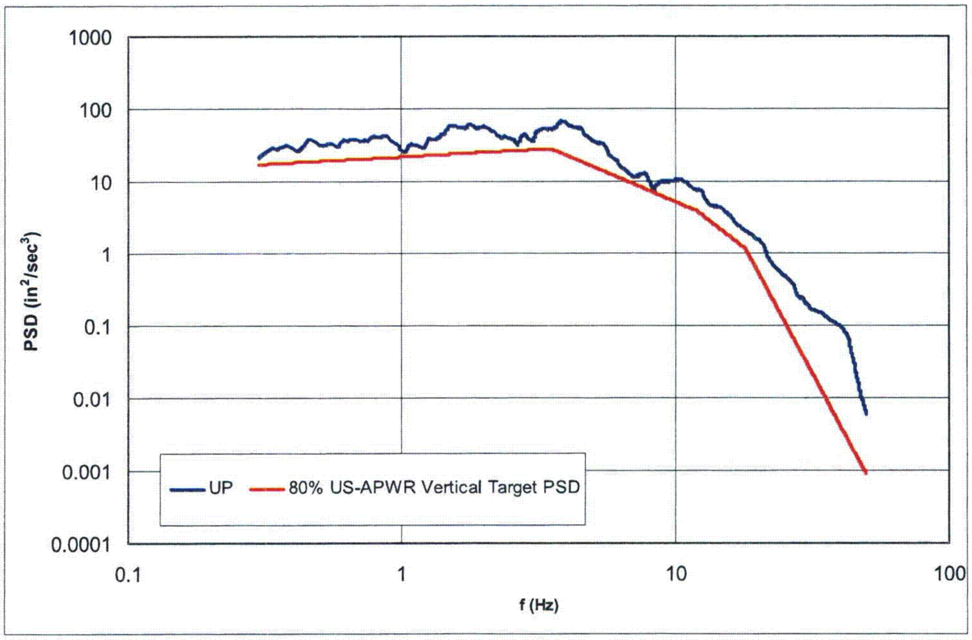


Figure 3.7.1-12 Smoothed Power Spectral Density Plots for Component V (UP)

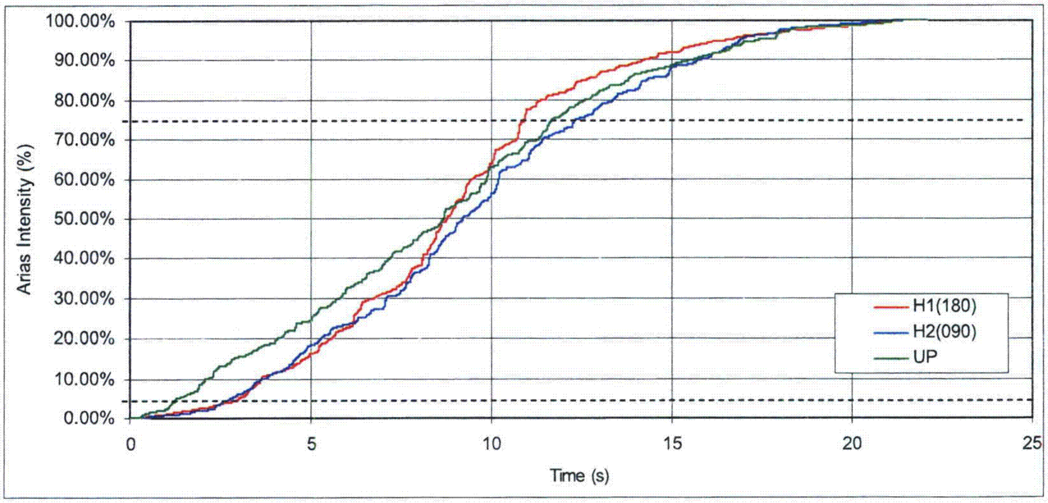


Figure 3.7.1-13 Arias Intensities of the Northridge – Mount Baldy Artificial Time History Components Showing 5%-75% Duration

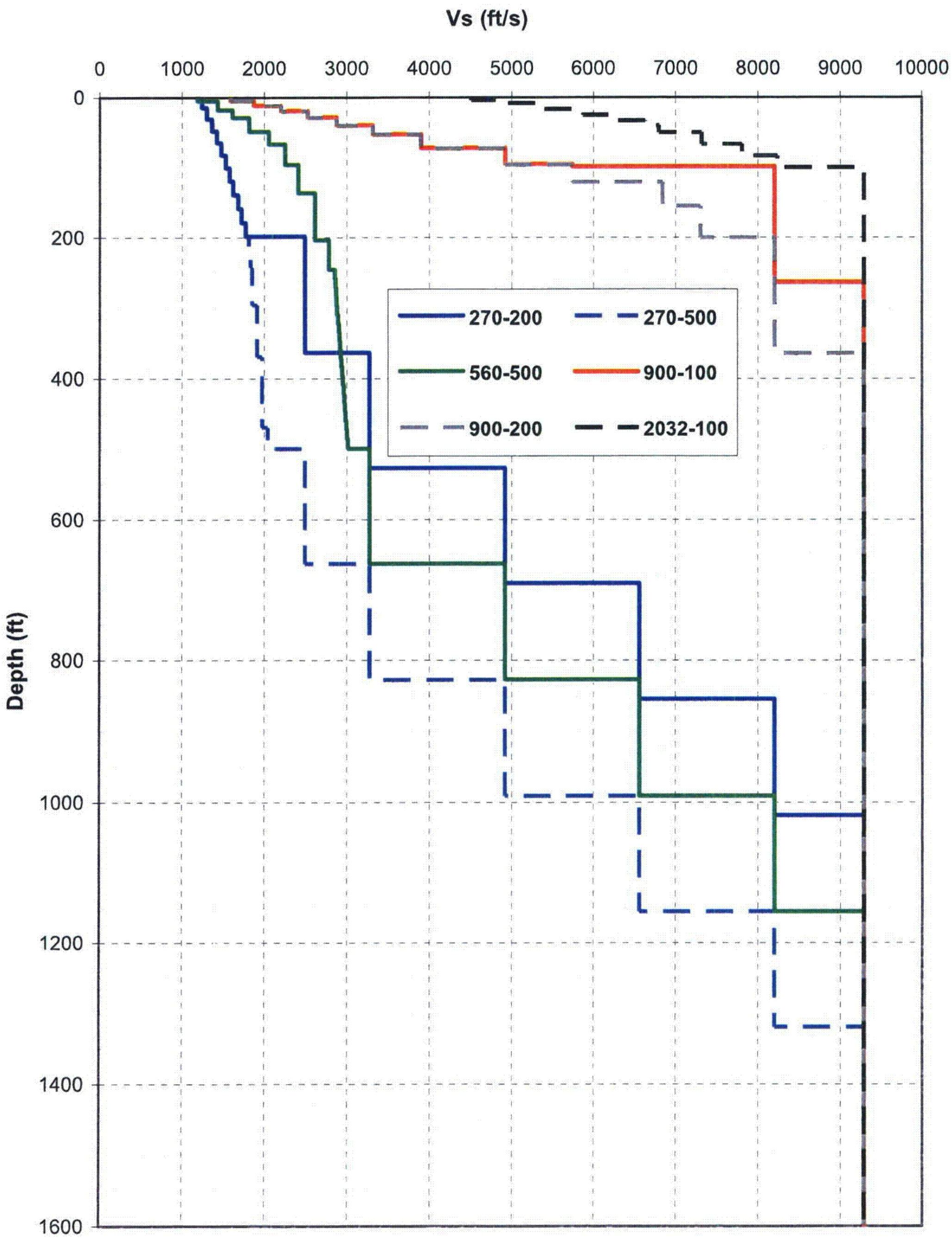


Figure 3.7.1-14 Six Generic Soil Profiles, Shear Wave Velocity $V_{(s)}$

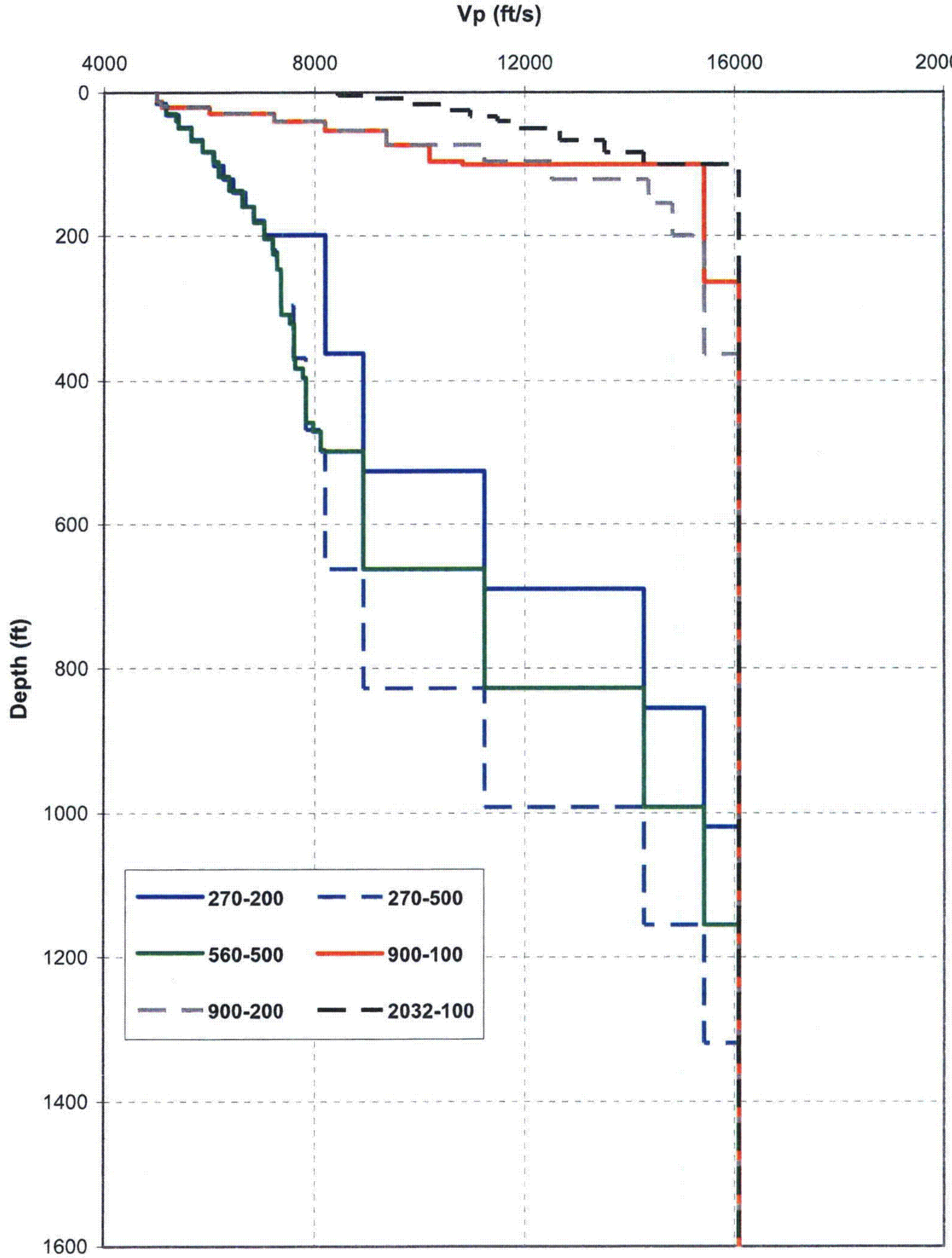


Figure 3.7.1-15 Six Generic Soil Profiles, Compression Wave Velocity (V_p)

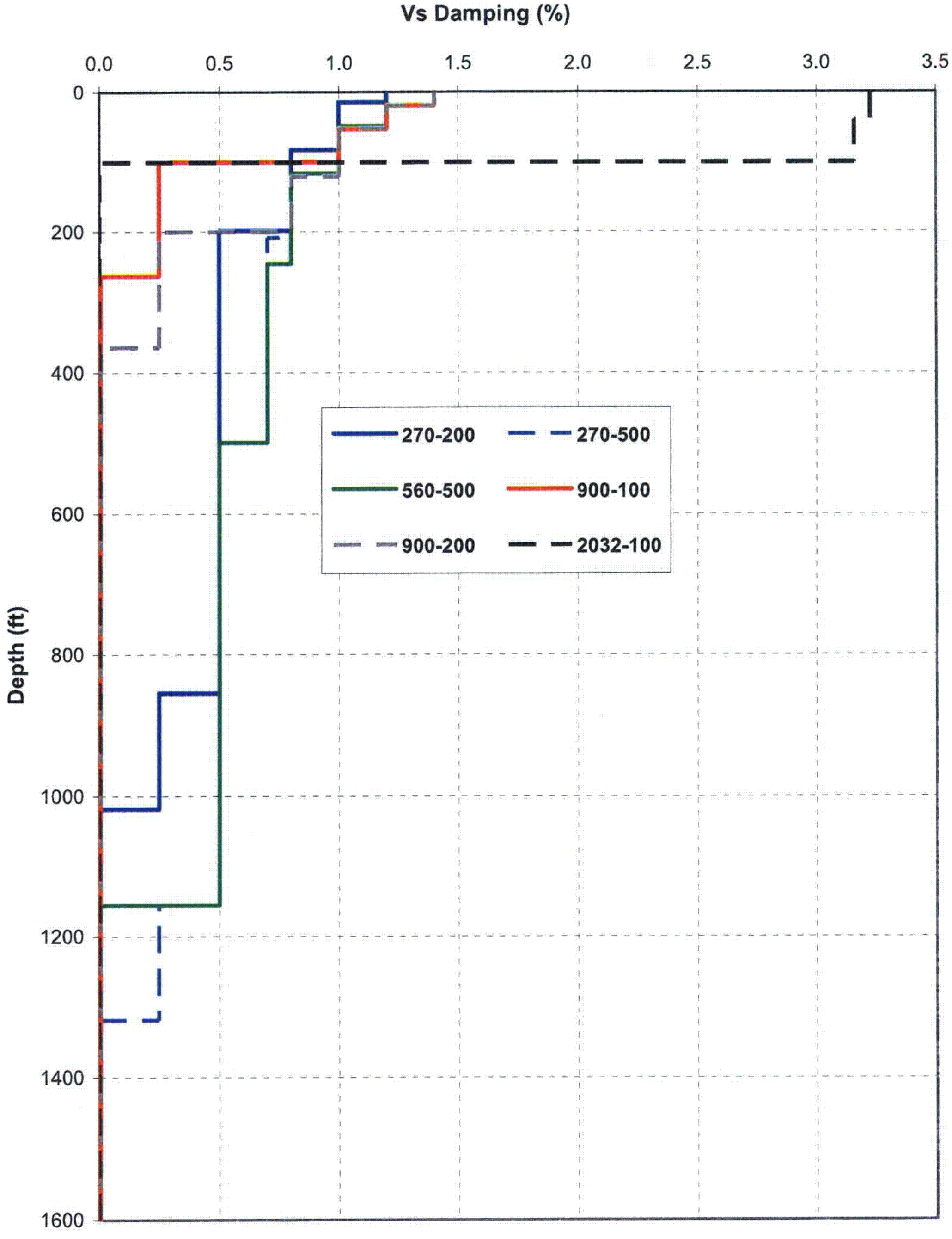


Figure 3.7.1-16 Six Generic Soil Profiles, Shear Wave Velocity Damping (%)

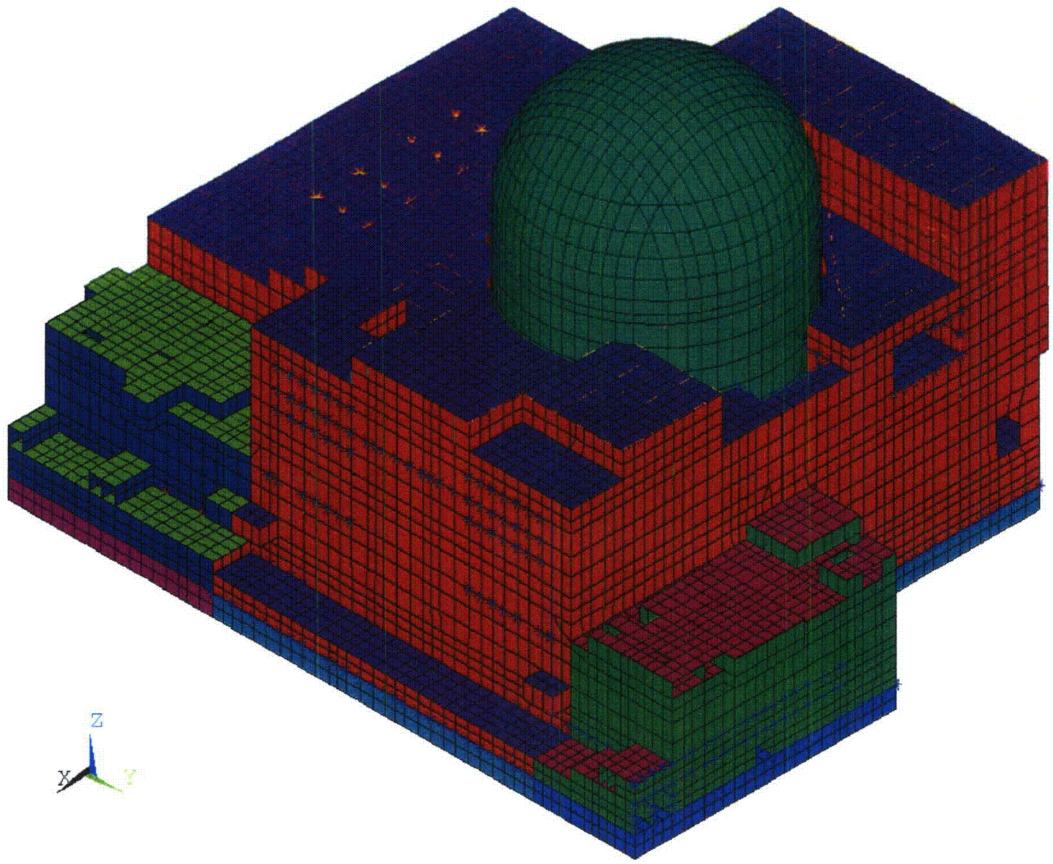


Figure 3.7.2-1 Integrated R/B Complex Dynamic FE Model

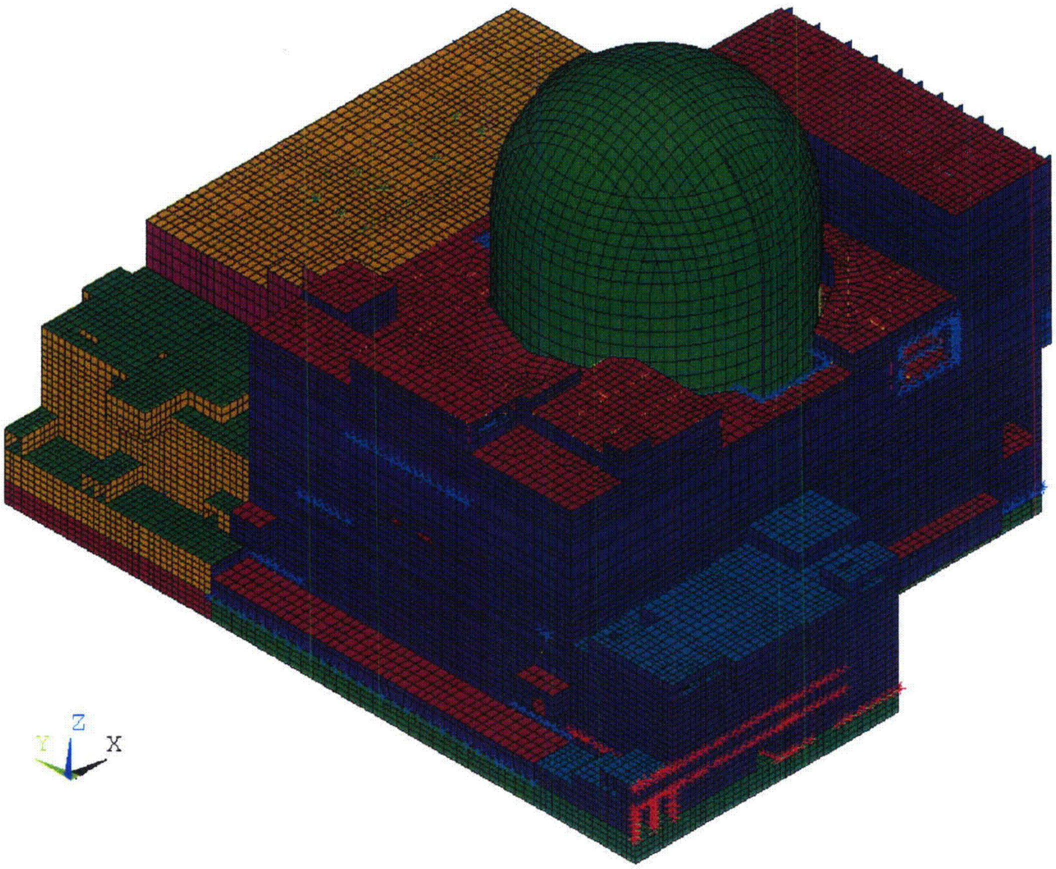


Figure 3.7.2-2 Integrated R/B Complex Detailed FE Model

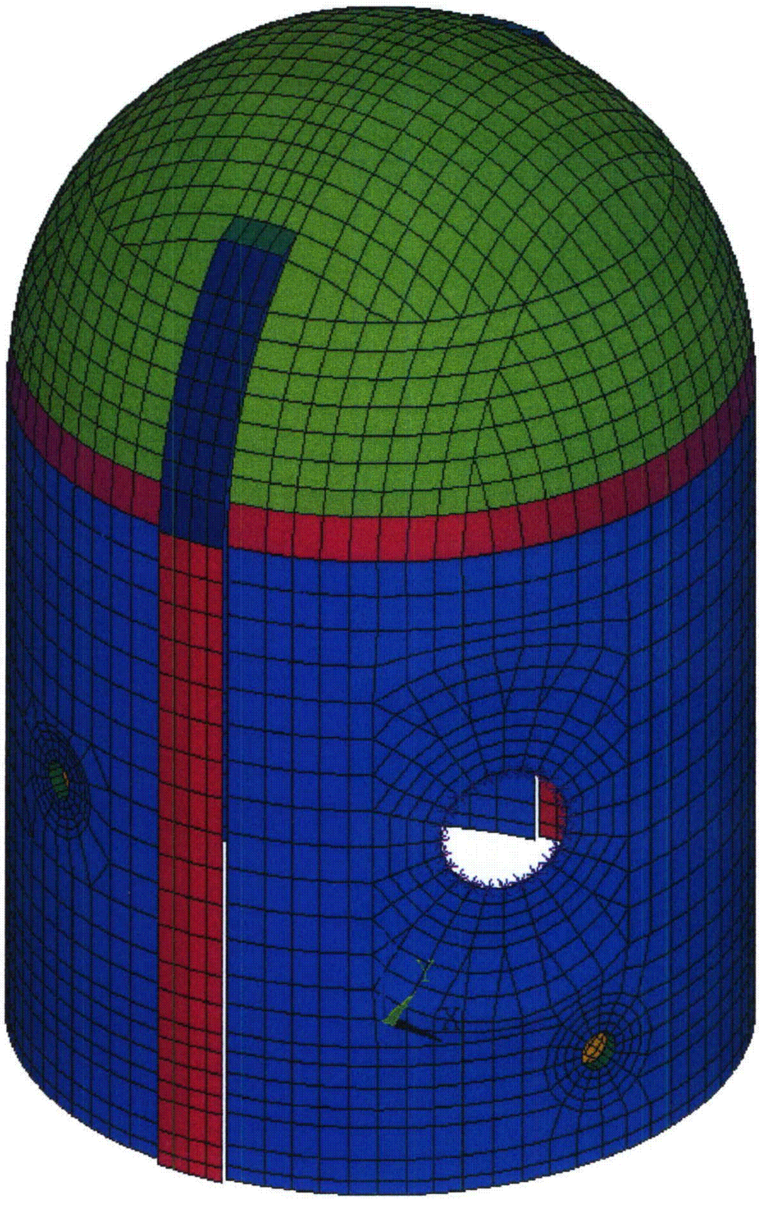


Figure 3.7.2-3 PCCV Detailed Model

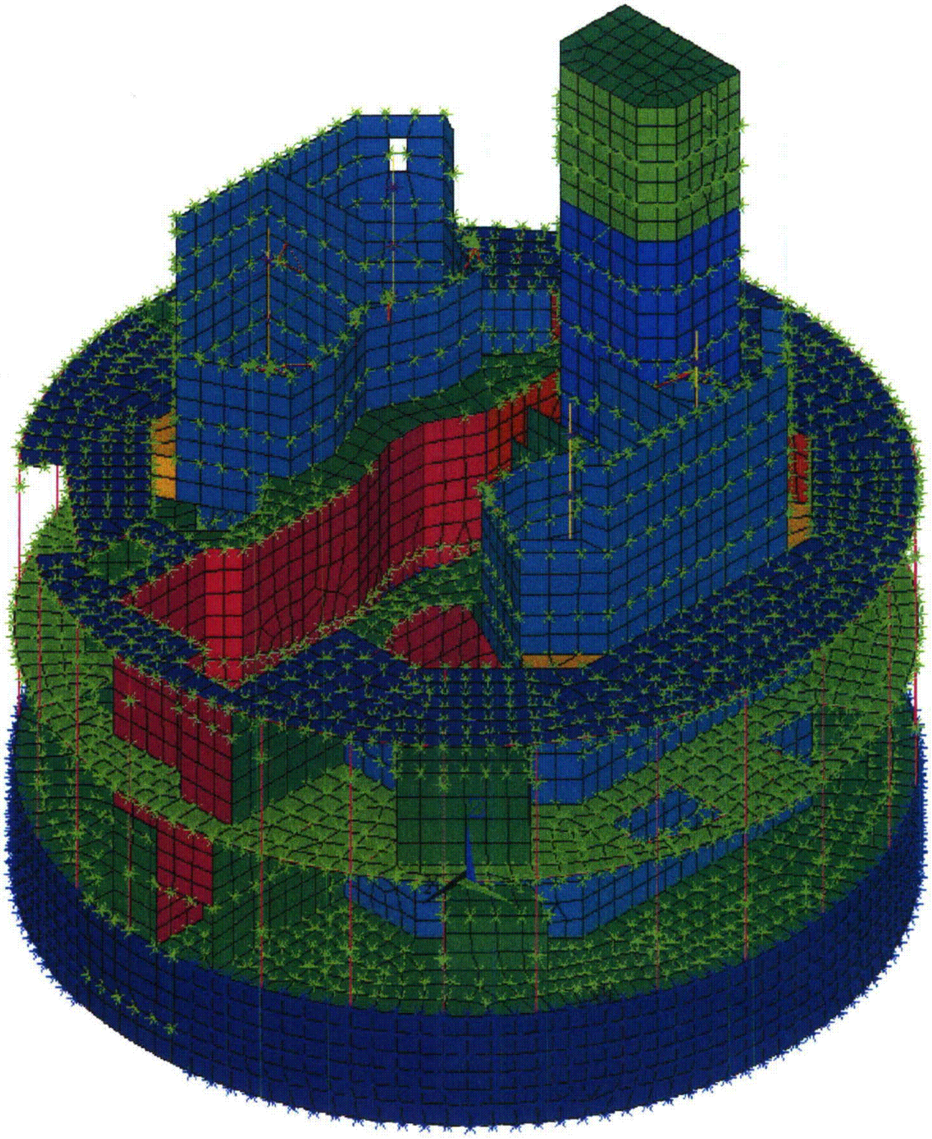


Figure 3.7.2-4 CIS Detailed Model
(includes the CIS and the RCL models)

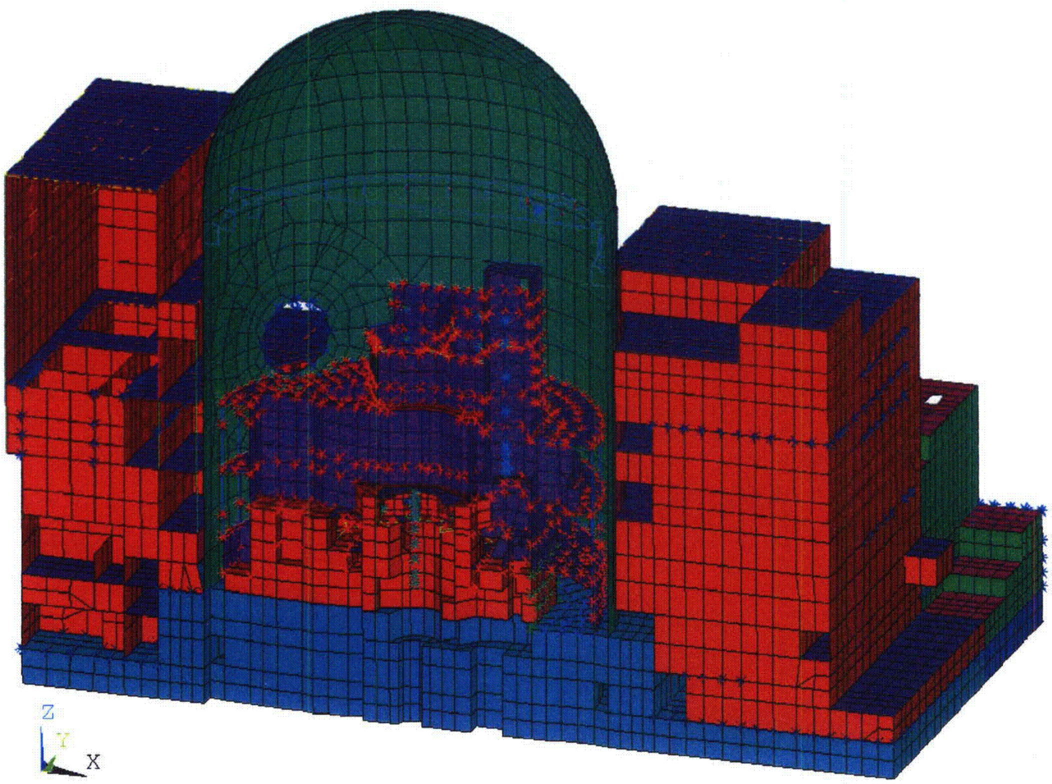


Figure 3.7.2-5 Section View of Dynamic R/B Complex FE Model Looking East

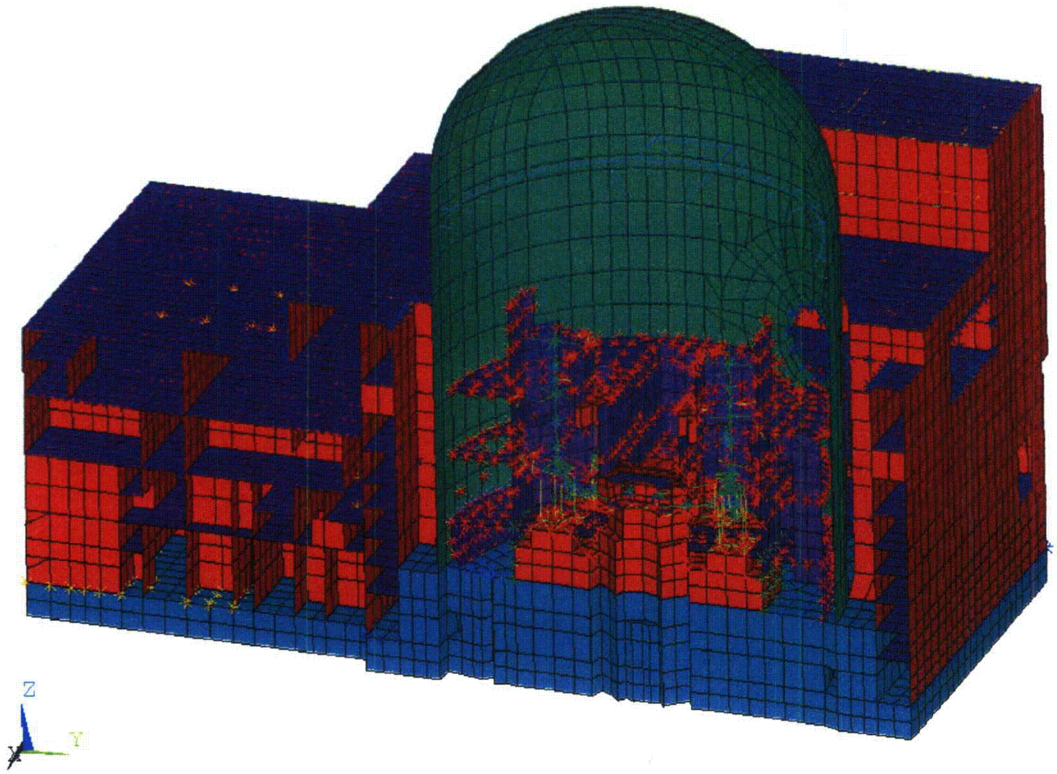


Figure 3.7.2-6 Section View of Dynamic R/B Complex FE Model Looking North

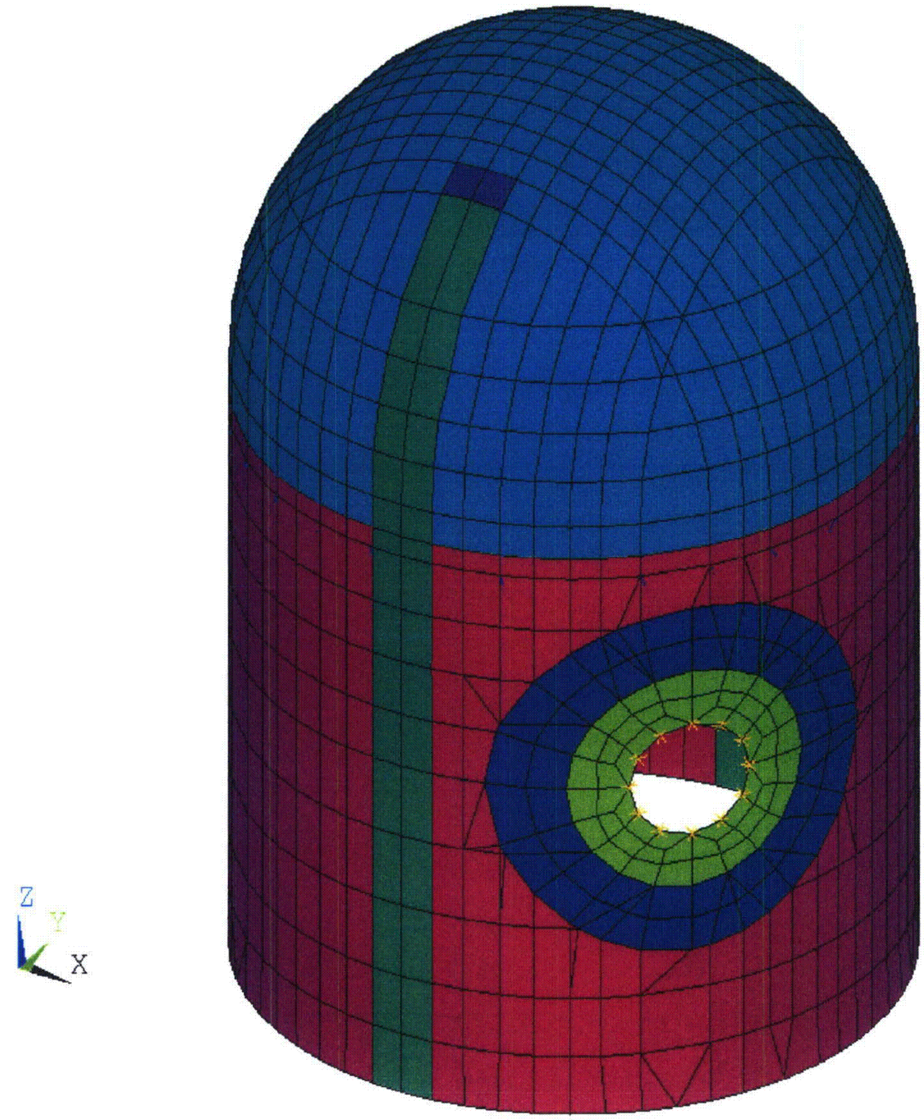


Figure 3.7.2-7 PCCV Dynamic Model

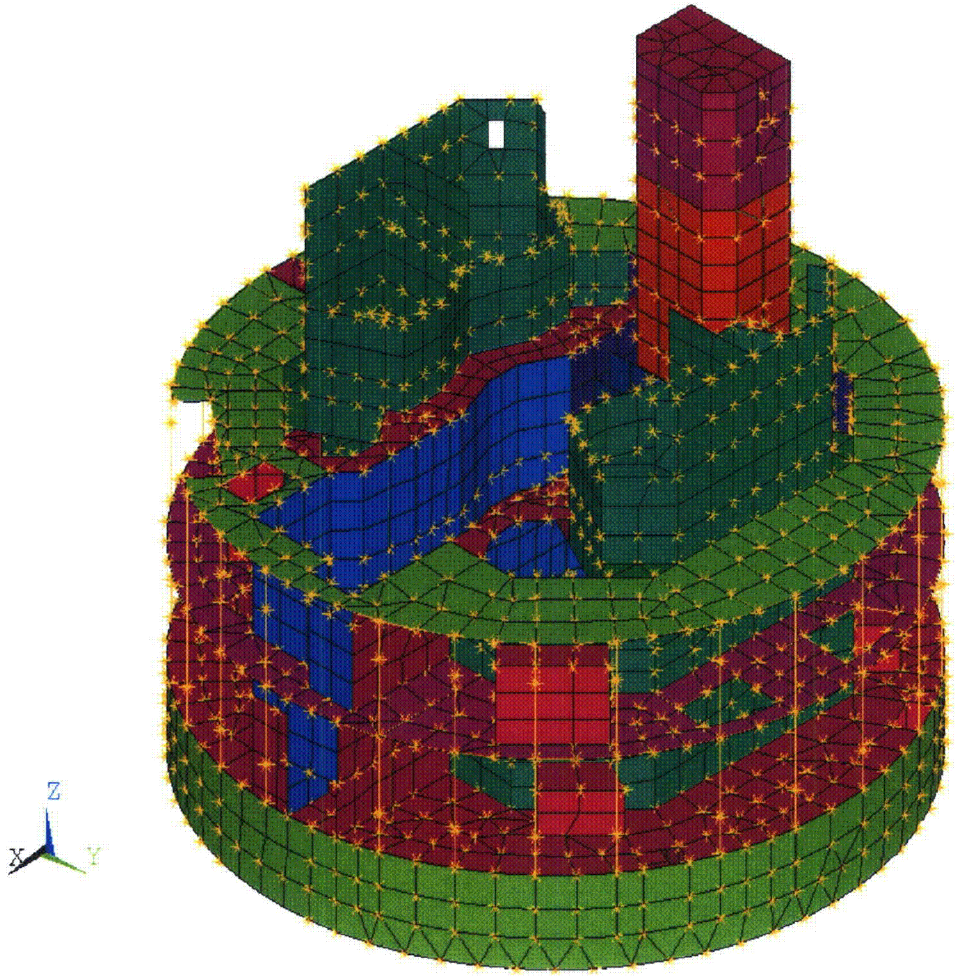


Figure 3.7.2-8 CIS Dynamic Model – Shell Elements (Excluding RCL)

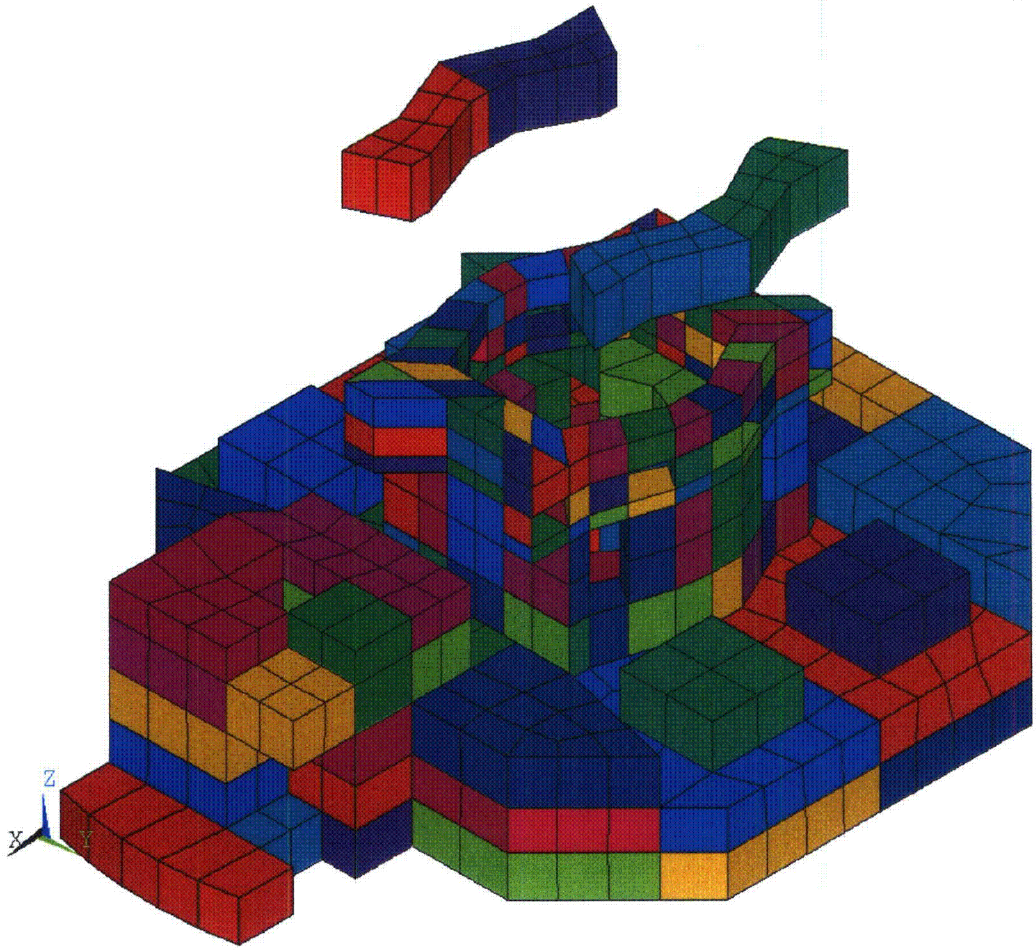


Figure 3.7.2-9 CIS Dynamic Model - Solid Elements

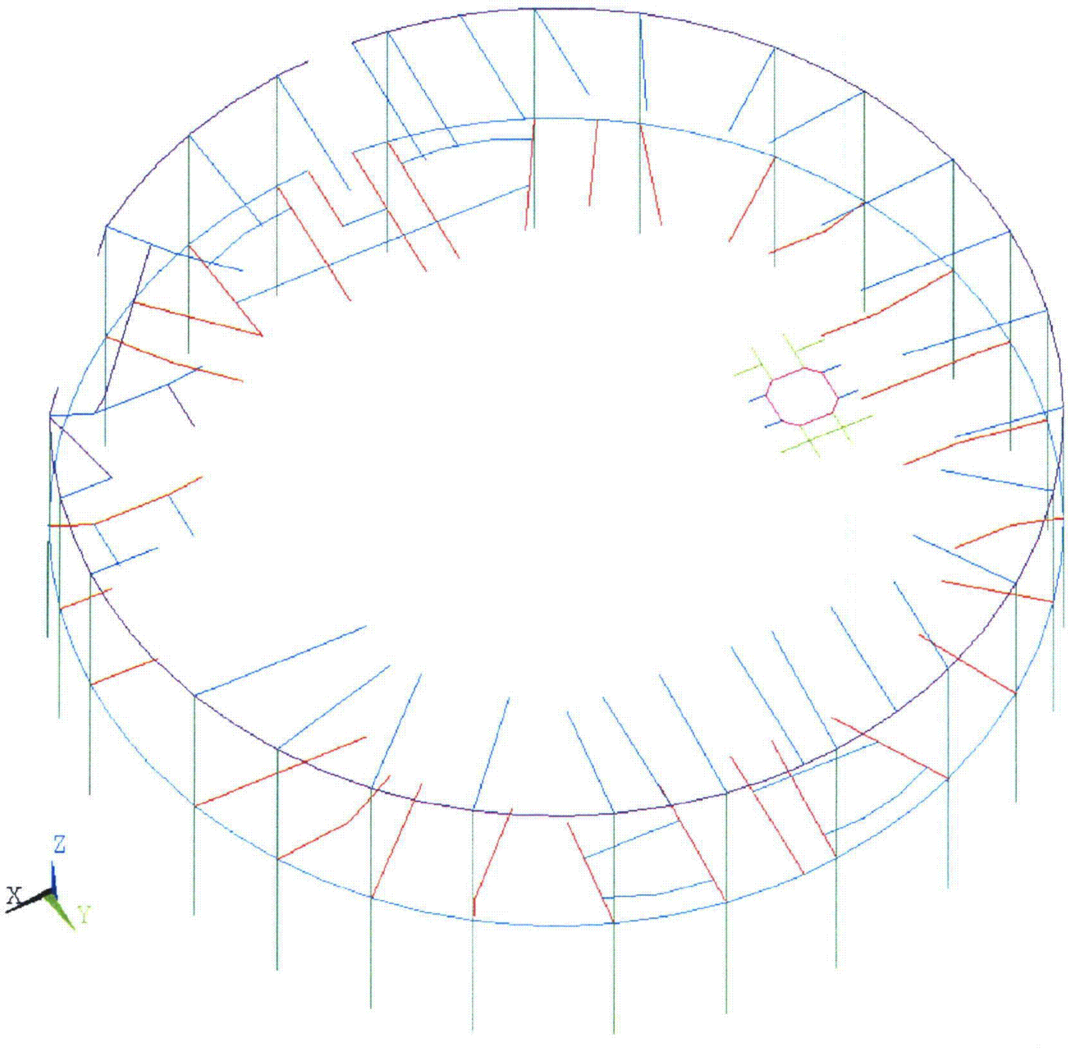


Figure 3.7.2-10 CIS Dynamic Model – Beam Elements (Excluding RCL)

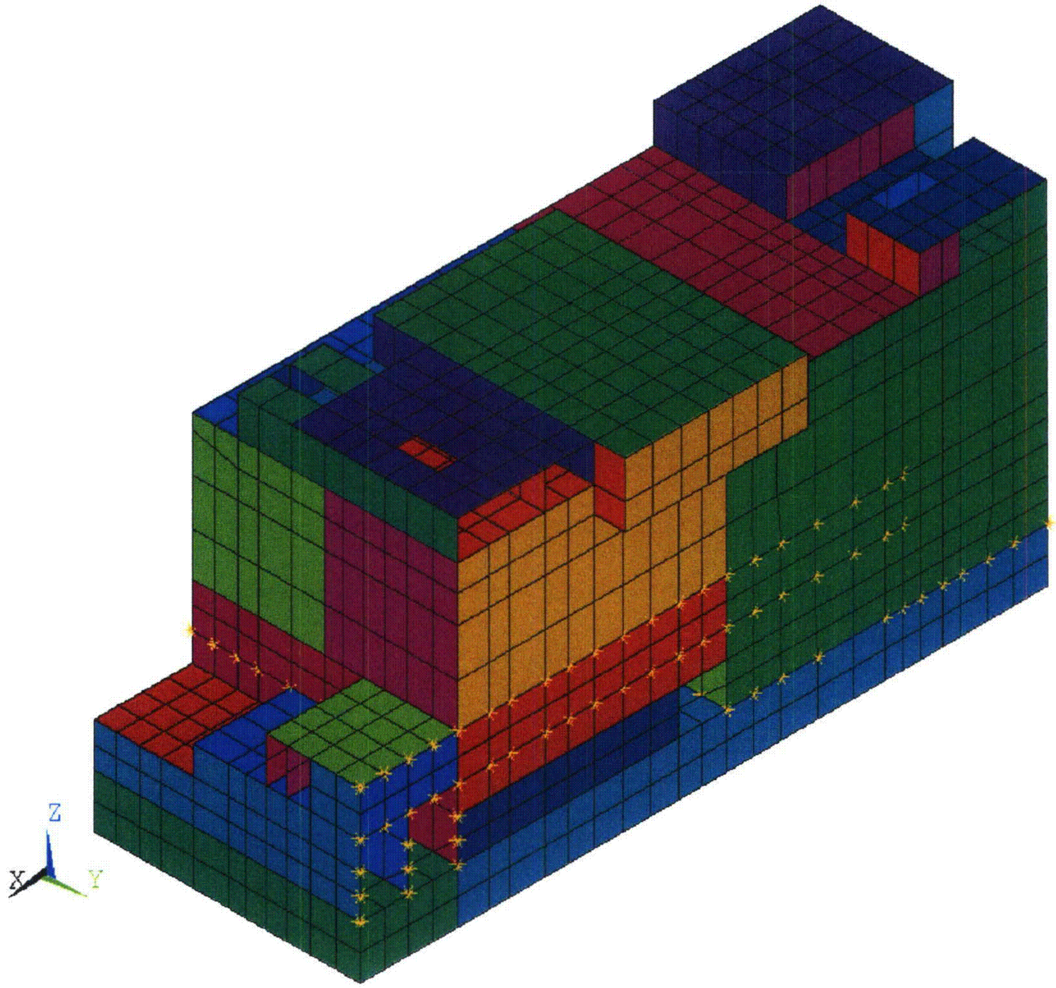


Figure 3.7.2-11 East PS/B Dynamic Model with ESWPC

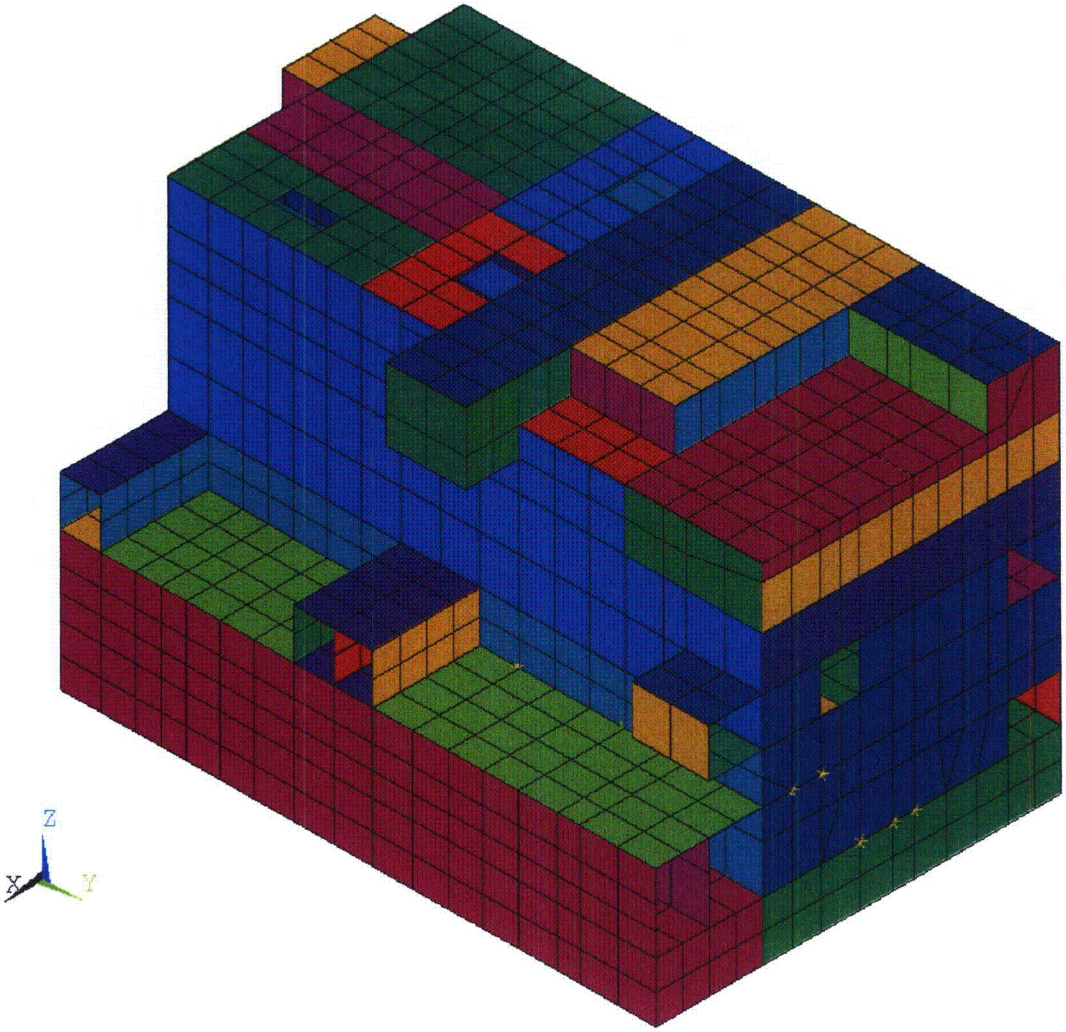


Figure 3.7.2-12 West PS/B Dynamic Model with ESWPC

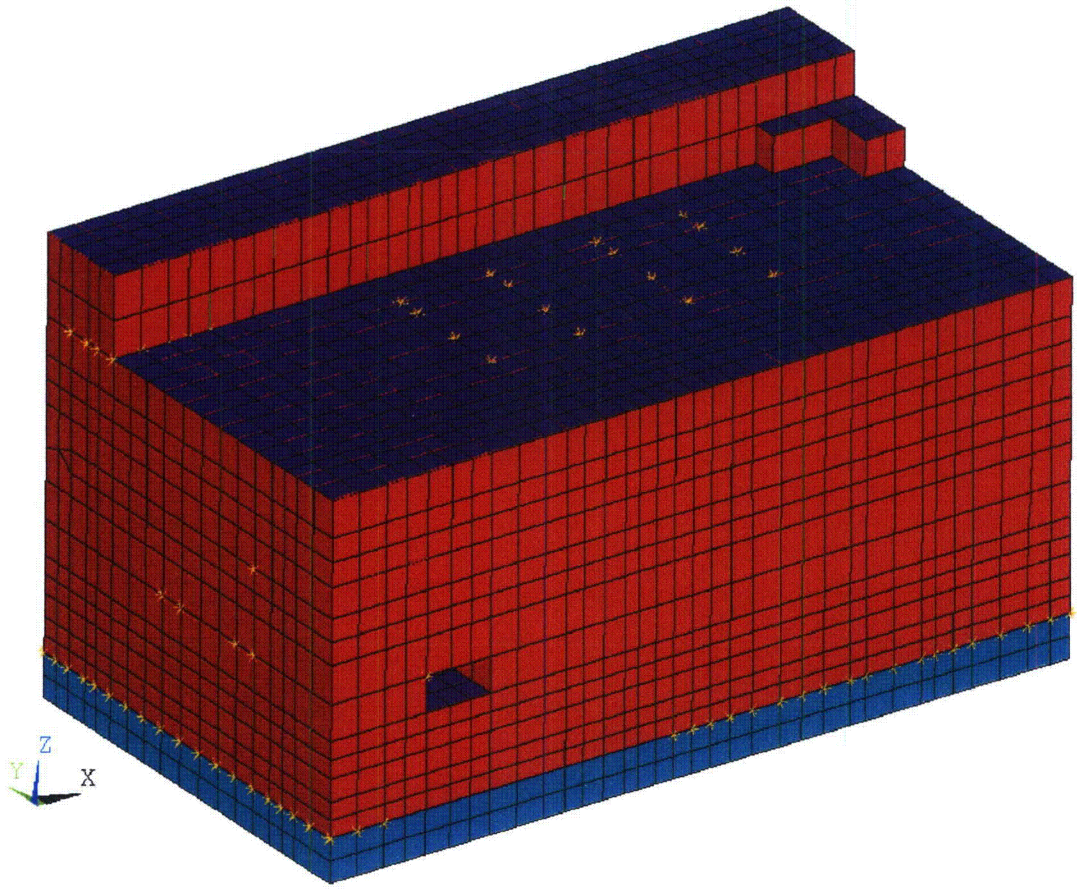


Figure 3.7.2-13 A/B Dynamic Model

Figure 3.7.2-14 Deleted

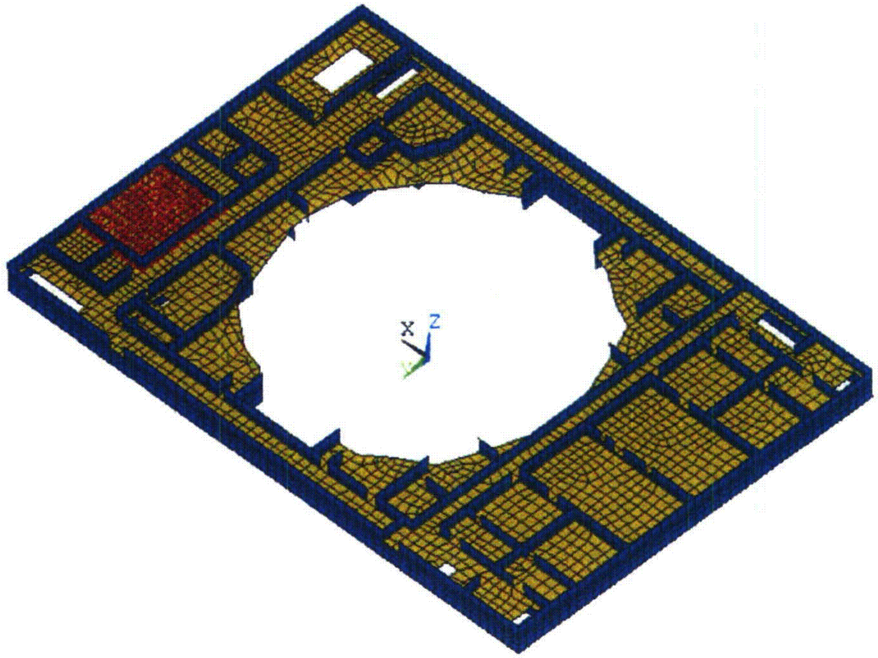


Figure 3.7.2-15 Extracted Detailed FE Model of Floor Slabs

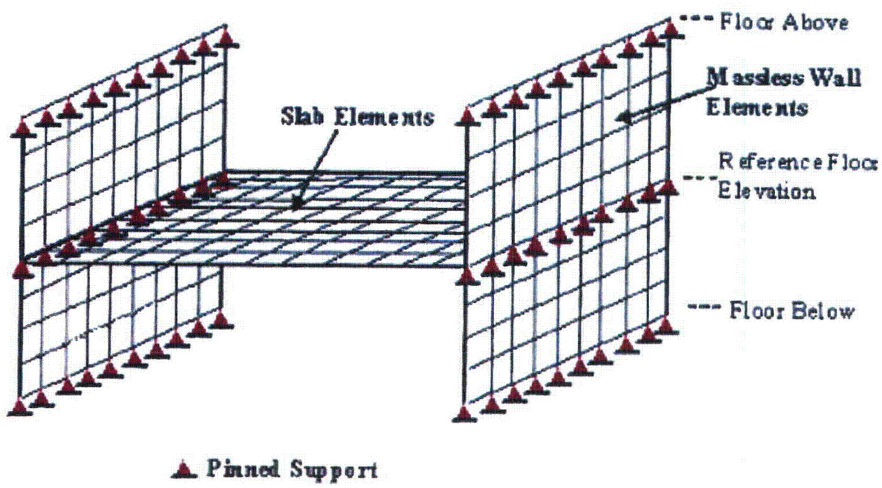


Figure 3.7.2-16 Floor Slab Model Boundary Conditions

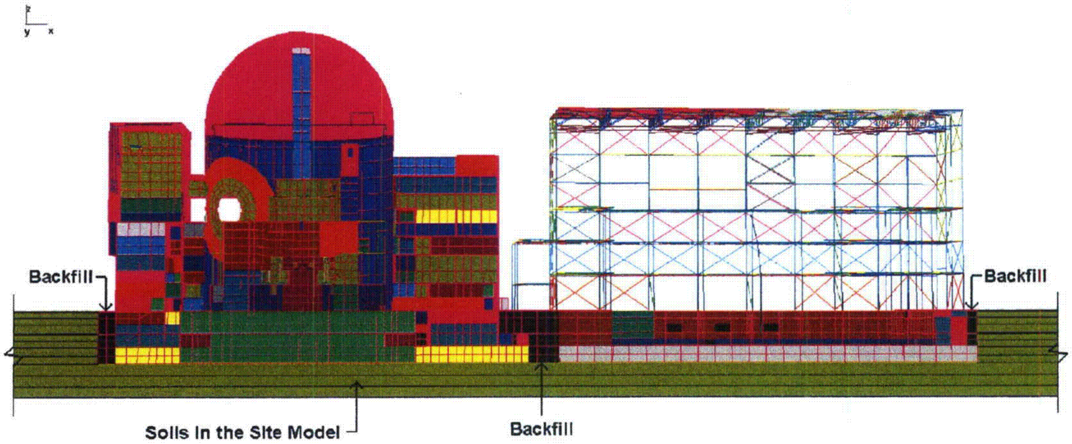


Figure 3.7.2-17 SSSI Model with Backfill and Free Field Soils (Looking East)

ENCLOSURE 2
Seismic Analyses and Structural Design
Tier 2* Selection Criteria

Background:

On May 15th, 2013, US-APWR Licensing and Engineering personnel presented the approach for identifying and presenting information on Critical Sections to the NRC staff. This paper provides the approach to be used for the identification of Tier 2* information and the applicable regulatory framework behind the implementation and processing of Tier 2* information.

Tier 2* Definition:

As defined by the NRC in NUREG-0800, Standard Review Plan (SRP), 14.3, "Inspections, Tests, Analyses and Acceptance Criteria" (ITAAC), Tier 2* information is that information in Tier 2 that, if considered to be changed by a combined license (COL) applicant or licensee, requires NRC approval prior to the change.

In accordance with 10 CFR 52, Appendix A, B, C, & D, "Design Certification Rule for the U.S. ABWR, System 80+, AP600, and AP1000," respectively, a departure from Tier 2, other than one affecting resolution of a severe accident issue identified in the plant-specific DCD or one affecting information required by 10 CFR 52.47(a)(28) to address 10 CFR 50.150, requires a license amendment if it would:

(1) Result in more than a minimal increase in the frequency of occurrence of an accident previously evaluated in the plant-specific DCD;

(2) Result in more than a minimal increase in the likelihood of occurrence of a malfunction of a structure, system, or component (SSC) important to safety previously evaluated in the plant-specific DCD:

(3) Result in more than a minimal increase in the consequences of an accident previously evaluated in the plant-specific DCD;

(4) Result in more than a minimal increase in the consequences of a malfunction of a SSC important to safety previously evaluated in the plant-specific DCD;

(5) Create a possibility for an accident of a different type than any evaluated previously in the plant-specific DCD;

(6) Create a possibility for a malfunction of an SSC important to safety with a different result than any evaluated previously in the plant-specific DCD;

(7) Result in a design basis limit for a fission product barrier as described in the plant-specific DCD being exceeded or altered; or

(8) Result in a departure from a method of evaluation described in the plant-specific DCD used in establishing the design bases or in the safety analyses.

A similar 10 CFR 52 Appendix will be promulgated for the US-APWR as part of NRC Licensing. Also note that the requirements above closely mirror the 10 CFR 50.59 evaluation criteria.

ENCLOSURE 2
Seismic Analyses and Structural Design
Tier 2* Selection Criteria

Tier 2* information identifies the design information that is important to the commercial nuclear power facility's design and material to the NRC's safety determination for use in the SER. "Material" in this context means that the NRC's safety case relies significantly on the information. Information identified as Tier 2* becomes part of the safety analysis report that cannot be changed by a license holder without prior NRC review and approval. Any revisions to the Tier 2* information will be subject to NRC review and approval to avoid unintended safety consequences in the construction and operations of a commercial nuclear power facility.

Selection Criteria:

Tier 2* information is the design data that is identified as key to the US-APWR basic design. The criteria for identifying information as Tier 2* includes the following:

1) Material properties used in basic design
   - The minimum/bounding required material properties used in the design (e.g., minimum compressive concrete strength, f'_c, at an identified number of days cure time)
2) Primary codes, standards, and guidance relied upon in design; including the applicable year, edition, addenda, etc. (not already identified as Tier 2*)
   - Not all such commitments should be identified as Tier 2*; only those necessary to maintain the US-APWR basic design essentially unchanged (i.e., ASME Code Section III, Div. 2, ACI 349-06, ANSI/AISC N690-94 including supplement 2)
3) Principal nominal dimensions that are important to building overall structural dimensions
   - Includes attributes such as member thicknesses (for concrete), sizes/sections (for structural steel), provided reinforcement ratio, etc. with tolerances to provide for construction deltas
4) Containment penetration details
   - For example, personnel airlocks, equipment hatch, etc. important to the safety analysis
5) CIS design methods that were utilized for the US-APWR which are justified or confirmed by laboratory tests. (These include CIS and PCCV Liner anchors)
6) Bounding design values important to US-APWR standard plant structural analyses and design
7) Models, methods, programs, codes, etc. used to assess design or calculate performance if considered an essential part of the NRC's safety review of the design

The NRC staff has acknowledged that requiring Tier 2* designations on analytical results (with several significant digits) is overly restrictive. Consistent with this NRC position, Tier 2* designations will not be applied to the results of applying a code requirement for reinforced steel areas. Careful consideration is given to avoid designating materials as Tier 2* when an alternative installation configuration or placement tolerance is typically allowed and is equally acceptable for safety.
The selection of Tier 2* information will rely heavily on engineering judgment of what is determined to be top level design features. Additionally, minimum thicknesses, nominal sizes, pointing to codes and standards for tolerances, etc., should be incorporated in the Tier 2* information, as appropriate, to prevent the need to designate actual measurements as Tier 2* that could unnecessarily constrain construction.

Per SRP 14.3, Appendix A, the NRC has the final authority to designate which material in the safety analysis report is Tier 2*. For all cases where the staff believes that Tier 2* applies, the cognizant NRC Technical Division Director must review and approve. Additionally, the SRP requires that for all information designated as Tier 2* that the staff documents the bases for the determination that changing the information would require prior NRC approval in the FSER. It is expected that the bases description will be based upon the criteria in SRP 14.3.