3.7 Seismic Design

The APR1400 structures, systems, and components (SSCs) important to safety are designed to withstand the effects of earthquakes without loss of capability to perform their safety functions, as required by 10 CFR 50, Appendix A, General Design Criterion (GDC) 2 (Reference 1).

The APR1400 SSCs are classified in one of the following three seismic categories:

a. Seismic Category I

b. Seismic Category II

c. Seismic Category III (non-seismic)

Seismic Category I SSCs are designed to withstand the effects of the earthquake event and to maintain their specified design functions. Seismic Category II SSCs do not perform safety-related functions, but structural failure or interaction could degrade the function of a seismic Category I SSC to an unacceptable safety level. Seismic Category III SSCs do not perform safety-related functions, and structural failure or interaction could not degrade the function of a seismic Category I SSC to an unacceptable safety level.

3.7.1 Seismic Design Parameters

This subsection describes seismic design parameters such as design ground motions and damping values that are used in the design of the SSCs important to safety and classified as seismic Category I in Section 3.2.

The APR1400 seismic Category I SSCs are designed for the safe shutdown earthquake (SSE). The SSE is defined as the maximum potential vibratory ground motion at the generic plant site. Since the operating basis earthquake (OBE) is defined as one third the SSE, an explicit analysis or design of the APR1400 seismic Category I SSCs based on OBE is not required in accordance with Appendix S of 10 CFR 50 (Reference 2).

3.7.1.1 Design Ground Motion

The design response spectra of the site independent SSE are now referred to as the certified seismic design response spectra (CSDRS). The CSDRS and design time histories compatible with CSDRS are described in the following subsections.

3.7.1.1.1 Design Ground Motion Response Spectra

The peak ground acceleration (PGA) of the CSDRS has been established as 0.3g for the APR1400 design for both the horizontal and vertical directions.

The horizontal and vertical CSDRS for the APR1400 are based on the NRC Regulatory Guide (RG) 1.60 (Reference 3) response spectra, enriched in the high frequency range in the following manner.

a. The spectral amplitudes of the horizontal and vertical response spectra at control frequencies 9 Hz and below are equal to those of the NRC RG 1.60 response spectra.

b. The control frequency at which the PGA is reached is changed from 33 Hz to 50 Hz for both the horizontal and vertical spectra.

c. A control frequency at 25 Hz is added. The spectral amplitudes at 25 Hz are set to the NRC RG 1.60 response spectra at 25 Hz scaled by a factor of 1.30 for both the horizontal and vertical spectra.

d. Linearly vary the modified spectra, on a log-log-scale, between the control frequencies 9 Hz, 25 Hz, and 50 Hz.

The digitized values of the resulting APR1400 horizontal and vertical CSDRS for 2, 3, 4, 5, 7, and 10 percent damping values are provided in Table 3.7-1. The APR1400 horizontal and vertical CSDRS are presented in Figures 3.7-1 and 3.7-2, respectively.

The CSDRS are applied at the finished grade in the free-field. As an additional requirement from 10 CFR 50, Appendix S. Figures 3.7A-12 and 3.7A-13 in Appendix

3.7A show that the horizontal components of CSDRS in the free-field at the foundation level of the APR1400 standard plant seismic Category I structures satisfy the PGA of at least 0.1g.

The site-specific seismic design can be developed for seismic Category I and II SSCs at the combined license (COL) stage. Seismic Category I and II SSCs are not included in the APR1400 standard plant design using the site-specific SSE derived from the ground motion response spectra (GMRS) in accordance with NRC RG 1.208 (Reference 4). In this case, the COL applicant is to determine the site-specific SSE and OBE that are applied to the seismic design of the site-specific seismic Category I and II SSCs and to the basis for the plant shutdown and is to verify the appropriateness of the site-specific SSE and OBE (COL 3.7(1)).

The COL applicant is to confirm that the horizontal components of the site-specific SSE ground motion in the free-field at the foundation level of the structures that are not included in the APR1400 standard plant design satisfy a PGA of at least 0.1g (COL 3.7(2)).

3.7.1.1.2 Design Ground Motion Time History

The three design acceleration time histories composed of two horizontal (“H1” and “H2”) and one vertical components (“VT”), which envelop the CSDRS, are applied in both soil-structure interaction analyses and fixed-base analyses of seismic Category I structures. The initial seed motions that were modified to create the design time histories are actual seed recorded Northridge earthquake time histories.

The design time histories are generated with an increment of time size of 0.005 second to provide a Nyquist frequency of 100 Hz. Figures 3.7-3, 3.7-4, and 3.7-5 show the acceleration, velocity, and displacement time histories for H1, H2, and VT components for each time step, respectively. The design time histories, H1, H2, and VT, are applied in the east-west (E-W) direction, north-south (N-S) direction, and vertical direction, respectively. The absolute values of correlation coefficients for each pair of the design time histories are as follows:

Correlation coefficient for H1 and H2 = 0.032

Correlation coefficient for H1 and VT = 0.079

Correlation coefficient for H2 and VT = 0.029

The design time histories are statistically independent because the correlation coefficients between the design time histories are less than 0.16 as specified in Standard Review Plan (SRP) 3.7.1 (Reference 5). Therefore, the representative maximum response of interest of the APR1400 SSCs can be obtained either by performing separate analyses for each of the three components of design time histories or by performing a single analysis with all three components of design time histories applied simultaneously.

The design time histories have a total time duration equal to 20.48 seconds and a corresponding stationary phase, which is the strong motion duration defined as the time required for the Arias Intensity rise from 5 percent to 75 percent in more than 6 seconds.

The design time histories are developed following the spectrum matching acceptance criteria of Option 1, Approach 1, in Section II of SRP 3.7.1. The comparison plots of the response spectra of the design time histories versus the design response spectra for 2, 3, 4, 5, 7, and 10 percent critical damping are shown in Figures 3.7-6, 3.7-7, and 3.7-8. The figures demonstrate that the design time histories envelop the design response spectra for those damping values, satisfying the requirement of SRP 3.7.1 that no more than 5 points fall below and by no more than 10 percent below the design response spectra. The response spectra are computed at the frequency intervals given in Table 3.7.1-1 of SRP 3.7.1.

According to SRP 3.7.1, the ratios V/A and AD/V^2, where A, V, D are peak ground acceleration, ground velocity, and ground displacement, respectively, should be consistent with characteristic values for the magnitude and distance of the appropriate controlling events defining the uniform hazard response spectra. The target and target ranges of values for the other design ground-motion time-history parameters are the median (m) values and the median (m) ± one standard deviation (σ) (i.e., m ± σ), ranges. The determination of these target and target ranges of values is based on the methodologies and ground motion databases as described in NRC RG 1.60 and relevant NUREG reports, namely, NUREG-0003 (Reference 6) and NUREG/CR-6728 (Reference 7). Table 3.7-2 shows comparison of the ratios V/A and AD/V^2 for the time histories with the guidance in NUREG/CR-6728 and that the ratios are between the target values, target median ± σ.

In SRP 3.7.1, the requirement of minimum power spectral density (PSD) to prevent the design ground acceleration time histories from having a deficiency of power over any frequency range is described. SRP 3.7.1 specifies that the use of a single time history is justified by satisfying a target PSD requirement in addition to the design response spectra enveloping requirements.

Since the original NRC RG 1.60 horizontal spectrum and the horizontal CSDRS are identical for frequencies of less than 9 Hz, no modification to the target horizontal PSD is done in this frequency range.

The time-history simulation method described in NUREG/CR-5347 (Reference 8) is used to develop the CSDRS-compatible horizontal target PSD in the higher frequency range above 9 Hz. The resulting piecewise log-log linear horizontal target PSD developed is given in Table 3.7-3. The minimum required horizontal PSD is then 0.8 times the horizontal target PSD.

The vertical target PSD, compatible with the vertical CSDRS, is obtained from the horizontal target PSD, compatible with the horizontal CSDRS using the following equation:

$$S_V(f) = [\, R_V(f\,,\ 2\ \%)\ /\ R_H(f\,,\ 2\ \%)]^2 \times S_H(f)$$

where $R_H(f, 2\ \%)$ and $R_V(f, 2\ \%)$ are, respectively, the 2 percent damped horizontal and vertical CSDRS values at the frequency (f). The detailed procedure for generating target PSD is described in Technical Report, APR1400-E-S-NR-13001-P (Reference 9). The minimum required vertical PSD is then 0.8 times the vertical target PSD.

The PSDs of the design acceleration time histories are presented in Figures 3.7-9 through 3.7-11. The PSDs of the design acceleration time histories exceed the minimum required PSD throughout the entire frequency range. The PSDs presented are the averaged PSD obtained over a moving frequent band of ± 20 percent centered at each frequency. The PSD amplitude at frequency (f) has the averaged PSD amplitude between the frequency range of $0.8f$ and $1.2f$ as stated in Appendix A of SRP 3.7.1.

3.7.1.1.3 Hard Rock High Frequency Seismic Input Motions

GMRS for some Central and Eastern United States rock sites show higher amplitude at high frequency than the CSDRS. The GMRS for such a site are called hard rock high frequency (HRHF) seismic input motions. The hard rock is defined as having the low-strain shear wave velocity greater than 2,084 m/sec (9,200 ft/sec) as described in Subsection 2.5.2.6. The PGA of the HRHF target response spectra are prescribed as 0.46g for the evaluation of the APR1400 standard plant design for both the horizontal and vertical directions. The HRHF horizontal and vertical target response spectra are shown in Figures 3.7-12 and 3.7-13, respectively. These HRHF response spectra exceed the CSDRS for frequencies about approximately 9 Hz.

The three acceleration time histories composed of two horizontal (H1H and H2H) and one vertical components (VTH), which envelop the HRHF response spectra, are generated. The time histories, H1H, H2H, and VTH are applied in the east-west (E-W) direction, north-south (N-S) direction, and vertical direction, respectively. The initial seed motions that were modified to create the time histories compatible with HRHF response spectra are actual seed recorded Nahanni earthquake time histories.

The time histories are generated with an increment of time size of 0.005 second. Figures 3.7-14, 3.7-15, and 3.7-16 show the acceleration, velocity, and displacement time histories for H1H, H2H, and VTH components for each time step, respectively. The absolute values of correlation coefficients for each pair of the time histories are as follows:

Correlation coefficient for H1H and H2H = 0.028

Correlation coefficient for H1H and VTH = 0.031

Correlation coefficient for H2H and VTH = 0.036

The time histories are statistically independent because the correlation coefficients between the design time histories are less than 0.16. The time histories have a total time duration equal to 20.48 seconds and a corresponding stationary phase, which is the strong motion duration defined as the time required for the Arias Intensity rise from 5 percent to 75 percent in more than 6 seconds.

The time histories are developed following the spectrum matching acceptance criteria of Option 1, Approach 1, in Section II of SRP 3.7.1. The comparison plots of the response spectra of the time histories versus the HRHF response spectra for 2, 3, 4, 5, 7, and 10 percent critical dampings are shown in Figures 3.7-17, 3.7-18, and 3.7-19. The figures demonstrate that the time histories envelop the HRHF response spectra for those damping values, satisfying the requirement of SRP 3.7.1 that no more than 5 points fall below and by no more than 10 percent below the HRHF response spectra.

According to SRP 3.7.1, the ratio V/A and AD/V^2 should be consistent with characteristic values for the magnitude and distance of the appropriate controlling events defining the uniform hazard response spectra. The target and target ranges of values for the other design ground-motion time-history parameters are the median (m) values and the median (m) ± one standard deviation (σ) (i.e., m ± σ) ranges. The determination of these target and target ranges of values is based on the methodologies and ground motion databases as described in NUREG/CR-6728. Table 3.7-4 shows comparison of the ratio V/A and AD/V^2 for the time histories with the guidance in NUREG/CR-6728 and that the ratios are between the target values, target median ± σ.

For the development of the HRHF-response spectra-compatible target PSDs in the frequency range from 0.3 to 80 Hz, the time-history simulation method described in NUREG/CR-5347 is used. The resulting piecewise log-log linear horizontal and vertical target PSD developed is given in Tables 3.7-5 and 3.7-6. The minimum required horizontal and vertical PSD is then 0.8 times the horizontal and vertical target PSD.

The PSDs of the acceleration time histories compatible with the HRHF response spectra are presented in Figures 3.7-20 through 3.7-22. The PSDs of the acceleration time histories exceed the minimum required PSD throughout the entire frequency range.

The evaluation methodology and results of the APR1400 for the HRHF seismic input motions are provided in Appendix 3.7B.

3.7.1.2 Percentage of Critical Damping Values

Damping values used for various nuclear safety-related SSCs are based on NRC RG 1.61 (Reference 10). These values are expressed in percentages of critical damping and are given in Table 3.7-7. Damping values of soil to be used in soil-structure interaction

analysis are obtained from generic modulus reduction and hysteretic damping curves recommended by EPRI TR-102293 (Reference 11) based on site response analysis of soil columns for the standard plant profiles considering shear strains compatibility.

3.7.1.3 Supporting Media for Seismic Category I Structures

Seismic Category I structures are founded directly on rock or competent soil. The nuclear island and emergency diesel generator building correspond to the seismic Category I structures of the APR1400 standard plant design. The nuclear island consists of the following seismic Category I structures, the reactor containment building and the auxiliary building, which are founded on a common basemat. The emergency diesel generator building and a diesel fuel oil storage tank room are also seismic Category I structures. The foundation embedment depth, foundation size, and total height of the seismic Category I structures are presented in Table 3.7-8.

For the design of seismic Category I structures, nine soil profiles and one fixed base condition are established with various shear wave velocities compared with soil depth.

The supporting media for the generic site are described in Appendix 3.7A about soil properties, layering characteristics, shear wave velocity, shear modulus, and density. Basically, soil-structure interaction analyses on soil sites for the APR1400 use the soil degradation curves recommended by EPRI TR-102293. The curves are used to generate the strain-compatible soil properties.

These nine profiles are considered representative to envelop sites where competent soil is defined by the shear wave velocity of the supporting medium at the foundation level exceeding 304.8 m/sec (1,000 ft/sec). The shear wave velocity profiles of the nine sites considered are shown in Figure 3.7-23. The nine soil profiles, S1 through S9, are developed as combinations of six soil layering categories, which are designated as 55, 100, 200, 500, 1,000 ft, and half-space, and five average-shear-wave-velocity categories, namely, 1,200, 2,000, 4,000, 6,000, and 9,200 ft/sec. The generic site soil profiles are described further in Technical Report, APR1400-E-S-NR-13001-P (Reference 9).

3.7.2 Seismic System Analysis

This subsection describes the seismic analysis methods and models for the seismic Category I structures of the APR1400 standard plant design such as the reactor containment building, auxiliary building and emergency diesel generator building. The three-dimensional finite element models (FEMs) having an adequate number of discrete mass degrees of freedom are developed to capture the global and local translational, rocking, and torsional responses of the structures.

The COL applicant is to provide the seismic design of seismic Category I SSCs that are part of the APR1400 standard plant design (COL 3.7(3)). The seismic Category I structures are as follows:

a. Seismic Category I essential service water pump house

b. Seismic Category I component cooling water heat exchanger building

3.7.2.1 Seismic Analysis Methods

Seismic Category I SSCs are identified in Section 3.2. The safety-related structures are modeled as three-dimensional FEMs for the seismic analysis. Table 3.7-9 summarizes the types of models, computer programs, and analysis methods that are used in the seismic analyses of the seismic Category I structures, as well as the purposes of the dynamic analyses. Figures 3.7-24, 3.7-25, 3.7-26, and 3.7-27 show FEMs of the safety-related structures for the reactor containment building containment structure, reactor containment building internal structure, auxiliary building, and emergency diesel generator building.

Further details of dynamic modeling of building structures for seismic analysis are described in Subsection 3.7.2.3. The structural model is analyzed with a separate consideration of the excitation in each of the three orthogonal directions, E-W, N-S, and vertical. The results are then combined as described in Subsection 3.7.2.6. The seismic analysis of the above structures is performed by one of the following methods described below.

3.7.2.1.1 Response Spectrum Analysis

The response of a multi-degree-of-freedom system subjected to seismic excitation is represented by the following equation of motion:

$$[M]\left\{\{\ddot{X}\} + \{\ddot{U}_g\}\right\} + [C]\{\dot{X}\} + [K]\{X\} = 0$$

Where:

$[M]$ = mass matrix $(n \times n)$

$[C]$ = damping matrix $(n \times n)$

$[K]$ = stiffness matrix $(n \times n)$

$\{X\}\{X\}$ = column vector of relative displacements $(n \times 1)$

$\{\dot{X}\}$ = column vector of relative velocities $(n \times 1)$

$\{\ddot{X}\}$ = column vector of relative accelerations $(n \times 1)$

n = number of dynamic degrees of freedom

$\{\ddot{U}_g\}$ = column vector of ground accelerations $(n \times 1)$

In the response spectrum analysis, the equations of motion are decoupled using the transformation:

$$\{X\}=\{\varphi\} \ \{Y\}$$

Where:

$\{\varphi\}$ = mode shape matrix

$\{Y\}$ = vector of normal, or generalized coordinates $(m \times 1)$

m = number of modes considered

The decoupled equation of motion for each mode is transformed to a single degree of freedom system:

$$\ddot{Y}_j + 2\lambda_j\omega_j\dot{Y}_j + \omega_j^2 Y_j = -\Gamma_j\ddot{U}_g$$

Where:

Y_j = generalized coordinate of the jth mode

λ_j = damping ratio for the jth mode expressed as fraction of critical damping

ω_j = circular frequency of the jth mode of the system

Γ_j = modal participation factor of the jth mode $= \dfrac{\{\emptyset_j\}^T\{M\}\{1\}}{\{\emptyset_j\}^T\{M\}\{\emptyset_j\}}$

The generalized maximum response of each mode is determined from:

$$Y_j(\max) = \Gamma_j \frac{S_{aj}}{\omega_j^2}$$

Where:

S_{aj} is the spectral acceleration corresponding to frequency ω_j.

The maximum displacement at node i relative to the base due to mode j is:

$$X_{ij}(\max) = \phi_{ij} Y_j(\max)$$

The modal response X_{ij} (max) is used to determine other modal response quantities, such as forces. The modal combination method is used to obtain the final response by the methods described in Subsection 3.7.2.7.

Response spectrum analysis is used to compute only the seismic design forces of the containment structure and internal structure in the reactor containment building using the in-structure response spectra at the top of basemat generated from seismic soil-structure interaction analysis. The seismic response forces obtained from the response spectrum analysis are then combined with other design loads to design structural members of the containment structure and internal structure.

3.7.2.1.2 Time History Methods

The solution of the equation of motion given in Subsection 3.7.2.1.1 is obtained using one of three methods: modal superposition, direct integration, or complex frequency response in the frequency domain.

The method utilizes mode superposition or direct integration for time history analysis and is used for as an alternative analysis option for seismic Category I systems and subsystems. The seismic responses of the systems and subsystems that are seismic Category I SSCs are obtained using the finite element method. The analyses of all of the systems are performed for three orthogonal (two horizontal and one vertical) components of in-structure response time histories at the points of attachment.

Modal Superposition Method

The modal superposition method is used when the equations of motion can be decoupled as given in Subsection 3.7.2.1.1. Then the decoupled equation of motion for each mode is integrated using a proven technique, such as those listed in Table 3.2-1 of ASCE 4-98 (Reference 12) and the total response is obtained by superposition method.

Direct Integration Method

In this method, the direct integration of the equations of motion by implicit or explicit methods of numerical integration is used to solve the equations of motion. In general implicit methods, ΔT is not larger than 1/10 of the shortest period of interest. The direct integration method is used to validate coarse mesh model to be used in the seismic analysis of the nuclear island structures versus fine mesh model under the fixed-base condition.

Complex Frequency Response Method

The equation of motion can also be solved in the frequency domain using the complex frequency response method. In this method, the transfer functions are first determined and the applied forces are then transformed into the frequency domain. The fast Fourier transform (FFT) algorithm is commonly used for the transformation between the time domain and frequency domain. To facilitate the FFT operation, the total number of

digitized points of the excitation time history that is used is a power of 2, which can be achieved by a process known as zero padding, which involves adding trailing zeros to the input ground motion. For damped systems, the trailing zeros also serve as a quiet zone, which allows the transient response motions to die out at the end of the duration to avoid cyclic overlapping in the discrete Fourier transform procedure.

The seismic responses of seismic Category I structures are obtained from site-independent analyses performed using three-dimensional soil-structure interaction models with the program ACS SASSI (Reference 13), which utilizes a time history analysis in the frequency domain with substructuring techniques and complex stiffness representation of stiffness and damping properties of the structures and subgrade. The global complex stiffness matrix of the structure is assembled from the stiffness matrices of the finite elements.

With the substructuring techniques, impedance, load vector, and complex dynamic stiffness matrices are developed separately for the structure and the subgrade. The input ground motion is transformed into the frequency domain using FFT. The equations of motion for the complex soil-structure interaction system are then developed by combining the equations of motion for the structure with those of the subgrade in the frequency domain. The seismic response is obtained in the frequency domain from solution of complex algebraic equations for a selected set of frequencies of analysis. The analysis solutions obtained for the selected set of frequencies are then interpolated and transformed into the time domain using inverse FFT.

3.7.2.2 Natural Frequencies and Responses

The modal analyses of the FEMs representing the seismic Category I structures are performed separately using the ANSYS (Reference 14) or GTSTRUDL (Reference 15) computer program. A total of 529 modes and their frequencies from the modal analysis of the FEM for the reactor containment building are computed. Figures 3.7-28, 3.7-29, and 3.7-30 show the first major X-, Y-, and Z-mode shapes of the containment structure, respectively, and modal frequencies and participating mass ratios are summarized in Table 3.7-10. Figures 3.7-31, 3.7-32, and 3.7-33 show the first major X-, Y- and Z-mode shapes of the internal structure, respectively, and modal frequencies and participating mass ratios are summarized in Table 3.7-11.

A total of 2,500 modes and their frequencies from the modal analysis of the FEM for the auxiliary building are computed. Figures 3.7-34, 3.7-35, and 3.7-36 show the first major X-, Y-, and Z-mode shapes of the auxiliary building, respectively, and modal frequencies and participating mass ratios are summarized in Table 3.7-12.

A total of 150 modes and their frequencies from the modal analysis of the FEM for the emergency diesel generator building are computed. Figures 3.7-37, 3.7-38, and 3.7-39 show the first major X-, Y-, and Z-mode shapes, respectively, and modal frequencies and participating mass ratios are summarized in Table 3.7-13.

The soil-structure interaction analyses for nine soil profiles developed to represent generic site conditions and one fixed-base analysis are performed for the seismic Category I structures. The final analysis results are obtained by enveloping both soil-structure interaction analysis results and fixed-base analysis results.

The seismic responses maximum absolute nodal accelerations, maximum displacements relative to the top of foundation mat, and maximum member forces for the reactor containment building containment structure, reactor containment building internal structure, auxiliary building, and emergency diesel generator building are presented in Tables 3.7-14 through 3.7-25.

3.7.2.3 Procedures Used for Analytical Modeling

3.7.2.3.1 Designation of Systems Versus Subsystems

The calculation of the dynamic response of a nuclear power plant subject to an earthquake loading is divided into two categories. The first is the safety-related main structural system and the second is the safety-related subsystem. The safety-related main structural system category refers to the analysis of standard plant buildings and structures that house and/or support safety-related systems. The safety-related subsystems category refers to smaller safety-related SSCs supported by the safety-related main structural systems.

The safety-related structures that are analyzed in the main structural system analysis are:

a. Reactor containment building prestressed concrete containment structure and reinforced concrete internal structure

b. Reinforced concrete auxiliary building

c. Reinforced concrete emergency diesel generator building

3.7.2.3.2 Decoupling Criteria for Subsystems

As recommended in the SRP 3.7.2 (Reference 16), the following decoupling criteria are used when a subsystem needs to be modeled:

a. If $R_m < 0.01$, decoupling is performed for any R_f

b. If $0.01 < R_m < 0.1$, decoupling is performed if $R_f \leq 0.8$ or $R_f \geq 1.25$

c. If $R_m > 0.1$, an approximate dynamic model of the subsystem is included in the main structural system

Where:

$$R_m = \frac{\text{total mass of the supported subsystem}}{\text{total mass of the supporting system}}$$

$$R_f = \frac{\text{fundamental frequency of the supported subsytem}}{\text{dominant frequency of the support motion}}$$

In general, most subsystems such as equipment and piping, with the exception of the reactor coolant system (RCS) and of the polar crane in reactor containment building, are decoupled from the floor that supports them.

3.7.2.3.3 Modeling of Safety-related Structures

Safety-related structures are modeled as three-dimensional FEMs. Major structural element systems such as floor slabs, foundation mat, roof slab, shear walls, and main

frames are included in the FEM. All subsystems such as equipment and piping are considered in accordance with the decoupling criteria described in Subsection 3.7.2.3.2. For all seismic analyses, the dead load, live load, and attachment load to piping, cable tray and miscellaneous equipment mass are assumed to contribute to the inertial forces. In addition, mass associated with all heavy equipment is also included in the computation of floor or wall masses.

The reactor containment building consists of the containment structure, internal structure, and RCS. The structures in the reactor containment building share a mat foundation with the auxiliary building. The containment structure is a prestressed concrete structure and the internal structures are made of reinforced concrete. The RCS is connected to the internal structure. Appendix 3A provides a detailed description of the RCS structural model. Each substructure is modeled separately, and they are then combined into a total model for the seismic analysis.

3.7.2.3.3.1 Models for Nuclear Island Structures

The models for seismic excitation of the safety-related structures for the APR1400 standard plant design consist of the following structures:

a. Reactor containment building containment structure

b. Reactor containment building internal structure

c. Auxiliary building

The modeling approach that is used for the reactor containment building and auxiliary building structural model consists of developing a three-dimensional FEM. The seismic analysis models of the containment structure, internal structure, and auxiliary building are shown in Figures 3.7-24, 3.7-25, and 3.7-26, respectively.

The detailed procedure for the development of the seismic analysis models regarding the nuclear island structures is provided in Technical Report, APR1400-E-S-NR-13002-P (Reference 17).

FEM of the Reactor Containment Building Containment Structure

The reactor containment building containment structure is a post-tensioned concrete structure that consists of a cylindrical wall and hemispherical dome with three buttresses spaced 120 degrees apart circumferentially. The reactor containment building containment structure is modeled using quadrilateral shell elements as shown in Figure 3.7-24.

FEM of the Reactor Containment Building Internal Structure

For the reactor containment building internal structure, the load-resisting elements consist of reinforced concrete walls. These walls are modeled with quadrilateral shell elements or 8-node solid elements depending on the thickness of the walls. Concrete slabs are modeled with quadrilateral shell elements. The internal structure basemat is modeled with 8-node solid and 4-node tetrahedral solid elements. Figure 3.7-25 shows the FEM of the reactor containment building internal structure.

FEM of the Auxiliary Building

The auxiliary building is composed of rectangular reinforced concrete walls and reinforced concrete slabs, which are lateral load-resisting systems and frames that support the vertical loads. The walls and slabs are modeled with quadrilateral shell elements. The frames are modeled with beam elements. Figure 3.7-26 shows the FEM of the auxiliary building.

Combined Model of Nuclear Island Structures

The dynamic model of the RCS is coupled to the internal structure FEM. The nuclear island structures share one common basemat, the reactor containment building and auxiliary building structural models are combined with each other on the common basemat to form a combined model used for the seismic analysis of the nuclear island structures.

3.7.2.3.3.2 Model for Emergency Diesel Generator Building

The emergency diesel generator building consists of a building structure to accommodate the emergency diesel generator and diesel fuel oil storage tank room, which is separated

from the building structure. The emergency diesel generator building and diesel fuel oil storage tank room FEMs are individually developed following the procedure used in the development of the auxiliary building model. Figure 3.7-27 shows FEM of the emergency diesel generator building.

3.7.2.3.4 Modeling for Three Component Input Motions

The three-dimensional FEMs of the safety-related structures as described in Subsection 3.7.2.3.3 are used in separate analyses for each of the three components of design time histories prescribed on the surface of the finished grade.

3.7.2.4 Soil-Structure Interaction

The soil-structure interaction analysis of the seismic Category I structures is performed using the substructuring method formulated in the frequency domain using the complex response method and the finite element method. For the soil-structure interaction analysis, the methodology of the ACS SASSI computer program (Reference 13) is used. ACS SASSI consists of a number of program modules used to solve dynamic soil-structure interaction problems in a seismic environment. The flexible volume method (the so-called direct method) of the ACS SASSI analysis methodology is used. For this analysis, the seismic environment is defined by vertically propagating S-wave for the horizontal excitation and by vertically propagating P-wave for the vertical excitation.

In a substructuring method, the soil strata and half-space are assembled and condensed first for computation of transfer functions in the frequency domain. From this, the impedances at the soil structure interface are established. Subsequently, the impedances are combined with a model of the structure, the control motion is applied to the combined system, and the equations of motion are solved for computation of final accelerations and displacements.

A detailed description of the seismic responses obtained from the soil-structure interaction analysis for the seismic Category I structures is summarized in Appendix 3.7A.

3.7.2.5 Development of In-structure Response Spectra

The time history analysis is used to generate the floor response spectra at wall and floor locations in the FEMs. The spectra are generated according to the procedure given in NRC RG 1.122 (Reference 18). As described in Subsection 3.7.2.3.4, the amplified responses spectra in horizontal and vertical directions are obtained by three separate analyses. The response spectra resulting from the combined effect of both horizontal and one vertical seismic input motions are obtained by using the square root of the sum of the squares (SRSS) method. The effects of vertical floor flexibility are explicitly included.

The spectra are generated for appropriate critical damping values. The peaks of the response spectra are broadened as described in NRC RG 1.122 and are referred to as in-structure response spectra (ISRS).

3.7.2.6 Three Components of Earthquake Motion

For dynamic analyses of the seismic Category I structures, three statistically independent orthogonal components of earthquake motion, two horizontal and one vertical, are applied to the structural models as separate loading cases. The models are analyzed using either the time history analysis method in time domain or frequency domain, or response spectrum analysis methods as appropriate. For the time history analysis, the total response can be obtained by algebraically summing the response parameters. For the response spectrum analysis, the total response of the structure due to the three input seismic motions is obtained by combining the directional responses using the SRSS method.

The peak responses due to the three earthquake components from the response spectrum analysis can also be combined using 100-40-40 percent rule, as described in NRC RG 1.92 (Reference 19). The 100-40-40 percent rule is based on the observation that the maximum increase in the resultant for two orthogonal forces occurs when these forces are equal. The maximum value is 1.4 times of one component. All possible combinations of the three orthogonal responses are considered. The 100-40-40 combination is expressed mathematically as:

$R = (\pm 1.0RX \pm 0.4RY \pm 0.4RZ)$, or

$R = (\pm 0.4RX \pm 1.0RY \pm 0.4RZ)$, or

$R = (\pm 0.4RX \ \pm \ 0.4RY \ \pm \ 1.0RZ).$

The 100-40-40 percent rule may apply to combining spatial components of responses in the same direction due to different components of motion.

3.7.2.7 Combination of Modal Responses

The total seismic response of a structure to an input response spectrum loading is obtained by combining the response of each individual mode of the structure in accordance with the requirements of NRC RG 1.92 (Reference 19). If the modes are not closely spaced, the significant modes are combined using the SRSS of the corresponding maximum values of the response of each element of the structure. This is expressed mathematically as:

$$R = (\sum_{K=1}^{N} R_K^2)^{1/2}$$

where R is the maximum response of a given element, R_K is the peak response of the element due to the Kth mode, and N is the number of significant modes. If some of the modes are closely spaced, the response of the individual modes is combined using the ten percent method from NRC RG 1.92. This can be expressed as:

$$R = (\sum_{K=1}^{N} R_K^2 + 2\sum |R_i R_j|)^{1/2}, \ i \neq j$$

where R, R_K and N are as previously defined. The second summation is performed on all i and j modes whose frequencies are closely spaced to one another.

The definition of modes with closely spaced frequencies is a function of the critical damping ratio as follows:

a. For critical damping ratios ≤ 2 percent, modes are considered closely spaced if the frequencies are within 10 percent of each other (i.e., for $f_i < f_j$, $f_j \leq 1.1\ f_i$).

b. For critical damping ratios > 2 percent, modes are considered closely spaced if the frequencies are within five times the critical damping ratio of each other (i.e., for $f_i < f_j$ and 5 percent damping, $f_j \leq 1.25\ f_i$; for $f_i < f_j$ and 10 percent damping, $f_j \leq 1.5\ f_i$).

The effect of missing mass for modes not included in the analysis is accounted for by using the method given in NRC RG 1.92.

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

The interfaces between seismic Category I and non-seismic Category I structures are designed for the dynamic loads and displacements produced by both the seismic Category I and non-seismic Category I structures.

To provide reasonable assurance that the failure of a non-seismic Category I structure under the effect of a seismic event does not impair the integrity of an adjacent seismic Category I structure, one of the following procedures is followed:

a. Maintenance of sufficient separation between non-seismic Category I structures and seismic Category I structures

b. Analysis and design of non-seismic Category I structures to prevent their failure under SSE conditions in such a manner that the margins of safety of these structures are equivalent to those of seismic Category I structures

c. Design of seismic Category I structures to withstand loads due to the collapse of the adjacent non-seismic Category I structures if sufficient spatial separation is not achieved

The turbine generator building and compound building are classified as non-seismic Category I structures and are located on the west side and south side of nuclear island with a 1.0m (3 ft) gap, respectively. Figures 3.7-40 and 3.7-41 show the FEMs of the turbine generator building and compound building, respectively. To evaluate the structure-soil-structure interaction effects on the nuclear island structures due to presence of adjacent non-seismic Category I structures, the structure-soil-structure interaction analysis using the coupled model for entire structures is performed. The interaction effects of these non-seismic Category I structures on the nuclear island are negligible as provided in Technical Report, APR1400-E-S-NR-13005-P (Reference 20).

The COL applicant is to confirm that the any site-specific non-seismic Category I SSCs are designed not to degrade the function of a seismic Category I SSC to an unacceptable safety level due to their structural failure or interaction (COL 3.7(4)).

3.7.2.9 Effects of Parameter Variations on Floor Response Spectra

To consider variations in the structural frequencies due to the uncertainties in material properties of the structure and approximations in modeling, the peaks of the computed floor response spectra are broadened by ± 15 percent and smoothed in accordance with NRC RG 1.122, as described in Subsection 3.7.2.5.

The effects of potential concrete cracking on the structural stiffness of reinforced concrete structures are considered as enveloping the floor response spectra for cracked concrete properties with 7 percent damping for the reinforced concrete structures and those for uncracked properties with 4 percent damping for the reinforced concrete structures.

Both uncracked and cracked concrete stiffnesses are considered separately in the seismic analysis models of the seismic Category I structures. For consideration of potential concrete cracking, the cracked concrete stiffness in horizontal and vertical seismic analysis models is reduced by half of the uncracked concrete stiffness except prestressed concrete containment structure and reinforced concrete columns and walls in the vertical models described in ASCE/SEI 43-05 (Reference 21). Therefore, for nine soil profiles and one fixed-base condition, a total of 20 analysis cases are performed in the seismic analysis to generate the floor response spectra of the seismic Category I structures.

The selected locations where the floor response spectra are obtained in analysis models and resultant floor response spectra enveloping the 20 analysis cases for the nuclear island structures are provided in Technical Report, APR1400-E-S-NR-13003-P (Reference 22).

3.7.2.10 Use of Constant Vertical Static Factors

The safety-related structures, systems, and components are analyzed in the vertical direction using the methods described in Subsection 3.7.2.1. The vertical component is considered to occur simultaneously with the two horizontal components and consistently combined with the horizontal components of the seismic motion as described in Subsection

3.7.2.6. Therefore, a constant vertical static factor is not used for the seismic design of seismic Category I SSCs.

3.7.2.11 Methods Used To Account for Torsional Effects

Because the structural models used for seismic Category I structures are constructed with finite elements containing 6 degrees of freedom per node, incorporating torsional effects into the models, the mathematical models include sufficient mass points and corresponding dynamic degrees of freedom to provide a three-dimensional representation of the dynamic characteristics of the structure.

Torsional effects are also accounted for in the structural models used to generate floor response spectra. An additional eccentricity of 5 percent of the maximum building dimension, perpendicular to load direction that results in an accidental torque, is applied to the static finite element structural model to calculate element forces due to accidental torsion. Accidental torsion is considered in both the E-W and N-S directions.

3.7.2.12 Comparison of Responses

Since only the time history analysis method, including the complex frequency response method, is used for the seismic Category I structures, comparison of the responses between the time history analysis method and the response spectrum method is not applicable.

3.7.2.13 Methods for Seismic Analysis of Dams

The COL applicant is to perform seismic analysis for any site-specific seismic Category I dams, if necessary (COL 3.7(5)).

3.7.2.14 Determination of Dynamic Stability of Seismic Category I Structures

The design overturning moments and base shears for seismic Category I structures are determined by time history analysis. The seismic motion is input separately to the structural models in three independent orthogonal directions. To check the overturning and sliding, the simultaneous action of horizontal and vertical seismic forces using methods described in Subsection 3.7.2.6 is incorporated.

The procedure to check the stability of seismic Category I structures is described in Subsection 3.8.5.

3.7.2.15 Analysis Procedure for Damping

For modal superposition method, composite modal damping values are used for structures with components of different damping characteristics. The composite modal damping values are based on weighting the damping factors according to the mass or the stiffness of each element. For the mass-weighted damping, the formulation is as follows:

$$\beta_{\mathrm{j}} = \frac{\sum_{\mathrm{i}=1}^{\mathrm{N}} \{\phi_{\mathrm{j}}\}^{T} \beta_{\mathrm{i}} \{\mathrm{M_i}\} \{\phi_{\mathrm{j}}\}}{\{\phi_{\mathrm{j}}\}^{T} [\mathrm{M}] \{\phi_{\mathrm{j}}\}}$$

Where:

N = total number of components

β_{j} = composite modal damping for mode j

β_{i} = critical modal damping associated with component i

ϕ_{j} = mode shape vector

$\{\mathrm{M_i}\}$ = subregion of mass matrix associated with component i

[M] = the mass matrix of the system

For the stiffness-weighted damping, the formulation is as follows:

$$\beta_{\mathrm{j}} = \frac{\sum_{\mathrm{i}=1}^{\mathrm{N}} \{\phi_{\mathrm{j}}\}^{T} \beta_{\mathrm{i}} \{K_{\mathrm{i}}\} \{\phi_{\mathrm{j}}\}}{\{\phi_{\mathrm{j}}\}^{T} [K] \{\phi_{\mathrm{j}}\}}$$

Where:

{Ki} = subregion of stiffness matrix associated with component i

[K] = the stiffness matrix of the system

For direct integration method, viscous damping proportional to the mass and stiffness matrix is used; thus

$$[C] = \alpha\ [M] + \beta\ [K]$$

where [C] is the damping matrix, [K] is the stiffness matrix, and [M] is the mass matrix. The values of α and β are selected so that the damping in the range of frequency of interest is approximately equal to the damping of the structure.

3.7.3 Seismic Subsystem Analysis

This subsection describes the seismic analysis methods for the APR1400 seismic Category I subsystems as civil structures that are not included in the main structural system, such as miscellaneous concrete and steel structures, buried piping, conduit, tunnel, dam, and above-ground tanks.

The seismic analysis of the seismic Category I mechanical subsystems, such as piping and equipment, is described in Section 3.9.

3.7.3.1 Seismic Analysis Methods

The seismic analysis of seismic Category I subsystems is performed using either the response spectrum analysis or time history analysis, as described in Subsection 3.7.2.1, or the equivalent static method described in Subsection 3.7.3.1.1.

3.7.3.1.1 Use of Equivalent Static Load Method of Analysis

In the seismic analyses of components, the equivalent static load method would be used if a dynamic analysis is not performed. The static seismic acceleration coefficient is equal to 1.5 times the peak g level in the applicable required response spectra. A value less than 1.5 times could be used if its conservatism and justification are verified. The equivalent seismic static load is the product of the equipment or component mass and the static seismic acceleration coefficient.

3.7.3.1.2 Determination of Number of Earthquake Cycles

The procedure used to account for the fatigue effect of cyclic motion associated with seismic excitation recognizes that the actual motion experienced during a seismic event consists of a single maximum or peak motion and some number of cycles of lesser magnitude. The total or cumulative usage factor can also be specified in terms of a finite number of cycles of the maximum or peak motion. Based on this consideration, seismic Category I subsystems, components, and equipment are designed for a total of two SSE events with 10 maximum stress cycles per event (20 full cycles of the maximum SSE stress range). Alternatively, an equivalent number of fractional vibratory cycles to that of 20 full SSE vibratory cycles may be used (but with an amplitude not less than one-third (1/3) of the maximum SSE amplitude) when derived in accordance with Appendix D of IEEE Std. 344-2004 (Reference 23).

3.7.3.2 Procedure Used for Analytical Modeling

The criteria and bases described in Subsection 3.7.2.3 are used to determine whether a component or structure will be analyzed as a subsystem. The modeling techniques incorporate either a single- or multi-degree of freedom subsystem consisting of discrete masses connected by spring elements. The associated damping coefficients are consistent with Table 3.7-7. The degree of complexity of each model is sufficient to accurately evaluate the dynamic behavior of the component.

3.7.3.3 Analysis Procedures for Damping

The analysis procedure used to account for the damping in subsystems complies with Subsections 3.7.1.2 and 3.7.2.15.

3.7.3.4 Three Components of Earthquake Motion

Seismic responses resulting from analysis of subsystems due to three components of earthquake motions are combined in the same manner as the seismic response resulting from the analysis of building structures as specified in Subsection 3.7.2.6.

3.7.3.5 Combination of Modal Responses

When a response spectrum method of analysis is used to analyze a subsystem, the maximum responses such as accelerations, shears, and moments in each mode are calculated independent of time. If the frequencies of the modes are well separated, the SRSS method of mode combination gives acceptable results; however, where the structural frequencies are not well separated, the modes are combined in accordance with NRC RG 1.92.

3.7.3.6 Use of Constant Vertical Static Factors

In general, seismic Category I subsystems are analyzed in the vertical direction using the methods specified in Subsection 3.7.3.1. No constant vertical static factors are used for subsystems.

3.7.3.7 Buried Seismic Category I Piping, Conduits, and Tunnels

During an earthquake, buried structures such as piping, conduits, and tunnels respond to various seismic waves propagating through the surrounding soil as well as to the dynamic differential movements of the buildings to which the structures are connected. The various waves associated with earthquake motion are P (compression) waves, S (shear) waves, and Rayleigh waves. The stresses in the buried structure are governed by the velocity and angle of incidence of these traveling waves. However, the wave types and their directions during an earthquake are very complex. For design purposes, the seismic-induced upper bound strains and corresponding stresses in the buried piping and concrete electrical ducts are calculated using expressions given by ASCE 4-98 (Reference 12).

Seismic design for buried seismic Category I structures takes the effect of wave propagation into consideration, based on the assumption that there is no movement of the buried structure remote from anchor points relative to the surrounding soil referred to in ASCE 4-98, Subsection 3.5.2. That is, the strain of the structure is the same as that of the surrounding soil medium, and the stress of the structure is calculated from the strain. Consideration of relative deformation between anchor points and the adjacent soil is applied to the design using the SRSS method for the three orthogonal stresses calculated from the relative displacements of the seismic analysis results.

The resistance effect of the surrounding soil for deformation or displacement of the buried structures, differential movement of the anchors, and shape or curvature changes of the bent

parts is taken into account in the analysis. The structures can be modeled by beam elements supported by an elastic foundation representing the stiffness of the adjacent soil.

Lateral dynamic soil pressure on buried seismic Category I structures is calculated in accordance with elastic theory by Wood referred to in ASCE 4-98, Subsection 3.5.3. The effect of underground water is considered by applying the equation proposed by Matuo and O'Hara based on the theory from Westergaard that is referred to in ASCE 4-98, Subsection C3.5.3.1.

The COL applicant is to perform a seismic analysis of buried seismic Category I piping, conduits, and tunnels (COL 3.7(6)).

3.7.3.8 Methods for Seismic Analysis of Category I Concrete Dams

The COL applicant is to perform seismic analysis for any site-specific seismic Category I dams, if required (COL 3.7(5)).

3.7.3.9 Methods for Seismic Analysis of Above-ground Tanks

Above-ground seismic Category I tanks are generally large, flat-bottomed, single-shell, free-standing cylindrical tanks anchored to reinforced concrete pads or directly on a building structure. Seismic analysis procedures address the issues described in NUREG/CR-1161RD (Reference 24), pages 28-30, based primarily on the methods of Haroun and Housner (Reference 25). The hydrodynamic mass effects following the procedures described in ASCE 4-98, Subsection 3.1.6, are considered in the seismic analysis model.

Because of the symmetry of these vertical tanks, the larger of the two horizontal earthquake components, if they are not equal in magnitude, is combined by the SRSS method with the vertical earthquake component.

The assessment of dynamic loading on storage tanks verifies stability of the tank wall against buckling behavior, accounting for hydrodynamic loads (impulsive and convective) and shell flexibility.

In the generation of dynamic loads, tanks are evaluated as filled, with consideration of convective (sloshing), impulsive (fluid-shell interaction), and rigid modes of behavior. For the convective mode, fluid damping is taken as 0.5 percent of critical damping in accordance with NRC RG 1.61. For the impulsive mode, structural (tank wall) damping is taken for the SSE, in accordance with SRP 3.7.3 (Reference 26). The effective mass, its location, and natural frequency for each mode of behavior are obtained from the equations and graphs in Haroun and Housner.

Using the site-specific foundation input response spectra developed at the base of the tank, spectral accelerations obtained for each mode at the appropriate damping and frequency are applied to the appropriate effective mass.

Structural adequacy of the anchorage provisions for the tank (e.g., anchor bolts, embedments) is developed assuming that the overturning moment on the tank is resisted only by compression in the shell and tension in the anchor bolts. The overturning moment at the base of the tank is computed as the sum of the flexible and rigid mode responses, each of which is the product of the applicable mass, height, and spectral acceleration.

The COL applicant is to perform seismic analysis for the seismic Category I above-ground tanks (COL 3.7(7)).

3.7.4 Seismic Instrumentation

3.7.4.1 Comparison with NRC RG 1.12

The proposed seismic instrumentation program is generally in accordance with NRC RG 1.12 (Reference 27) and NRC RG 1.166 (Reference 28) and consistent with the methodology used for seismic analysis that is described in Subsection 3.7.2. The seismic design of the APR1400 standard plant is based on site-independent seismic response analysis of basemats resting on generic supporting media that are the surface of elastic half-space that is subjected to the SSE input a control motion.

3.7.4.2 Location and Description of Instrumentation

State-of-the-art digital instrumentation that enables the prompt processing of data at the plant site is used. A triaxial time-history accelerometer is provided at each of the following locations. Two horizontal axes of the accelerometer are aligned with the plant north-south and east-west direction, as follows:

a. One at the finished grade on the competent foundation in the free-field.

b. Four in the nuclear island: one on the common basemat in reactor containment building, one on the common basemat in auxiliary building, one on the spring line near the polar crane, and one at the reactor containment building operating floor.

c. Two in the component cooling water heat exchanger building: one on the foundation mat and one at the roof.

3.7.4.3 Main Control Room Operator Notification

Activation of the seismic trigger causes an audible and visual annunciation in the main control room to alert the plant operator that an earthquake has occurred.

When a seismic event that exceeds the maximum acceleration of the OBE or design basis event (DBE) occurs, an audible and visible alarm in the I&C equipment room is initiated and the audible and visible alarms are annunciated in the main control room. The loss of power of the seismic cabinet is annunciated in the main control room.

3.7.4.4 Comparison of NRC RG 1.166

The seismic instrumentation and OBE exceedance checks meet the intent of NRC RG 1.166.

3.7.4.5 Instrument Surveillance

Each seismic instrument is demonstrated to be operable by the performance of the channel check, channel calibration, and channel functional test operations. The channel checks are performed every 2 weeks for the first 3 months of service after startup. After the initial 3-month period and three consecutive successful checks, the channel checks are performed

monthly. The channel calibration is performed during each refueling. The channel functional test is performed every 6 months (COL 3.7(8)).

3.7.4.6 Program Implementation

The COL applicant is to identify the implementation milestone for the seismic instrumentation implementation program.

3.7.5 Combined License Information

COL 3.7(1) The COL applicant is to determine the site-specific SSE and OBE that are applied to the seismic design of the site-specific seismic Category I and II SSCs and the basis for the plant shutdown. The COL applicant is also to verify the appropriateness of the site-specific SSE and OBE.

COL 3.7(2) The COL applicant is to confirm that the horizontal components of the SSE site-specific ground motion in the free-field at the foundation level of the structure satisfy a peak ground acceleration of at least 0.1g.

COL 3.7(3) The COL applicant is to provide the seismic design of the seismic Category I SSCs that are part of the APR1400 standard plant design. The seismic Category I structures are as follows:

a. Seismic Category I essential service water pump house

b. Seismic Category I component cooling water heat exchanger building

COL 3.7(4) The COL applicant is to confirm that the any site-specific non-seismic Category I SSCs are designed not to degrade the function of a seismic Category I SSC to an unacceptable safety level due to their structural failure or interaction.

COL 3.7(5) The COL applicant is to perform any site-specific seismic design for dams that is required.

COL 3.7(6) The COL applicant is to perform seismic analysis of buried seismic Category I piping, conduits, and tunnels.

COL 3.7(7) The COL applicant is to perform seismic analysis for the seismic Category I above-ground tanks.

COL 3.7(8) The COL applicant is to identify the implementation milestone for the seismic instrumentation implementation program.

3.7.6 References

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2. 10 CFR 50, Appendix S, “Earthquake Engineering Criteria for Nuclear Power Plants.”

3. NRC RG 1.60, “Design Response Spectra for Seismic Design of Nuclear Power Plants,” Rev. 1, U.S. Nuclear Regulatory Commission, December 1973.

4. NRC RG 1.208, “A Performance-based Approach to Define the Site-specific Earthquake Ground Motion,” Rev. 4, U.S. Nuclear Regulatory Commission, March 2007.

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18. NRC RG 1.122, "Development of Floor Design Response Spectra for Seismic Design of Floor-supported Equipment or Components," Rev. 1, U.S. Nuclear Regulatory Commission, February 1978.

19. NRC RG 1.92, "Combining Modal Responses and Spatial Components in Seismic Response Analysis," Rev. 3, U.S. Nuclear Regulatory Commission, September 2012.

20. APR1400 Technical Report, APR1400-E-S-NR-13005-P, "Evaluation of Structure-soil-structure Interaction (SSSI) Effects of the APR1400 Standard Plant," Rev. 0, September 2013.

21. ASCE/SEI 43-05, “Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities,” American Society of Civil Engineers, 2005.

22. APR1400 Technical Report, APR1400-E-S-NR-13003-P, “SSI Analysis Methodology and Results of NI Buildings of the APR1400 Standard Plant,” Rev. 0, September 2013.

23. IEEE Std. 344-2004 (Reaffirmed 2009), “Recommended Practice for Seismic Qualification of Class 1E Equipment for Nuclear Power Generating Stations,” Institute of Electrical and Electronics Egineers (IEEE), June 2005.

24. NUREG/CR-1161RD, “Recommended Revisions to Nuclear Regulatory Commission Seismic Design Criteria,” NUREG/CR-1161RD. Lawrence Livermore Laboratory, May 1980.

25. M.A. Haroun and G.W. Housner, “Seismic Design of Liquid Storage Tanks,” Journal of the Technical Councils of ASCE, Vol. 107, No. TC1, American Society of Civil Engineers, pp. 191-207, April 1981.

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27. NRC RG 1.12, “Nuclear Power Plant Instrumentation for Earthquakes,” Rev. 2, U.S. Nuclear Regulatory Commission, March 1997.

28. NRC RG 1.166, “Pre-earthquake Planning and Immediate Nuclear Power Plant Operator Postearthquake Actions,” U.S. Nuclear Regulatory Commission, March 1997.

Table 3.7-1

Spectral Amplitude of CSDRS for Control Points

Horizontal

Damping Ratio (%)	Amplification Factor for Control Points						
	0.1 Hz	0.2 Hz	0.25 Hz	2.5 Hz	9 Hz	25 Hz	50 Hz
2	0.0276	0.111	0.171	1.275	1.062	0.511	0.300
3	0.0254	0.102	0.159	1.125	0.939	0.498	0.300
4	0.0238	0.096	0.147	1.020	0.852	0.487	0.300
5	0.0226	0.090	0.141	0.939	0.783	0.479	0.300
7	0.0207	0.084	0.129	0.816	0.681	0.464	0.300
10	0.0188	0.075	0.117	0.684	0.570	0.464	0.300

Vertical

Damping Ratio (%)	Amplification Factor for Control Points						
	0.1 Hz	0.2 Hz	0.25 Hz	3.5 Hz	9 Hz	25 Hz	50 Hz
2	0.0184	0.075	0.114	1.215	1.062	0.511	0.300
3	0.0170	0.069	0.105	1.074	0.939	0.498	0.300
4	0.0159	0.063	0.099	0.972	0.852	0.487	0.300
5	0.0151	0.060	0.093	0.894	0.783	0.479	0.300
7	0.0138	0.057	0.087	0.777	0.681	0.464	0.300
10	0.0125	0.051	0.078	0.651	0.570	0.447	0.300

Table 3.7-2

Comparison of Ratios V/A and AD/V^2

Component	V/A, cm/sec/g (in/sec/g)	Target V/A, cm/sec/g (in/sec/g)			AD/V^2	Target AD/V^2		
		med-σ	median	med+σ		med-σ	median	med+σ
H1	148.08 (58.3)	87.63 (34.5)	121.92 (48.0)	169.67 (66.8)	6.2	4.2	6.0	8.6
H2	151.64 (59.7)	87.63 (34.5)	121.92 (48.0)	169.67 (66.8)	5.8	4.2	6.0	8.6
VT	160.02 (63.0)	87.63 (34.5)	121.92 (48.0)	169.67 (66.8)	4.2	4.2	6.0	8.6

Table 3.7-3

Target PSD Compatible with Horizontal CSDRS

Frequency (f) Range f (Hz or cps)	Piecewise Linear Target PSD $S_H(f)$ (in^2/sec^4/cps)
$0.2 < f \le 2.5$ Hz	$S_H(f) = 2\pi \times 58.4\ (f / 2.5)^{0.2}$
$2.5 < f \le 9.0$ Hz	$S_H(f) = 2\pi \times 58.4\ (2.5 / f)^{1.8}$
$9.0 < f \le 16$ Hz	$S_H(f) = 2\pi \times 5.82\ (9.0 / f)^{2.2}$
$16 < f \le 25$ Hz	$S_H(f) = 2\pi \times 1.64\ (16 / f)^{3.8}$
$25 < f \le 35$ Hz	$S_H(f) = 2\pi \times 0.30\ (25 / f)^{4.8}$
$35 < f \le 50$ Hz	$S_H(f) = 2\pi \times 0.06\ (35 / f)^{10}$

Table 3.7-4

Comparison of Ratios for HRHF Seismic Input Motions V/A and AD/V^2

Component	V/A, cm/sec/g (in/sec/g)	Target V/A, cm/sec/g (in/sec/g)			AD/V^2	Target AD/V^2		
		med-σ	median	med+σ		med-σ	median	med+σ
H1	48.62 (19.14)	30.43 (11.98)	42.34 (16.67)	58.88 (23.18)	8.26	3.92	6.14	9.63
H2	57.61 (22.68)	30.43 (11.98)	42.34 (16.67)	58.88 (23.18)	5.69	3.92	6.14	9.63
VT	54.53 (21.47)	30.43 (11.98)	42.34 (16.67)	58.88 (23.18)	7.53	3.92	6.14	9.63

Table 3.7-5

Target PSD Compatible with Horizontal HRHF Seismic Input Motions

Frequency (f) Range f (Hz or cps)	Piecewise Linear Target PSD $S_H(f)$ (in^2/sec^4/cps)
$0.3 < f \leq 1.5$ Hz	$S_0(f) = 2\pi \times 6.85\ (0.3 / f)^{-0.4}$
$1.5 < f \leq 4.0$ Hz	$S_0(f) = 2\pi \times 13.04\ (1.5 / f)^{-0.2}$
$4.0 < f \leq 19$ Hz	$S_0(f) = 2\pi \times 15.86\ (4.0 / f)^{0.25}$
$19 < f \leq 40$ Hz	$S_0(f) = 2\pi \times 10.75\ (19.0 / f)^{1.1}$
$40 < f \leq 55$ Hz	$S_0(f) = 2\pi \times 4.75\ (40.0 / f)^{2.3}$
$55 < f \leq 70$ Hz	$S_0(f) = 2\pi \times 2.28\ (55.0 / f)^{4.5}$
$70 < f \leq 80$ Hz	$S_0(f) = 2\pi \times 0.76\ (70.0 / f)^{7.1}$

Table 3.7-6

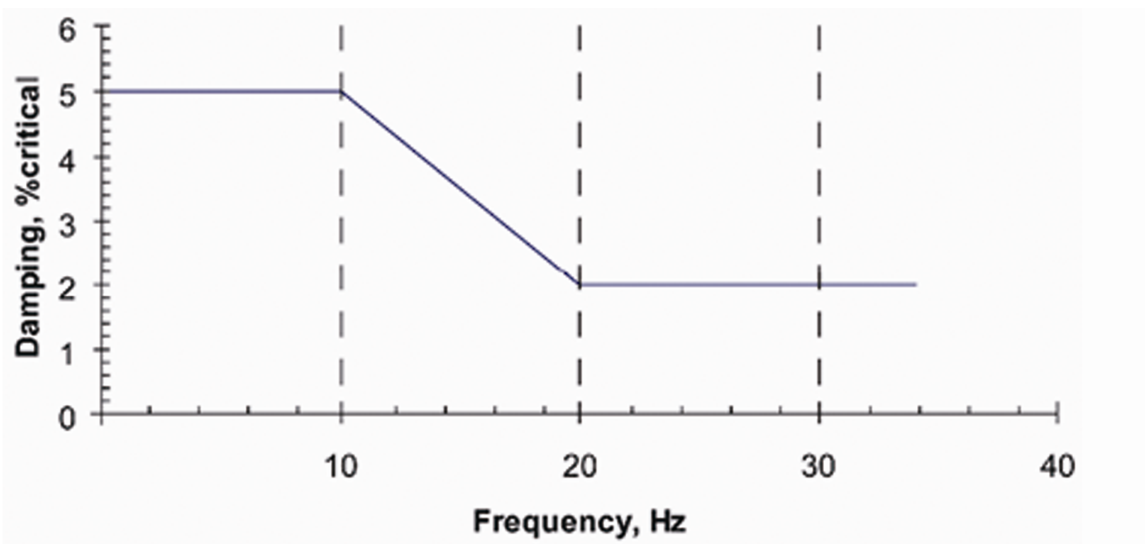
Target PSD Compatible with Vertical HRHF Seismic Input Motions

Frequency (f) Range f (Hz or cps)	Piecewise Linear Target PSD S_H (f) (in^2/sec^4/cps)
$0.3 < f \le 1.5$ Hz	$S_0(f) = 2\pi \times 3.44\ (0.3 / f)^{-0.5}$
$1.5 < f \le 4.0$ Hz	$S_0(f) = 2\pi \times 7.69\ (1.5 / f)^{-0.1}$
$4.0 < f \le 19$ Hz	$S_0(f) = 2\pi \times 8.49\ (4.0 / f)^{0.15}$
$19 < f \le 40$ Hz	$S_0(f) = 2\pi \times 6.72\ (19.0 / f)^{0.3}$
$40 < f \le 55$ Hz	$S_0(f) = 2\pi \times 5.38\ (40.0 / f)^{1.5}$
$55 < f \le 70$ Hz	$S_0(f) = 2\pi \times 3.34\ (55.0 / f)^{3.9}$
$70 < f \le 80$ Hz	$S_0(f) = 2\pi \times 1.31\ (70.0 / f)^{6.2}$

Table 3.7-7 (2 of 2)

(1) As an alternative for response spectrum analysis 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 NRC RG 1.61.
- 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 dissipated 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, 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 Regulatory Authority's review for acceptance on a case-by-case basis.

(3) Use 0.5 % damping for sloshing mode for tanks.

Table 3.7-7 (2 of 2)

(1) As an alternative for response spectrum analysis 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 NRC RG 1.61.
- 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.
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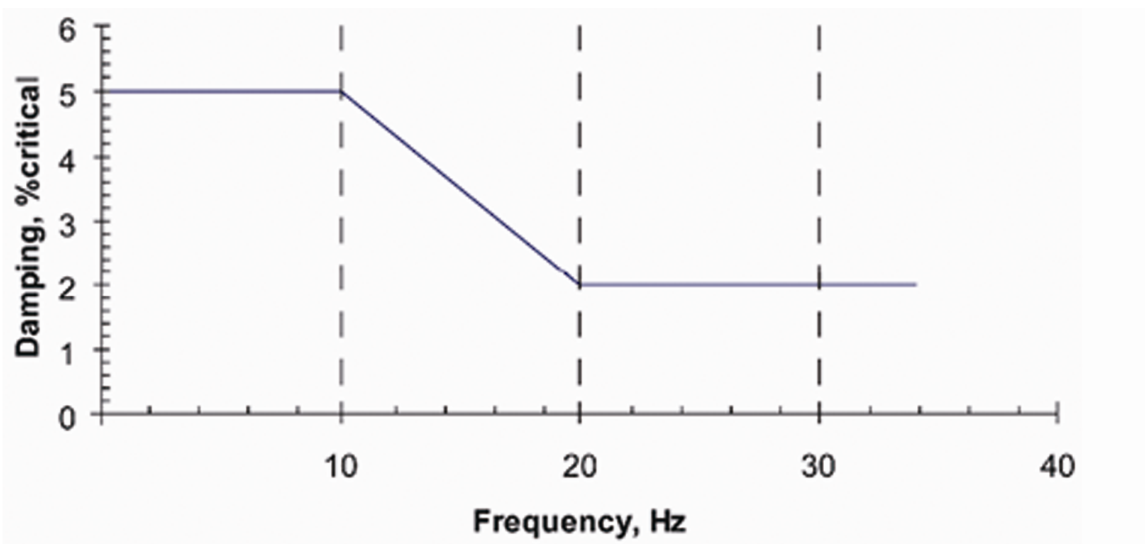

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(3) Use 0.5 % damping for sloshing mode for tanks.

Table 3.7-8

Foundation Embedment Depth, Foundation Size, and Total Height of Seismic Category I Structures

Structures	Foundation Embedment Depth, m (ft)	Foundation Size, m (ft)	Maximum Height, m (ft)
Nuclear Island – Reactor Containment Building – Auxiliary Building	See note 1. 16.4 (53′-8″)	Radius 25.6 (84′-0″) 107.3 × 88.1 (352′-0″ × 289′-0″)	87.9 (288′-6″) 56.4 (185′-0″)
Emergency Diesel Generator (EDG) Building	2.6 (8′-6″)	39.9 × 18.3 (131′-0” × 60′-0″)	17.8 (58′-6″)
Diesel Fuel Oil Storage Tank (DFOST) Room	1.2 (4′-0″)	20.3 × 18.3 (66′-6″ × 60′-0″)	18.7 (61′-6″)

(1) The auxiliary building wraps around the reactor containment building with a minimum of 2 inches seismic gap.

Table 3.7-9 (1 of 2)

Summary of Models and Analysis Methods

Model	Analysis Method	Program	Type of Dynamic Response/Purpose
Reactor containment building fine-mesh model	Modal analysis Direct integration time history analysis	ANSYS	To verify the mesh sizes of reactor containment building coarse-mesh model
Auxiliary building fine-mesh model	Modal analysis Direct integration time history analysis	ANSYS	To verify the mesh sizes of reactor containment building coarse-mesh model
Reactor containment building coarse-mesh model	Modal analysis Direct integration time history analysis	ANSYS	To create and verify SASSI reactor containment building model
Auxiliary building coarse-mesh model	Modal analysis Direct integration time history analysis	ANSYS	To create and verify SASSI auxiliary building model
SASSI reactor containment building model	Time history analysis Complex frequency response analysis	ACS SASSI	To create SASSI combined nuclear island model
SASSI auxiliary building model	Time history analysis Complex frequency response analysis	ACS SASSI	To create SASSI combined nuclear island model
SASSI combined nuclear island model	Time history analysis Complex frequency response analysis	ACS SASSI	Performed for nine generic soils and one fixed base case. To develop time histories for generating plant design in-structure response spectra. To obtain maximum absolute nodal acceleration. To obtain maximum displacements relative to basemat and free-field. To obtain maximum member forces and moments.

Table 3.7-9 (2 of 2)

Model	Analysis Method	Program	Type of Dynamic Response/Purpose
EDG building (include DFOST room) model	Modal analysis	GTSTRUDL	To create and verify SASSI EDG building (include DFOST room) model.
SASSI EDG building (include DFOST room) model	Time history analysis Complex frequency response analysis	ACS SASSI	Performed for nine generic soils and one fixed base case. To develop time histories for generating plant design in-structure response spectra. To obtain maximum absolute nodal acceleration. To obtain maximum displacements relative to basemat and free-field. To obtain maximum member forces and moments.

Table 3.7-10

Summary of Modal Properties of Reactor Containment Building
Containment Structure

Modes	Natural Frequency (Hz)	Mass Ratio	Direction
11	3.49	18.2 %	X (E-W)
12	3.58	15.7 %	Y (N-S)
30	10.60	22.5 %	Z (Vertical)
98	19.55	1.1 %	Drumming

Table 3.7-11

Summary of Modal Properties of Reactor Containment Building Internal Structure

Modes	Natural Frequency (Hz)	Mass Ratio	Direction
18	6.27	4.1 %	Y (N-S)
28	9.72	6.1 %	X (E-W)
25	8.61	1.5 %	Y (N-S)
29	9.93	5.1 %	X (E-W)
132	23.41	8.0 %	Z (Vertical)

Table 3.7-12

Summary of Modal Properties of Auxiliary Building

Modes	Natural Frequency (Hz)	Mass Ratio	Direction
1	5.35	37.5 %	Y (N-S)
2	5.16	18.5 %	X (E-W)
3	5.79	13.9 %	X (E-W)
4	5.16	12.3 %	Y (N-S)
5	5.35	10.7 %	X (E-W)

Table 3.7-13

Summary of Modal Properties of Emergency Diesel Generator Building

EDG Building

Modes	Natural Frequency (Hz)	Mass Ratio	Direction
1	11.79	61.3 %	X (E-W)
2	13.74	56.9 %	Y (N-S)
7	17.87	13.69 %	Z (Vertical)

DFOST Room

Modes	Natural Frequency (Hz)	Mass Ratio	Direction
1	13.35	31.87 %	X (E-W)
3	17.01	51.17 %	Y (N-S)
10	28.92	14.74 %	Z (Vertical)

Table 3.7-14

Maximum Response Accelerations of Reactor Containment Building
Containment Structure

Maximum Response Accelerations of Reactor Containment Building (g)				
Structure	Elevation (ft)	Direction		
		X	Y	Z
Containment Structure	78	0.386	0.368	0.402
	90	0.411	0.403	0.408
	104	0.466	0.483	0.418
	118	0.507	0.551	0.441
	124	0.542	0.569	0.458
	132	0.579	0.606	0.484
	160	0.731	0.686	0.619
	180	0.834	0.783	0.703
	196	0.904	0.838	0.768
	216	0.957	0.918	0.835
	241	1.110	1.033	0.917
	255	1.221	1.126	0.957
	275	1.219	1.255	1.006
	302	1.379	1.413	1.133
	328	1.521	1.534	1.336
	332	1.538	1.548	1.363

Table 3.7-15

Maximum Relative Displacements of Reactor Containment Building Containment Structure

Elevation (ft)	Maximum Displacements Relative to Basemat (in)		
	X-direction	Y-direction	Z-direction
78	0.0906	0.0965	0.3322
90	0.1806	0.1962	0.3429
104	0.3372	0.3239	0.3807
118	0.4908	0.4593	0.4161
124	0.5520	0.5243	0.4290
132	0.6339	0.6011	0.4480
160	0.9273	0.8712	0.5062
180	1.1415	1.0991	0.5454
196	1.3047	1.2676	0.5745
216	1.5239	1.4815	0.6057
241	1.7880	1.7381	0.6342
255	1.9243	1.8747	0.6454
275	2.1183	2.0807	0.6302
302	2.3529	2.3466	0.5183
332	2.6010	2.6096	0.1349

Table 3.7-16

Maximum Member Forces of Reactor Containment Building
Containment Structure

Seismic Member Force and Moment (kips, ft)						
Elevation (ft)	Fx	Fy	Fz	Mx	My	Mz
307.5	1.195E+04	1.224E+04	1.169E+04	1.762E+05	1.779E+05	1.130E+04
281	2.305E+04	2.365E+04	2.187E+04	7.281E+05	7.241E+05	3.730E+04
254.5	3.047E+04	3.147E+04	2.887E+04	1.562E+06	1.541E+06	6.021E+04
241	3.459E+04	3.586E+04	3.294E+04	2.102E+06	2.067E+06	7.395E+04
220	4.543E+04	4.722E+04	4.172E+04	3.222E+06	3.116E+06	1.034E+05
200	5.117E+04	5.327E+04	4.842E+04	4.326E+06	4.184E+06	1.247E+05
178	5.587E+04	5.823E+04	5.461E+04	5.634E+06	5.456E+06	1.432E+05
156	5.959E+04	6.205E+04	6.000E+04	7.023E+06	6.798E+06	1.640E+05
136	6.163E+04	6.554E+04	6.357E+04	8.315E+06	8.054E+06	1.779E+05
130	6.255E+04	6.716E+04	6.489E+04	8.713E+06	8.439E+06	1.837E+05
125	6.255E+04	6.716E+04	6.489E+04	9.037E+06	8.747E+06	1.837E+05
114	6.497E+04	7.007E+04	6.726E+04	9.777E+06	9.458E+06	1.929E+05
100	6.660E+04	7.202E+04	6.908E+04	1.071E+07	1.036E+07	1.989E+05
78	6.795E+04	7.382E+04	7.098E+04	1.223E+07	1.177E+07	2.037E+05

Table 3.7-17

Maximum Response Accelerations of Reactor Containment Building Internal Structure

Maximum Response Accelerations of Internal Structure (g)				
Structure	Elevation (ft)	Direction		
		X	Y	Z
Primary Shield Wall	66	0.396	0.354	0.418
	78	0.424	0.360	0.419
	100	0.476	0.378	0.442
	107	0.484	0.386	0.485
	114	0.505	0.423	0.499
	130	0.510	0.471	0.509
	137	0.539	0.546	0.538
	156	0.897	0.964	0.814
	191a	1.031	4.531	0.677
	191b	1.314	1.617	0.856
Secondary Shield Wall	78	0.407	0.363	0.430
	100a	0.475	0.403	0.465
	100b	0.470	0.420	0.393
	107	0.507	0.423	0.503
	114	0.545	0.484	0.541
	130	0.634	0.658	0.625
	137	0.678	0.721	0.643
	156	0.937	1.022	0.753
	191	2.060	4.475	0.918
Slabs	66	-	-	0.381
	100	-	-	0.800
	107	-	-	0.452
	111	-	-	0.858
	114	-	-	0.780
	125	-	-	0.768

191a : Primary Shield Wall (PSW) top
191b : Pressurizer Room Corners
100a : Secondary Shield Wall (SSW) interface with concrete pedestal top
100b : In-containment Refueling Water Storage Tank walls

Table 3.7-18

Maximum Relative Displacements of Reactor Containment Building Internal Structure

Structure	Elevation (ft)	Maximum Displacements Relative to Basemat (inches)		
		X-direction	Y-direction	Z-direction
Primary Shield Wall	66	0.0396	0.0435	0.1305
	78	0.0853	0.0906	0.1310
	100	0.1741	0.1825	0.1613
	107	0.2015	0.2103	0.1670
	114	0.2337	0.2449	0.1685
	130	0.2952	0.3119	0.1701
	137	0.3268	0.3499	0.1616
	156	0.4515	0.4838	0.2548
	191a	0.5719	1.5790	0.1755
	191b	0.6315	0.6136	0.2573
Secondary Shield Wall	78	0.0843	0.0939	0.2155
	100a	0.1754	0.1842	0.2247
	100b	0.1730	0.1792	0.2820
	107	0.2039	0.2134	0.2275
	114	0.2362	0.2494	0.2286
	130	0.3115	0.3294	0.2337
	137	0.3424	0.3622	0.2356
	156	0.4492	0.4672	0.2411
	191	0.6630	1.5572	0.2440

Table 3.7-19

Maximum Member Forces of Reactor Containment Building
Internal Structure

Seismic Member Force and Moment (kips, ft)							
Structure	Elevation (ft)	Fx	Fy	Fz	Mx	My	Mz
Primary Shield Wall	156	2.207E+03	4.327E+03	1.634E+03	1.043E+05	5.266E+04	5.207E+04
	136.5	5.137E+03	5.070E+03	4.599E+03	1.882E+05	1.885E+05	9.742E+04
	130	6.243E+03	5.756E+03	5.861E+03	2.139E+05	2.543E+05	1.177E+05
	114	8.532E+03	7.179E+03	8.580E+03	2.983E+05	4.092E+05	1.424E+05
	100	1.048E+04	8.982E+03	1.128E+04	3.991E+05	5.531E+05	1.566E+05
	78	1.181E+04	1.097E+04	1.385E+04	5.920E+05	7.697E+05	1.529E+05
	66	1.224E+04	1.152E+04	1.445E+04	7.244E+05	9.092E+05	1.520E+05
Secondary Shield Wall	191	2.409E+02	2.644E+02	1.394E+02	4.395E+03	6.691E+03	1.002E+04
	156	5.259E+03	5.172E+03	3.506E+03	1.580E+05	1.619E+05	7.571E+04
	136.5	1.045E+04	7.637E+03	8.318E+03	3.285E+05	4.248E+05	1.151E+05
	130	1.245E+04	8.644E+03	1.070E+04	4.007E+05	5.440E+05	1.306E+05
	114	1.395E+04	9.768E+03	1.275E+04	5.306E+05	7.640E+05	1.467E+05
	100	1.532E+04	1.104E+04	1.507E+04	6.847E+05	9.775E+05	1.626E+05
	78	2.229E+04	2.011E+04	2.517E+04	9.991E+05	1.460E+06	1.805E+05

Table 3.7-20

Maximum Response Accelerations of Auxiliary Building

Maximum Response Accelerations of Auxiliary Building (g)					
Shear Wall				Floor	
Elevation (ft)	Direction			Elevation (ft)	Direction
	X	Y	Z		Z
55	0.304	0.314	0.402	55	0.359
68	0.385	0.527	0.479	68	0.749
78	0.641	0.537	0.468	78	1.267
100	0.677	0.801	0.526	100	1.452
120	0.932	1.19	0.687	120	2.107
137.5	1.093	1.499	0.794	137.5	1.974
156	1.34	1.905	0.892	156	2.241
174	2.085	2.126	0.963	174	2.83
195	2.099	2.187	0.931	195	1.543
195a	2.067	1.312	0.884	195a	1.389
216.75	2.423	2.688	1.003	216.75	3.477
213	2.262	1.53	0.941	213	2.326
213.5	2.154	1.702	1.035	213.5	2.443
195b	1.826	1.52	1.013	195c	2.884
114	0.754	0.669	0.67	114	0.926

195a : Main Control Room Roof
195b : Fuel Handling Area
195c : Control Area 2 Emergency Exhaust ACU and Normal Exhaust ACU

Table 3.7-21

Maximum Relative Displacements of Auxiliary Building

Floor Label	Floor Elevation	Maximum Displacement Relative to Basemat (inches)			
		X-direction	Y-direction	Z-direction (VT)	Z-direction (VS)
1-F	55'-0"	0.0857	0.0762	0.2525	0.1864
1-M	68'-0"	0.0982	0.1446	0.2790	0.2446
2-F	78'-0"	0.1333	0.1674	0.2966	0.2804
3-F	100'-0"	0.2882	0.3747	0.3302	0.2154
4-F	120'-0"	0.4525	0.6397	0.3458	0.2581
5-F	137'-6"	0.5694	0.7879	0.3560	0.2947
6-F	156'-0"	0.6872	0.9541	0.3666	0.2622
7-F	174-'0"	0.8761	1.0461	0.3751	0.2679
8-1	195'-0"	0.9516	0.9842	0.2280	0.1590
8-M	195'-0"	0.9410	0.6651	0.1129	0.1161
8-2	216'-9"	1.0706	1.3213	0.2452	0.3583
8-3	213'-0"	1.0344	0.7538	0.1326	0.1704
8-4	213'-6"	0.9695	0.9637	0.3854	0.6291
8-5	195'-0"	0.8499	0.9060	0.3825	0.2353
3-M	114'-0"	0.3610	0.3703	0.2035	0.1468

Table 3.7-22

Maximum Member Forces of Auxiliary Building

Seismic Member Force and Moment (kips, ft)						
Elevation (ft)	Fx	Fy	Fz	Mx	My	Mz
213.5	5425	7174	2920	314174	197979	637117
213	14787	13126	5504	478184	711285	1502367
195	19263	17401	6656	765341	945752	1981261
195	10174	7661	5262	184377	550634	706529
174	68233	66361	30736	2430827	2518524	2257363
156	101579	111413	54516	4762013	4780395	3313104
137.5	143267	165299	88733	7929215	7729526	4421197
120	193300	222571	124868	12005966	11199955	5614660
98.5	240185	282559	162898	18111608	16597000	6657538
77	277236	335937	203768	25105085	22507797	7326407
67	293250	358622	225936	28399238	25433525	7608524
55	300911	369591	238280	32392109	28921247	7725579

Table 3.7-23

Maximum Response Accelerations of EDG Building

EDG Building

Slab Elevation	E-W(g)	N-S(g)	Vertical(g)
100′-0″	0.59	0.49	0.33
135′-0″	0.82	0.82	0.45

DFOST Room

Slab Elevation	E-W(g)	N-S(g)	Vertical(g)
63′-0″	0.48	0.37	0.38
100′-0″	0.81	0.90	0.45

Table 3.7-24

Maximum Relative Displacements of EDG Building

EDG Building

Elevation	E-W Direction (in)	N-S Direction (in)	Vertical Direction (ins)
135’-0”	0.139	0.085	0.003

DFOST Room

Elevation	E-W Direction (in)	N-S Direction (in)	Vertical Direction (ins)
100’-0”	0.075	0.081	0.004

Table 3.7-25

Maximum Member Forces of EDG Building

EDG Building

EDG	SHEAR FORCE AND MOMENT (kips, ft)					
ELEVATION	Fx	Fy	Fz	Mx	My	Mz
135’-0”	2860	2847	1576	50245	37590	19318
100’-0”	9995	8745	5578	347885	342807	66603

DFOST Room

DFOST	SHEAR FORCE AND MOMENT (kips, ft)					
ELEVATION	Fx	Fy	Fz	Mx	My	Mz
100’-0”	1051	1170	591	19570	15292	22701
63’-0”	4569	3865	3361	138428	155258	39058