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U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

ATTENTION: MR. T. R. QUAY

SUBJECT: SSAR SECTION 3.7

Dear Mr. Quay:

Enclosed is a preliminary copy of Section 3.7 of the AP600 SSAR text and revised tables and figures. It is provided as a preliminary copy for discussion at the meeting on seismic and structural issues during the week of June 12-16, 1995.

This section will be included in a future revision of the SSAR.

If you have any questions, please contact Brian A. McIntyre (412) 374-4334.

Sincerely,

N. J. Liparulo, Manager
Nuclear Safety Regulatory and Licensing Activities

/nja

Enclosure

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3.7 Seismic Design

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

Each plant structure, system, equipment, and component is classified in an applicable seismic category depending on its function. A three-level seismic classification system is used for the AP600: seismic Category I, seismic Category II, and nonseismic. The definitions of the seismic classifications and a seismic classifications listing of structures, systems, equipment, and components are presented in Section 3.2.

Seismic design of the AP600 seismic Categories I and II structures, systems, equipment, and components is based on the safe shutdown earthquake (SSE). The safe shutdown earthquake is defined as the maximum potential vibratory ground motion at the generic plant site as identified in Section 2.5.

The operating basis earthquake (OBE) has been eliminated as a design requirement for the AP600. Low-level seismic effects are 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 safe shutdown earthquake. Criteria for evaluating the need to shut down the plant following an earthquake are established using the cumulative absolute velocity approach according to EPRI Report NP-5930 (Reference 1) and EPRI Report TR-100082 (Reference 17).

Seismic Category I structures, systems, and components are designed to withstand the effects of the safe shutdown earthquake event and to maintain the specified design functions. Seismic Category II and nonseismic structures are designed or physically arranged (or both) so that the safe shutdown earthquake could not cause unacceptable structural interaction with or failure of seismic Category I structures, systems, and components.

3.7.1 Seismic Input

The geologic and seismologic considerations of the generic plant site are discussed in Section 2.5.

The peak ground acceleration of the safe shutdown earthquake has been established as 0.30g. The vertical peak ground acceleration is conservatively assumed to equal the horizontal value of 0.30g as discussed in Section 2.5.

3.7.1.1 Design Response Spectra

The AP600 design response spectra of the safe shutdown earthquake are provided in Figures 3.7.1-1 and 3.7.1-2 for the horizontal and the vertical components, respectively.

The horizontal design response spectra for the AP600 plant are developed, using the Regulatory Guide 1.60 spectra as the base and several evaluations to investigate the high frequency amplification effects. These evaluations included:

1) Comparison of Regulatory Guide 1.60 spectra with the spectra predicted by recent eastern U.S. spectral velocity attenuation relations (References 23, 24, 25, and 26) using a suite of magnitudes and distances giving a 0.3 g peak acceleration,

2) Comparison of Regulatory Guide 1.60 spectra with the 10^{-4} annual probability uniform hazard spectra developed for eastern U.S. nuclear power plants by both Lawrence Livermore National Laboratory (Reference 27) and Electric Power Research Institute (Reference 28), and

3) Comparison of Regulatory Guide 1.60 spectra with the spectra of 79 additional old and newer components of strong earthquake time histories not considered in the original derivation of Regulatory Guide 1.60.

Based on the above described evaluations, it is concluded that the eastern U.S. seismic data exceed Regulatory Guide 1.60 spectra by a modest amount in the 15 to 33 hertz frequency range when derived either from published attenuation relations or from the 10^{-4} annual probability of exceedance uniform hazard spectra at eastern U.S. sites. This conclusion is consistent with findings of other investigators that eastern North American earthquakes have more energy at high frequencies than western earthquakes. Exceedance of Regulatory Guide 1.60 spectra at the high frequency range, therefore, would be expected since Regulatory Guide 1.60 spectra are based primarily on western U.S. earthquakes. The evaluation shows that, at 25 hertz (approximately in the middle of the range of high frequencies being considered, and a frequency for which spectral amplitudes are explicitly evaluated) the mean-plus-one-standard-deviation spectral amplitudes for 5 percent damping range from about 2.1 to 4 cm/sec and average 2.7 cm/sec. Whereas, the Regulatory Guide 1.60 spectral amplitude at the same frequency and damping value equal just over 2 cm/sec.

It is concluded, therefore, that an appropriate augmented 5 percent damping horizontal design velocity response spectrum for the AP600 project is one with spectral amplitudes equal to the Regulatory Guide 1.60 spectrum at control frequencies 0.25, 2.5, 9 and 33 hertz augmented by an additional control frequency at 25 hertz with an amplitude equal to 3 cm/sec. This spectral amplitude equals 1.3 times the Regulatory Guide 1.60 amplitude at the same frequency. The additional control point's spectral amplitude of other damping values were determined by increasing the Regulatory Guide 1.60 spectral amplitude by 30 percent.

The AP600 design vertical response spectrum is, similarly, based on the Regulatory Guide 1.60 vertical spectra at lower frequencies but is augmented at the higher frequencies.

The AP600 design response spectra's relative values of spectrum amplification factors for control points are presented in Table 3.7.1-3.

The design response spectra are applied at the finished grade in the free field.

3.7.1.2 Design Time History

A "single" set of three mutually orthogonal, statistically independent, synthetic acceleration time histories is used as the input in the dynamic analysis of seismic Category I structures. The synthetic time histories were generated by modifying a set of actual recorded "TAFT" earthquake time histories. The design time histories include a total time duration equal to 20 seconds and a corresponding stationary phase, strong motion duration greater than 6 seconds. The acceleration, velocity, and displacement time-history plots for the three orthogonal earthquake components, "H1," "H2," and "V", are presented in Figures 3.7.1-3, 3.7.1-4, and 3.7.1-5. Design horizontal time history, H1, is applied in the north-south (Global X or 1) direction; design horizontal time history, H2, is applied in the east-west (global Y or 2) direction; and design vertical time history is applied in the vertical (global Z or 3) direction. The cross-correlation coefficients between the three components of the design time histories are as follows:

$$\rho_{12} = 0.05, \quad \rho_{23} = 0.043, \text{ and } \rho_{31} = 0.140$$

where 1, 2, 3 are the three global directions.

Since the three coefficients are less than 0.16 as recommended in Reference 30, which was referenced by NRC Regulatory Guide 1.92, Revision 1, it is concluded that these three components are statistically independent. The design time histories are applied at the finished grade in the free field.

The ground motion time histories (H1, H2, and V) are generated with time step size of 0.010 second for applications in soil structure interaction analyses. For applications in the fixed-base mode superposition time-history analyses, the time step size is reduced to 0.005 second by linear interpolation. The cutoff frequency used in the horizontal and vertical seismic analysis of the nuclear island for the hard rock site is 34 hertz. The cutoff frequencies used in the soil structure interaction analyses are 33 hertz for the soft rock site, and 15 hertz and 21 hertz for the soft-to-medium stiff soil site in the horizontal and vertical directions, respectively. The maximum "cut-off" frequency for both the soil structure interaction analyses and the fixed-base analyses is well within the Nyquist frequency limit.

The comparison plots of the acceleration response spectra of the time histories versus the design response spectra for 2, 3, 4, 5, and 7 percent critical damping are shown in Figures 3.7.1-6, 3.7.1-7, and 3.7.1-8. The SRP 3.7.1, Table 3.7.1-1, provision of frequency intervals is used in the computation of these response spectra.

In SRP 3.7.1 the NRC introduced the requirement of minimum power spectral density to prevent the design ground acceleration time histories from having a deficiency of power over any frequency range. SRP 3.7.1, Revision 2, specifies that the use of a single time history is justified by satisfying a target power spectral density (PSD) requirement in addition to the design response spectra enveloping requirements. Furthermore, it specifies that when spectra other than Regulatory Guide 1.60 spectra are used, a compatible PSD shall be developed using procedures outlined in NUREG/CR-5347 (Reference 29).

The NUREG/CR-5347 procedures involve ad hoc hybridization of two earlier PSD envelopes. Since the modification to the RG 1.60 design spectra adopted for AP600 (see subsection 3.7.1.1) is relatively small (compared to the uncertainty in the fit to RG 1.60 of PSD-compatible time histories referenced in NUREG/CR-5347) and occurs only in the frequency range between 9 to 33 hertz, a project-specific PSD is developed using a slightly different hybridization for the higher frequencies.

Since the original RG 1.60 spectrum and the project-specific modified RG 1.60 spectrum are identical for frequencies less than 9 hertz, no modification to the PSD is done in this frequency range. At frequencies above 9 hertz, the third and the fourth legs of the PSD are slightly modified as follows:

- The frequency at which the design response spectrum inflected towards a 1.0 amplification factor at 33 hertz takes place at 25 hertz in the AP600 spectrum rather than at 9 hertz as in the RG 1.60 spectrum. The third leg of the PSD, therefore, is extended to about 25 hertz rather than 16 hertz.

- The lead coefficient to the fourth leg of the PSD is changed to connect with the extended third leg.

The AP600 augmented PSD, anchored to 0.3 g, is as follows:

$S_0(f) \quad = 58.5\ (f/2.5)^{0.2}\ in^2/sec^3,\ \ f \leq 2.5$ hertz

$S_0(f) \quad = 58.5\ (2.5/f)^{1.8}\ in^2/sec^3,\ \ 2.5$ hertz $\leq f \leq 9$ hertz

$S_0(f) \quad = 5.832\ (9/f)^3\ in^2/sec^3,\ \ 9$ hertz $\leq f \leq 25$ hertz

$S_0(f) \quad = 0.27\ (25/f)^8\ in^2/sec^3,\ \ 25$ hertz $\leq f$

The AP600 Minimum Power Spectral Density (PSD) is presented in Figure 3.7.1-9. This AP600 target PSD is compatible with the AP600 horizontal design response spectra and envelops a target PSD compatible with the AP600 vertical design response spectra. This AP600 target PSD, therefore, is conservatively applied to the vertical response spectra.

The comparison plots of the power spectral density curve of the AP600 acceleration time histories versus the target power spectral density curve are presented in Figures 3.7.1-10, 3.7.1-11, and 3.7.1-12. The PSD functions of the design time histories are calculated at

uniform frequency steps of 0.0489 hertz. The PSDs presented in Figures 3.7.1-10 through 3.7.1-12 are the averaged PSD obtained over a moving frequency band of ±20 percent centered at each frequency. The PSD amplitude at frequency (f) has the averaged PSD amplitude between the frequency range of 0.8 f and 1.2 f as stated in appendix A of Revision 2 of SRP 3.7.1.

3.7.1.3 Critical Damping Values

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 safe shutdown earthquake damping values used in the dynamic analysis are presented in Table 3.7.1-1. The damping values are based on Regulatory Guide 1.61, ASCE Standard 4-86 (Reference 3), and 5 percent damping for piping, except for the damping values of the primary coolant loop piping, which is based on Reference 22, and conduits, cable trays and their related supports.

The damping values for conduits, cable trays and their related supports are shown in Table 3.7.1-1 and Figure 3.7.1-13. The damping value of conduit, empty cable trays, and their related supports is similar to that of a bolted structure, namely 7 percent of critical. The damping value of filled cable trays and supports increases with increased cable fill and level of seismic excitation. The damping value for cable trays and supports is based on test results (Reference 19).

For structures or components composed of different material types, the composite modal damping is calculated using the strain energy method. The strain energy dependent modal damping values are computed based on Reference 20. The modal damping values equal:

$$\beta_n = \sum_{i=1}^{nc} \frac{\{\phi_n\}^T \beta_i [K_i]_i \{\phi_n\}}{\{\phi_n\}^T [K_t] \{\phi_n\}}$$

where:

β_n = ratio of critical damping for mode n

nc = number of elements

$\{\phi_n\}$ = mode n (eigenvector)

$[K_i]_i$ = stiffness matrix of element i

β_i = ratio of critical damping associated with element i

$[K_t]$ = total system stiffness matrix

Strain-dependent damping values are used for the foundation material in accordance with Reference 5 and 6 for rock sites and Reference 33 for soil sites. The strain-dependent damping curves for the foundation materials are presented in Figures 3.7.1-14 and 3.7.1-15 for rock material and soil material, respectively. The strain-dependent soil material damping is limited to 15 percent of critical damping.

3.7.1.4 Supporting Media for Seismic Category I Structures

The seismic design basis for the AP600 is to provide design coverage for as many plant sites as practical. For the design of seismic Category I structures, a set of four design soil profiles of various shear wave velocities is established in Appendices 2A and 2B. The four design soil profiles include a hard rock site, a soft rock site, an upper bound soft-to-medium stiff soil site and a soft-to-medium soil site. The shear wave velocity profiles and related governing parameters of the three sites considered are the following:

- For the hard rock site, an upper bound case for firm sites using fixed base seismic analysis

- For the soft rock site, a shear wave velocity of 2400 feet per second at the ground surface, increasing linearly to 3200 feet per second at a depth of 240 feet, and base rock at the depth of 120 feet

- For the soft-to-medium soil site, a shear wave velocity of 1000 feet per second at ground surface, increasing parabolically to 2400 feet per second at 240 feet, base rock at the depth of 120 feet, and ground water at grade level.

- For the upper bound soft-to-medium soil site, a shear wave velocity of 1414 feet per second at ground surface, increasing parabolically to 3394 feet per second at 240 feet, base rock at the depth of 120 feet, and ground water at grade level. The initial soil shear modulus profile is twice that of the soft-to-medium soil site.

The strain-dependent shear modulus curves for the foundation materials, together with the corresponding damping curves, are shown in Figures 3.7.1-14 and 3.7.1-15 for rock material and soil material, respectively. The shear wave velocity profile for the design soil profiles, with variation of depth to base rock, is shown in Figure 3.7.1-17.

The AP600 nuclear island consists of three seismic Category I structures founded on a common basemat. The three structures that make up the nuclear island are the coupled auxiliary and shield buildings, the steel containment vessel, and the containment internal structures. The nuclear island is shown in Figure 3.7.1-16. The foundation embedment depth, foundation size, and total height of the seismic Category I structures are presented in Table 3.7.1-2.

A coupled nuclear island stick model and design soil profile finite element models are used in the three-dimensional soil-structure interaction analysis described in subsection 3.7.2.4.

3.7.2 Seismic System Analysis

Seismic Category I structures, systems, and components are classified according to Regulatory Guide 1.29. Seismic Category I building structures of AP600 consist of the containment building (the steel containment vessel and the containment internal structures), the shield building, and the auxiliary building. These structures are founded on a common basemat and are collectively known as the nuclear island or nuclear island structures. Key dimensions, such as thickness of the basemat, floor slabs, roofs and walls, of the seismic Category I building structures are shown in Figure 3.7.2-28.

Seismic systems are defined, according to SRP 3.7.2, Section II.3.a, as the seismic Category I structures that are considered in conjunction with their foundation and supporting media to form a soil-structure interaction model. The following subsections describe the seismic analyses performed for the nuclear island. Other seismic Category I structures, systems, equipment, and components not designated as seismic systems (that is, heating, ventilation, and air-conditioning systems; electrical cable trays; piping systems) are designated as seismic subsystems. The analysis of seismic subsystems is presented in subsection 3.7.3.

Seismic Category I building structures are on the nuclear island. Other building structures are classified nonseismic or seismic Category II. Nonseismic structures are analyzed and designed for seismic loads according to the Uniform Building Code (Reference 2) requirements for Zone 2A. Seismic Category II building structures are designed for the safe shutdown earthquake using the same methods as are used for seismic Category I structures. The acceptance criteria are based on ACI 349 for concrete structures and on AISC N690 for steel structures. The seismic Category II building structures are constructed to the same requirements as the nonseismic building structures, ACI 318 for concrete structures and AISC-S355 for steel structures.

Separate seismic analyses are performed for the nuclear island, one for each of the four design soil profiles defined in subsection 3.7.1.4. The analyses generate one set of in-structure responses for each of the design soil profiles. The four sets of in-structure seismic responses are enveloped to obtain the seismic design envelope (design member forces, nodal accelerations, nodal displacements, and floor response spectra) used in the design and analysis of seismic Category I structures, components, and seismic subsystems.

Table 3.7.2-14 summarizes the types of models and analysis methods that are used in the seismic analyses of the nuclear island. It also summarizes the type of results that are obtained and where they are used in the design.

3.7.2.1 Seismic Analysis Methods

Seismic analyses of the nuclear island are performed in conformance with the criteria within SRP 3.7.2.

Seismic analyses, using the response spectrum method, the mode superposition time-history method, and the complex frequency response analysis method, are performed for the SSE to

determine the seismic force distribution for use in the design of the nuclear island structures, and to develop in-structure seismic responses (accelerations, displacements, and floor response spectra) for use in the analysis and design of seismic subsystems.

3.7.2.1.1 Response Spectrum Analysis

Response spectrum analyses, using computer program BSAP (Reference 7), are performed to obtain the seismic forces and moments required for the structural design of the auxiliary building, the shield building, and the containment internal structures on the nuclear island. The response spectrum analyses consider modes up to 33 hertz using the double sum modal combination method, and consider high frequency responses using the procedure given in Appendix A to SRP 3.7.2, Revision 2.

The analyses are performed using the three-dimensional, finite element models of the coupled shield and auxiliary buildings and the containment internal structures developed and discussed in subsection 3.7.2.3. Figure 3.7.2-1 shows the finite element model of the coupled shield and auxiliary buildings without the shield building roof stick model. The finite element model of the containment internal structures is shown in Figure 3.7.2-2. In addition, two typical wall sections of the coupled shield and auxiliary buildings are presented in Figure 3.7.2-3.

Response spectrum analyses are performed only for the hard rock site where the soil-structure interaction effect is negligible, as described in Appendix 2B. Therefore, the response spectrum analyses are performed using the fixed-base, three-dimensional, finite element models. The in-plane forces obtained from the analyses are used for the design of floors and walls.

A comparison of the member forces and moments obtained in the three-dimensional analyses of the lumped-mass stick models, Tables 3.7.2-11 through 3.7.2-13, shows that the hard rock profile does not always govern design of the nuclear island structures. In cases where other design soil profiles give higher element forces than the hard rock profile, the in-plane forces obtained from the response spectrum analyses of the finite element models for the hard rock site are increased by a scaling factor. The scaling factor, at a given plant elevation, is equal to the ratio of the largest three-dimensional stick model element force over the three-dimensional stick model element force for the hard rock profile.

3.7.2.1.2 Time-History Analysis and Complex Frequency Response Analysis

Mode superposition time-history analyses using computer program BSAP and complex frequency response analysis using computer program SASSI (Reference 8) are performed to obtain the in-structure seismic response (accelerations, displacements, and floor response spectra) needed in the analysis and design of seismic subsystems.

The three-dimensional, lumped-mass stick models of the nuclear island structures developed as described in subsection 3.7.2.3 are used in conjunction with the design soil profiles presented in subsection 3.7.1.4 to obtain the in-structure responses. The lumped-mass stick models of the nuclear island structures are presented in Figure 3.7.2-4 for the coupled shield

and auxiliary buildings, in Figure 3.7.2-5 for the steel containment vessel, in Figure 3.7.2-6 for the containment internal structures, and in Figure 3.7.2-7 for the reactor coolant loop model. The individual building lumped-mass stick models are interconnected with rigid linking elements to form the overall dynamic model of the nuclear island. The nuclear island basemat and the periphery walls of the embedded portion of the nuclear island are represented by a three-dimensional, finite element model, as shown in Figure 3.7.2-8.

For the hard rock site the soil-structure interaction effect is negligible, as described in Appendix 2B. Therefore, for the hard rock site, the nuclear island is analyzed as a fixed-base structure, using computer program BSAP without the foundation media. The three components of earthquake (two horizontal and one vertical time histories) are applied simultaneously in the analysis.

For the remaining design soil profiles, the three-dimensional, nuclear island stick model is coupled with the foundation media to form a soil-structure interaction model to account for the effects of embedment and foundation rocking, torsion, and translation. The seismic soil-structure interaction analysis of the coupled nuclear island and soil foundation model is performed using computer program SASSI. The soil-structure interaction analyses are performed with the three statistically independent acceleration time histories of earthquake applied separately. The total seismic response is then obtained by combining the responses of the three components of earthquake algebraically in each time step. Subsection 3.7.2.4 provides details of the soil-structure interaction analysis.

Seismic responses of the nuclear island structures for the various design soil profiles are enveloped and the resulting response spectra are used in the design and analysis for most of the seismic subsystems. Certain subsystems, as described in subsection 3.7.3.6, are analyzed using the time histories obtained from a series of soil-specific analyses for the design soil profiles presented in subsection 3.7.1.4.

3.7.2.2 Natural Frequencies and Response Loads

Modal analyses are performed for the lumped-mass stick models of the seismic Category I structures on the nuclear island developed in subsection 3.7.2.3. Table 3.7.2-1 summarizes the modal properties of the stick model representing the coupled shield and auxiliary buildings. Table 3.7.2-2 shows the modal properties of the steel containment vessel. Table 3.7.2-3 shows the modal properties for both the containment internal structures without the reactor coolant loop stick model (sheet 1) and the coupled containment internal structures and reactor coolant loop stick model (sheets 2 and 3). Table 3.7.2-4 shows the modal properties of the overall stick model of the nuclear island.

The seismic analysis of the nuclear island considers 74 vibration modes, up to the frequency limit of 34 hertz, shown in Table 3.7.2-4. The total cumulative mass participating in the seismic response constitute 90, 90, and 83 percent of the total mass, excluding the building mass within the embedded portion of the nuclear island.

Table 3.7.2-3, sheet 1, demonstrates the large stiffness of the containment internal structures. The table shows, for frequencies up to 33 hertz, a total cumulative mass of 42 percent in the north-south direction, 39 percent in the east-west direction, and negligible amount in the vertical direction. For frequencies up to 60 hertz, the table shows the total cumulative mass increased to 99, 99 and 43 percent in the three respective directions. Because of the high frequency modal participation, the seismic force and moment responses of the containment internal structures are determined from a response spectrum analysis of the fixed-based nuclear island lumped-mass stick model. The response spectrum analysis considers 74 vibration modes, up to 34 hertz, using the double sum method and, above 34 hertz, high frequency responses using the procedure given in Appendix A to SRP 3.7.2, Revision 2.

Figures 3.7.2-9 through 3.7.2-11 show, respectively, representative vibration mode shapes for the coupled shield and auxiliary buildings, the steel containment vessel and the containment internal structures.

Maximum absolute acceleration (ZPA) responses of the design soil profiles at selected locations on the coupled shield and auxiliary buildings, the steel containment vessel, and the containment internal structures are summarized in Tables 3.7.2-5, 3.7.2-6, and 3.7.2-7, respectively. These maximum absolute acceleration responses are plotted in Figures 3.7.2-12 through 3.7.2-14. Similarly, maximum displacement responses relative to the base of the lumped-mass nuclear island stick model at top of basemat, for the design soil profiles, are summarized and plotted in Tables 3.7.2-8 through 3.7.2-10 and in Figures 3.7.2-15 through 3.7.2-17, respectively, for the coupled shield and auxiliary buildings, the steel containment vessel, and the containment internal structures.

Maximum seismic response forces and moments determined in the lumped-mass stick model for the design soil profiles are summarized in Tables 3.7.2-11 through 3.7.2-13 and plotted in Figures 3.7.2-18 through 3.7.2-23 respectively, for the coupled shield and auxiliary buildings, the steel containment vessel, and the containment internal structures.

3.7.2.3 Procedure Used for Modeling

Based on the general plant arrangement, three-dimensional, finite element models are developed for the nuclear island structures—a finite element model of the coupled shield and auxiliary buildings, a finite element model of the containment internal structures, and axisymmetric shell models of the steel containment vessel and shield building roof. These three-dimensional, finite element models provide the basis for the development of the lumped-mass stick model of the nuclear island structures.

Three-dimensional, lumped-mass stick models are developed to represent the steel containment vessel, the containment internal structures, and the coupled shield and auxiliary buildings. Discrete mass points are provided at major floor elevations and at locations of structural discontinuities. The structural eccentricities between centers of rigidity and the centers of mass of the structures are considered. These seismic models consist of lumped masses connected by elastic structural elements together with rigid elements to simulate eccentricity.

The individual building lumped-mass stick models are interconnected with rigid linking elements to form the overall dynamic model of the nuclear island.

Seismic subsystems coupled to the overall dynamic model of the nuclear island include the coupling of the reactor coolant loop model to the model of the containment internal structures, and the coupling of the polar crane model to the model of the steel containment vessel. The criteria used for decoupling seismic subsystems from the nuclear island model is according to Section II.3.b of SRP 3.7.2, Revision 2. The total mass of other major subsystems and equipment are less than one percent of their respective supporting nuclear island structures, therefore, the mass of other major subsystems and equipment are included as concentrated lumped-mass only.

3.7.2.3.1 Coupled Shield and Auxiliary Buildings and Containment Internal Structures

The finite element models of the coupled shield and auxiliary buildings and the reinforced concrete portions of the containment internal structures are based on the gross concrete section with the modulus based on the specified compressive strength of concrete of contributing structural walls and slabs. The properties of the concrete-filled structural modules are computed using the combined gross concrete section and the transformed steel face plates of the structural modules. Furthermore, the weight density of concrete plus the uniformly distributed miscellaneous dead-weights are considered by adjusting the material mass density of the structural elements. An equivalent tributary slab area load of 50 pounds per square feet is considered to represent miscellaneous dead-weight such as minor equipment, piping and raceways. Live load is not included in the mass of the global seismic models. Major equipment weights are included as concentrated lumped masses at the equipment locations. Figures 3.7.2-1 and 3.7.2-2 show, respectively, the finite element models of the coupled shield and auxiliary buildings and the containment internal structures. A lumped-mass stick model of the shield building roof structure is coupled with the finite element model and the stick model of the coupled auxiliary and shield buildings. The stick model of the shield building roof structure is included in the seismic analyses. The lumped-mass stick model of the shield building roof is not shown in Figure 3.7.2-1 to maintain visual clarity of the finite element model.

Because of the irregular structural configuration, the properties of the three-dimensional, lumped-mass stick models are determined using building sections extracted from the three-dimensional building finite element models. Figure 3.7.2-3, sheets 1 and 2, show two typical building sections from the coupled shield and auxiliary buildings finite element model. The properties of the stick model beam elements, including the location of centroid, center of rigidity and center of mass, and equivalent sectional areas and moment of inertia, are computed using specific finite element sections representing the walls and columns between principal floor elevations of the structures. The equivalent translation and rotational stiffness (sectional areas and moment of inertia) of the three-dimensional beams are computed by applying unit forces and moments at the top of the specific finite element sections.

The eccentricities between the centroids (the neutral axis for axial and bending deformation), the centers of rigidity (the neutral axis for shear and torsional deformation), and the centers

of mass of the structures are represented by a combination of two sticks in the seismic model. One stick represents only the axial areas of the structural member and is located at the centroid. This stick model is developed to resist the vertical seismic input motion. The other stick represents other beam element properties except the axial area of the structural member and is located at the center of rigidity. This stick model is developed to resist the horizontal seismic input motions. At a typical model elevation, there are four rigid beam elements connecting the center of mass node to the sticks located at the shear centers and the centroids of the wall sections above and below.

The shield building roof including the passive containment cooling system water storage tank is represented by a lumped-mass stick model simulating the dynamic behavior of this portion of the roof structure. This lumped-mass stick model is combined with the lumped-mass stick model representing the lower portion of the shield building. In the three-dimensional finite element model, the lumped-mass stick model of the shield building roof is located at the center of the shield building represented using cylindrical shell elements. The lumped-mass stick model of the shield building roof is connected to the three-dimensional shell elements using 18 horizontal rigid beams.

The in-containment refueling water storage tank (IRWST) is included in the three-dimensional FEM used in the development of the lumped-mass stick model representing the containment internal structures (CIS). Therefore, the lumped-mass stick model of the CIS includes the stiffness and mass effect of the IRWST.

Figures 3.7.2-4 and 3.7.2-6 show, respectively, the lumped-mass stick models of the coupled shield and auxiliary buildings and the containment internal structures.

A simplified reactor coolant loop model is developed and coupled with the containment internal structures model for the seismic analysis. The reactor coolant loop stick model is presented in Figure 3.7.2-7.

3.7.2.3.2 Steel Containment Vessel

The steel containment vessel is a freestanding, cylindrical, steel shell structure with ellipsoidal upper and lower steel domes. The three-dimensional, lumped-mass stick model of the steel containment vessel is developed based on the axisymmetric shell model. Figure 3.7.2-5 presents the steel containment vessel stick model. In the stick model, the properties are calculated as follows:

- Members representing the cylindrical portion are based on the properties of the actual circular cross section of the containment vessel.
- Members representing the bottom head are based on equivalent stiffnesses calculated from the shell of revolution analyses for static 1.0g in vertical and horizontal directions.
- Shear, bending and torsional properties for members representing the top head are based on the average of the properties at the successive nodes, using the actual circular cross

section. These are the properties that affect the horizontal modes. Axial properties, which affect the vertical modes, are based on equivalent stiffnesses calculated from the shell of revolution analyses for static 1.0g in the vertical direction.

This method used to construct a stick model from the axisymmetric shell model of the containment vessel is verified by comparison of the natural frequencies determined from the stick model and the shell of revolution model as shown in Table 3.7.2-15. The shell of revolution vertical model (n = 0 harmonic) has a series of local shell modes of the top head between 23 and 30 hertz. These modes are predominantly in a direction normal to the shell surface and cannot be represented by a stick model. These local modes have small contribution to the total response to a vertical earthquake as they are at a high frequency where seismic excitation is small.

The containment air baffle, presented in subsection 3.8.4.1.3, is supported from the steel containment vessel at regular intervals so that a gap is maintained for airflow. It is constructed with individual panels which do not contribute to the stiffness of the containment vessel. The fundamental frequency of the baffle panels and supports is about twice the fundamental frequency of the containment vessel. The mass of the air baffle is small, equal to approximately 10 percent of the vessel plates to which it is attached. The air baffle, therefore, is assumed to have negligible interaction with the steel containment vessel. Only the mass of the air baffle is considered and added at the appropriate elevations of the steel containment vessel stick model.

The polar crane is supported on a ring girder which is an integral part of the steel containment vessel at elevation 209'-0". It is modelled as a single degree of freedom system attached to the steel containment shell as shown in Figure 3.7.2-5. The polar crane model includes the flexibility of the crane bridge girders and truck assembly, and the containment shell's local flexibility.

During plant operating conditions, the polar crane is parked in the direction 10 degrees off the plant north-south direction with the trolley located at one end near the containment shell. In the seismic model, however, the slight offset of the polar crane is neglected by assuming the crane bridge spanning in the north-south direction and the mass eccentricity of the trolley is considered by locating the mass of the trolley at the northern limit of travel of the main hook. Furthermore, the mass eccentricity of the two equipment hatches and the two personnel airlocks are considered by placing their mass at their respective center of mass as shown in Figure 3.7.2-5.

3.7.2.3.3 Nuclear Island Seismic Model

The various building lumped-mass stick models are interconnected with rigid linking elements to form the overall dynamic model of the nuclear island. The nuclear island seismic model consists of 80 mass points and 249 dynamic degrees of freedom. The mass properties of the

lumped-mass stick models include all tributary mass expected to be present during plant operating conditions. This includes the dead weight of walls and slabs, weight of major equipment, and equivalent tributary slab area loads representing miscellaneous equipment, piping and raceways.

The nuclear island seismic model includes the effect of flexibility of typical floor slabs. Typical floor slabs with vertical frequency less than 33 hertz are simulated in the dynamic model using single degree of freedom vertical oscillators attached to their respective elevations on the nuclear island lumped mass stick model. The masses of the vertical oscillators are deducted from the corresponding nodal masses in the building lumped mass stick model.

Furthermore, the hydrodynamic mass effect of the water within the passive containment cooling system water tank on the shield building roof and the in-containment refueling water storage tank within the containment internal structures is evaluated. The convective (sloshing) effect of the water mass is found to be negligible. Hence, only the impulsive effect of the water mass is included in the nuclear island seismic model.

For the soil-structure interaction analyses, the nuclear island basemat and the periphery walls of the embedded portion of the nuclear island are represented by a three-dimensional, finite element model, as shown in Figure 3.7.2-8.

3.7.2.4 Soil-Structure Interaction

Soil-structure interaction (SSI) analyses of the nuclear island are performed to generate its SSI responses. The nuclear island SSI responses generated for the analysis and design of seismic subsystems include nodal displacements, nodal accelerations, and floor response spectra.

The nuclear island SSI analyses are performed for the design soil profiles described in subsection 3.7.1.4, except for the hard rock site condition, where the possibility of SSI is negligible. Furthermore, the effects of the adjacent structures (turbine, annex and radwaste buildings) on the seismic response of the nuclear island are negligible. Therefore, the adjacent structures are not included in the SSI analyses.

SSI analyses are performed using the complex frequency-response method with computer program SASSI. Computer program SHAKE (Reference 9) is used to compute the safe shutdown earthquake dynamic strain compatible soil properties, such as shear modulus and damping. The material (hysteretic) damping ratio for soil in the SSI analyses is limited not to exceed 15 percent. The SSI analyses of the nuclear island are performed using the program SASSI, which is capable of handling two- and three-dimensional SSI problems involving multiple structures with rigid or flexible embedded foundations of arbitrary shape.

SSI analyses are performed using the three-dimensional model of the soil profiles coupled with the nuclear island lumped-mass stick model developed in subsection 3.7.2.3. The nuclear island lumped-mass stick model consists of lumped masses connected to elastic structural elements by horizontal rigid beam elements to simulate eccentricity. For the SSI analyses using program SASSI, the rigid elements have the following properties:

- The area to length ratio of the rigid element is within the range of 10^3 to 10^5 times the largest area to length ratio of its connecting elastic structural elements, and

- The moment of inertia to length^3 ratio of the rigid element is within the range of 10^3 to 10^5 times the largest moment of inertia to length^3 ratio of its connecting elastic structural elements.

Furthermore, the stiffness and mass contributed by the periphery walls in the embedded portion of the nuclear island are subtracted from the model properties of the lumped-mass stick model. The mass and stiffness properties adjustment is accomplished by recalculating the properties of the embedded portion of the 3D lumped-mass stick model based on the finite element model without the periphery walls. To form the soil-structure interaction model, the lumped-mass stick models are coupled to the three-dimensional, finite element foundation model through rigid connections at elevations 82′-6″, 100′-0″ (see Figure 3.7.2-29).

The SSI effects on the seismic Category I structures due to embedment of the nuclear island, the location of the ground water, and the layering of soil profiles selected are considered in modeling of the soil medium. A technical selection process has been used to determine the representative soil conditions for the generic plant sites as described in Appendices 2A and 2B.

3.7.2.5 Development of Floor Response Spectra

The design floor response spectra are generated according to Regulatory Guide 1.122.

Seismic floor response spectra are computed using time-history responses determined from the nuclear island seismic analyses with the various design soil profiles. The time-history responses for the hard rock condition are determined from a mode superposition time history analysis using computer program BSAP. The time history responses for the soft rock and the soft-to-medium soil cases are obtained from a complex frequency response analysis using computer program SASSI. Floor response spectra for damping values equal to 2, 3, 4, 5, 7, 10, and 20 percent of critical damping are computed at the required locations.

The floor response spectra for the design of subsystems and components are generated by enveloping the nodal response spectra determined for the different design soil profiles. The envelopes of the floor response spectra for the four design soil profiles are developed as follows:

- The spectral acceleration is calculated at the same frequencies for all four of the design soil profiles

- The maximum spectral acceleration at each frequency from any of the four design soil profiles is then selected for the envelope

- The enveloped floor response spectra is then broadened by ±15 percent

The enveloped floor response spectra are smoothed, and the spectral peaks associated with the structural frequencies are broadened by fifteen percent (±15 percent) to account for the variation in the structural frequencies, due to the uncertainties in parameters such as material and mass properties of the structure and soil, damping values, seismic analysis technique, and the seismic modeling technique. Figure 3.7.2-24 shows the smoothing and broadening procedure used to generate the design floor response spectra.

The safe shutdown earthquake floor response spectra for 5 percent damping, at representative locations of the coupled auxiliary and shield buildings, the steel containment vessel, and the containment internal structures are presented in Figures 3.7.2-25 through 3.7.2-27. The representative response spectra figures include the acceleration response spectra computed for the individual design soil profiles and the corresponding enveloped and widened floor response spectrum.

3.7.2.6 Three Components of Earthquake Motion

Seismic system analyses are performed considering the simultaneous occurrences of the two horizontal and the vertical components of earthquake.

In mode superposition time-history analyses using computer program BSAP, the three components of earthquake are applied either simultaneously or separately. In the BSAP analyses with the three earthquake components applied simultaneously, the effect of the three components of earthquake motion is included within the analytical procedure so that further combination is not necessary.

In analyses with the earthquake components applied separately and in the response spectrum analyses, the effect of the three components of earthquake motion are combined using one of the following methods:

- For seismic analyses with the statistically independent earthquake components applied separately, the time-history responses from the three earthquake components are combined algebraically at each time step to obtain the combined response time-history. This method is used in the BSAP time-history and SASSI analyses.

- The peak responses due to the three earthquake components from the response spectrum analyses are combined using the square root of the sum of squares (SRSS) method. This method is used in the BSAP response spectrum analyses.

- 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 percent of the peak (100 percent-40 percent-40 percent method). Combinations of seismic responses from the three earthquake components, together with variations in sign (plus or minus), are considered. This method is used in the nuclear island basemat analyses and in the containment vessel stability analyses.

The containment vessel is analyzed using axisymmetric finite element models. These axisymmetric building structures are analyzed for one horizontal seismic input from any horizontal direction and one vertical earthquake component. Responses are combined by either the SRSS method or by a modified 100 percent-40 percent-40 percent method in which one component is taken at 100 percent of its maximum value and the other is taken at 40 percent of its maximum value.

For the seismic responses presented in subsection 3.7.2.2, the effect of three components of earthquake are considered as follows:

- Response Spectrum Analysis - the responses from the three components of earthquake motion are combined using the square root of the sum of square (SRSS) technique.

- Mode Superposition Time History Analysis (program BSAP) and the Complex Frequency Response Analysis (program SASSI) - the time history responses from the three components of earthquake motion are combined algebraically at each time step.

A summary of the dynamic analyses performed and the combination techniques used are presented in Table 3.7.2-16.

3.7.2.7 Combination of Modal Responses

The modal responses of the response spectrum system structural analysis are combined using the square root of the sum of squares method. When closely spaced modes are present, these modes are considered using either the grouping method, the 10 percent method, or the double sum method shown in Section C of Regulatory Guide 1.92, Revision 1. When high frequency effects are significant, they are included using the procedure given in Appendix A to SRP 3.7.2. In the fixed base mode superposition time history analysis of the hard rock site, the total seismic response is obtained by superposing the modal responses within the analytical procedure so that further combination is not necessary.

A summary of the dynamic analyses performed and the combination techniques used are presented in Table 3.7.2-16.

3.7.2.8 Interaction of Seismic Category II and Nonseismic Structures with Seismic Category I Structures, Systems or Components

Nonseismic structures are evaluated to determine that their seismic response does not preclude the safety functions of seismic Category I structures, systems or components. This is accomplished by satisfying one of the following:

- The collapse of the nonseismic structure will not cause the nonseismic structure to strike a seismic Category I structure, system or component.

- The collapse of the nonseismic structure will not impair the integrity of seismic Category I structures, systems or components.

- The structure is classified as seismic Category II and is analyzed and designed to prevent its collapse under the safe shutdown earthquake.

The structures adjacent to the nuclear island are the annex building, the radwaste building, and the turbine building. The annex building is classified as seismic Category II and is designed to prevent its collapse under the safe shutdown earthquake. The minimum space required between the annex building and the nuclear island to avoid contact is obtained by absolute summation of the deflections of each structure obtained from either a time history or a response spectrum analysis for each structure.

The radwaste building is classified as nonseismic and is designed to the seismic requirements of the Uniform Building Code, Zone 2A with an Importance Factor of 1.25. As shown in the radwaste building general arrangement in Figure 1.2-29, it is a small steel framed building. If it were to impact the nuclear island or collapse in the safe shutdown earthquake, it would not impair the integrity of the reinforced concrete nuclear island. The minimum clearance between the structural elements of the radwaste building and the nuclear island is four inches.

The turbine building is classified as nonseismic. As shown on the turbine building general arrangement in Figures 1.2-30 through 1.2-34, the major structure of the turbine building is separated from the nuclear island by approximately eighteen feet. Floors between the turbine building main structure and the nuclear island provide access to the nuclear island. The floor beams are supported on the outside face of the nuclear island with a minimum horizontal clearance of four inches between the structural elements of the turbine building and the nuclear island. These beams are of light construction such that they will collapse if the differential deflection of the two buildings exceeds the clearance and will not jeopardize the two foot thick walls of the nuclear island. The roof in this area rests on the roof of the nuclear island and could slide relative to the roof of the nuclear island in a large earthquake. The seismic design is upgraded from Zone 2A, Importance Factor of 1.25, to Zone 3 with an Importance Factor of 1.0 in order to provide margin against collapse during the safe shutdown earthquake. The turbine building is a concentrically braced steel frame structure designed to meet the following criteria:

- The turbine building is designed in accordance with ACI-318 for concrete structures and with AISC for steel structures. Seismic loads are defined in accordance with the Uniform Building Code for Zone 3 with an Importance Factor of 1.0.

- The minimum horizontal clearance between the structural elements of the turbine building and the nuclear island and annex building is 4 inches.

- Steel structural bracing connections are designed with sufficient strength to develop tensile yield in the bracing before the connection fails.

3.7.2.9 Effects of Parameter Variations on Floor Response Spectra

Seismic model uncertainties due to, among other things, uncertainties in material properties, mass properties, damping values, the effect of concrete cracking, and the modeling techniques are accounted for in the widening of floor response spectra, as described in subsection 3.7.2.5.

3.7.2.10 Use of Constant Vertical Static Factors

The vertical component of the safe shutdown earthquake is considered to occur simultaneously with the two horizontal components in the seismic analyses. Therefore, constant vertical static factors are not used for the design of seismic Category I structures.

3.7.2.11 Method Used to Account for Torsional Effects

The seismic analysis models of the nuclear island incorporate the mass and stiffness eccentricities of the seismic Category I structures and the torsional degrees of freedom. An accidental torsional moment is included in the design of the nuclear island structures. The accidental torsional moment due to the eccentricity of each mass is determined using the following:

- Horizontal mass properties of the building stick models shown in Figures 3.7.2-4, 3.7.2-5, and 3.7.2-6,
- The enveloping value of the north-south and east-west nodal accelerations shown in Tables 3.7.2-5, 3.7.2-6, and 3.7.2-7.
- An assumed accidental eccentricity equal to ±5 percent of the maximum building dimensions at the elevation of the mass.
- The torsional moments due to eccentricities of the masses at each elevation are assumed to act in the same direction on each structure. Both positive and negative values are considered.

3.7.2.12 Comparison of Responses

In the seismic analyses, the response spectrum analysis method is used in conjunction with the finite element models, while the mode superposition time-history and the complex frequency response method are applied to the lumped-mass stick model of the nuclear island. Therefore, a comparison of responses calculated by alternative methods is not necessary.

3.7.2.13 Methods for Seismic Analysis of Dams

Seismic analysis of dams is site specific design.

3.7.2.14 Determination of Seismic Category I Structure Overturning Moments

Subsection 3.8.5.5.4 describes the effects of seismic overturning moments.

3.7.2.15 Analysis Procedure for Damping

Subsection 3.7.1.3 presents the damping values used in the seismic analyses. For structures comprised of different material types, the composite modal damping approach utilizing the

strain energy method is used to determine the composite modal damping values. Subsection 3.7.2.4 presents the damping values used in the soil-structure interaction analysis.

3.7.3 Seismic Subsystem Analysis

This subsection describes the seismic analysis methodology for subsystems, which are those structures and components that do not have an interface with the soil-structure interaction analyses. Structures and components considered as subsystems include the following:

- Structures, such as floor slabs, miscellaneous steel platforms and framing
- Equipment modules consisting of components, piping, supports, and structural frames
- Equipment including vessels, tanks, heat exchangers, valves, and instrumentation
- Distributive systems including: piping and supports, electrical cable trays and supports, HVAC ductwork and supports, instrumentation tubing and supports, and conduits and supports

Subsection 3.9.2 describes dynamic analysis methods for the reactor internals. Subsection 3.9.3 describes dynamic analysis methods for the primary coolant loop support system. Subsection 3.7.2 describes the analysis methods for seismic systems, which are those structures and components that are considered with the foundation and supporting media. Section 3.2 includes the seismic classification of building structures, systems, and components.

3.7.3.1 Seismic Analysis Methods

The methods used for seismic analysis of subsystems include, modal response spectrum analysis, time-history analysis, and equivalent static analysis. The methods described in this subsection are acceptable for any subsystem. The particular method used is selected by the designer based on its appropriateness for the specific item. Items analyzed by each method are identified in the descriptions of each method in the following paragraphs.

3.7.3.2 Determination of Number of Earthquake Cycles

Seismic Category I structures, systems, and components are evaluated for one occurrence of the safe shutdown earthquake (SSE). In addition, subsystems sensitive to fatigue are evaluated for cyclic motion due to earthquakes smaller than the safe shutdown earthquake. Using analysis methods, these effects are considered by inclusion of seismic events with an amplitude not less than one-third of the SSE amplitude. The number of cycles is calculated based on IEEE-344-1987 (Reference 21) to provide the equivalent fatigue damage of two SSE events with 10 high-stress cycles per event. Typically, there are five seismic events with an amplitude equal to one-third of the SSE response. Each event has 63 high-stress cycles. For ASME Class 1 piping, the fatigue evaluation is performed based on five seismic events with an amplitude equal to one-third of the SSE response. Each event has 63 high-stress cycles.

When seismic qualification is based on dynamic testing for structures, systems, or components containing mechanisms that must change position in order to function, operability testing is performed for the safe shutdown earthquake preceded by one or more earthquakes. The number of preceding earthquakes is calculated based on IEEE-344-1987 (Reference 21) to provide the equivalent fatigue damage of one SSE event. Typically, the preceding earthquake is one SSE event or five one-half SSE events.

3.7.3.3 Procedure Used for Modeling

The dynamic analysis of any complex system requires the discretization of its mass and elastic properties. This is accomplished by concentrating the mass of the system at distinct characteristic points or nodes, and interconnecting them by a network of elastic springs representing the stiffness properties of the systems. The stiffness properties are computed either by hand calculations for simple systems or by finite element methods for more complex systems.

Nodes are located at mass concentrations and at additional points within the system. They are selected in such a way as to provide an adequate representation of the mass distribution and high-stress concentration points of the system.

At each node, degrees of freedom corresponding to translations along three orthogonal axes, and rotations about these axes are assigned. The number of degrees of freedom is reduced by the number of constraints, where applicable. For equipment qualification, reduced degrees of freedom are acceptable provided that the analysis adequately and conservatively predicts the response of the equipment.

The size of the model is reviewed so that a sufficient number of masses or degrees of freedom are used to compute the response of the system. A model is considered adequate provided that additional degrees of freedom do not result in more than a 10 percent increase in response, or the number of degrees of freedom equals or exceeds twice the number of modes with frequencies less than 33 hertz.

3.7.3.4 Basis for Selection of Frequencies

The effect of the building amplification on equipment and components is addressed by the floor response spectra method or by a coupled analysis of the building and equipment. Certain components are designed for a natural frequency greater than 33 hertz. In those cases where it is practical to avoid resonance, the fundamental frequencies of components and equipment are selected to be less than one-half or more than twice the dominant frequencies of the support structure.

3.7.3.5 Equivalent Static Load Method of Analysis

The equivalent static load method involves 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 method are adjusted to account for the relative motion between points of support when significant.

3.7.3.5.1 Single Mode Dominant or Rigid Structures or Components

For rigid structures and components, or for cases where the response can be classified as single mode dominant, the following procedures are used. Examples of these systems, structures, and components are equipment, and piping lines, instrumentation tubing, cable trays, HVAC, and floor beams modeled on a span by span basis.

- For rigid systems, structures, and components (fundamental frequency $\geq$ 33 hz) an equivalent seismic load is defined for the direction of excitation as the product of the component mass and the zero period acceleration value obtained from the applicable floor response spectra.

- A rigid component (fundamental frequency $\geq$ 33 hz), whose support can be represented by a flexible spring, can be modeled as a single degree of freedom 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 at the natural frequency from the applicable floor response spectra. If the frequency is not determined, the peak acceleration from the applicable floor response spectrum is used.

- If the component has a distributed mass whose dynamic response will be 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 floor response spectra times a factor of 1.5. A factor of less than 1.5 may be used if justified. A factor of 1.0 is used for structures or equipment that can be represented as uniformly loaded cantilever, simply supported, fixed-simply supported, or fixed-fixed beams (References 10 and 11). If the frequency is not determined, the peak acceleration from the applicable floor response spectrum is used.

3.7.3.5.2 Multiple Mode Dominant Response

This procedure applies to piping, instrumentation tubing, cable trays, and HVAC that are multiple span models. The equivalent static load method of analysis can be used for design of piping systems, 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 floor response spectra. 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.

3.7.3.6 Three Components of Earthquake Motion

Two horizontal components and one vertical component of seismic response spectra are employed as input to a modal response spectrum analysis. The spectra are associated with the safe shutdown earthquake. In the response spectrum 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 spectrum analyses are combined using the square root of the sum of squares (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 percent of the peak (100 percent-40 percent-40 percent method). Combinations of seismic responses from the three earthquake components, together with variations in sign (plus or minus), are considered.

One set of three mutually orthogonal artificial time histories is used when time-history analyses are performed. When the responses from the three components of motion are calculated simultaneously, each component is statistically independent of the other two. For this case, the components are combined by algebraic sum.

In addition, an optional method for combining the response of the three components of earthquake motion is presented in the following paragraphs.

The time-history safe shutdown earthquake 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 time-history safe shutdown earthquake analysis of the system is performed by applying three mutually orthogonal and statistically independent, artificial time histories. Possible examples of the use of this method of seismic analysis include the following:

- The subsystem analysis is a flexible floor or miscellaneous structural steel frame. The corresponding system analysis is the soil-structure interaction analysis of the nuclear island structures.

- The subsystem analysis is the primary loop piping system and interior concrete building structure. The interface point is the top of the basemat. The corresponding system analysis is the soil-structure interaction analysis of the nuclear island structures.

- The subsystem analysis is the reactor coolant pump and internal components. The interface points are the welds on the pump suction and discharge nozzles. The corresponding system analysis is the primary loop piping system and interior concrete building structure.

3.7.3.7 Combination of Modal Responses

For the seismic response spectra analyses, the zero period acceleration cut-off frequency is 33 hertz. High frequency or rigid modes are considered using the Left-Out-Force method or the Missing Mass Method described in subsection 3.7.3.7.1. The method to combine the low frequency modes is described in subsection 3.7.3.7.2. The rigid mode results in the three perpendicular directions of the seismic input are combined by square-root-sum-of-the-squares (SRSS) method. The resultant response of the rigid modes is combined by SRSS with the flexible mode results. The combination of modal responses in time history analyses of piping systems is described in subsection 3.7.3.17. Modal responses in time history analyses of other subsystems are combined as described in subsection 3.7.2.6.

3.7.3.7.1 Combination of High-Frequency Modes

This subsection describes alternative methods of accounting for high-frequency modes (generally greater than 33 hertz) in seismic response spectrum analysis. Higher-frequency modes can be excluded from the response calculation if the change in response is less than or equal to 10 percent.

3.7.3.7.1.1 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 left-out-force method is used in program PS+CAEPIPE.

The left-out-force vector, $\{Fr\}$, is calculated based on lower modes:

$$\{Fr\} = \left[1 - \Sigma M e_j e_j^T\right] f(t)$$

where $f(t)$ = the applied load vector,

M = the mass matrix

e_j = the eigenvector

Note that Σ is only for all 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:

$$\{Fr\} = Am[M]\left[\{r\} - \Sigma P_j e_j\right]$$

where:

Am = 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

Since,

$$P_j = e_j^T [M] \{r\}, \quad \{Fr\} = Am [M] \{r\} \left[1 - \Sigma M e_j e_j^T\right]$$

In PS+CAEPIPE, the low frequency modes are combined by one of the Reg Guide 1.92 methods in the response spectrum analysis. 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. This factor is usually the ZPA (zero period acceleration) of the response spectrum for the corresponding direction. The resultant low frequency responses are combined by square root of the sum of the squares with the high frequency responses (rigid modes results).

In GAPPIPE, the results from the high frequency responses are also combined by the square root of the sum of the squares with those from the resultant loads contributed by lower ~~loads~~ modes. The missing mass correction for an independent support motion or multiple response spectra analysis is exactly the same as that for the single enveloped response spectrum analysis except that Am used is the envelope of all the zero period accelerations of all the independent support inputs.

3.7.3.7.1.2 SRP 3.7.2 Method

The method described in SRP Section 3.7.2, may also be used for combination of high-frequency modes.

The following is the procedure for incorporating responses associated with high-frequency modes.

Step 1 Determine the modal responses only for those modes having natural frequencies less than that at which the spectral acceleration approximately returns to the zero period acceleration (33 hertz for the Regulatory Guide 1.60 response spectra). Combine such modes according to the methods discussed in subsection 3.7.3.7.2.

Step 2 For each degree of freedom included in the dynamic analysis, determine the fraction of degree of freedom mass included in the summation of all modes included in Step 1. This fraction d_i for each degree of freedom is given by:

$$d_i = \sum_{n=1}^{N} C_n \times \phi_{n,i}$$

where:

n = order of mode under consideration
N = number of modes included in Step 1
$\phi_{n,i}$ = nth natural mode of the system

C_n is the participation factor given by:

$$C_n = \frac{(\phi_n)^T (1)}{(\phi_n)^T [m] (\phi_n)}$$

Next, determine the fraction of degree of freedom mass not included in the summation of these modes:

$$e_i = d_i - \delta_{ij}$$

where δ_{ij} is the Kronecker delta, which is 1 if degree of freedom i is in the direction of the earthquake motion and 0 if degree of freedom i is a rotation or not in the direction of the earthquake input motion.

If, for any degree of freedom i, the absolute value of this fraction e_i exceeds 0.1, the response from higher modes is included with those included in Step 1.

Step 3 Higher modes can be assumed to respond in phase with the zero period acceleration and, thus, with each other. Hence, these modes are combined algebraically, which is equivalent to pseudostatic response to the inertial forces from these higher modes excited at the zero period acceleration. The pseudostatic inertial forces associated with the summation of all higher modes for each degree of freedom i are given by:

$$P_i = ZPA \times M_i \times e_i$$

where:

P_i = force or moment to be applied by degree of freedom i

M_i = mass or mass moment of inertia associated with degree of freedom i.

The subsystem is then statically analyzed for this set of pseudo static inertial forces applied to all degrees of freedom to determine the maximum responses associated with high-frequency modes not included in Step 1.

Step 4 The total combined response to high-frequency modes (Step 3) is combined by the square root of sum of the squares method with the total combined response from lower-frequency modes (Step 1) to determine the overall structural peak responses.

3.7.3.7.2 Combination of Low-Frequency Modes

This subsection describes the method for combining modal responses in the seismic response spectra analysis. The total unidirectional seismic response for subsystems is obtained by combining the individual modal responses using the square root sum of the squares method. 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 percent of the lower frequency.

Combined total response for systems having such closely spaced modal frequencies is obtained by adding to the square root sum of squares of all modes the product of the responses of the modes in each group of closely spaced modes and coupling factor. This can be represented mathematically as:

$$R_T^2 = \sum_{i=1}^{N} R_i^2 + 2 \sum_{j=1}^{S} \sum_{k=Mj}^{Nj-1} \sum_{\ell=k+1}^{Nj} R_k \, R_\ell \, \varepsilon_{k\ell}$$

where:

R_T = total unidirectional response
R_i = absolute value of response of mode i
N = total number of modes considered
S = number of groups of closely spaced modes
M_j = lowest modal number associated with group j of closely spaced modes
N_j = highest modal number associated with group j of closely spaced modes
$\varepsilon_{k\ell}$ = coupling factor, defined as follows:

$$\varepsilon_{k\ell} = \left(1 + \frac{(w'_k - w'_\ell)^2}{(\beta'_k \, w_k + \beta'_\ell \, w_\ell)^2}\right)^{-1}$$

and,

$$w'_k = w_k \left[1 - (\beta_k)^2\right]^{1/2}$$

$$\beta'_k = \beta_k + \frac{2}{w_k t_d}$$

where:

w_k = frequency of closely spaced mode k
β_k = fraction of critical damping in closely spaced mode k
t_d = duration of the earthquake (= 30 seconds)

Alternatively, a more conservative 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 is no greater than 10 percent. Therefore,

$$R_T^2 = \sum_{i=1}^{N} R_i^2 + 2\sum \varepsilon_{kl} R_k R_l$$

where:

$$\frac{|w_k - w_\ell|}{w_\ell} \leq 0.1$$

All other terms for the modal combination remain the same. The 10 percent grouping method is more conservative than the grouping method because the same mode can appear in more than one group.

In addition to the above methods, any of the other methods in Regulatory Guide 1.92 may be used for modal combination.

3.7.3.8 Analytical Procedure for Piping

This subsection describes the modeling methods and analytical procedures for piping systems.

The piping system is modeled as beam elements with lump masses connected by a network of elastic springs representing the stiffness properties of the piping system. Concentrated weights such as valves or flanges are also modeled as lump masses. The effects of torsion (including eccentric masses), bending, shear, and axial deformations, and effects due to the changes in stiffness values of curved members are accounted for in the piping dynamic model.

The lump masses are selected so that the maximum spacing is not greater than the length that would produce a natural frequency equal to the zero period acceleration (ZPA) frequency of the seismic input when calculated based on a simply supported beam. As a minimum, the number of degrees of freedom is equal to twice the number of modes with frequencies less than the ZPA frequency.

The stiffness matrix of the piping system is calculated based on the stiffness values of the pipe elements and support elements. Minimum rigid or calculated support stiffness values are used (see subsections 3.9.3.1.5 and 3.9.3.4). When the support deflections are limited to 1/8 inches in the combined faulted condition, minimum rigid support stiffness values are used. If the

combined faulted condition deflection for any support exceeds 1/8 inches, calculated support stiffness values are used for the piping system.

Valves, equipment and piping modules are considered as rigid if the natural frequencies are greater than 33 hertz. Valves with lower frequencies are included in the piping system model. See subsection 3.7.3.8.2.1 for flexible equipment and subsection 3.7.3.8.3 for flexible modules

See subsection 3.9.3.1.4 for the primary loop piping and support system.

3.7.3.8.1 Supporting Systems

This subsection deals with the analysis of piping systems that provide support to other piping systems. The supported piping system may be excluded from the analysis of the supporting piping system when the ratio of the supported pipe to supporting pipe moment of inertia is less than or equal to 0.04.

If the ratio of the run piping outside diameter to the branch piping outside diameter (nominal pipe size) exceeds or equals 3.0, the branch piping can be excluded from the analysis of the run piping. The mass and stiffness effects of the branch piping are considered as described below.

Stiffness Effect

The stiffness effect of the decoupled branch pipe is considered significant when the distance from the run pipe outside diameter to the first rigid or seismic support on the decoupled branch pipe is less than or equal to one half the deadweight span of the branch pipe (given in ASME III Code Subsection NF).

Mass Effect

Considering one direction at a time, the mass effect is significant when the weight of half the span (from the decoupling point) of the branch pipe in one direction is more than 20 percent the weight of the main run pipe span in the same direction. Concentrated weights in the branch pipe are considered. A branch pipe span in x direction is the span between the decoupled branch point and the first seismic or rigid support in the x direction. A main run pipe span in the x direction is the piping bounded by the first seismic or rigid support in the x direction on both sides of the decoupled branch point. Similarly, the same definition applies to the spans in other directions (y and z).

If the calculated branch pipe weight is less than 20 percent but more than 10 percent of the main run pipe weight, this weight is lumped at the decoupling point of the run pipe for the run pipe analysis. This weight can be neglected if it is less than 10 percent of the main run pipe weight.

Required Coupled Branch Piping

If the stiffness and/or mass effects are considered significant, the branch piping is included in the piping analysis for the run pipe analysis. The portion of branch piping considered in the analysis adequately represents the behavior of the run pipe and branch pipe. The branch line model ends in one of the following ways: a) the first six-way anchor; b) four rigid/seismic supports in each of the three perpendicular directions; or c) a rigidly supported zone as described in subsection 3.7.3.13.4.2.

3.7.3.8.2 Supported Systems

This subsection deals with the analysis of piping systems that are supported by other piping systems or by equipment.

3.7.3.8.2.1 Large Diameter Auxiliary Piping

This subsection deals with ASME Class 1 piping larger than one inch nominal pipe size and ASME Class 2 and 3 piping with nominal pipe size larger than two inches. The response spectra methodology is used.

When the supporting system is a piping system, the supported pipe (branch) can be decoupled from the supporting pipe (run) when the ratio of the run piping nominal pipe size to branch pipe nominal pipe size is greater than or equal to three to one. Decoupling can also be done when the moment of inertia of the branch pipe is less than or equal to 4 percent of the moment of inertia of the run pipe.

When the run pipe is decoupled from the analytical model of the branch pipe, the connection point is considered to be anchored for seismic inertia analysis. The response spectra for this analytical anchor are the spectra at the building floor location corresponding to run pipe supports near the connection point. The motions of the connection point are determined from a separate seismic inertia analysis of the run pipe. These motions are applied as static anchor motions to the branch pipe. This criterion is accepted industry practice. For example, a 3-inch branch pipe can be decoupled from a 10-inch or a larger run pipe. When this approach is used the safe shutdown earthquake inertial displacement of the run pipe branch location (or equipment nozzle) is limited to 1 inch in each of three coordinate directions to minimize the amplification effects of the run pipe (or equipment) on the building floor response spectra.

During the analysis of the branch piping, resulting values of tee anchor reactions are checked against the capabilities of the tee.

When the supporting system is equipment, the supported pipe can be decoupled from the supporting equipment using the same criteria as when the supporting system is a piping

system with the run pipe stiffness replaced by the stiffness of the equipment. The equipment stiffness at the nozzle to pipe weld must satisfy the following:

$$K \geq \frac{200EI}{L^3}$$

where:

K = Equipment stiffness along each of three orthogonal directions

E = Young's Modulus

I = Moment of inertia of equivalent run pipe that can be decoupled for branch pipe analysis

L = Span length of equivalent run pipe based on Table NF-3611-1, ASME Code, Section III

3.7.3.8.2.2 Small-Diameter Auxiliary Piping

This subsection deals with ASME Code Class 1 piping equal to or less than 1-inch nominal pipe size and ASME Class 2 and 3 piping with nominal pipe sizes less than or equal to two inches. This includes instrumentation tubing. These piping systems may be supported by equipment or primary loop piping or other auxiliary piping or both. The response spectra or equivalent static load methodology is used. One of the following methods may be used for these systems:

- Same method as described in subsection 3.7.3.8.2.1; or,
- Equivalent static analysis based on appropriate load factors applied to the response spectra acceleration values.

3.7.3.8.3 Piping Systems on Modules

Many portions of the systems for the AP600 are assembled as modules offsite and shipped to the plant as completed units. This method of construction does not result in any unique requirements for the analysis of these structures, systems, or components. Existing industry standards and regulatory requirements and guidelines are appropriate for the evaluation of structures, systems, and components included in modules.

The modules are constructed using a structural steel framework to support the equipment, pipe, and pipe supports in the module. The structural steel framework is designed as part of the building structure according to the criteria given in subsection 3.8.4.

One exception is the pressurizer and safety relief valve module, which is attached to the top of the pressurizer. For this module the structures and piping arrangements support valves off the pressurizer and not the building structure. The structural steel frame is designed as a component support according to ASME Code, Section III, Subsection NF. Piping in modules is routed and analyzed in the same manner as in a plant not employing modules. Piping is analyzed from anchor point to anchor point, which are not necessarily at the boundaries of the module. This is consistent with the manner in which room walls are treated in a nonmodule plant.

The supported piping or component may be decoupled from the seismic analysis of the structural frame based on the following criteria. The mass ratio, Rm, and the frequency ratio, Rf, are defined as follows:

Rm = mass of supported component or piping/mass of supporting structural frame

Rf = frequency of the component or piping/frequency of the structural frame

Decoupling may be done when:

- Rm < 0.01, for any Rf, or
- Rm ≥ 0.01 and ≤ 0.10, if Rf ≤ 0.8 or if Rf is ≥ 1.25.

In addition, supported piping may be decoupled if analysis shows that the effect on the structural frame is small, that is, when the change in response is less than 10 percent. When piping or components are decoupled from the analysis of the frame, the contributory mass of the piping and components is included as a rigid mass in the model of the structural frame.

When piping or components are decoupled from the analysis of the frame using the preceding criteria, the effect of the frame is accounted for in the analysis of the decoupled components or piping. Either an amplified response spectra or a coupled model is used. The amplified response spectra are obtained from the time history SSE analysis of the frame. The coupled model consists of a simplified mass and stiffness model of the frame connected to the seismic model of the components or piping.

Alternative criteria may be applied to simple frames that behave as pipe support miscellaneous steel. Decoupling may be done when the deflection of the frame due to combined faulted condition loading is less than or equal to 1/8 inch. These deflections are defined with respect to the structure to which the structural frame is attached. The stiffness of the intervening elements between the frame and the supported piping or component is considered as follows: Rigid stiffness values are used for fabricated supports, and vendor stiffness values are used for standard supports such as snubbers and rigid gapped supports. The mass of the structural frame is evaluated as a self-weight excitation loading on the frame and the structures supporting the frame. The same approach is used for pipe support miscellaneous steel, as described in subsection 3.9.3.4.

When the supported components or piping cannot be decoupled, they are included in the analysis model of the structural frame. The interaction between the piping and the frame is incorporated by including the appropriate stiffness and mass properties of the components, piping, and frame in the coupled model.

3.7.3.8.4 Piping Systems with Gapped Supports

This subsection describes the analysis methods for piping systems with rigid gapped supports. These supports may be used to minimize the number of pipe support snubbers and the corresponding inservice testing and maintenance activities.

The analysis consists of an iterative response spectra analysis of the piping and support system. Iterations are performed to establish calculated piping displacements that are compatible with the stiffness and gap of the rigid gapped supports. The results of the computer program GAPPIPE, which uses this methodology, are supported with test data (Reference 13).

The method implemented in GAPPIPE to analyze piping systems supported by rigid gapped supports is based on the equivalent linearization technique. GAPPIPE analysis is performed whenever snubber supports are replaced by rigid gapped supports.

The basis of the concept is to find an equivalent linear spring with a response-dependent stiffness for each nonlinear rigid gapped support, or Limit Stop, in the mathematical model of the piping system. The equivalent linearized stiffness minimizes the mean difference in force in the support between the equivalent spring and the corresponding original gapped support. The mean difference is estimated by an averaging process in the time domain, that is, across the response duration, using the concept of random vibration. Details of the design and analysis methods and modeling assumptions are described in Reference 34.

3.7.3.9 Combination of Support Responses

This subsection describes alternative methods for combining the responses from the individual support or attachment points that connect the supported system or subsystem to the supporting system or subsystem. There are two aspects to the responses from the support or attachment points: seismic anchor motions and envelope or multiple-input response spectra methodology.

Seismic Anchor Motions - The response due to differential seismic anchor motions is calculated using static analysis (without including a dynamic load factor). In this analysis, the static model is identical to the static portion of the dynamic model used to compute the seismic response due to inertial loading. In particular, the structural system supports in the static model are identical to those in the dynamic model.

The effect of relative seismic anchor displacements is obtained either by using the worst combination of the peak displacements or by proper representation of the relative phasing characteristics associated with different support inputs. For components supported by a single concrete building (coupled shield and auxiliary buildings, or containment internal structures), the seismic motions at all elevations above the basemat are taken to be in phase. When the

component supports are in the same structure, the relative seismic anchor motions are small and the effects are neglected. This is applicable to building structures and to those supplemental steel frames that are rigid in comparison to the components. Supplemental steel frames that are flexible can have significant seismic anchor motions which are considered. When the components supports are in different structures, the relative seismic anchor motion between the structures is taken to be out-of-phase and the effects are considered. The results of the modal spectra analysis (multiple input or envelope) are combined with the results from seismic anchor motion by the absolute sum method.

Response Spectra Methods - The envelope broadened uniform-input response spectra can lead to excessive conservatism and unnecessary pipe supports. The peak shifting method and independent support motion spectra method are used to avoid unnecessary conservatism.

Seismic Response Spectra Peak Shifting

The peak shifting method may be used in place of the broadened spectra method, as described below.

Determine the natural frequencies $(f_e)_n$ of the system to be qualified in the broadened range of the maximum spectrum acceleration peak.

If no equipment or piping system natural frequencies exist in the ±15 percent interval associated with the maximum spectrum acceleration peak, then the interval associated with the next highest spectrum 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 then evaluated by performing N + 3 separate analyses using the envelope unbroadened floor design response spectrum and the envelope unbroadened spectrum 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, etc.) at any given point in the system. If three different floor response spectrum 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 spectrum curves.

Independent Support Response Spectrum Methods

The use of multiple-input response spectra accounts for the phasing and interdependence characteristics of the various support points. The following alternative methods are used for the AP600 plant. These are based on the guidelines provided by the "Pressure Vessel Research Committee Technical Committee on Piping Systems" (Reference 14).

Envelope Uniform Response Spectra - Method A - The seismic response spectrum that envelopes the supports is used in place of the spectra at each support in the envelope uniform response spectra. Also, the contribution from the seismic anchor motion of the support points is assumed to be in phase and is added algebraically as follows:

$$q_i = d_i \sum_{j=1}^{N} P_{ij}$$

where:

q_i	=	combined displacement response in the normal coordinate for mode i
d_i	=	maximum value of d_{ij}
d_{ij}	=	displacement spectral value for mode i associated with support "j"
P_{ij}	=	participation factor for mode i associated with support j
N	=	number of support points

Enveloped response spectra are developed as the seismic input in three perpendicular directions of the piping 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 spectrum 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 consideration of closely spaced modes and highly frequency modes to obtain the resultant forces, moments, displacements, accelerations, and support loads. The total seismic responses are combined by square-root-sum-of-the-squares method for all three earthquake directions.

Independent Support Motion - Method B - When there are more than one supporting structure, the independent support motion (ISM) method for seismic response spectra may be used.

Each support group is considered to be in a random-phase relationship to the other supports groups. The responses caused by each support group are combined by the square-root-sum-of-the-square method. The displacement response in the modal coordinate becomes:

$$q_i = [\sum_{j=1}^{N} (P_{ij} d_{ij})^2]^{1/2}$$

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.

3.7.3.10 Vertical Static Factors

Constant static factors can be used in some cases for the design of seismic Category I subsystems and equipment. The criteria for using this method are presented in subsection 3.7.3.5.

3.7.3.11 Torsional Effects of Eccentric Masses

The methods used to account for the torsional effects of valves and other eccentric masses (for example, valve operators) in the seismic subsystem analyses are as follows:

- When valves and other eccentric masses are considered rigid, the mass of the operator and valve body or other eccentric mass are located at their respective center of gravity. The eccentric components (that is, yoke, valve body) are modeled as rigid members.

- When valves and other eccentric masses are not considered rigid, the dynamic models are simulated by the lumped masses in discrete locations (that is, center of gravity of valve body and valve operator), coupled by elastic members with properties of the eccentric components.

3.7.3.12 Seismic Category I Buried Piping Systems and Tunnels

There are no seismic Category I buried piping systems and tunnels in the AP600 design.

3.7.3.13 Interaction of Other Systems with Seismic Category I Systems

The safety functions of seismic Category I structures, systems, and components are protected from interaction with nonseismic structures, systems, and components; or their interaction is evaluated. The safety-related systems and components required for safe shutdown are described in Section 7.4. This equipment is located in selected areas of the auxiliary building and inside containment. The primary means of protecting safety-related structures, systems, and components from adverse seismic interactions are discussed in the following paragraphs in the order of preference.

- Separation - separation with the use of physical barriers

- Segregation - routing away from location of seismic Category I systems, structures, and components

- Impact Evaluation - contact with seismic Category I systems, structures, and components may occur, and there is insufficient energy in the impact to cause loss of safety function.

- Support as seismic Category II

Interaction of connected systems with seismic Category I piping is considered by including the other piping in the analysis of the seismic Category I system. Interaction of piping systems that are adjacent to Category I structures, systems, and components is also considered. This is discussed in subsection 3.7.3.13.4.

The containment and each room outside containment containing safety-related systems or equipment, as identified in Table 3.7.3-1, are reviewed for potential adverse seismic interactions to demonstrate that systems, structures, and components are not prevented from performing their required safe shutdown functions. In addition, the review identifies the protection features required to mitigate the consequences of seismic interaction in an area that contains safety-related equipment.

The evaluation steps to address seismic interaction taken for each room or building area containing seismic Category I systems, structures, and components are:

- Define targets susceptible to damage (sensitive targets)

  Sensitive targets are those seismic Category I components for which adverse spatial interaction can result in loss of safety function

- Define sources which can potentially interact in an adverse manner with the target

- If possible, assure adequate free space to eliminate the possibility of seismically-induced damaging impacts for the sensitive targets

- Assess impact effects (interaction) when adequate free space is not present

- Correct adverse seismic interaction conditions

The three-dimensional computer model and composites developed for the nuclear island are used during the design process of the systems and components in the nuclear island, to aid in evaluating and documenting the review for seismic interactions. This review is performed using the design criteria and guidelines described in Subsections 3.7.3.13.1 through 3.7.3.13.4.

The seismic interaction review will be updated by the Combined License applicant. This review is performed in parallel with the seismic margin evaluation (see PRA Report,

Appendix H, Subsection H.2.5). The review is based on as-procured data, as well as the as-constructed condition.

3.7.3.13.1 Separation and Segregation

Separation - The general plant arrangement provides physical separation between the seismic Category I and nonseismic structures, systems, and components to the maximum extent practicable in the nuclear island. The objective is to assist in the preclusion of a potential adverse interaction if the nonseismic structures, systems and components were to fail during a seismic event. Whenever possible nonseismic pipe, electrical raceway, or ductwork is not routed above or adjacent to safety-related equipment, pipe, electrical raceway, or ductwork thereby eliminating the possibility of seismic interaction.

Workstations and other equipment in the Main Control Room are separated from piping. Further, as stated in Section 3.2.1.1.2, structures, systems, and components that are located overhead in the Main Control Room are supported as seismic Category II.

Segregation - Where separation by physical means cannot be accomplished and it becomes necessary to locate or route nonseismic structures, systems, and components in or through safety-related areas, the nonseismic structures, systems and components are segregated from the seismic Category I items to the extent practicable.

Nonseismic cabinets are separated or segregated from seismic Category I cabinets. Also, if a cabinet is a source or a target, the cabinet doors must be secured by latches or fasteners to assure they do not open during a seismic event.

3.7.3.13.2 Impact Analysis

Adverse spatial interaction (i.e., loss of structural integrity or function effecting safety) can potentially occur when two items are in close proximity. Adverse spatial interaction can result from contact or impact from overturning. Seismic Category I systems, structures, and components that are sensitive to seismic interaction are identified as potential targets. Sources are structures or components that can have adverse spatial interaction with the seismic Category I systems, structures, and components. Identification and evaluation of spatial interactions includes the following considerations:

- Proximity of the source to the target. That is, the location of the source within the impact evaluation zone (shown in Figure 3.7.3-1)

  If a source is outside the impact evaluation zone, and does not enter this zone if overturning occurs, no adverse spatial interaction can occur with the identified target. If the source is within the impact evaluation zone and the supports of the source fail, the source could free fall potentially impacting the target.

- Robustness of target

  If a target has significant structural integrity, and its function is not an issue, adverse spatial interaction could not occur with the identified source.

- Energy of impact

  The energy of the source impacting the target may be so low as not to cause adverse spatial interaction with the target.

A specific nonseismic structure, system, or component identified as a source to a specific safety-related component can be acceptable without being supported as seismic Category II, if an analysis demonstrates that the weight and configuration of the source, relative to the target, and the trajectory of the source are such that the interaction would not cause unacceptable damage to the target. For example, a nonseismic instrument tube routed above a seismic Category I electrical cable tray would not pose a hazard and would be acceptable.

Nonseismic equipment can overturn as a result of a safe shutdown earthquake. The trajectory of its fall is evaluated to determine if it poses a potential impact hazard to a safety-related structure, system, or component. If it poses a hazard, the equipment is relocated, or it is supported as described in subsection 3.7.3.13.3.

Nonseismic walls, platforms, stairs, ladders, grating, handrail installations, or other structures next to safety-related structures, systems, and components are evaluated to determine if their failure is credible.

Should a nonseismic structure, system, or component be capable of being dislodged from its supports, the trajectory of its fall is evaluated for potential adverse impacts. If these present a hazard, the structure, system or component is relocated or supported as described in subsections 3.7.3.13.3 and 3.7.3.13.4. Impact is assumed for sources within an impact evaluation zone around the safety-related equipment. The impact evaluation zone is defined as the envelope around the target for which a source, if located outside of the envelope, would not impact the target during a safe shutdown earthquake in the event the supports of the source were to fail and allow the source to fall. The impact evaluation zone is defined by the volume extending 6 feet horizontally from the perimeter of the seismic Category I object up to a height of 35 feet. The impact evaluation zone above 35 feet is defined by a 10-degree cone radiating vertically from the foot of the object, projected from its perimeter. This definition of the impact evaluation zone is illustrated in Figure 3.7.3-1. The impact evaluation zone need not extend beyond seismic Category I structures such as walls or floor slabs.

The following seismic Category I equipment (potential targets) are not sensitive to piping, HVAC ducts, and cable tray interaction because they are robust to these types of impact:

- Tanks, "heavy" equipment (e.g., heat exchangers, etc.)
- Mechanical or electrical penetrations
- HVAC
- Adjacent Piping

- Conduits
- Cable trays
- Structures

3.7.3.13.3 Seismic Category II Supports

Where the preceding approaches of separation, segregation, or impact analysis cannot prevent unacceptable interaction, the source is classified and supported as seismic Category II. The seismic Category II designation provides confidence that these nonseismic structures, systems, and components can withstand the forces of a safe shutdown earthquake in addition to the loading imparted on the seismic Category II supports due to failure of the remaining nonseismically supported portions. This includes nozzle loads from the nonseismic piping. Design methods and stress criteria for systems, structures, and components classified as seismic Category II are the same as for seismic Category I systems, structures, and components, except for piping which is described in subsection 3.7.3.13.4.2. However, the functionality of these seismic Category II sources does not have to be maintained following a safe shutdown earthquake.

HVAC duct and/or cable trays within the impact evaluation zone are seismically supported using the criteria given in Appendices 3G and 3H for seismic Category I assuring that the HVAC and cable tray segments identified as a source will not fall or adversely impact the sensitive target. Adequate free space between the source and target is assured using the load combination that includes the safe shutdown earthquake. The seismic displacement of the HVAC duct and/or cable tray is 6 inches or the calculated displacement.

Nonseismic equipment identified as a source within the impact evaluation zone is supported as seismic Category II. Support seismic loads include seismic inertia loads of the equipment determined as described in subsection 3.7.3.5 and nozzle loads from attached piping determined as described in subsection 3.7.3.13.4.2. Adequate free space is assessed considering a six inch deflection envelope for equipment identified as a source, or calculated deflections obtained using the safe shutdown earthquake load combination and elastic analysis.

3.7.3.13.4 Interaction of Piping with Seismic Category I Piping Systems, Structures, and Components

This subsection describes the design methods for piping to prevent adverse spatial interactions.

3.7.3.13.4.1 Seismic Category I Piping

The safe shutdown earthquake piping displacements obtained for the seismic Category I piping are used for the evaluation of seismic interaction with sensitive equipment. Adequate free space between a source and a target is checked adding absolutely the piping safe shutdown earthquake deflection and the safe shutdown earthquake target deflection along with the other loads (eg., dead weight, thermal) that are in the appropriate design criteria load combinations. Sensitive equipment for piping as the source is seismic Category I equipment shown in

Table 3.7.3-2 along with the portion that must be protected ("zone of protection"). Supports may be added to limit seismic movement to eliminate potential adverse interaction.

3.7.3.13.4.2 Seismic Category II Piping

This subsection describes the methods and criteria for piping that is connected to seismic Category I piping. Interaction of seismic Category I piping and nonseismic Category I piping connected to it is achieved by incorporating into the analysis of the seismic Category I system a length of pipe that represents the actual dynamic behavior of the complete run of the nonseismic Category I system. The length considered is classified as seismic Category II and extends to the interface anchor or rigid support as described below.

The seismic Category II portion of the line, up to the interface anchor or interface rigid support (last seismic support), is analyzed according to Equation 9 of ASME Code, Section III, Class 3, with a stress limit equal to the smaller of 4.5 S_h and 3.0 S_y. In either case, the nonseismic piping is isolated from the seismic Category I piping by anchors or seismic supports. The anchor or seismic Category II supports are designed for loads from the nonseismic piping. This includes three plastic moment components (M_{p1}, M_{p2}, or M_{p3}) in each of three local coordinate directions. The responses to the three moments are evaluated independently. The seismic Category II portion of the line is analyzed by the response spectrum or equivalent static load method for safe shutdown earthquake.

Single Interface Anchor

The seismic Category II piping may be terminated at a single interface anchor (six-way). This anchor and the supports on the seismic Category II piping are evaluated for safe shutdown earthquake loadings using the rules of ASME III Subsection NF. If the anchor is an equipment nozzle, then the equipment load path through the equipment supports are evaluated to the same acceptance criteria as seismic Category I equipment.

Anchor Followed by a Series of Seismic Supports

The seismic Category II piping may be terminated at the last seismic support which follows a six-way anchor on the seismic Category II piping. This last seismic support and the supports on the seismic Category II piping are evaluated for safe shutdown earthquake loadings using the rules of ASME III Subsection NF. From the anchor to the last seismic support the response to the plastic moments (M_{p1}, M_{p2}, or M_{p3}) is combined with the responses to seismic anchor motions and equivalent static seismic inertia of the piping system by the absolute sum method. The responses to these moments are evaluated independently. The support and anchor loads due to the plastic moments (M_{p1}, M_{p2}, or M_{p3}) of the seismically analyzed and supported section can be reduced if the elbow/bend resultant moments have exceeded the plastic limit moments of the elbow/bend. The value of the reduction factor RF is as follows:

RF = Multiplier used to reduce the interface anchor and support loads

RF = < 1, (if RF > 1, no reduction is applicable)

RF	=	M_L/M_a
M_a	=	Resultant moment at elbow/bend. Use maximum value if several elbows/bends are within seismically supported region.
M_L	=	$0.8h^{0.6} D^2t\ Sy$ for $h < 1.45$
M_L	=	$D^2t\ Sy$ for $h > 1.45$
h	=	Flexibility characteristic of elbow/bend
D	=	Outside diameter of elbow/bend
t	=	Thickness of elbow/bend
R	=	Bend radius of elbow/bend

Rigid Region

The seismic Category II piping may be terminated at the last seismic support of a rigidly supported region of the piping system. The rigid region is typically defined as either four bi-lateral supports around an elbow or six bilateral supports around a tee. The structural behavior of the rigid region is similar to that of a six-way anchor. This last seismic support in the rigid region and the supports on the seismic Category II piping are evaluated for safe shutdown earthquake loadings using the rules of ASME III Subsection NF.

3.7.3.13.4.3 Nonseismic Piping

Nonseismic piping within the impact evaluation zone is seismically supported, thereby ensuring that the pipe segment identified as a source will not fall or adversely impact the sensitive target (Table 3.7-2). This situation is shown in Figure 3.7.3-2, and the seismic supported piping criteria described below:

- Supports within the impact evaluation zone, plus 1 transverse support in each transverse direction beyond the impact evaluation zone, are classified as seismic Category II, and are evaluated for the safe shutdown earthquake loading using the rules of ASME III, Subsection NF.

- Piping within the impact evaluation zone plus one transverse support in each transverse direction are evaluated to Equation 9 of ASME Code, Section III, Class 3, with a stress limit equal to the smaller of 4.5 S_h and 3.0 S_y. Outside the impact evaluation zone, the nonseismic piping meets ASME/ANSI B31.1 requirements.

- The nonseismic piping and seismic Category II supports are designed for loads from the nonseismic piping beyond the impact evaluation zone. This includes three plastic moment components (M_{p1}, M_{p2}, or M_{p3}) in each of three local coordinate directions applied at the first and last seismic Category II support. The responses to the three

moments are evaluated independently. The response from the moments applied at the first seismic Category II support is combined with the response from the moments applied at the last seismic Category II support and with the responses to seismic anchor motions and equivalent static seismic inertia of the piping system by the absolute sum method. The support and anchor loads due to the plastic moments (M_{p1}, M_{p2}, or M_{p3}) of the seismically analyzed and supported section can be reduced if the elbow/bend resultant moments have exceeded the plastic limit moments of the elbow/bend. The value of the reduction factor RF is the same as the value for connected seismic Category II piping described above.

- The piping segment identified as the source has at least one effective axial support.

- Adequate free space between a source and a target is checked adding absolutely the piping safe shutdown earthquake deflections (defined following seismic Category II piping analysis methodology) and the safe shutdown earthquake target deflection. Also included are the displacements associated with the appropriate load cases.

- When the anchor is an equipment nozzle, the equipment is supported as seismic Category II as described in subsection 3.7.3.13.3.

3.7.3.14 Seismic Analyses for Reactor Internals

See subsection 3.9.2 for the dynamic analyses of reactor internals.

3.7.3.15 Analysis Procedure for Damping

Damping values used in the seismic analyses of subsystems are presented in subsection 3.7.1.3. SSE damping values used for different types of analysis are provided in Table 3.7.1-1. 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. Composite modal damping for coupled building and piping systems is used for piping systems that are coupled to the primary coolant loop system and the interior concrete building. Piping systems analyzed by the uniform envelope response spectra method, including coupled equipment, and valves, can be evaluated with 5 percent damping. Five percent damping is not used in piping systems that are susceptible to stress corrosion cracking.

For the time history dynamic analysis and independent support motion response spectra analysis of piping systems, 4 percent, 3 percent, and 2 percent damping values are used as described in Table 3.7.1-1.

When piping systems and nonsimple module steel frames (simple frames are described in subsection 3.7.3.8.3) are in a single coupled model, composite damping, as described in subsection 3.7.1.3 is used.

3.7.3.16 Analysis of Seismic Category I Tanks

This subsection describes the seismic analyses for the large, atmospheric seismic Category I pools and tanks. These are reinforced concrete structures with stainless steel liners, as discussed in subsections 3.8.3 and 3.8.4 or with structural modules, as discussed in Appendix 3A. They include the spent fuel pit in the auxiliary building, the in-containment refueling water storage tank, and the passive containment cooling water tank incorporated into the shield building roof. There are no other seismic Category I tanks.

The seismic analyses of the tank consider the impulsive and convective forces of the water as well as the flexibility of the walls. For the spent fuel pit, cask loading pit and fuel transfer canal the impulsive loads are calculated by considering a portion of the water mass responding with the concrete walls. The impulsive forces are calculated by conventional methods for rigid tanks. The passive containment cooling water tank is analyzed using methods described in Reference 15 for toroidal tanks. It is also analyzed by finite element methods. The in-containment refueling water storage tank is irregular in plan and is analyzed by finite element methods.

3.7.3.17 Time History Analysis of Piping Systems

The time history dynamic analysis is an alternate seismic analysis method for response spectrum analysis when time history seismic input is used. This method is also used for dynamic analyses of piping systems subjected to time history hydraulic transient loadings or forcing functions induced by postulated pipe breaks. Modal superposition method is used to solve the equations of motion. The computer programs used are GAPPIPE, PS+CAEPIPE, and WECAN.

The modal superposition method is based on the equations of motion which can be decoupled as long as the piping system is within its elastic limit. The modal responses are obtained from integrating the decoupled equations. The total responses are obtained by the algebraic sum of the individual responses of the individual modes at each time step. The time steps used are no larger than the time history input time steps.

For time history analysis using the PS+CAEPIPE program, low frequency modes are combined by absolute sum in the bounded solution analysis and are combined by algebraic sum in the selective true time history analysis. The resultant low frequency responses are combined by square-root-of-the-sum-of-the-squares with the high frequency responses in the time history analysis. Composite modal damping is used with PS+CAEPIPE program.

For time history analysis using the GAPPIPE or WECAN programs, the number of modes used in the modal analysis is chosen so that the results of the dynamic analysis based on the chosen number of modes are within 10 percent of the results of the dynamic analysis based on the next higher number of modes used. The number of modes analyzed is selected to account for the principal vibration modes of the piping system. The modes are combined by algebraic sum. Composite modal damping is used with WECAN program.

3.7.4 Seismic Instrumentation

3.7.4.1 Comparison with Regulatory Guide 1.12

Compliance with Regulatory Guide 1.12 is discussed in this section and in subsection 1.9.1.

3.7.4.1.1 Safety Design Basis

The seismic instrumentation serves no safety-related function and therefore has no nuclear safety design basis.

3.7.4.1.2 Power Generation Design Basis

The seismic instrumentation is designed to provide the following:

- Collection of seismic data in digital format
- Analysis of seismic data after a seismic event

- Operator notification that a seismic event exceeding a preset value has occurred
- Operator notification (after analysis of data) that a predetermined cumulative absolute velocity value has been exceeded.

3.7.4.2 Location and Description of Instrumentation

The following instrumentation and associated equipment are used to measure plant response to earthquake motion.

Four triaxial acceleration sensor units, located as stated in subsection 3.7.4.2.1, are connected to a time-history analyzer. The time-history analyzer recording and playback system is located in a panel in the nuclear island in a room near the main control room. Seismic event data from these sensors are recorded on a solid-state digital recording system at 200 samples per second per data channel.

This solid-state recording system has internal batteries and a charger to prevent the loss of data during a power outage, and to allow data collection in a seismic event during which the power fails. Normally 120 volt alternating current power is supplied from the non-Class 1E dc and uninterruptible power supply system. Recording capacity is 72 hours of data after system initiation. The triaxial acceleration sensors have a dynamic range of 1000 to 1 (0.0001 to 1.0g) and a frequency range of 0.2 to 50 hertz.

Seismic triggers, as recommended by Regulatory Guide 1.12, are not provided since the system uses triaxial acceleration sensor input signals to initiate the time-history analyzer recording and main control room alarms. The system initiation value is adjustable from 0.002g to 0.02g.

The time-history analyzer starts recording triaxial acceleration data from each of the triaxial acceleration sensors after the initiation value has been exceeded. Pre-event recording time is adjustable from 1.2 to 15.0 seconds, and will be set to record at least 3 seconds of pre-event signal. Post-event run time is adjustable from 10 to 90 seconds. Each recording channel has an associated timing mark record with 2 marks per second, with an accuracy of about 0.02 percent.

The instrumentation components are qualified to IEEE 344-1987 (Reference 16).

The sensor installation anchors are rigid so that the vibratory transmissibility over the design spectra frequency range is essentially unity.

3.7.4.2.1 Triaxial Acceleration Sensors

Each sensor unit contains three accelerometers mounted in a mutually orthogonal array mounted with one horizontal axis parallel to the major axis assumed in the seismic analysis.

One sensor unit will be located in the free field. Because this location is site-specific, the planned location will be determined by the Combined License applicant. The AP600 seismic monitoring system will provide for signal input from the free field sensor.

A second sensor unit is located on the nuclear island basemat in the spare battery charger room at elevation 66'-6" near column lines 9 and L.

A third sensor unit is located on the shield building structure at elevation 229' near column lines 4-1 and K.

The fourth sensor unit is located on the containment internal structure on the east wall of the east steam generator compartment just above the operating floor at elevation 138' close to column lines 6 and K.

Seismic instrumentation is not located on equipment, piping, or supports since experience has shown that data obtained at these locations are obscured by vibratory motion associated with normal plant operation.

3.7.4.2.2 Time-History Analyzer

The time-history analyzer receives input from the triaxial acceleration sensors and, when activated as described in subsection 3.7.4.3, begins recording the triaxial data from each triaxial acceleration sensor and initiates audio and visual alarms in the main control room.

This recorded data will be used to evaluate the seismic acceleration of the structure on which the triaxial acceleration sensors are mounted.

The time-history analyzer is a multichannel, digital recording system with the capability to automatically download the recorded acceleration data to a dedicated computer for data storage, playback, and analysis after a seismic event.

The operator may select the analysis of either cumulative absolute velocity or the response spectrum. Analysis results are printed out on a dedicated graphics printer that is part of the system and is located in the same panel as the time-history analyzer.

3.7.4.3 Control Room Operator Notification

The time-history analyzer provides for initiation of audible and visual alarms in the main control room when predetermined seismic acceleration values sensed by any of the triaxial acceleration sensors are exceeded and when the system is activated to record a seismic event. In addition to alarming when the system is activated, the analyzer portion of the system will provide a second alarm if the predetermined cumulative absolute velocity value has been exceeded by any of the sensors. Alarms are annunciated in the main control room.

3.7.4.4 Comparison of Measured and Predicted Responses

The recorded seismic data is used by the combined license applicant operations and engineering departments to evaluate the effects of the earthquake on the plant structures and equipment.

The criterion for initiating a plant shutdown following a seismic event will be exceedance of a specified response spectrum limit or a cumulative absolute velocity limit. The seismic instrumentation system is capable of computing the cumulative absolute velocity as described in EPRI Report NP-5930 (Reference 1) and EPRI Report TR-100082 (Reference 17).

3.7.4.5 Tests and Inspections

Periodic testing of the seismic instrumentation system is accomplished by the functional test feature included in the software of the time-history recording accelerograph. The system is modular and is capable of single-channel testing or single channel maintenance without disabling the remainder of the system.

3.7.5 Combined License Information

3.7.5.1 Seismic Analysis of Dams

Combined License applicants referencing the AP600 certified design will evaluate dams whose failure could affect the site interface flood level specified in subsection 2.4.1.2. The evaluation of the safety of existing and new dams will use the site-specific safe shutdown earthquake.

3.7.5.2 Post-Earthquake Procedures

Combined License applicants referencing the AP600 certified design will prepare site-specific procedures for activities following an earthquake. These procedures will be used to accurately determine both the response spectrum and the cumulative absolute velocity of the recorded earthquake ground motion from the seismic instrumentation system. The procedures and the data from the seismic instrumentation system will provide sufficient information to guide the operator on a timely basis to determine if the level of earthquake ground motion requiring shutdown has been exceeded. The procedures will follow the guidance of EPRI Reports NP-5930 (Reference 1), TR-100082 (Reference 17), and NP-6695 (Reference 18), as modified by the NRC staff (Reference 32).

3.7.6 References

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Table 3.7.1-1

SAFE SHUTDOWN EARTHQUAKE DAMPING VALUES

Welded aluminum structures (%)	8
Welded and friction-bolted steel structures and equipment (%)	4
Bearing bolted structures and equipment (%)	7
Prestressed concrete structures (%)	5
Reinforced concrete structures (%)	7
Concrete filled steel plate structures (%)	5
Primary coolant loop (%)	5
Piping systems (for uniform envelope response spectra analysis)	5
Piping systems (alternative for time history analysis and independent support motion response spectra analysis)	
Less than or equal to 12-inch diameter (%)	2
Greater than 12-inch diameter (%)	3
Primary coolant loop (%)	4
Fuel assemblies (%)	20
Control rod drive mechanisms (%)	5
Cable trays & related supports (%) (see Figure 3.7.1-13)	20
Conduits & related supports (%)	7
HVAC ductwork (%)	7
HVAC welded ductwork (%)	4
Cabinets and panels for electrical equipment (%)	5
Equipment such as welded instrument racks and tanks (%)	3

Table 3.7.1-2

EMBEDMENT DEPTH AND RELATED DIMENSIONS OF CATEGORY I STRUCTURES

Structure	Foundation Embedment Depth (ft)	Least Foundation Width (ft)	Structure Height (ft)
Shield Building	See Note	See Note	246.75
Steel Containment Vessel	See Note	See Note	189.83
Auxiliary Building	See Note	See Note	119.50

Note:
1. The seismic Category I structures are founded on a common basemat embedded 39.5 feet, with dimensions shown in Figure 3.7.1-16.

Table 3.7.1-3

AP600 HORIZONTAL DESIGN RESPONSE SPECTRA RELATIVE VALUES OF SPECTRUM AMPLIFICATION FACTORS FOR CONTROL POINTS

Percent of Critical Damping	Amplification Factors for Control Points				
	Acceleration[1]				Displacement[1]
	A (33 cps)	B' (25 cps)[2]	B (9 cps)	C (2.5 cps)	D (0.25 cps)
2.0	1.0	1.70	3.54	4.25	2.50
3.0	1.0	1.66	3.13	3.76	2.34
4.0	1.0	1.63	2.84	3.41	2.19
5.0	1.0	1.60	2.61	3.13	2.05
7.0	1.0	1.55	2.27	2.72	1.88

AP600 VERTICAL DESIGN RESPONSE SPECTRA RELATIVE VALUES OF SPECTRUM AMPLIFICATION FACTORS FOR CONTROL POINTS

Percent of Critical Damping	Amplification Factors for Control Points				
	Acceleration[1]				Displacement[1]
	A (33 cps)	B' (25 cps)[2]	B (9 cps)	C (3.5 cps)	D (0.25 cps)
2.0	1.0	1.70	3.54	4.05	1.67
3.0	1.0	1.66	3.13	3.58	1.56
4.0	1.0	1.63	2.84	3.25	1.46
5.0	1.0	1.60	2.61	2.98	1.37
7.0	1.0	1.55	2.27	2.59	1.25

Note:

1. Maximum ground displacement is taken proportional to maximum ground acceleration, and is 36 inches for ground acceleration of 1.0 gravity.
2. The 5% damping amplification factor for control point B' is derived per discussion in subsection 3.7.1.1. This 5% damping amplification factor equals 1.3 times the RG 1.60 response spectra at 25 Hertz. The amplification factors at control point B' for other damping values are determined by increasing the RG 1.60 response spectra at 25 hertz by 30 percent.

Table 3.7.2-1

COUPLED SHIELD AND AUXILIARY BUILDINGS LUMPED-MASS STICK MODEL MODAL PROPERTIES

Mode No.	Freq. (CPS)	Genl. Mass	Participation Factors X	Participation Factors Y	Participation Factors Z	Modal Masses X	Modal Masses Y	Modal Masses Z	Cumulative Mass, % X	Cumulative Mass, % Y	Cumulative Mass, % Z
1	4.04	1.00	-0.861	34.698	-1.764	0.742	1203.954	3.112	0.0	28.2	0.1
2	4.42	1.00	-32.920	-0.730	1.099	1083.739	0.534	1.208	25.4	28.2	0.1
3	6.55	1.00	0.406	7.720	-21.618	0.165	59.602	467.357	25.4	29.6	11.1
4	6.66	1.00	-4.325	19.851	10.858	18.701	394.082	117.886	25.9	38.9	13.8
5	6.76	1.00	-22.265	-4.455	-4.144	495.711	19.845	17.175	37.5	39.3	14.2
6	8.62	1.00	3.738	-2.308	-0.082	13.969	5.326	0.007	37.8	39.4	14.2
7	12.10	1.00	2.129	-31.038	-0.983	4.532	963.357	0.965	37.9	62.0	14.2
8	12.45	1.00	-23.372	-2.274	-0.434	546.271	5.171	0.189	50.7	62.1	14.2
9	13.35	1.00	22.072	0.128	0.421	487.160	0.016	0.177	62.1	62.1	14.2
10	19.63	1.00	-1.071	0.504	35.702	1.147	0.254	1274.604	62.1	62.1	44.1
11	22.06	1.00	9.727	10.760	2.204	94.616	115.778	4.857	64.4	64.9	44.2
12	22.13	1.00	12.887	-9.994	1.828	166.077	99.887	3.340	68.3	67.2	44.3
13	23.72	1.00	2.581	6.996	-0.578	6.661	48.940	0.334	68.4	68.3	44.3
14	28.39	1.00	-0.920	11.099	-4.892	0.846	123.177	23.933	68.4	71.2	44.9
15	29.12	1.00	-11.866	-1.044	1.536	140.796	1.089	2.358	71.7	71.3	44.9
16	34.26	1.00	2.756	4.151	28.835	7.593	17.232	831.467	71.9	71.7	64.4
17	35.09	1.00	-7.754	-7.272	10.281	60.120	52.881	105.707	73.3	72.9	66.9
18	35.45	1.00	9.210	-8.158	0.975	84.830	66.546	0.952	75.3	74.5	66.9
19	37.22	1.00	4.194	0.004	-0.705	17.590	0.000	0.496	75.7	74.5	66.9
20	40.20	1.00	9.007	-7.609	0.364	81.128	57.892	0.132	77.6	75.8	66.9
21	40.84	1.00	7.203	9.432	-2.069	51.890	88.955	4.282	78.8	77.9	67.0
22	42.40	1.00	-0.407	-1.151	0.140	0.166	1.324	0.020	78.8	77.9	67.0
23	42.77	1.00	2.039	-3.162	-1.925	4.157	9.996	3.707	78.9	78.2	67.1
24	43.85	1.00	3.069	1.949	5.535	9.419	3.800	30.634	79.2	78.3	67.8
25	46.28	1.00	13.138	1.923	-0.497	172.605	3.698	0.247	83.2	78.3	67.8
26	47.26	1.00	-3.669	9.330	-0.229	13.462	87.041	0.053	83.5	80.4	67.8
27	49.38	1.00	4.575	3.363	-0.057	20.934	11.308	0.003	84.0	80.6	67.8
28	53.87	1.00	-0.441	-8.312	-0.298	0.194	69.089	0.089	84.0	82.3	67.8
29	55.60	1.00	-0.444	0.500	-4.233	0.197	0.250	17.917	84.0	82.3	68.3
30	56.13	1.00	0.385	-0.301	-2.298	0.148	0.091	5.282	84.0	82.3	68.4
31	57.10	1.00	-13.915	2.197	-0.153	193.640	4.827	0.023	88.6	82.4	68.4
32	59.94	1.00	-1.731	-9.937	-1.460	2.997	98.737	2.130	88.6	84.[illegible]	68.4
33	60.92	1.00	0.107	0.209	15.294	0.012	0.044	233.908	88.6	84.7	73.9
34	64.94	1.00	0.845	13.262	-0.572	0.713	175.868	0.328	88.6	88.8	73.9
35	64.98	1.00	0.134	-7.695	1.723	0.018	59.216	2.969	88.6	90.2	74.0
36	65.03	1.00	-2.522	-18.980	-0.610	6.360	360.257	0.372	88.8	98.7	74.0
37	65.81	1.00	20.342	1.163	0.116	413.802	1.351	0.013	98.5	98.7	74.0
					SUMMATIONS	4203.107	4211.416	3158.233			
					TOTAL MASS	4267.520	4267.520	4267.521			

Table 3.7.2-2

STEEL CONTAINMENT
VESSEL LUMPED-MASS STICK MODEL
MODAL PROPERTIES

Mode No.	Freq. (CPS)	Genl. Mass	Participation Factors X	Participation Factors Y	Participation Factors Z	Modal Masses X	Modal Masses Y	Modal Masses Z	Cumulative Mass, % X	Cumulative Mass, % Y	Cumulative Mass, % Z
1	2.19	1.00	0.000	-4.840	0.000	0.000	23.422	0.000	0.0	11.2	0.0
2	4.46	1.00	0.000	0.000	4.686	0.000	0.000	21.955	0.0	11.2	10.4
3	5.06	1.00	-7.687	0.000	0.000	59.086	0.000	0.000	28.4	11.2	10.4
4	7.61	1.00	0.000	-11.595	0.000	0.000	134.453	0.000	28.4	75.8	10.4
5	8.03	1.00	-9.937	0.000	0.000	98.752	0.000	0.000	75.8	75.8	10.4
6	14.92	1.00	0.000	0.000	0.000	0.000	0.000	0.000	75.8	75.8	10.4
7	18.38	1.00	0.000	0.000	-11.509	0.000	0.000	132.456	75.8	75.8	73.3
8	22.02	1.00	0.000	5.528	0.000	0.000	30.563	0.000	75.8	90.5	73.3
9	22.03	1.00	5.534	0.000	0.000	30.628	0.000	0.000	90.5	90.5	73.3
10	30.08	1.00	0.000	0.000	-5.862	0.000	0.000	34.358	90.5	90.5	89.6
11	35.16	1.00	0.000	1.647	0.000	0.000	2.711	0.000	90.5	91.8	89.6
12	35.16	1.00	-1.640	0.000	0.000	2.689	0.000	0.000	91.8	91.8	89.6
13	44.15	1.00	0.000	2.311	0.000	0.000	5.340	0.000	91.8	94.4	89.6
14	44.20	1.00	-2.311	0.000	0.000	5.342	0.000	0.000	94.4	94.4	89.6
15	44.94	1.00	0.000	0.000	0.000	0.000	0.000	0.000	94.4	94.4	89.6
					SUMMATIONS	196.498	196.489	188.769			
					TOTAL MASS	208.206	208.206	210.762			

Note:

1. The first three modes of vibration are principally polar crane response modes.

Table 3.7.2-3 (Sheet 1 of 3)

CONTAINMENT INTERNAL STRUCTURES, WITHOUT RCL LUMPED-MASS STICK MODEL MODAL PROPERTIES

Mode No.	Freq. (CPS)	Genl. Mass	Participation Factors X	Participation Factors Y	Participation Factors Z	Modal Masses X	Modal Masses Y	Modal Masses Z	Cumulative Mass, % X	Cumulative Mass, % Y	Cumulative Mass, % Z
1	13.38	1.00	-1.601	18.512	-0.389	2.563	342.681	0.151	0.2	30.0	0.0
2	15.72	1.00	20.540	4.827	-0.498	421.873	23.295	0.248	37.1	32.0	0.0
3	17.13	1.00	-7.719	9.207	-0.169	59.578	84.770	0.029	42.3	39.4	0.0
4	39.44	1.00	-0.982	-9.211	-2.733	0.963	84.833	7.471	42.4	46.8	0.7
5	41.67	1.00	0.264	24.386	-1.008	0.070	594.653	1.016	42.4	98.8	0.8
6	42.03	1.00	-24.053	0.577	-4.210	578.525	0.333	17.726	93.0	98.8	2.3
7	44.81	1.00	-6.260	0.793	10.407	39.193	0.630	108.308	96.4	98.9	11.8
8	46.21	1.00	5.401	0.769	-8.562	29.174	0.591	73.305	99.0	98.9	18.2
9	47.16	1.00	-0.833	-0.649	-16.664	0.694	0.421	277.673	99.0	99.0	42.4
10	58.40	1.00	0.509	-0.023	2.087	0.259	0.001	4.354	99.0	99.0	42.8
11	61.81	1.00	0.072	-0.051	0.236	0.005	0.003	0.056	99.0	99.0	42.8
12	63.02	1.00	0.058	-0.269	-2.640	0.003	0.072	6.969	99.0	99.0	43.4
13	78.69	1.00	-0.017	0.013	-0.046	0.000	0.000	0.002	99.0	99.0	43.4
14	84.52	1.00	-0.006	0.009	1.417	0.000	0.000	2.007	99.0	99.0	43.6
15	90.93	1.00	0.050	0.172	-21.741	0.003	0.030	472.666	99.0	99.0	84.8
					SUMMATIONS	1132.903	1132.313	971.981			
					TOTAL MASS	1143.900	1143.900	1146.500			

Table 3.7.2-3 (Sheet 2 of 3)

CONTAINMENT INTERNAL STRUCTURES, INCLUDING RCL LUMPED-MASS STICK MODEL MODAL PROPERTIES

Mode No.	Freq. (CPS)	Genl. Mass	Participation Factors X	Participation Factors Y	Participation Factors Z	Modal Masses X	Modal Masses Y	Modal Masses Z	Cumulative Mass,% X	Cumulative Mass,% Y	Cumulative Mass,% Z
1	4.26	1.00	-0.120	1.250	-0.005	0.014	1.561	0.000	0.0	0.1	0.0
2	4.26	1.00	1.139	0.120	-0.004	1.298	0.014	0.000	0.1	0.1	0.0
3	5.05	1.00	0.020	6.998	-0.030	0.000	48.975	0.001	0.1	4.0	0.0
4	6.20	1.00	-0.001	0.000	0.221	0.000	0.000	0.049	0.1	4.0	0.0
5	6.61	1.00	-6.598	-0.199	0.032	43.539	0.040	0.001	3.5	4.0	0.0
6	6.73	1.00	2.457	-0.325	-0.003	6.036	0.106	0.000	4.0	4.0	0.0
7	11.89	1.00	-0.052	-1.907	-0.001	0.003	3.636	0.000	4.0	4.2	0.0
8	12.20	1.00	0.025	-0.131	-8.731	0.001	0.017	76.226	4.0	4.2	6.0
9	12.26	1.00	-0.287	0.842	-1.239	0.082	0.708	1.535	4.0	4.3	6.1
10	12.40	1.00	3.371	0.753	-0.029	11.362	0.567	0.001	4.9	4.3	6.1
11	13.33	1.00	-0.907	21.239	-0.358	0.823	451.075	0.128	4.9	39.6	6.1
12	13.65	1.00	-9.717	0.816	0.003	94.427	0.666	0.000	12.3	39.7	6.1
13	14.11	1.00	-2.748	-0.115	-0.144	7.551	0.013	0.021	12.9	39.7	6.1
14	14.43	1.00	-10.106	-0.112	0.089	102.134	0.012	0.008	20.9	39.7	6.1
15	15.05	1.00	-0.688	-1.326	-0.014	0.473	1.759	0.000	20.9	39.8	6.1
16	15.16	1.00	-0.831	-1.267	0.028	0.690	1.606	0.001	21.0	39.9	6.1
17	15.55	1.00	1.796	0.234	-0.072	3.224	0.055	0.005	21.2	39.9	6.1
18	15.89	1.00	-17.416	-4.755	0.496	303.310	22.612	0.246	45.0	41.7	6.1
19	17.29	1.00	-7.727	8.248	-0.160	59.704	68.022	0.026	49.6	47.0	6.1
20	17.34	1.00	0.370	-0.395	0.047	0.137	0.156	0.002	49.6	47.0	6.1
21	17.68	1.00	0.924	-0.547	0.050	0.854	0.299	0.003	49.7	47.1	6.1
22	18.87	1.00	-0.025	0.025	0.352	0.001	0.001	0.124	49.7	47.1	6.1
23	20.18	1.00	0.058	-0.220	0.020	0.003	0.048	0.000	49.7	47.1	6.1
24	20.29	1.00	0.096	0.866	0.041	0.009	0.750	0.002	49.7	47.1	6.1
25	20.50	1.00	-0.089	0.034	0.013	0.008	0.001	0.000	49.7	47.1	6.1
26	23.20	1.00	-0.009	0.024	-1.186	0.000	0.001	1.407	49.7	47.1	6.2
27	23.86	1.00	-0.006	0.029	-0.110	0.000	0.001	0.012	49.7	47.1	6.2
28	25.34	1.00	0.060	0.076	2.812	0.004	0.006	7.909	49.7	47.1	6.8
29	26.02	1.00	0.396	-0.450	0.064	0.157	0.202	0.004	49.7	47.1	6.8
30	26.33	1.00	-0.693	-0.344	-0.157	0.480	0.118	0.025	49.7	47.1	6.8
31	29.41	1.00	-0.002	0.536	-0.222	0.000	0.287	0.049	49.7	47.2	6.8
32	30.21	1.00	0.414	-0.214	-0.080	0.172	0.046	0.006	49.8	47.2	6.8
33	34.35	1.00	-0.020	-0.019	0.005	0.000	0.000	0.000	49.8	47.2	6.8
34	35.10	1.00	-0.247	-0.143	-9.038	0.061	0.021	81.679	49.8	47.2	13.2
35	36.52	1.00	0.017	-0.171	2.299	0.000	0.029	5.286	49.8	47.2	13.6
36	37.14	1.00	0.655	5.790	0.099	0.430	33.523	0.010	49.8	49.8	13.6
37	37.36	1.00	4.760	-0.797	0.123	22.660	0.635	0.015	51.6	49.8	13.6
38	37.99	1.00	-0.251	0.029	-0.160	0.063	0.001	0.026	51.6	49.8	13.6
39	38.10	1.00	-3.808	0.806	-0.025	14.500	0.650	0.001	52.7	49.9	13.6
40	38.95	1.00	-0.079	-0.301	-1.376	0.006	0.091	1.894	52.7	49.9	13.8
41	39.21	1.00	0.939	10.082	1.930	0.882	101.651	3.724	52.8	57.8	14.1
42	40.34	1.00	0.677	-1.138	0.148	0.458	1.295	0.022	52.8	58.0	14.1
43	41.82	1.00	18.869	-0.321	4.230	356.051	0.103	17.891	80.7	58.0	15.5
44	42.30	1.00	-0.576	-22.739	1.216	0.332	517.042	1.480	80.7	98.4	15.6
45	44.23	1.00	14.100	-1.271	-2.806	198.822	1.615	7.873	96.2	98.5	16.2

Table 3.7.2-3 (Sheet 3 of 3)

CONTAINMENT INTERNAL STRUCTURES, INCLUDING RCL LUMPED-MASS STICK MODEL MODAL PROPERTIES

Mode No.	Freq. (CPS)	Genl. Mass	Participation Factors X	Participation Factors Y	Participation Factors Z	Modal Masses X	Modal Masses Y	Modal Masses Z	Cumulative Mass, % X	Cumulative Mass, % Y	Cumulative Mass, % Z
46	45.94	1.00	0.749	2.299	14.118	0.561	5.286	199.320	96.3	98.9	31.8
47	46.42	1.00	-6.026	-0.763	7.458	36.310	0.582	55.616	99.1	99.0	36.1
48	46.75	1.00	-0.410	-1.226	10.693	0.168	1.504	114.343	99.1	99.1	45.0
49	48.77	1.00	0.067	-0.212	-7.457	0.004	0.045	55.611	99.1	99.1	49.4
50	55.43	1.00	-0.005	0.115	-0.004	0.000	0.013	0.000	99.1	99.1	49.4
51	55.45	1.00	-0.001	-0.001	-0.339	0.000	0.000	0.115	99.1	99.1	49.4
52	58.42	1.00	-0.512	0.024	-2.046	0.262	0.001	4.185	99.1	99.1	49.7
					SUMMATIONS	1268.068	1267.448	636.881			
					TOTAL MASS	1279.110	1279.110	1281.740			

Table 3.7.2-4 (Sheet 1 of 2)

NUCLEAR ISLAND
COMBINED LUMPED-MASS STICK MODEL
MODAL PROPERTIES

Mode	Freq.	Genl.	Participation Factors			Modal Masses			Cumulative Mass, %		
No.	(CPS)	Mass	X	Y	Z	X	Y	Z	X	Y	Z
1	2.01	1.00	-0.002	-0.006	2.983	0.000	0.000	8.896	0.0	0.0	0.1
2	2.19	1.00	-0.003	4.953	-0.004	0.000	24.533	0.000	0.0	0.4	0.1
3	3.06	1.00	0.018	-0.005	0.935	0.000	0.000	0.874	0.0	0.4	0.2
4	3.63	1.00	1.046	-0.374	3.574	1.094	0.140	12.775	0.0	0.4	0.4
5	4.13	1.00	-0.929	32.296	-1.997	0.863	1043.062	3.988	0.0	17.6	0.4
6	4.26	1.00	-0.125	1.058	0.012	0.016	1.120	0.000	0.0	17.6	0.4
7	4.26	1.00	1.224	0.119	-0.007	1.498	0.014	0.000	0.1	17.6	0.4
8	4.46	1.00	-0.909	0.264	4.879	0.826	0.070	23.809	0.1	17.6	0.8
9	4.50	1.00	30.543	0.751	-1.433	932.894	0.564	2.054	15.4	17.6	0.9
10	5.01	1.00	0.040	6.364	0.013	0.002	40.504	0.000	15.4	18.3	0.9
11	5.06	1.00	6.944	0.007	0.033	48.219	0.000	0.001	16.2	18.3	0.9
12	5.19	1.00	0.001	0.000	-0.208	0.000	0.000	0.043	16.2	18.3	0.9
13	6.54	1.00	-0.451	-3.479	24.185	0.203	12.102	584.897	16.2	18.5	10.5
14	6.60	1.00	6.852	-0.330	0.042	46.946	0.109	0.002	17.0	18.5	10.5
15	6.72	1.00	2.701	-1.037	-0.126	7.296	1.076	0.016	17.1	18.5	10.5
16	6.77	1.00	-5.119	19.367	6.323	26.209	375.081	39.985	17.6	24.7	11.2
17	6.87	1.00	-20.300	-5.652	-3.493	412.105	31.940	12.204	24.4	25.2	11.4
18	7.55	1.00	0.028	9.757	0.297	0.001	95.190	0.088	24.4	26.8	11.4
19	7.88	1.00	0.010	-0.527	4.384	0.000	0.278	19.221	24.4	26.8	11.7
20	7.88	1.00	-0.011	0.047	1.873	0.000	0.002	3.509	24.4	26.8	11.8
21	8.00	1.00	-8.932	-0.141	0.068	79.782	0.020	0.005	25.7	26.8	11.8
22	8.63	1.00	-3.130	1.818	0.433	9.794	3.304	0.188	25.8	26.8	11.8
23	8.89	1.00	1.412	-0.128	5.626	1.993	0.016	31.648	25.9	26.8	12.3
24	9.50	1.00	0.628	-0.019	4.870	0.394	0.000	23.722	25.9	26.8	12.7
25	10.68	1.00	-2.231	-13.215	-0.031	4.978	174.626	0.001	25.9	29.7	12.7
26	11.67	1.00	0.147	0.742	-3.779	0.021	0.551	14.281	25.9	29.7	12.9
27	11.91	1.00	0.339	0.357	0.046	0.115	0.127	0.002	26.0	29.7	12.9
28	12.07	1.00	0.046	-0.299	-8.403	0.002	0.090	70.617	26.0	29.7	14.1
29	12.25	1.00	-0.118	0.755	-3.851	0.014	0.570	14.828	26.0	29.7	14.3
30	12.43	1.00	-2.975	2.976	0.052	8.850	8.858	0.003	26.1	29.9	14.3
31	12.72	1.00	5.180	2.513	0.196	26.829	6.313	0.038	26.5	30.0	14.3
32	13.56	1.00	6.929	-18.312	-0.241	48.009	335.343	0.058	27.3	35.5	14.3
33	13.67	1.00	7.380	12.210	0.484	54.472	149.082	0.234	28.2	38.0	14.3
34	13.70	1.00	-2.135	20.730	1.318	4.559	429.720	1.737	28.3	45.0	14.3
35	13.82	1.00	0.351	-4.165	3.515	0.123	17.348	12.353	28.3	45.3	14.6
36	14.36	1.00	-0.736	-0.540	6.472	0.541	0.291	41.885	28.3	45.3	15.2
37	14.42	1.00	12.283	0.508	0.827	150.867	0.258	0.684	30.8	45.3	15.3
38	14.60	1.00	24.809	0.842	0.630	615.494	0.709	0.396	40.9	45.4	15.3
39	14.92	1.00	-4.976	0.213	0.022	24.759	0.045	0.000	41.3	45.4	15.3
40	14.99	1.00	0.149	0.362	-0.020	0.022	0.131	0.000	41.3	45.4	15.3
41	15.08	1.00	-0.736	-1.858	-0.457	0.541	3.454	0.209	41.4	45.4	15.3
42	15.55	1.00	1.985	0.174	-0.004	3.939	0.030	0.000	41.4	45.4	15.3
43	15.81	1.00	16.208	5.365	0.091	262.697	28.786	0.008	45.7	45.9	15.3
44	16.74	1.00	-0.380	0.381	0.872	0.145	0.146	0.761	45.7	45.9	15.3
45	17.22	1.00	7.946	-8.418	-0.795	63.133	70.858	0.632	46.8	47.1	15.3

Table 3.7.2-4 (Sheet 2 of 2)

NUCLEAR ISLAND
COMBINED LUMPED-MASS STICK MODEL
MODAL PROPERTIES

Mode No.	Freq. (CPS)	Genl. Mass	Participation Factors X	Participation Factors Y	Participation Factors Z	Modal Masses X	Modal Masses Y	Modal Masses Z	Cumulative Mass, % X	Cumulative Mass, % Y	Cumulative Mass, % Z
46	17.66	1.00	-0.590	0.631	1.043	0.348	0.398	1.088	46.8	47.1	15.3
47	17.71	1.00	0.642	-0.116	28.566	0.413	0.013	816.031	46.8	47.1	28.8
48	18.06	1.00	-0.145	0.015	-1.918	0.021	0.000	3.678	46.8	47.1	28.8
49	18.06	1.00	0.477	-0.116	-1.298	0.227	0.014	1.686	46.8	47.1	28.9
50	18.10	1.00	0.759	-0.336	-11.372	0.576	0.113	129.322	46.8	47.1	31.0
51	18.17	1.00	-0.173	0.213	-0.107	0.030	0.045	0.011	46.8	47.1	31.0
52	18.91	1.00	0.166	-0.171	13.289	0.028	0.029	176.610	46.8	47.1	33.9
53	19.71	1.00	-1.314	0.103	23.634	1.728	0.011	558.548	46.8	47.1	43.1
54	20.18	1.00	-0.056	0.218	-0.068	0.003	0.047	0.005	46.8	47.1	43.1
55	20.29	1.00	0.087	0.804	0.058	0.008	0.646	0.003	46.8	47.1	43.1
56	20.49	1.00	0.100	0.085	-0.024	0.010	0.007	0.001	46.8	47.1	43.1
57	21.33	1.00	0.250	0.001	-3.765	0.063	0.000	14.174	46.8	47.1	43.3
58	21.57	1.00	-0.042	0.064	-1.436	0.002	0.004	2.061	46.8	47.1	43.4
59	21.96	1.00	-3.588	4.216	-0.046	12.874	17.776	0.002	47.1	47.4	43.4
60	22.00	1.00	3.914	3.832	-0.409	15.320	14.687	0.167	47.3	47.6	43.4
61	22.60	1.00	-0.211	-2.147	0.094	0.044	4.609	0.009	47.3	47.7	43.4
62	22.94	1.00	-1.353	-1.045	-3.270	1.832	1.092	10.696	47.3	47.7	43.6
63	23.05	1.00	2.847	5.301	1.136	8.106	28.103	1.290	47.5	48.2	43.6
64	23.96	1.00	12.253	-2.441	1.106	150.139	5.957	1.223	49.9	48.3	43.6
65	24.70	1.00	-1.161	-9.013	-1.234	1.347	81.242	1.523	50.0	49.6	43.6
66	25.33	1.00	-0.080	-0.121	-3.105	0.006	0.015	9.638	50.0	49.6	43.8
67	25.98	1.00	-0.367	0.589	0.032	0.134	0.347	0.001	50.0	49.6	43.8
68	26.30	1.00	0.683	0.136	0.157	0.466	0.019	0.025	50.0	49.6	43.8
69	28.05	1.00	-0.793	-0.453	4.376	0.628	0.205	19.151	50.0	49.6	44.1
70	29.41	1.00	-0.012	-0.403	-0.045	0.000	0.163	0.002	50.0	49.6	44.1
71	29.69	1.00	0.087	2.207	-12.985	0.008	4.870	168.620	50.0	49.7	46.9
72	30.21	1.00	-0.271	-1.070	0.538	0.073	1.145	0.290	50.0	49.7	46.9
73	30.25	1.00	-0.831	7.810	-2.977	0.691	60.998	8.860	50.0	50.7	47.0
74	30.73	1.00	-7.977	-0.272	1.821	63.639	0.074	3.314	51.1	50.7	47.1
					SUMMATIONS	3099.010	3079.093	2854.682			
					TOTAL MASS	6070.271	6070.271	6062.211			

Table 3.7.2-5 (Sheet 1 of 3)

MAXIMUM ABSOLUTE NODAL ACCELERATION (ZPA)
COUPLED AUXILIARY & SHIELD BUILDINGS

HARD ROCK SITE CONDITION

Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g.)					
	N-S Direction		E-W Direction		Vertical Direction	
307.25	1.34		1.26		0.90	
297.07	1.17		1.08		0.90	
284.41	0.96		0.95		0.89	
271.58	0.84		0.87		0.88	
246.00	0.80		0.78		0.57	
241.00	0.78		0.75		0.57	
230.00	0.72	(0.75)	0.69	(0.72)	0.56	(0.64)
210.00	0.61	(0.64)	0.73	(0.74)	0.52	(0.59)
180.00	0.49	(0.52)	0.72	(0.74)	0.43	(0.53)
160.50	0.49	(0.52)	0.63	(0.67)	0.38	(0.48)
153.00	0.48	(0.51)	0.61	(0.64)	0.37	(0.45)
135.25	0.43	(0.44)	0.48	(0.54)	0.35	(0.40)
117.50	0.37	(0.37)	0.38	(0.41)	0.34	(0.36)
100.00	0.30	(0.30)	0.30	(0.31)	0.32	(0.33)
82.50	0.30	(0.30)	0.30	(0.30)	0.30	(0.31)
66.50	0.30		0.30		0.30	

Note:
1. Enveloped response results at the north, south, east and west edge nodes of the structure are shown in parentheses. This is the maximum value of the response at any of these edge nodes.

Table 3.7.2-5 (Sheet 2 of 3)

MAXIMUM ABSOLUTE NODAL ACCELERATION (ZPA) COUPLED AUXILIARY & SHIELD BUILDINGS

SOFT ROCK SITE CONDITION

Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g.)					
	N-S Direction		E-W Direction		Vertical Direction	
307.25	1.38		1.62		0.94	
297.07	1.20		1.38		0.94	
284.41	1.04		1.17		0.94	
271.58	0.93		1.14		0.93	
246.00	0.84		1.05		0.54	
241.00	0.82		1.02		0.53	
230.00	0.77	(0.83)	0.99	(0.99)	0.52	(0.62)
210.00	0.68	(0.69)	0.77	(0.79)	0.49	(0.59)
180.00	0.53	(0.55)	0.64	(0.67)	0.40	(0.53)
160.50	0.44	(0.45)	0.56	(0.58)	0.36	(0.49)
153.00	0.41	(0.43)	0.54	(0.55)	0.36	(0.47)
135.25	0.36	(0.38)	0.43	(0.46)	0.34	(0.44)
117.50	0.34	(0.34)	0.34	(0.35)	0.33	(0.41)
100.00	0.31	(0.32)	0.31	(0.31)	0.32	(0.38)
82.50	0.30	(0.31)	0.30	(0.31)	0.31	(0.35)
66.50	0.30		0.30		0.32	

Note:

1. Enveloped response results at the north, south, east and west edge nodes of the structure are shown in parentheses. This is the maximum value of the response at any of these edge nodes.

Table 3.7.2-5 (Sheet 3 of 3)

MAXIMUM ABSOLUTE NODAL ACCELERATION (ZPA)
COUPLED AUXILIARY & SHIELD BUILDINGS

SOFT-TO-MEDIUM STIFF SOIL CONDITION

Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g.)					
	N-S Direction		E-W Direction		Vertical Direction	
307.25	0.86		0.98		0.74	
297.07	0.79		0.90		0.73	
284.41	0.69		0.76		0.73	
271.58	0.60		0.72		0.73	
246.00	0.56		0.65		0.50	
241.00	0.55		0.63		0.49	
230.00	0.52	(0.53)	0.58	(0.59)	0.48	(0.57)
210.00	0.46	(0.48)	0.50	(0.50)	0.47	(0.55)
180.00	0.40	(0.41)	0.40	(0.41)	0.42	(0.52)
160.50	0.35	(0.37)	0.36	(0.36)	0.39	(0.50)
153.00	0.34	(0.35)	0.37	(0.38)	0.37	(0.49)
135.25	0.30	(0.31)	0.32	(0.33)	0.36	(0.48)
117.50	0.27	(0.28)	0.30	(0.30)	0.36	(0.45)
100.00	0.26	(0.26)	0.29	(0.29)	0.35	(0.43)
82.50	0.24	(0.24)	0.29	(0.29)	0.34	(0.41)
66.50	0.23		0.29		0.35	

Note:

1. Enveloped response results at the north, south, east and west edge nodes of the structure are shown in parentheses. This is the maximum value of the response at any of these edge nodes.

Table 3.7.2-6 (Sheet 1 of 3)

MAXIMUM ABSOLUTE NODAL ACCELERATION (ZPA)
STEEL CONTAINMENT VESSEL

HARD ROCK SITE CONDITION

Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g.)					
	N-S Direction		E-W Direction		Vertical Direction	
256.33	0.87		1.15		1.30	
248.33	0.85		1.11		1.04	
240.33	0.82	(0.85)	1.07	(1.07)	0.92	(0.93)
229.52	0.78		1.02		0.81	
218.71	0.73		0.96		0.77	
205.33	0.69	(0.71)	0.89	(0.89)	0.75	(0.78)
205.33 (Polar Crane)	1.82		1.09		1.17	
190.00	0.64		0.80		0.71	
170.00	0.55		0.66		0.65	
162.00	0.51	(0.51)	0.60	(0.60)	0.62	(0.65)
144.50	0.41		0.48		0.55	
132.25	0.36		0.39		0.50	
116.86	0.33	(0.33)	0.34	(0.34)	0.43	(0.44)
112.50	0.32		0.33		0.41	
104.13	0.31		0.31		0.37	
100.00	0.30		0.30		0.32	

Note:
1. Enveloped response results at the north and west edge nodes of the structure are shown in parentheses. This is the maximum value of the response at any of these edge nodes.

Table 3.7.2-6 (Sheet 2 of 3)

MAXIMUM ABSOLUTE NODAL ACCELERATION (ZPA) STEEL CONTAINMENT VESSEL

SOFT ROCK SITE CONDITION

Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g.)					
	N-S Direction		E-W Direction		Vertical Direction	
256.33	0.66		0.90		0.78	
248.33	0.64		0.86		0.63	
240.33	0.63	(0.63)	0.83	(0.83)	0.57	(0.57)
229.52	0.61		0.78		0.49	
218.71	0.59		0.74		0.46	
205.33	0.56	(0.57)	0.67	(0.67)	0.45	(0.47)
205.33 (Polar Crane)	1.31		1.15		1.10	
190.00	0.54		0.59		0.44	
170.00	0.47		0.49		0.39	
162.00	0.45	(0.45)	0.45	(0.45)	0.41	(0.44)
144.50	0.40		0.38		0.37	
132.25	0.37		0.34		0.37	
116.86	0.35	(0.35)	0.33	(0.33)	0.36	(0.41)
112.50	0.34		0.31		0.36	
104.13	0.32		0.31		0.35	
100.00	0.31		0.31		0.34	

Note:
1. Enveloped response results at the north and west edge nodes of the structure are shown in parentheses. This is the maximum value of the response at any of these edge nodes.

Table 3.7.2-6 (Sheet 3 of 3)

MAXIMUM ABSOLUTE NODAL ACCELERATION (ZPA) STEEL CONTAINMENT VESSEL

SOFT-TO-MEDIUM STIFF SOIL CONDITION

Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g.)					
	N-S Direction		E-W Direction		Vertical Direction	
256.33	0.47		0.63		0.69	
248.33	0.45		0.61		0.57	
240.33	0.43	(0.44)	0.59	(0.59)	0.51	(0.53)
229.52	0.41		0.56		0.45	
218.71	0.38		0.53		0.42	
205.33	0.35	(0.37)	0.49	(0.49)	0.42	(0.50)
205.33 (Polar Crane)	0.69		1.12		1.35	
190.00	0.34		0.44		0.41	
170.00	0.31		0.38		0.41	
162.00	0.30	(0.30)	0.36	(0.36)	0.40	(0.48)
144.50	0.29		0.31		0.40	
132.25	0.28		0.32		0.39	
116.86	0.27	(0.28)	0.30	(0.30)	0.38	(0.44)
112.50	0.27		0.30		0.38	
104.13	0.26		0.29		0.38	
100.90	0.26		0.29		0.37	

Note:
1. Enveloped response results at the north and west edge nodes of the structure are shown in parentheses. This is the maximum value of the response at any of these edge nodes.

Table 3.7.2-7 (Sheet 1 of 3)

MAXIMUM ABSOLUTE NODAL ACCELERATION (ZPA) CONTAINMENT INTERNAL STRUCTURE

HARD ROCK SITE CONDITION

Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g.) N-S Direction		E-W Direction		Vertical Direction	
158.00 (PRZ Compartment)	0.84		0.69		0.30	
148.00 (SG-West Comprt.)	0.78		0.59		0.31	
148.00 (SG-East Comprt.)	0.71		0.57		0.31	
135.25	0.69	(0.75)	0.56	(0.81)	0.30	(0.32)
107.17	0.35	(0.37)	0.31	(0.32)	0.30	(0.31)
103.00	0.34		0.31		0.30	
98.10	0.33		0.30		0.30	
82.50	0.30		0.30		0.30	

Note:

1. Enveloped response results at the north, south, east and south edge nodes of the structure are shown in parentheses. This is the maximum value of the response at any of these edge nodes.

Table 3.7.2-7 (Sheet 2 of 3)

MAXIMUM ABSOLUTE NODAL ACCELERATION (ZPA) CONTAINMENT INTERNAL STRUCTURE

SOFT ROCK SITE CONDITION

Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g.)					
	N-S Direction		E-W Direction		Vertical Direction	
158.00 (PRZ Compartment)	0.45		0.44		0.34	
148.00 (SG-West Comprt.)	0.42		0.40		0.34	
148.00 (SG-East Comprt.)	0.38		0.39		0.31	
135.25	0.38	(0.40)	0.37	(0.43)	0.33	(0.36)
107.17	0.31	(0.31)	0.31	(0.31)	0.32	(0.35)
103.00	0.31		0.31		0.33	
98.10	0.31		0.31		0.32	
82.50	0.30		0.30		0.32	

Note:
1. Enveloped response results at the north, south, east and south edge nodes of the structure are shown in parentheses. This is the maximum value of the response at any of these edge nodes.

Table 3.7.2-7 (Sheet 3 of 3)

MAXIMUM ABSOLUTE NODAL ACCELERATION (ZPA) CONTAINMENT INTERNAL STRUCTURE

SOFT-TO-MEDIUM STIFF SOIL CONDITION

Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g.)					
	N-S Direction		E-W Direction		Vertical Direction	
158.00 (PRZ Compartment)	0.27		0.40		0.38	
148.00 (SG-West Comprt.)	0.27		0.37		0.38	
148.00 (SG-East Comprt.)	0.27		0.36		0.34	
135.25	0.26	(0.28)	0.35	(0.39)	0.37	(0.41)
107.17	0.25	(0.25)	0.28	(0.29)	0.35	(0.40)
103.00	0.25		0.28		0.37	
98.10	0.25		0.28		0.36	
82.50	0.24		0.28		0.36	

Note:

1. Enveloped response results at the north, south, east and south edge nodes of the structure are shown in parentheses. This is the maximum value of the response at any of these edge nodes.

Table 3.7.2-8 (Sheet 1 of 3)

MAXIMUM DISPLACEMENT RELATIVE TO TOP OF BASEMAT COUPLED AUXILIARY & SHIELD BUILDINGS

HARD ROCK SITE CONDITION

Elevation (ft)	Maximum Relative Displacement (inches)					
	N-S Direction		E-W Direction		Vertical Direction	
307.25	0.66		0.70		0.20	
297.07	0.58		0.63		0.20	
284.41	0.47		0.53		0.20	
271.58	0.37		0.45		0.20	
246.00	0.34		0.40		0.05	
241.00	0.33		0.39		0.05	
230.00	0.30	(0.31)	0.35	(0.37)	0.05	(0.17)
210.00	0.24	(0.26)	0.29	(0.30)	0.05	(0.16)
180.00	0.15	(0.17)	0.19	(0.21)	0.02	(0.12)
160.50	0.10	(0.11)	0.13	(0.15)	0.02	(0.10)
153.00	0.08	(0.09)	0.12	(0.13)	0.02	(0.09)
135.25	0.05	(0.06)	0.07	(0.09)	0.01	(0.07)
117.50	0.02	(0.03)	0.04	(0.05)	0.01	(0.05)
100.00	0.	(0.)	0.	(0.01)	0.01	(0.02)
82.50	0.	(0.)	0.	(0.)	0.	(0.01)
66.50	0.		0.		0.	

Note:

1. Enveloped response results at the north, south, east and west edge nodes of the structure are shown in parentheses. This is the maximum value of the response at any of these edge nodes.

Table 3.7.2-8 (Sheet 2 of 3)

MAXIMUM DISPLACEMENT RELATIVE TO TOP OF BASEMAT COUPLED AUXILIARY & SHIELD BUILDINGS

SOFT ROCK SITE CONDITION

Elevation (ft)	Maximum Relative Displacement (inches)					
	N-S Direction		E-W Direction		Vertical Direction	
307.25	0.71		0.94		0.24	
297.07	0.62		0.83		0.24	
284.41	0.53		0.74		0.24	
271.58	0.45		0.65		0.24	
246.00	0.41		0.59		0.07	
241.00	0.40		0.56		0.07	
230.00	0.36	(0.40)	0.51	(0.53)	0.06	(0.21)
210.00	0.30	(0.32)	0.41	(0.43)	0.06	(0.20)
180.00	0.20	(0.22)	0.27	(0.28)	0.03	(0.16)
160.50	0.14	(0.15)	0.19	(0.20)	0.02	(0.14)
153.00	0.12	(0.13)	0.17	(0.17)	0.02	(0.13)
135.25	0.08	(0.09)	0.11	(0.12)	0.02	(0.11)
117.50	0.05	(0.06)	0.07	(0.07)	0.01	(0.08)
100.00	0.02	(0.02)	0.02	(0.02)	0.01	(0.05)
82.50	0.01	(0.01)	0.01	(0.01)	0.01	(0.04)
66.50	0.		0.	(0.01)	0.	(0.04)

Note:

1. Enveloped response results at the north, south, east and west edge nodes of the structure are shown in parentheses. This is the maximum value of the response at any of these edge nodes.

Table 3.7.2-8 (Sheet 3 of 3)

MAXIMUM DISPLACEMENT RELATIVE TO TOP OF BASEMAT COUPLED AUXILIARY & SHIELD BUILDINGS

SOFT-TO-MEDIUM STIFF SOIL CONDITION

Elevation (ft)	Maximum Relative Displacement (inches)					
	N-S Direction		E-W Direction		Vertical Direction	
307.25	0.60		0.95		0.20	
297.07	0.54		0.88		0.20	
284.41	0.46		0.76		0.20	
271.58	0.40		0.68		0.20	
246.00	0.36		0.60		0.06	
241.00	0.35		0.58		0.06	
230.00	0.32	(0.34)	0.53	(0.54)	0.06	(0.26)
210.00	0.25	(0.27)	0.45	(0.46)	0.05	(0.25)
180.00	0.19	(0.22)	0.31	(0.33)	0.02	(0.23)
160.50	0.13	(0.17)	0.26	(0.28)	0.04	(0.21)
153.00	0.12	(0.15)	0.23	(0.24)	0.04	(0.20)
135.25	0.08	(0.11)	0.18	(0.19)	0.04	(0.19)
117.50	0.09	(0.09)	0.12	(0.13)	0.04	(0.17)
100.00	0.03	(0.05)	0.07	(0.10)	0.04	(0.15)
82.50	0.01	(0.04)	0.04	(0.14)	0.02	(0.14)
66.50	0.	(0.03)	0.	(0.06)	0.	(0.13)

Note:

1. Enveloped response results at the north, south, east and west edge nodes of the structure are shown in parentheses. This is the maximum value of the response at any of these edge nodes.

Table 3.7.2-9 (Sheet 1 of 3)

MAXIMUM DISPLACEMENT RELATIVE TO TOP OF BASEMAT STEEL CONTAINMENT VESSEL

HARD ROCK SITE CONDITION

Elevation (ft)	Maximum Relative Displacement (inches)					
	N-S Direction		E-W Direction		Vertical Direction	
256.33	0.19		0.21		0.05	
248.33	0.18		0.20		0.04	
240.33	0.18	(0.18)	0.19	(0.19)	0.04	(0.06)
229.52	0.17		0.18		0.03	
218.71	0.16		0.17		0.03	
205.33	0.14	(0.15)	0.15	(0.15)	0.03	(0.05)
205.33 (Polar Crane)	0.59		2.23		0.57	
190.00	0.12		0.13		0.03	
170.00	0.09		0.10		0.02	
162.00	0.08	(0.09)	0.09	(0.09)	0.02	(0.05)
144.50	0.06		0.06		0.02	
132.25	0.04		0.04		0.01	
116.86	0.02	(0.02)	0.02	(0.02)	0.01	(0.03)
112.50	0.02		0.02		0.01	
104.13	0.01		0.01		0.01	
100.00	0.		0.		0.01	

Note:

1. Enveloped response results at the north and west edge nodes of the structure are shown in parentheses. This is the maximum value of the response at any of these edge nodes.

Table 3.7.2-9 (Sheet 2 of 3)

MAXIMUM DISPLACEMENT RELATIVE TO TOP OF BASEMAT STEEL CONTAINMENT VESSEL

SOFT ROCK SITE CONDITION

Elevation (ft)	Maximum Relative Displacement (inches)					
	N-S Direction		E-W Direction		Vertical Direction	
256.33	0.20		0.27		0.03	
248.33	0.19		0.26		0.03	
240.33	0.18	(0.19)	0.24	(0.24)	0.03	(0.08)
229.52	0.17		0.22		0.02	
218.71	0.17		0.20		0.02	
205.33	0.15	(0.16)	0.18	(0.18)	0.02	(0.08)
205.33 (Polar Crane)	0.49		2.34		0.54	
190.00	0.13		0.17		0.02	
170.00	0.11		0.13		0.02	
162.00	0.10	(0.10)	0.11	(0.12)	0.02	(0.07)
144.50	0.07		0.09		0.02	
132.25	0.06		0.07		0.02	
116.86	0.04	(0.04)	0.05	(0.05)	0.01	(0.05)
112.50	0.04		0.04		0.01	
104.13	0.02		0.03		0.01	
100.00	0.02		0.02		0.01	

Note:

1. Enveloped response results at the north and west edge nodes of the structure are shown in parentheses. This is the maximum value of the response at any of these edge nodes.

Table 3.7.2-9 (Sheet 3 of 3)

MAXIMUM DISPLACEMENT RELATIVE TO TOP OF BASEMAT STEEL CONTAINMENT VESSEL

SOFT-TO-MEDIUM STIFF SOIL CONDITION

Elevation (ft)	Maximum Relative Displacement (inches)					
	N-S Direction		E-W Direction		Vertical Direction	
256.33	0.21		0.42		0.04	
248.33	0.20		0.40		0.04	
240.33	0.19	(0.21)	0.38	(0.38)	0.03	(0.18)
229.52	0.19		0.36		0.03	
218.71	0.17		0.33		0.03	
205.33	0.16	(0.17)	0.30	(0.30)	0.03	(0.18)
205.33 (Polar Crane)	0.33		2.32		0.64	
190.00	0.14		0.26		0.03	
170.00	0.11		0.22		0.02	
162.00	0.10	(0.12)	0.20	(0.20)	0.02	(0.17)
144.50	0.08		0.17		0.02	
132.25	0.07		0.14		0.02	
116.86	0.07	(0.08)	0.11	(0.11)	0.02	(0.15)
112.50	0.05		0.10		0.02	
104.13	0.04		0.08		0.02	
100.00	0.03		0.07		0.02	

Note:

1. Enveloped response results at the north and west edge nodes of the structure are shown in parentheses. This is the maximum value of the response at any of these edge nodes.

Table 3.7.2-10 (Sheet 1 of 3)

MAXIMUM DISPLACEMENT RELATIVE TO TOP OF BASEMAT CONTAINMENT INTERNAL STRUCTURE

HARD ROCK SITE CONDITION

Elevation (ft)	Maximum Relative Displacement (inches)					
	N-S Direction		E-W Direction		Vertical Direction	
158.00 (PRZ Compartment)	0.04		0.05		0.01	
148.00 (SG-West Comprt.)	0.04		0.04		0.01	
148.00 (SG-East Comprt.)	0.03		0.04		0.	
135.25	0.03	(0.04)	0.04	(0.05)	0.	(0.01)
107.17	0.	(0.01)	0.01	(0.01)	0.	(0.01)
103.00	0.		0.		0.	
98.10	0.		0.		0.	
82.50	0.		0.		0.	

Note:

1. Enveloped response results at the north, south, east and west edge nodes of the structure are shown in parentheses. This is the maximum value of the response at any of these edge nodes.

Table 3.7.2-10 (Sheet 2 of 3)

MAXIMUM DISPLACEMENT RELATIVE TO TOP OF BASEMAT CONTAINMENT INTERNAL STRUCTURE

SOFT ROCK SITE CONDITION

Elevation (ft)	Maximum Relative Displacement (inches)					
	N-S Direction		E-W Direction		Vertical Direction	
158.00 (PRZ Compartment)	0.05		0.07		0.02	
148.00 (SG-West Comprt.)	0.04		0.06		0.02	
148.00 (SG-East Comprt.)	0.04		0.06		0.01	
135.25	0.04	(0.04)	0.06	(0.06)	0.01	(0.04)
107.17	0.02	(0.02)	0.02	(0.02)	0.01	(0.03)
103.00	0.01		0.02		0.01	
98.10	0.01		0.02		0.01	
82.50	0.01		0.01		0.01	

Note:

1. Enveloped response results at the north, south, east and west edge nodes of the structure are shown in parentheses. This is the maximum value of the response at any of these edge nodes.

Table 3.7.2-10 (Sheet 3 of 3)

MAXIMUM DISPLACEMENT RELATIVE TO TOP OF BASEMAT
CONTAINMENT INTERNAL STRUCTURE

SOFT-TO-MEDIUM STIFF SOIL CONDITION

Elevation (ft)	Maximum Relative Displacement (inches)					
	N-S Direction		E-W Direction		Vertical Direction	
158.00 (PRZ Compartment)	0.08		0.17		0.08	
148.00 (SG-West Comprt.)	0.07		0.15		0.07	
148.00 (SG-East Comprt.)	0.05		0.15		0.04	
135.25	0.08	(0.09)	0.13	(0.14)	0.04	(0.13)
107.17	0.03	(0.05)	0.08	(0.08)	0.01	(0.12)
103.00	0.03		0.07		0.04	
98.10	0.02		0.06		0.02	
82.50	0.01		0.04		0.02	

Note:

1. Enveloped response results at the north, south, east and west edge nodes of the structure are shown in parentheses. This is the maximum value of the response at any of these edge nodes.

Table 3.7.2-11 (Sheet 1 of 3)

MAXIMUM MEMBER FORCES AND MOMENTS COUPLED AUXILIARY & SHIELD BUILDINGS

HARD ROCK SITE CONDITION

Elevation	Maximum Forces ($x10^3$ Kips)				Maximum Moment ($x10^3$ K-ft)	
(ft)	Axial	N-S Shear	E-W Shear	Torque	about N-S Axis	about E-W Axis
307.25					24.70	32.40
	1.67	2.65	2.49	5.02		
297.07					72.80	88.20
	3.69	5.47	5.07	11.06		
284.41					190.00	222.50
	7.09	9.33	8.47	26.56		
271.58					391.00	443.10
	11.17	9.33	8.47	51.92		
246.00					631.00	659.00
	13.18	14.45	14.64	53.32		
241.00					697.20	722.20
	14.29	15.54	15.83	53.63		
230.00					843.30	891.60
	16.46	17.58	17.85	68.05		
210.00					1135.00	1234.00
	19.52	20.37	20.14	92.89		
180.00					1807.00	1893.00
	22.17	24.03	22.66	784.80		
160.50					2341.00	2316.00
	23.62	26.41	25.42	952.60		
153.00					2513.00	2428.00
	25.26	29.41	29.63	694.70		
135.25					2981.00	2940.00
	27.92	33.81	36.41	958.10		
117.50					3535.00	3422.00
	29.94	36.62	41.13	1173.00		
100.00					4200.00	3937.00

Table 3.7.2-11 (Sheet 2 of 3)

MAXIMUM MEMBER FORCES AND MOMENTS
COUPLED AUXILIARY & SHIELD BUILDINGS

SOFT ROCK SITE CONDITION

Elevation	Maximum Forces ($x10^3$ Kips)			Maximum Moment ($x10^3$ K-ft)		
(ft)	Axial	N-S Shear	E-W Shear	Torque	about N-S Axis	about E-W Axis
307.25					22.40	23.40
	1.80	2.59	2.97	3.01		
297.07					78.10	77.20
	3.97	5.38	6.06	6.62		
284.41					220.00	212.00
	7.63	9.30	10.10	15.90		
271.58					462.00	436.00
	12.00	9.30	10.10	31.10		
246.00					757.00	642.00
	13.80	15.30	16.90	34.20		
241.00					836.00	713.00
	14.50	16.50	18.40	35.20		
230.00					1010.00	890.00
	16.10	18.60	21.00	41.60		
210.00					1360.00	1250.00
	18.50	21.30	24.50	56.30		
180.00					1980.00	2090.00
	21.40	24.90	28.00	872.00		
160.50					2600.00	2580.00
	23.30	27.20	29.90	941.00		
153.00					2800.00	2700.00
	25.50	30.00	32.90	735.00		
135.25					3380.00	3310.00
	29.70	34.90	37.40	995.00		
117.50					4050.00	3940.00
	34.80	40.30	42.90	1200.00		
100.00					4740.00	4640.00

Table 3.7.2-11 (Sheet 3 of 3)

MAXIMUM MEMBER FORCES AND MOMENTS COUPLED AUXILIARY & SHIELD BUILDINGS

SOFT-TO-MEDIUM STIFF SOIL CONDITION

Elevation (ft)	Maximum Forces ($x10^3$ Kips)			Maximum Moment ($x10^3$ K-ft)		
	Axial	N-S Shear	E-W Shear	Torque	about N-S Axis	about E-W Axis
307.75					10.70	8.77
	1.44	1.64	1.93	1.45		
297.07					43.10	32.60
	3.20	3.49	4.04	3.18		
284.41					126.00	98.70
	6.15	6.20	7.08	7.83		
271.58					271.00	216.00
	9.68	6.20	7.08	15.40		
246.00					467.00	387.00
	11.10	10.50	12.20	17.40		
241.00					525.00	442.00
	11.80	11.40	13.00	18.00		
230.00					664.00	558.00
	13.20	13.00	14.70	22.20		
210.00					947.00	824.00
	15.40	15.30	16.90	29.00		
180.00					1440.00	1560.00
	18.30	18.20	19.50	616.00		
160.50					1780.00	1890.00
	20.30	19.90	21.40	658.00		
153.00					1950.00	1990.00
	22.90	22.30	24.10	545.00		
135.25					2380.00	2470.00
	27.80	26.50	28.70	660.00		
117.50					2830.00	2950.00
	33.30	31.00	32.20	848.00		
100.00					3380.00	3470.00

Table 3.7.2-12 (Sheet 1 of 3)

MAXIMUM MEMBER FORCES AND MOMENTS
STEEL CONTAINMENT VESSEL

HARD ROCK SITE CONDITION

Elevation (ft)	Maximum Forces (x10^3 Kips)				Maximum Moment (x10^3 K-ft)	
	Axial	N-S Shear	E-W Shear	Torque	about N-S Axis	about E-W Axis
256.33					0.00	0.00
	0.25	0.16	0.20	0.00		
248.33					2.50	2.10
	0.51	0.43	0.56	0.51		
240.33					8.70	7.20
	0.83	0.71	0.93	1.45		
229.52					21.10	17.10
	1.15	0.98	1.29	2.78		
218.71					37.80	30.20
	1.47	1.25	1.66	4.24		
205.33					66.10	52.80
	2.66	2.47	2.45	7.48		
190.00					107.50	93.40
	3.08	2.83	2.94	9.52		
170.00					169.90	151.50
	3.42	3.10	3.31	11.04		
162.00					199.50	178.30
	3.73	3.30	3.59	12.22		
144.50					265.80	238.70
	4.01	3.50	3.85	13.36		
132.25					315.60	283.40
	4.22	3.62	4.02	14.11		
116.86					378.[illegible]	339.60
	4.29	3.65	4.06	14.32		
112.50					397.20	356.20
	4.36	3.68	4.10	14.51		
104.13					432.00	387.10
	4.40	3.68	4.10	14.57		
100.00					449.00	402.20

Table 3.7.2-12 (Sheet 2 of 3)

MAXIMUM MEMBER FORCES AND MOMENTS
STEEL CONTAINMENT VESSEL

SOFT ROCK SITE CONDITION

Elevation	Maximum Forces ($x10^3$ Kips)				Maximum Moment ($x10^3$ K-ft)	
(ft)	Axial	N-S Shear	E-W Shear	Torque	about N-S Axis	about E-W Axis
256.33					0.00	0.00
	0.15	0.13	0.17	0.00		
248.33					2.30	1.70
	0.36	0.35	0.46	0.17		
240.33					7.90	5.60
	0.57	0.59	0.76	0.48		
229.52					18.90	13.30
	0.75	0.83	1.05	0.92		
218.71					34.00	23.40
	0.91	1.07	1.35	1.41		
205.33					59.30	40.70
	1.87	2.02	1.98	2.49		
190.00					93.60	72.50
	2.15	2.44	2.36	3.18		
170.00					143.00	129.00
	2.37	2.99	2.66	3.77		
162.00					167.00	148.00
	2.59	2.99	2.89	4.21		
144.50					220.00	210.00
	2.81	3.01	3.11	4.70		
132.25					259.00	248.00
	3.00	3.01	3.22	5.02		
116.86					311.00	292.00
	3.08	3.01	2.99	5.15		
112.50					327.00	305.00
	3.17	3.06	2.99	5.24		
104.13					355.00	329.00
	3.23	3.08	3.11	5.41		
100.00					368.00	341.00

Table 3.7.2-12 (Sheet 3 of 3)

MAXIMUM MEMBER FORCES AND MOMENTS
STEEL CONTAINMENT VESSEL

SOFT-TO-MEDIUM STIFF SOIL CONDITION

Elevation (ft)	Maximum Forces ($x10^3$ Kips) Axial	N-S Shear	E-W Shear	Torque	Maximum Moment ($x10^3$ K-ft) about N-S Axis	about E-W Axis
256.33					0.00	0.00
	0.13	0.09	0.12	0.00		
248.33					1.42	1.08
	0.33	0.24	0.34	0.14		
240.33					5.03	3.75
	0.52	0.40	0.57	0.40		
229.52					12.60	9.12
	0.69	0.55	0.80	0.77		
218.71					22.80	16.30
	0.85	0.70	1.03	1.18		
205.33					39.90	28.40
	1.73	1.35	1.80	2.13		
190.00					70.30	43.80
	2.05	1.36	1.99	2.78		
170.00					116.00	71.90
	2.27	1.58	2.05	3.35		
162.00					138.00	85.70
	2.46	1.73	2.24	3.87		
144.50					187.00	117.00
	2.63	1.93	2.43	4.52		
132.25					223.00	144.00
	2.78	2.10	2.57	5.10		
116.86					265.00	178.00
	2.85	2.18	2.64	5.37		
112.50					273.00	183.00
	2.93	2.26	2.72	5.67		
104.13					318.00	209.00
	2.97	2.32	2.76	5.84		
100.00					332.00	214.00

Table 3.7.2-13 (Sheet 1 of 3)

MAXIMUM MEMBER FORCES AND MOMENTS
CONTAINMENT INTERNAL STRUCTURES

HARD ROCK SITE CONDITION

Elevation (ft)	Maximum Forces ($x10^3$ Kips) Axial	N-S Shear	E-W Shear	Torque	Maximum Moment ($x10^3$ K-ft) about N-S Axis	about E-W Axis
Above Elevation 135.25', West SG Compartment						
158.00					0.02	0.
	0.08	0.23	0.32	0.52		
148.00					3.55	2.82
	0.23	0.70	0.90	7.73		
135.25					15.05	11.67
Above Elevation 135.25', East SG Compartment						
148.00					0.12	0.07
	0.11	0.34	0.35	1.35		
135.25					4.40	4.40
Below Elevation 135.25'						
135.25					27.90	25.60
	2.15	7.37	9.64	363.90		
107.17					306.90	223.20
	4.03	8.69	10.29	405.90		
103.00					348.50	257.40
	6.74	9.57	11.12	364.90		
98.10					394.10	299.10
	10.85	13.22	13.50	418.70		
82.50					582.20	475.80

Table 3.7.2-13 (Sheet 2 of 3)

MAXIMUM MEMBER FORCES AND MOMENTS CONTAINMENT INTERNAL STRUCTURES

SOFT ROCK SITE CONDITION

Elevation (ft)	Maximum Forces ($x10^3$ Kips) Axial	N-S Shear	E-W Shear	Torque	Maximum Moment ($x10^3$ K-ft) about N-S Axis	about E-W Axis
Above Elevation 135.25', West SG Compartment						
158.00					0.02	0.
	0.09	0.10	0.11	0.06		
148.00					1.26	1.34
	0.26	0.34	0.37	2.64		
135.25					5.95	5.73
Above Elevation 135.25', East SG Compartment						
148.00					0.12	0.07
	0.11	0.13	0.14	0.30		
135.25					1.80	1.70
Below Elevation 135.25'						
135.25					18.30	23.40
	2.51	4.44	4.77	69.40		
107.17					162.00	135.00
	4.69	6.64	6.73	139.00		
103.00					188.00	151.00
	7.78	8.74	8.71	61.90		
98.10					216.00	191.00
	12.60	13.10	13.90	58.60		
82.50					416.00	407.00

Table 3.7.2-13 (Sheet 3 of 3)

MAXIMUM MEMBER FORCES AND MOMENTS
CONTAINMENT INTERNAL STRUCTURES

SOFT-TO-MEDIUM STIFF SOIL CONDITION

Elevation	Maximum Forces ($x10^3$ Kips)				Maximum Moment ($x10^3$ K-ft)	
(ft)	Axial	N-S Shear	E-W Shear	Torque	about N-S Axis	about E-W Axis
Above Elevation 135.25', West SG Compartment						
158.00					0.02	0.
	0.10	0.07	0.10	0.05		
148.00					1.14	1.31
	0.28	0.25	0.29	1.89		
135.25					4.89	4.50
Above Elevation 135.25', East SG Compartment						
148.00					0.13	0.07
	0.12	0.10	0.13	0.21		
135.25					1.67	1.30
Below Elevation 135.25'						
135.25					19.70	25.10
	2.78	3.44	4.10	55.60		
107.17					155.00	112.00
	5.16	5.19	5.88	124.00		
103.00					175.00	119.00
	8.62	6.80	7.71	58.40		
98.10					203.00	154.00
	13.90	10.50	11.70	54.20		
82.50					381.00	316.00

Table 3.7.2-14 (Sheet 1 of 2)

SUMMARY OF MODELS AND ANALYSIS METHODS

Model	Analysis Method	Program	Type of Dynamic Response/Purpose
2D lumped mass stick models coupled with 2D model of the foundation	Complex frequency response analysis	SASSI	To identify governing site properties and design soil profiles.
2D lumped mass stick, fixed base models	Mode superposition time history analysis	BSAP	To identify governing site properties and design soil profiles.
3D lumped mass stick, fixed base models	Mode superposition time history analysis	BSAP	Performed for hard rock profile. To develop time histories for generating floor response spectra. To obtain the following: Maximum absolute nodal accelerations (ZPA). Maximum displacements relative to basemat. Maximum member forces and moments for all structures, except the containment internal structures.
	Response spectrum analysis	BSAP	To obtain the seismic force and moment response of the containment internal structures (Subsection 3.7.2.2) including the high frequency modal effect. Member forces are used also to determine the SSI scaling factor (see note 1).
3D lumped mass stick models coupled with 3D model of the foundation	Complex frequency response analysis	SASSI	Performed for the soft rock and soft-to-medium soil profiles. To develop time histories for generating floor response spectra. To obtain the following: Maximum absolute nodal accelerations (ZPA). Maximum displacements relative to basemat. Maximum member forces and moments. Member forces are used also to determine the SSI scaling factor (see note 1).
3D finite element, fixed base models, coupled Aux/Shield buildings and Cont. internal structures	Response spectrum analysis	BSAP	Performed for the hard rock profile. To obtain the in-plane forces[1] for the design of floors and walls.

Table 3.7.2-14 (Sheet 2 of 2)

SUMMARY OF MODELS AND ANALYSIS METHODS

Model	Analysis Method	Program	Type of Dynamic Response/Purpose
3D finite element model of the nuclear island basemat	Equivalent static analysis using nodal accelerations and member forces from 3D stick model	ANSYS	To obtain SSE bearing reactions and member forces in the basemat
3D shell of revolution model of steel containment vessel	Equivalent static analysis using nodal accelerations from 3D stick model	CBI 0781	To obtain SSE Stress for the containment vessel
3D finite element model of the shield building roof	Equivalent static analysis using nodal accelerations from 3D stick model	ANSYS STRUDL	To obtain SSE member forces for the shield building roof

(1) The in-plane forces for the hard rock profiles are increased by an SSI scaling factor when, based on a comparison of force responses of the 3D lumped-mass stick model, either the soft rock or soft-to-medium stiff soil cases give higher element forces than the hard rock case. The SSI scaling factor, at a given plant elevation, is equal to the ratio of the largest 3D stick model element forces over the 3D stick model element force for the hard rock case.

Table 3.7.2-15

COMPARISON OF FREQUENCIES FOR CONTAINMENT VESSEL SEISMIC MODEL

Mode No.	Vertical Model		Horizontal Model	
	Shell of Revolution Model	Stick Model	Shell of Revolution Model	Stick Model
1	17.71 Hertz	18.33 Hertz	7.39 Hertz	7.56 Hertz
2	23.59 Hertz	30.06 Hertz	20.88 Hertz	22.0 Hertz

Table 3.7.2-16

SUMMARY OF DYNAMIC ANALYSES & COMBINATION TECHNIQUES

Model	Analysis Method	Program	3 Components Combination	Modal Combination
3D lumped mass stick, fixed base models	Mode superposition time history analysis	BSAP	Algebraic Sum	n/a
	Response spectrum analysis	BSAP	SRSS	SRSS w/ Double Sum
3D lumped mass stick models coupled with 3D model of the foundation	Complex frequency response analysis	SASSI	Algebraic Sum	n/a
3D finite element, fixed base models, coupled Aux/Shield buildings and Cont. internal structures	Response spectrum analysis	BSAP	SRSS	SRSS w/ Double Sum
3D finite element model of the nuclear island basemat	Equivalent static analysis using nodal accelerations & Member forces from 3D stick model	ANSYS	100%,40%,40%	n/a
3D shell of revolution model of steel containment vessel	Equivalent static analysis using nodal accelerations from 3D stick model	CBI 0781	SRSS or 100%,40%	n/a
3D finite element model of the shield building roof	Equivalent static analysis using nodal accelerations from 3D stick model	ANSYS STRUDL	SRSS	n/a

Table 3.7.3-1 (Sheet 1 of 3)

SEISMIC CATEGORY I EQUIPMENT OUTSIDE CONTAINMENT BY ROOM NUMBER

Room No.	Room Name	Equipment Description
12101	Division A battery room 1	Batteries
12102	Division C battery room 1	Batteries
12103	Spare battery room	Spare batteries
12104	Division B battery room 1	Batteries
12105	Division D battery room 1	Batteries
12113	Spare battery charger room	
12162	RNS pump room A	RNS pressure boundary
12163	RNS pump room B	RNS pressure boundary
12201	Division A DC equipment room	DC equipment
12202	Division C battery room 2	Batteries
12203C	Division C DC equipment room	DC equipment
12203B	Division B DC equipment room	DC equipment
12204	Division B battery room 2	Batteries
12205	Division D DC equipment room	DC equipment room
12211	Corridor	Divisional cables
12212	RCP trip switchgear room B	RCP trip switchgear
12253	Pipe chase	RNS containment isolation valves
12255	CVS makeup pump room	CVS isolation valves
12256	Lower annulus	CVS/WLS containment isolation valves RNS piping
12257	Pipe chase	CVS/WLS containment isolation valves RNS piping
12300	Corridor	Divisional cable
12301	Division A I&C room	Divisional I&C
12302	Division C I&C room	Divisional I&C
12303	Remote shutdown workstation	Remote shutdown workstation
12304	Division B I&C/penetration room	Divisional I&C/electrical penetrations

Table 3.7.3-1 (Sheet 2 of 3)

SEISMIC CATEGORY I EQUIPMENT OUTSIDE CONTAINMENT BY ROOM NUMBER

Room No.	Room Name	Equipment Description
12305	Division D I&C/penetration room	Divisional I&C/electrical penetrations
12306	Valve/piping penetration room	CCS/CVS/DWS/FPS/SGS containment isolation valves
12311	Corridor	Divisional cabling
12312	RCP trip switchgear room A	RCP trip switchgear
12313	Division C I&C/penetration room	Divisional I&C/electrical penetrations
12321	I&C/non 1E penetration room	Divisional cabling
12351	Maintenance floor and staging area	Divisional cabling (ceiling)
12352	Personnel hatch	Personnel airlock (interlocks)
12354	Rad pipe chase	PSS/SFS containment isolation valves
12356	Middle annulus	Class 1E electrical penetrations Various mechanical piping penetrations
12362	RNS HX room A	RNS pressure boundary
12363	RNS HX room B	RNS pressure boundary
12400	Control room vestibule	Control room access
12401	Main control room	Main control panels VBS HVAC dampers VES isolation valves Lights
12404	Lower MSIV compartment B	SGS containment isolation valves, instrumentation and controls
12405	VBS B and D equipment room	VWS/PXS/CAS containment isolation valves
12406	Lower MSIV compartment A	SGS containment isolation valves, instrumentation and controls
12412	Electrical penetration room Division A	Divisional electrical penetrations

Table 3.7.3-1 (Sheet 3 of 3)

SEISMIC CATEGORY I EQUIPMENT OUTSIDE CONTAINMENT BY ROOM NUMBER

Room No.	Room Name	Equipment Description
12421	RCC/non 1E penetration room	Divisional cabling
12422	Reactor trip switchgear II	Reactor trip switchgear
12423	Reactor trip switchgear I	Reactor trip switchgear
12452	VFS penetration room	VFS containment isolation valves, divisional cabling
12454	Rad pipe chase	SFS/PSS/VFS/CVS cont. isolation valves
12504	Upper MSIV compartment B	SGS CIVs, instrumentation and controls
12506	Upper MSIV compartment A	VWS/PXS/CAS containment isolation valves
12552	Personnel hatch	Personnel airlock (interlocks)
12553	Operating deck staging area	VES high pressure air bottles
12556	Upper annulus	PCS piping and cabling PCS air baffle
12561	Fuel handling area	Spent fuel storage racks
12701	PCS valve room	PCS isolation valves/instrumentation
	PCS Water storage tank	Level and temperature instrumentation

Table 3.7.3-2

EQUIPMENT CLASSIFIED AS SENSITIVE TARGETS FOR SEISMICALLY ANALYZED PIPING, HVAC DUCTING, CABLE TRAYS

Component	Discussion	Zone of Protection
Seismic Category I Valve No Class 1E Electrical Equipment Not pressure sensitive	These are manual valves. The actuator must be protected from impact.	Valve body and actuator area
Seismic Category I Valve Class 1E Electrical Equipment Pressure sensitive	These valves have sensitive Class 1E equipment (eg., Position indicators, limit switches, motor operator) or solenoid valves.	One support (acting in direction of impact) on each side of valve
Seismic Category I Dampers	The actuator must be protected along with any Class 1E equipment.	Within one support (acting in direction of impact) on each side of HVAC
Monitors	This includes Neutron Detectors, Radiation Monitors, Resistance Temperature Detectors, Speed Sensors, Thermocouples, Transmitters	Monitors and associated wiring
Sensitive Electrical Equipment Housed in Cabinets, Panels or Boards	This includes: relays, contractors, breakers, and switchgear.	Cabinets, panels, and boards housing sensitive devices
Class 1E exposed cables and wiring	Cables and wiring which are not housed in cable trays or conduits must be protected.	Exposed cables and wiring
Device or Instrument Tubing	Any device or tubing that could be damaged resulting in the loss of the pressure boundary of a safety class line.	Device or tubing
Penetrations	Rigid penetrations are considered robust. Floating penetrations with bellows are considered sensitive	Floating penetration and associated bellows

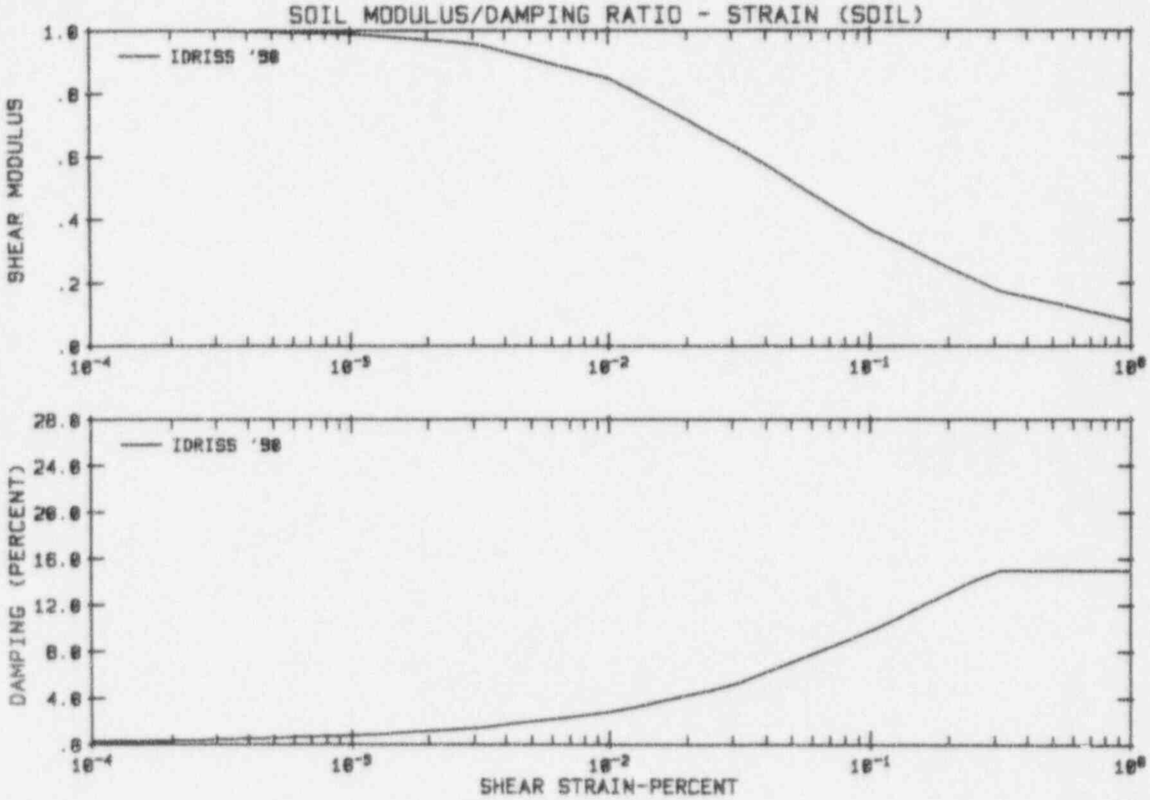
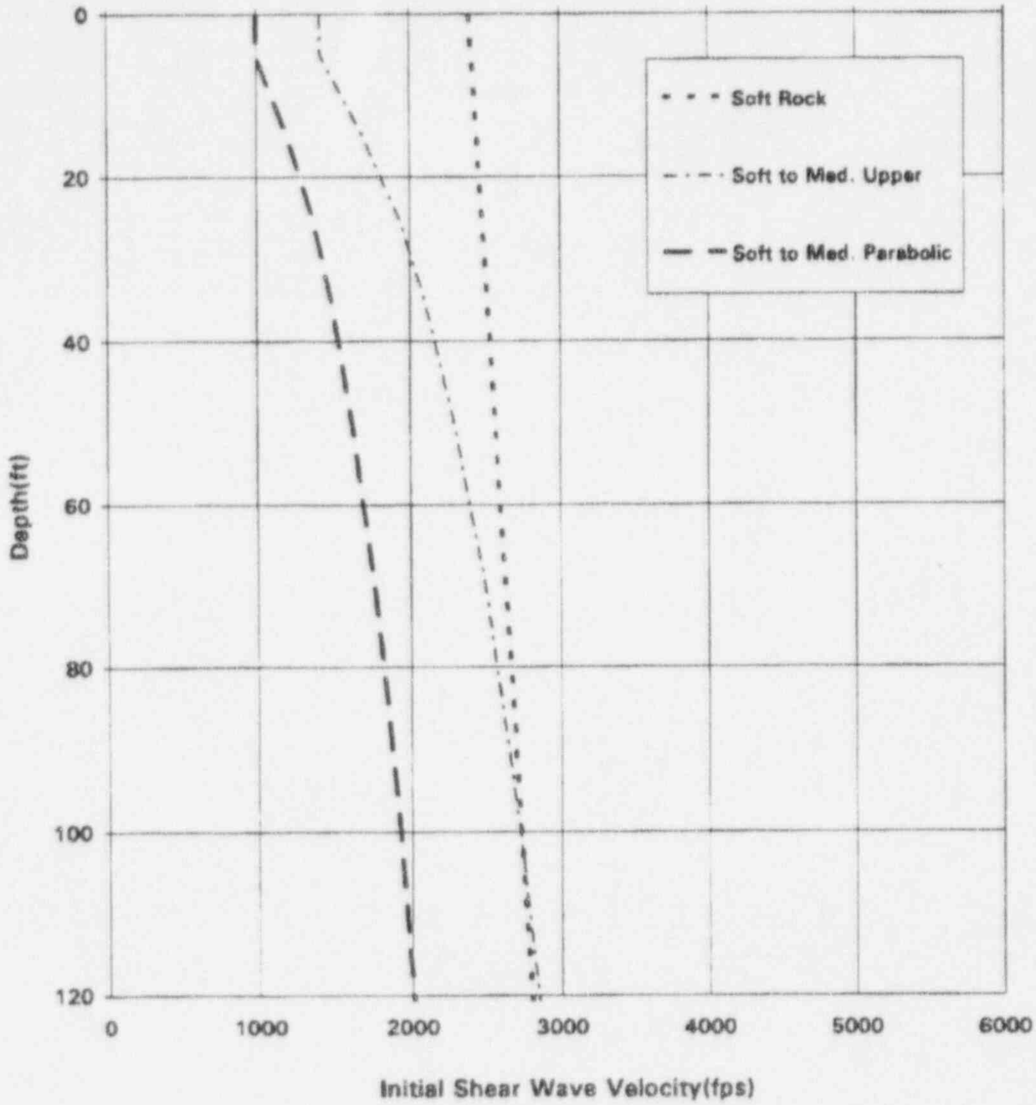


Figure 3.7.1-15
Strain Dependent Properties of Soil Material



** The design soil profiles also include a hard rock site, which represents an upper bound case for firm sites, using fixed base seismic analysis.

Figure 3.7.1-17
Shear Wave Velocity of Design Soil Profiles

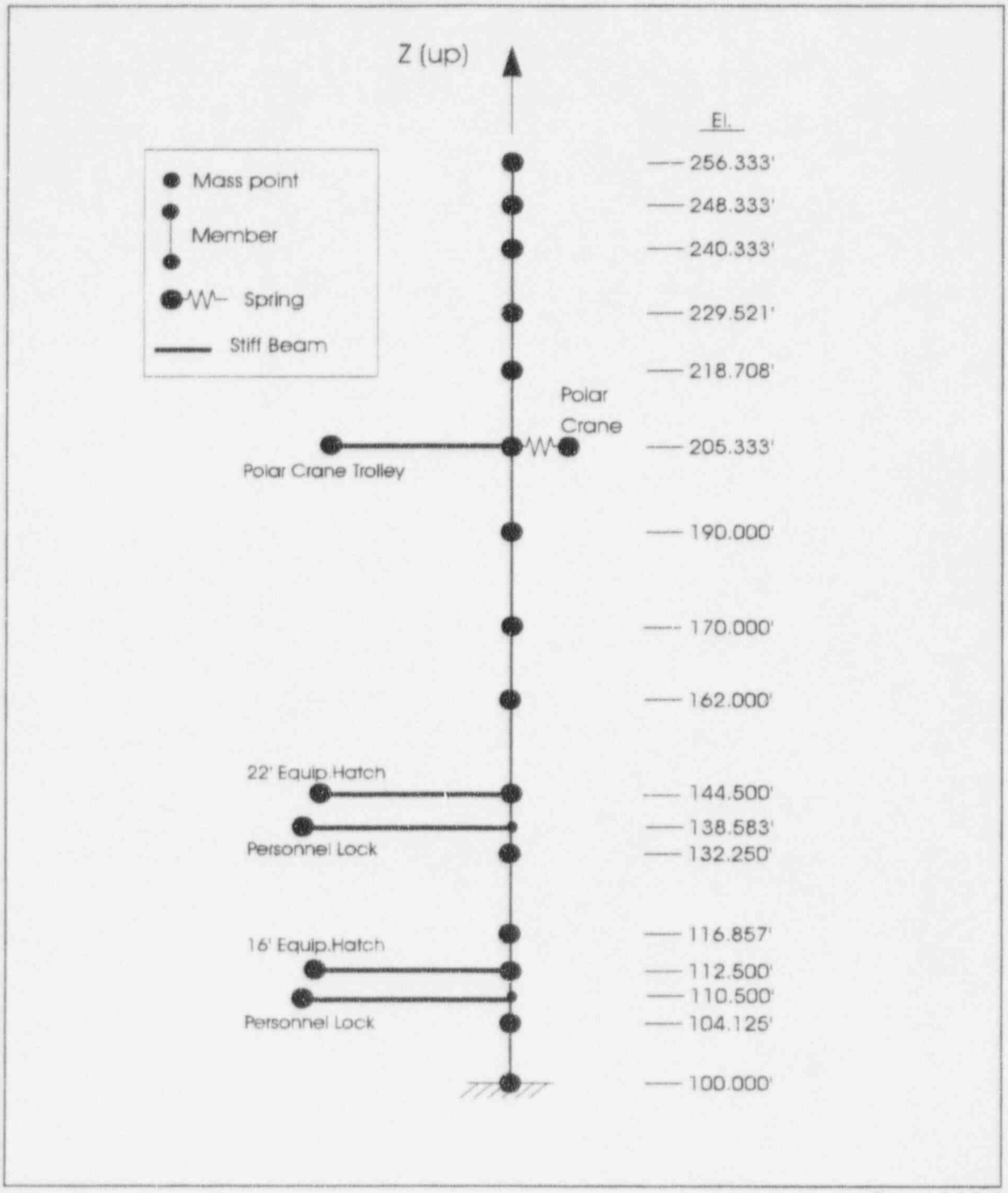
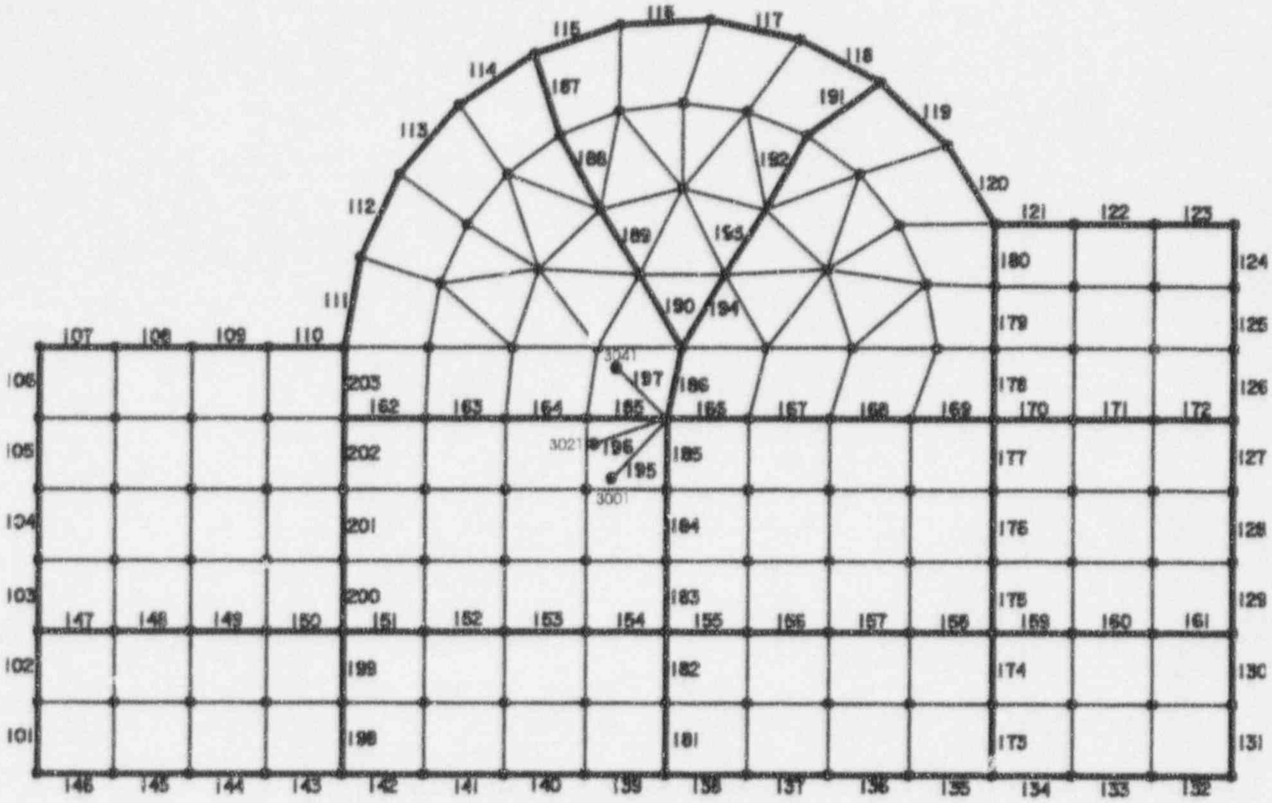


Figure 3.7.2-5

Steel Containment Vessel Lumped Mass Stick Model



LOCATION OF RIGID BEAM ELEMENTS @ EL. 66'-6"
(TOP OF BASEMAT)

NOTE:
NUMBERED LINES INDICATE RIGID BEAM ELEMENTS

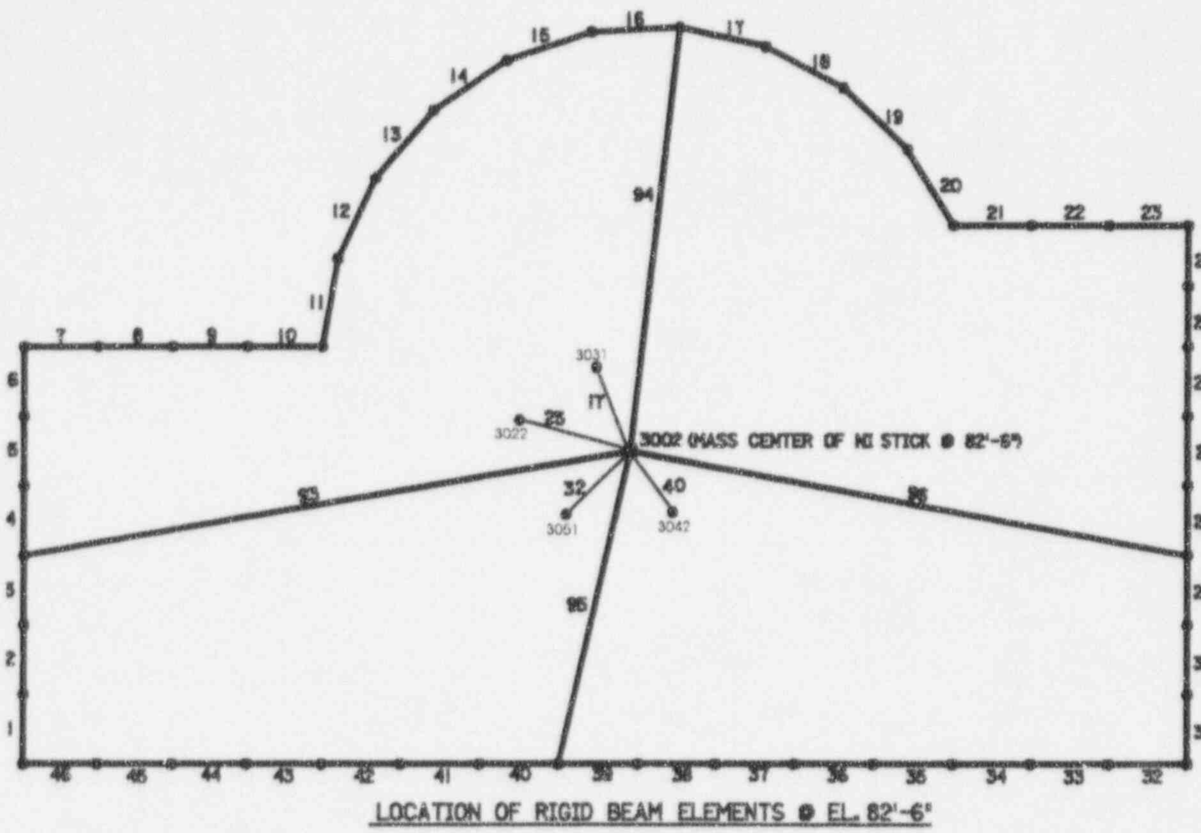
* *

The following three nodes of the 3D lumped mass stick model are located at Elevation 66.5':

Node# 3001 - Mass Center of coupled auxiliary/shield building @ Elev. 66.5',
Node# 3021 - Shear Center of building section between Elev. 66.5' to 82.5',
Node# 3041 - Centroid of building section between Elev. 66.5' to 82.5'.

Figure 3.7.2-29 (Sheet 1 of 3)

3D Seismic Analysis Model, Plan at Elev. 66.5'



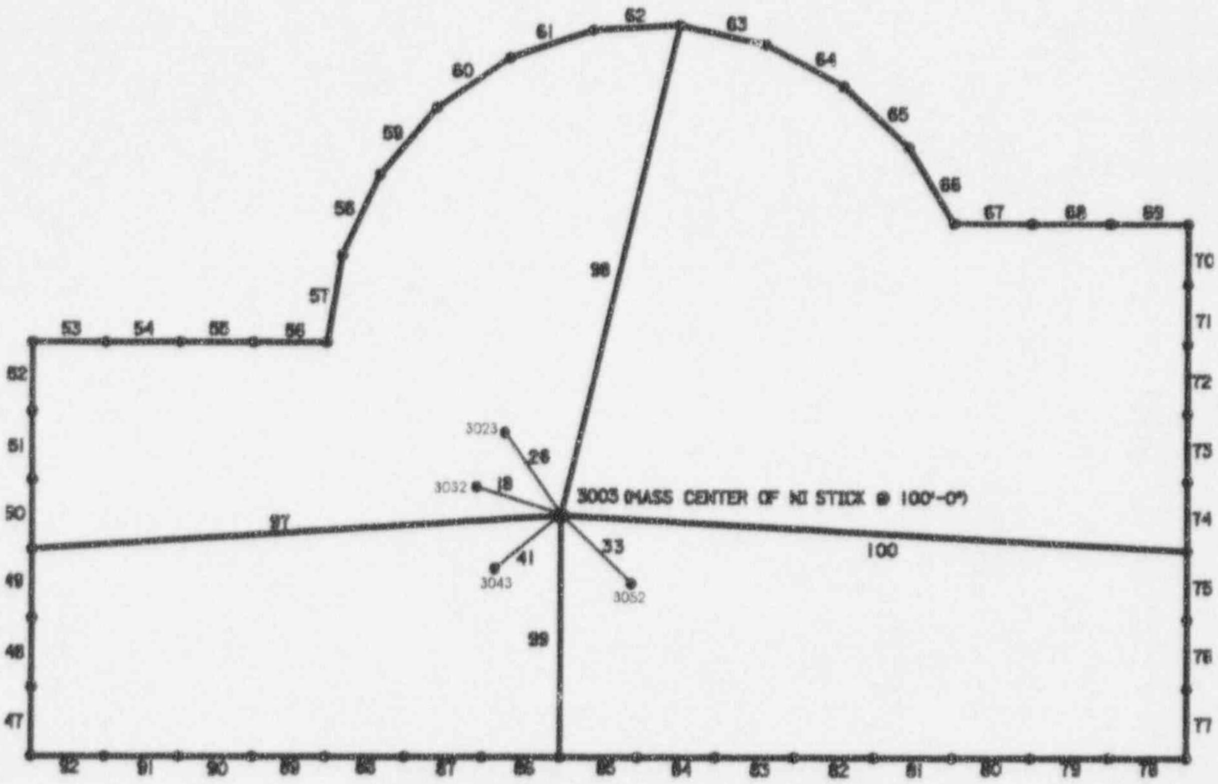
NOTE:
NUMBERED LINES INDICATE RIGID BEAM ELEMENTS

*   *

The following four nodes of the 3D lumped mass stick model are located at Elevation 82.5':
Node# 3031 - Shear Center of building section between Elev. 66.5' to 82.5',
Node# 3022 - Shear Center of building section between Elev. 82.5' to 100.0',
Node# 3051 - Centroid of building section between Elev. 66.5' to 82.5'.
Node# 3042 - Centroid of building section between Elev. 82.5' to 100.1'.

Figure 3.7.2-29 (Sheet 2 of 3)

3D Seismic Analysis Model, Plan at Elev. 82.5'



LOCATION OF RIGID BEAM ELEMENTS @ EL. 100'-0"

NOTE:
NUMBERED LINES INDICATE RIGID BEAM ELEMENTS

Figure 4

* The following four nodes of the 3D lumped mass stick model are located at Elevation 100.0': *

Node# 3032 - Shear Center of building section between Elev. 82.5' to 100.0',
Node# 3023 - Shear Center of building section between Elev. 100.0' to 117.5',
Node# 3052 - Centroid of building section between Elev. 82.5' to 100.0'.
Node# 3043 - Centroid of building section between Elev. 100.0' to 117.5'.

Figure 3.7.2-29 (Sheet 3 of 3)

3D Seismic Analysis Model, Plan at Elev. 100.0'

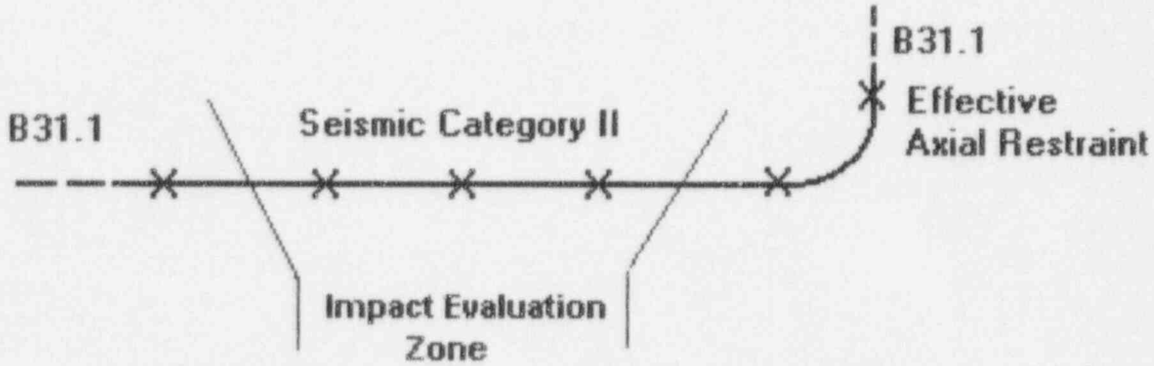


Figure 3.7.3-2

Impact Evaluation Zone and Seismic Supported Piping