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). For the purposes of the shutdown criteria in Reference 1 the operating basis earthquake for shutdown is considered to be one-third of the safe shutdown earthquake.

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. Qualification of a site where the soil characteristics are outside the range of the generic site interface is discussed in subsection 2.5.4.

The peak ground acceleration of the safe shutdown earthquake has been established as 0.30g for the AP600 design. 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:

- 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

- 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)

- 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 equal to the horizontal response spectrum.

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. Site specific time histories may be used as defined in subsection 2.5.4.5.5. 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 33 hertz. The cutoff frequencies used in the soil structure interaction analyses are 33 hertz for the soft rock site, and 15 hertz horizontal and 21 hertz vertical for the soft-to-medium soil site and 20 hertz horizontal and 33 hertz vertical for the upper bound soft-to-medium soil site. The maximum "cut-off" frequency for 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 power spectral density shall be developed using procedures outlined in NUREG/CR-5347 (Reference 29).

The NUREG/CR-5347 procedures involve ad hoc hybridization of two earlier power spectral density 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 power spectral density- compatible time histories referenced in NUREG/CR-5347) and occurs only in the frequency range between 9 to 33 hertz, a project-specific power spectral density 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 power spectral density is done in this frequency range. At frequencies above 9 hertz, the third and the fourth legs of the power spectral density 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 power spectral density, therefore, is extended to about 25 hertz rather than 16 hertz.

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

The AP600 augmented power spectral density, anchored to 0.3 g, is as follows:

$S_0(f) = 58.5\ (f/2.5)^{0.2}\ \text{in}^2/\text{sec}^3,\quad f \leq 2.5$ hertz
$S_0(f) = 58.5\ (2.5/f)^{1.8}\ \text{in}^2/\text{sec}^3,\quad 2.5 \text{ hertz} \leq f \leq 9$ hertz
$S_0(f) = 5.832\ (9/f)^{3}\ \text{in}^2/\text{sec}^3,\quad 9 \text{ hertz} \leq f \leq 25$ hertz
$S_0(f) = 0.27\ (25/f)^{8}\ \text{in}^2/\text{sec}^3,\quad 25 \text{ hertz} \leq f$

The AP600 Minimum Power Spectral Density is presented in Figure 3.7.1-9. This AP600 target power spectral density is compatible with the AP600 horizontal design response spectra and envelops a target power spectral density compatible with the AP600 vertical design response spectra. This AP600 target power spectral density, 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 power spectral density functions of the design time histories are calculated at uniform frequency steps of 0.0489 hertz. The power spectral densities presented in Figures 3.7.1-10 through 3.7.1-12 are the averaged power spectral density obtained over a moving frequency band of ±20 percent centered at each frequency. The power spectral density amplitude at frequency (f) has the averaged power spectral density 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 value 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. For cable trays and supports demonstrated to be similar to those tested, damping values of Figure 3.7.1-13 may be used. These are 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:

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

where:

β_n = ratio of critical damping for mode n

nc = number of elements

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

$[K_t]_i$ = 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 for rock sites in accordance with Reference 5 and 6 and for soil sites in accordance with Reference 33. 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 soil site and a soft-to-medium soil site. The shear wave velocity profiles and related governing parameters of the four 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 is assumed 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 is assumed 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.

Free field analyses are performed for each design soil profile and acceleration response spectra are calculated at the foundation level of the nuclear island as described in subsection 2A-6. The envelopes of the horizontal and vertical spectra for the four design soil cases are shown in Figures 3.7.1-18 and 3.7.1-19 respectively. These envelopes represent the seismic input at foundation level for which the AP600 is designed. These spectra are used for comparison with the results of site specific soil amplification studies as described in subsection 2.5.2.1.

The AP600 design for the site parameters given in Table 2-1 is implemented using analyses for the four soil conditions described above. These four soil conditions consist of competent material. They do not include profiles consisting of one or more thin layers overlying competent material (Standard Review Plan 3.7.2). The shear wave velocity parameter in Table 2-1 excludes these shallow soil sites from consideration as a part of design certification.

The AP600 may also be suitable at sites outside the bounds of the site parameters. These sites may be demonstrated to be acceptable using the methodology in subsection 2.5.2.2. These site specific analyses demonstrating acceptability of the site would be submitted as part of the Combined License application. The evaluation of the suitability of these sites is not included in design certification.

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 3.7.2-12.*]*

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 and design allowables 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 including the supplemental requirements described in subsections 3.8.4.4.1 and 3.8.4.5. 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.

*NRC Staff approval is required prior to implementing a change in this information; see DCD Section 3.5.

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.

The seismic analyses of the nuclear island are summarized in a seismic analysis summary report. This report describes the development of the finite element models, the soil structure interaction analyses, and the results thereof. A separate report provides the floor response spectra for the nuclear island.

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 safe shutdown earthquake 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.

Coupled Shield and Auxiliary Buildings on Fixed Base

The analyses are performed using the three-dimensional, finite element model of the coupled shield and auxiliary buildings and the stick models of the shield building roof, the steel containment vessel 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. 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 for the hard rock site where the soil-structure interaction effect is negligible, as described in Appendix 2B. Response spectrum analyses are performed using the fixed-base, three-dimensional, finite element models. The support provided by the embedment below grade is not considered in these response spectrum analyses.

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 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.

Coupled Shield and Auxiliary Buildings on Flexible Base

Response spectrum analyses are also performed using the Coupled Auxiliary and Shield Buildings on a flexible base. The model is the same as that used for the fixed-base hard rock site response spectrum analyses described above, except that plate elements representing the basemat and horizontal and vertical springs are added to represent the flexibility of the subgrade. As in the fixed-base hard rock site response spectrum analyses, the support provided by the embedment below grade is not considered.

The response spectrum analysis performed for the flexible base overestimates the seismic response because of the conservative treatment of soil structure interaction. It provides the relative distribution of loads to the various shear walls when the plant is located at a soil site. Adjustment factors are applied so that the overall forces in the structure match corresponding results from the SSI analyses performed previously using SASSI.

The envelope of the in-plane forces obtained from the response spectrum analyses on the fixed base and on the flexible base is used for the design of floors and walls.

Containment Internal Structures

Response spectrum analyses of the containment internal structures on a fixed base are performed using the three-dimensional, finite element model of the containment internal structures developed and discussed in subsection 3.7.2.3. Figure 3.7.2-2 shows the finite element model of the containment internal structures. The forces obtained from the response spectrum analyses of the finite element models for the hard rock site are increased by a scaling factor to account for other soil profiles as described for the coupled shield and auxiliary buildings.

Response spectrum analysis of the fixed-base nuclear island lumped-mass stick model is discussed in subsection 3.7.2.2.

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 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 stiff beam 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 70 vibration modes, up to the frequency limit of 33 hertz, shown in Table 3.7.2-4. The total cumulative mass participating in the seismic response constitute 88, 89, and 88 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 32 percent in the north-south direction, 29 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 83, 83, and 41 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-base nuclear island lumped-mass stick model. The response spectrum analysis considers 70 vibration modes, up to 33 hertz, using the double sum modal combination method and, above 33 hertz, non-participating mass are considered 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 the vibration mode shapes for the combined lumped-mass stick model consisting of 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. 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 in Tables 3.7.2-8 through 3.7.2-10, 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, respectively, for the coupled shield and auxiliary buildings, the steel containment vessel, and the containment internal structures.

3.7.2.2.1 Seismic Model Modifications

Additional analyses are performed to evaluate the effects of added water inventory in the Passive Containment Cooling System tank on top of the shield building and the addition of mass due to snow and live loads. These analyses use the nuclear island lumped mass stick seismic model described in subsection 3.7.2.3.3 modified as follows:

- The elevation of the top nodes of the coupled shield and auxiliary building stick shown in Figure 3.7.2-4 are raised from elevations 306.25' and 297.08' to elevations 307.25' and 297.58'. The lumped mass stick model for the shield building roof includes the increase in tank volume and the added water inventory in the Passive Contaiment Cooling Tank.

- 75 percent of the snow load and 25 percent of the live load are added as mass to the coupled shield and auxiliary building stick and to the containment internal structures stick.

Modal analyses using the lumped mass stick model (designated Model B in the tables) are performed using computer program BSAP. Modal frequencies for the coupled auxiliary and shield buildings are summarized in Table 3.7.2-20 and compared with the frequencies of the models described in subsection 3.7.2.3.3 (designated as Model A in the tables). The comparison demonstrates that the modifications have only minor effects on the fixed-base seismic analysis which represents the hard rock site condition.

Of the three design soil cases, the upper bound of the soft to medium (2G) soil case is the most controlling for the design of the AP600. This case is selected to evaluate the effect of the modifications to the seismic model. SASSI analyses using the modified model are performed for the upper bound of the soft to medium (2G) soil case. Maximum nodal accelerations and member forces are compared in Tables 3.7.2-21, 3.7.2-22 and 3.7.2-23.

Floor response spectra for the 2G soil case are calculated at selected locations. The differences in the floor response spectra are small, except in the shield building roof in the vertical direction. The peak of the vertical broadened spectra is increased to envelope the results for the modified seismic model as discussed in section 3.7.2.5. The broadened floor response spectra at the base of the passive containment cooling water storage tank (elevation 272.42') are shown in Figure 3.7.2-20. The analyses confirm the adequacy of the seismic responses used in the design of the structures and the adequacy of the floor response spectra.

Site specific evaluation, if required in accordance with section 2.5.2.2, will use the modified lumped mass stick model.

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, a finite element model of the shield building roof, and an axisymmetric shell model of the steel containment vessel. 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 to vertical elastic structural elements by horizontal stiff beam elements to simulate eccentricity. The individual building lumped-mass stick models are interconnected with other stiff beam 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 is less than one percent of the respective supporting nuclear island structures; therefore, the mass of other major subsystems and equipment is 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 foot is considered to represent miscellaneous deadweight such as minor equipment, piping and raceways. 25 percent of the floor live load or 75 percent of the roof snow load, whichever is applicable, is considered as mass in the global seismic models (these masses are only included in the modified model described in subsection 3.7.2.2.1). 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 horizontal stiff 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. The member properties of the stick model are selected to match the frequencies and mode shapes from the finite element model. The portion of the roof from the bottom of the air inlets to the bottom of the passive containment cooling system tank is modelled by an equivalent beam. 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 stiff beams.

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

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 above elevation 240′ 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 only seismic Category I components attached to this portion of the top head are the water distribution weirs of the passive containment cooling system. These weirs are designed such that their fundamental frequencies are outside the 23 to 30 hertz range of the local shell modes.

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 stiff beam elements to form the overall dynamic model of the nuclear island as shown in Figure 3.7.2-18. For the fixed-base analysis, the nuclear island seismic model consists of 71 mass points and 178 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 hydrodynamic mass effect of the water within the passive containment cooling system water tank on the shield building roof, the in-containment refueling water storage tank within the containment internal structures, and the spent fuel pool in the auxiliary building is evaluated. The convective (sloshing) effect of the water mass within the passive containment cooling system water tank on the shield building roof is found to be negligible. Hence, only the impulsive effect of the water mass is included in the nuclear island seismic model. The total mass of the water in the in-containment refueling water storage tank within the containment internal structures, and the spent fuel pool in the auxiliary building 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 soil-structure interaction responses. The nuclear island soil-structure interaction responses generated for the analysis and design of seismic subsystems include nodal displacements, nodal accelerations, and floor response spectra.

The nuclear island soil-structure interaction analyses using three-dimensional models 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 soil-structure interaction 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 soil-structure interaction analyses using three-dimensional models. The effect of the adjacent structures is included in the analysis for lateral earth pressure as described in Appendix 2C.

Soil-structure interaction 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 soil-structure interaction analyses is limited not to exceed 15 percent. The soil-structure interaction analyses of the nuclear island are performed using the program SASSI, which is capable of handling two- and three-dimensional soil-structure interaction problems involving multiple structures with rigid or flexible embedded foundations of arbitrary shape.

Soil-structure interaction 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 (1) vertical elastic beam elements between floor elevations to represent wall stiffness and (2) lumped masses at the center of mass of each floor elevation. At each floor elevation, these vertical beam elements are connected with the lumped masses through horizontal stiff beam elements. For the soil-structure interaction analyses using program SASSI, these horizontal stiff beams have the following properties:

- The area to length ratio of the stiff beam element is within the range of 10^3 to 10^5 times the largest area to length ratio of its connecting elastic structural elements.
- The moment of inertia to length^3 ratio of the stiff beam 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 three-dimensional 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 stiff beams at elevations 82′-6″ and 100′-0″ (see Figure 3.7.2-13). The stiffness of each of these stiff beams is based on the lower stiffness of the connecting members.

The soil-structure interaction effects on the seismic Category I structures due to embedment of the nuclear island, 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.

Two-dimensional seismic soil-structure interaction analyses are performed as described in Appendix 2C to obtain lateral earth pressures on the exterior walls below grade.

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 ±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 3.7.2-14 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 3.7.2-15 through 3.7.2-17. 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.

The broadened floor response spectra for the shield building roof in the vertical direction are based on soil structure interaction analyses which include added inventory in the Passive Containment Cooling System tank. These analyses use the modified nuclear island seismic model described in subsection 3.7.2.2.1. The peak of the vertical broadened spectra for the shield building roof are increased to envelope the results of the additional analysis as shown in sheet 9 of Figure 3.7.2-15.

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 square root of the sum of squares 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 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.

3.7.2.8.1 Annex Building

The annex building is classified as seismic Category II. The structural configuration is shown in Figure 3.7.2-19. The annex building is analyzed for the safe shutdown earthquake for the four sites described in subsection 3.7.1.4. Seismic input is defined by response spectra applied at the base of a dynamic model of the annex building. The horizontal spectra are obtained from the 2D SASSI analyses described in Appendix 2C and account for soil-structure and structure-soil-structure interaction. Input in the east-west direction uses the response spectra obtained from the two dimensional analyses for the annex building mat. Input in the north-south direction uses the response spectra obtained from the two dimensional analyses for the turbine building mat. Vertical input is obtained from 2D FLUSH finite element soil-structure interaction analyses. The seismic response spectra input at the base of the annex building are the envelopes of the four sites and also envelope the AP600 design free field ground spectra shown in Figures 3.7.1-1 and 3.7-1-2. The envelope of the maximum building response acceleration values is applied as equivalent static loads to a more detailed static model.

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 maximum displacement of the roof of the annex building is 1.6 inches in the east-west direction. The minimum clearance between the structural elements of the annex building above grade and the nuclear island is 4 inches.

3.7.2.8.2 Radwaste Building

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 1.2-22, 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 above grade and the nuclear island is 4 inches.

Three methods are used to demonstrate that a potential radwaste building impact on the nuclear island during a seismic event will not impair its structural integrity:

- The maximum kinetic energy of the impact during a seismic event considers the maximum radwaste building and nuclear island velocities. The total kinetic energy is considered to be absorbed by the nuclear island and converted to strain energy. The deflection of the nuclear island is less than 0.2″. The shear forces in the nuclear island walls are less than the ultimate shear strength based on a minus one standard deviation of test data.

- Stress wave evaluation shows that the stress wave resulting from the impact of the radwaste building on the nuclear island has a maximum compressive stress less than the concrete compressive strength.

- An energy comparison shows that the kinetic energy of the radwaste building is less than the kinetic energy of tornado missiles for which the exterior walls of the nuclear island are designed.

3.7.2.8.3 Turbine Building

The turbine building is classified as nonseismic. As shown on the turbine building general arrangement in Figures 1.2-23 through 1.2-30, the major structure of the turbine building is separated from the nuclear island by approximately 18 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 nominal horizontal clearance of 12 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 an eccentrically 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 1991 Uniform Building Code provisions for Zone 3 with an Importance Factor of 1.0. For an eccentrically braced structure the resistance modification factor is 10 (UBC-91, reference 1) using allowable stress design. When using allowable stress design, the allowable stresses are not increased by one third for seismic loads. The resistance modification factor is reduced to 7 for load and resistance factor design (ASCE 7-93, reference 35).

- The nominal horizontal clearance between the structural elements of the turbine building above grade and the nuclear island and annex building is 12 inches.

- The design of the lateral bracing system complies with the seismic requirements for eccentrically braced frames given in section 9.3 of the AISC Seismic Provisions for Structural Steel Buildings. (reference 34). Quality assurance is in accordance with ASCE 7-93 (reference 35) for the lateral bracing system.

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. Stresses in the concrete structural elements due to the safe shutdown earthquake are below the tensile strength of the concrete. The effect of cracking of the concrete-filled structural modules inside containment due to thermal loads is discussed in subsection 3.8.3.4.2.

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

The three-dimensional lumped mass fixed base stick model of the nuclear island was analyzed by mode superposition time history analysis and by the response spectrum analysis method for the hard rock site condition. Tables 3.7.2-17, 3.7.2-18, and 3.7.2-19 compare the maximum absolute nodal accelerations, member forces, and moments, respectively. Both analyses considered vibration modes up to 33 hertz. In the response spectrum analyses, the combination of modal responses used the double sum method and included high frequency effects as discussed in subsection 3.7.2.7 and summarized in Table 3.7.2-16. The two methods of analysis give similar results with the response spectrum analysis being generally more conservative. Investigations of the two analyses showed that the conservatism in the response spectrum analyses is due to cross coupling of the directions in the multistick model. The double sum modal combination method used in the response spectrum analysis is very conservative when there are closely spaced modes some of which are out-of-phase.

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, walls, 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 safe shutdown earthquake amplitude. The number of cycles is calculated based on IEEE-344-1987 (Reference 21) to provide the equivalent fatigue damage of two safe shutdown earthquake events with 10 high-stress cycles per event. Typically, there are five seismic events with an amplitude equal to one-third of the safe shutdown earthquake 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 safe shutdown earthquake 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 safe shutdown earthquake event. Typically, the preceding earthquake is one safe shutdown earthquake event or five one-half safe shutdown earthquake 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.

Dynamic models of floor and roof slabs and miscellaneous steel platforms and framing include masses equal to 25 percent of the floor live load or 75 percent on the roof snow load, whichever is applicable.

Dynamic models are prepared for the following seismic Category I steel structures. Response spectrum or time history analyses are performed for structural design.

- Passive containment cooling valve room (room number 12701)
- Steel framing around steam generators
- Containment air baffle

Seismic input for the subsystem and component design are the enveloped floor response spectra described in subsection 3.7.2.5 or the response time histories for each of the four design soil profiles as described in subsection 3.7.2.1. Where amplified response spectra are required on the subsystem for design of components, such as for use in the decoupled analyses of piping or components described in subsection 3.7.3.8.3, the amplified response spectra are generated and enveloped as described in subsection 3.7.2.5.

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 hertz), 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 hertz), whose support can be represented by a flexible spring, can be modelled 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. Static factors smaller than 1.5 are not used for piping systems. 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 which are rigid in the axial direction (fundamental frequency greater than or equal to 33 hertz), 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. This method is not used for piping systems.

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 the 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 Regulatory 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 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:

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

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. For piping systems, the methods in Regulatory Guide 1.92 are used for modal combinations. For other subsystems, the methods in Regulatory Guide 1.92 or the following alternative methods may be used. 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,

$$w_k^{'} = w_k \; [1 - (\beta_k)^2]^{1/2}$$

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

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,

$$R_T^2 = \sum_{i=1}^{N} R_i^2 + 2\sum \varepsilon_{k\ell} R_k R_\ell$$

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 zero period acceleration frequency.

The piping system analysis model includes the effect of piping support mass when the contributory mass of the support is greater than 10 percent of the total mass of the effected piping spans. The contributory mass of the support is the portion of the support mass that is attached to the piping; such as clamps, bolts, trunnions, struts, and snubbers. Supports that are not directly attached to the piping, such as box frames, need not be considered for mass effects. The mass of the applicable support will not affect the response of the system in the supported direction, therefore only the unsupported direction needs to be considered. Based on this reasoning, the mass of full anchors can be neglected. The total mass of each effected piping span includes the mass of the piping, fluid contents, insulation, and any concentrated masses (for example, valves or flanges) between the adjacent supports in each unrestrained direction on both sides of the applicable support. For example; the contributory mass of an X direction support must be compared to the mass of the piping spans in the unrestrained Y and Z directions. A contributory support mass that is less than 10 percent of the masses of the effected spans will have insignificant effect on the response of the piping system and can be neglected.

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:

- First six-way anchor
- Four rigid/seismic supports in each of the three perpendicular directions
- 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 1-inch nominal pipe size and ASME Class 2 and 3 piping with nominal pipe size larger than 2 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.

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

The seismic inertia effects of equipment and piping that provide support to supported (branch) piping systems are considered when significant. When the frequency of the supporting equipment is less than 33 hertz. then either a coupled dynamic model of the piping and equipment is used, or the amplified response spectra at the equipment connection point is used with a decoupled model of the supported piping. When supported piping is supported by larger piping, one of the following methods is used:

- A coupled dynamic model of the supported piping and the supporting piping
- Amplified response spectra at the connection point to the supporting piping with a decoupled model of the supported piping

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 2 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
- Equivalent static analysis based on appropriate load factors applied to the response spectra acceleration values

The Combined License applicants will complete the final design of the small-bore piping and address the as-built reconciliation in accordance with the criteria outlined in subsections 3.9.3 and 3.9.8.2.

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
- $Rm \geq 0.01$ and ≤ 0.10, if $Rf \leq 0.8$ 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 safe shutdown earthquake 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 12.

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 the 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 high 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 support 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:

1. 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.

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

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

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

5. 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. 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 subsection 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 (for example, heat exchangers)
- Mechanical or electrical penetrations
- Heating, ventilation, and air conditioning (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 3F and 3A 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 6-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. The frequency of the piping system in the rigid region is greater than or equal to 33 hertz. 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 one 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. Safe shutdown earthquake 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 the weighted 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. Composite modal damping is used for piping systems that are coupled to flexible equipment or flexible valves. Piping systems analyzed by the uniform envelope response spectra method with rigid 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 or with structural modules, as discussed in subsections 3.8.3 and 3.8.4. 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. WECAN is not used for linear time history analyses or response spectra analyses of piping systems.

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 cutoff frequency is selected based on the frequency content of the input forcing function and the highest significant frequency of the piping system. The integration time step is no larger than 10 percent of the period of the cutoff frequency.

Four separate soil cases, as described in subsection 3.7.1.4 are considered in the seismic analysis. When time history analysis is used, a time history analysis for each soil case is performed. For time history analysis of piping system models that include a dynamic model of the supporting concrete building either the building stiffness is varied by + or - 30 percent, or the time scale is shifted by + or - 15 percent, to account for uncertainties. Alternately, when uniform enveloping time history analysis is performed, modelling uncertainties are accounted for by the spreading that is included in the broadened response spectra. In this case, the four soils are accounted for in the broadened response spectra.

For dynamic analysis, other than seismic analysis, three separate analyses are performed for each loading case to account for uncertainties. The three analyses correspond to three different time scales: normal time, time expanded by 15 percent, and time compressed by 15 percent. Alternatively, when the results are shown to be acceptable based on comparison with test data, one time history analysis is performed using normal time.

For time history analysis using the PS+CAEPIPE program, the response from the high frequency modes above the cutoff frequency is calculated based on the static response to the left-out-forces. This response is combined with the response from the low frequency modes by algebraic sum at each time step. Composite modal damping is used with PS+CAEPIPE program. The damping of the individual components is as listed in Table 3.7.1-1.

Alternately, for time history analysis using the PS+CAEPIPE, 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. The damping of the individual components is as listed in Table 3.7.1-1.

3.7.4 Seismic Instrumentation

3.7.4.1 Comparison with Regulatory Guide 1.12

Compliance with Regulatory Guide 1.12 and draft Regulatory Guide DG-1016 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 and analysis system has internal batteries and a charger to prevent the loss of data during a power outage, and to allow data collection and analysis 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. 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. A minimum of 25 minutes of continuous recording is provided. 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. The triaxial acceleration sensors have a dynamic range of 1000 to 1 (0.001 to 1.0g) and a frequency range of 0.2 to 50 hertz.

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 time-history analyzer can compute cumulative absolute velocity (CAV) and the 5 percent of critical damping response spectrum for frequencies between 1 and 10 Hz. The operator may select the analysis of either CAV 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 holder 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).

3.7.5.3 Seismic Interaction Review

The seismic interaction review will be updated by the Combined License applicant. This review is performed in parallel with the seismic margin evaluation. The review is based on as-procured data, as well as the as-constructed condition.

3.7.5.4 Reconciliation of Seismic Analyses of Nuclear Island Structures

The Combined License applicant will reconcile the seismic analyses described in subsection 3.7.2 for detail design changes such as those due to as-procured equipment information. Deviations are acceptable based on an evaluation consistent with the methods and procedure of Section 3.7 provided the amplitude of the seismic floor response spectra including the effect due to these deviations, do not exceed the design basis floor response spectra by more than 10 percent. If it is necessary to update the soil structure interaction analyses, these analyses should be performed with site specific soil properties using seismic input defined by the response spectra given in Figures 3.7.1-1 and 3.7.1-2.

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Table 3.7.1-1

SAFE SHUTDOWN EARTHQUAKE DAMPING VALUES

	Percent
Welded aluminum structures	4
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
Piping (for uniform envelope response spectra analysis)	5
Piping (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
Full cable trays and related supports	10[1]
Empty cable trays and related supports	7
Conduits and 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

Notes

1. Cable tray systems similar to those tested in Reference 19 may use the damping values given in Figure 3.7.1-13.

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.*]*

*NRC Staff approval is required prior to implementing a change in this information; see DCD Introduction Section 3.5.

Table 3.7.1-3

AP600 DESIGN RESPONSE SPECTRA
AMPLIFICATION FACTORS FOR CONTROL POINTS

HORIZONTAL

Percent of Critical Damping	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

VERTICAL

Percent of Critical Damping	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

<u>Note</u>:

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 percent damping amplification factor for control point B' is derived per discussion in subsection 3.7.1.1. This 5 percent 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			Modal Masses			Cumulative Mass, %		
			X	Y	Z	X	Y	Z	X	Y	Z
1	0.12	1.00	0.003	-0.343	0.000	0.000	0.118	0.000	0.0	0.0	0.0
2	0.12	1.00	0.264	0.001	0.000	0.069	0.000	0.000	0.0	0.0	0.0
3	0.12	1.00	-0.037	1.354	0.000	0.001	1.833	0.000	0.0	0.0	0.0
4	0.12	1.00	-0.730	-0.011	0.000	0.533	0.000	0.000	0.0	0.0	0.0
5	0.12	1.00	0.316	-6.395	0.000	0.100	40.891	0.000	0.0	1.0	0.0
6	0.12	1.00	6.005	0.297	0.000	36.057	0.088	0.000	0.8	1.0	0.0
7	0.12	1.00	-0.231	5.371	0.000	0.053	28.843	0.000	0.8	1.6	0.0
8	0.12	1.00	5.815	0.253	0.000	33.816	0.064	0.000	1.6	1.6	0.0
9	0.12	1.00	-0.114	3.152	0.000	0.013	9.933	0.000	1.6	1.9	0.0
10	0.12	1.00	-3.333	-0.123	0.000	11.110	0.015	0.000	1.9	1.9	0.0
11	0.12	1.00	0.000	0.046	0.000	0.000	0.002	0.000	1.9	1.9	0.0
12	0.12	1.00	-0.040	0.000	0.000	0.002	0.000	0.000	1.9	1.9	0.0
13	4.31	1.00	-1.380	35.789	-1.662	1.904	1280.844	2.761	1.9	31.1	0.1
14	4.77	1.00	-34.559	-1.422	1.417	1194.356	2.022	2.008	29.1	31.1	0.1
15	6.77	1.00	1.889	3.126	24.525	3.567	9.773	601.462	29.2	31.3	13.8
16	8.84	1.00	2.018	11.455	-0.090	4.072	131.206	0.008	29.3	34.3	13.8
17	9.17	1.00	-7.506	-16.983	-0.232	56.342	288.407	0.054	30.6	40.9	13.8
18	9.27	1.00	-19.100	8.662	0.254	364.820	75.035	0.065	38.9	42.6	13.8
19	11.94	1.00	0.820	27.981	0.884	0.672	782.936	0.782	38.9	60.5	13.8
20	12.56	1.00	30.115	-0.850	0.125	906.937	0.722	0.016	59.6	60.5	13.8
21	14.81	1.00	-0.211	-1.646	-0.153	0.045	2.709	0.023	59.6	60.6	13.8
22	18.50	1.00	-1.862	15.634	7.629	3.467	244.411	58.200	59.7	66.1	15.2
23	19.10	1.00	-17.757	-1.624	-3.621	315.324	2.638	13.110	66.9	66.2	15.5
24	19.50	1.00	-0.554	-2.655	28.532	0.307	7.051	814.057	66.9	66.4	34.0
25	22.19	1.00	-3.638	4.774	-11.694	13.232	22.792	136.745	67.2	66.9	37.2
26	23.06	1.00	0.402	3.951	-3.622	0.162	15.614	13.119	67.2	67.2	37.5
27	23.38	1.00	-3.023	-0.628	26.338	9.139	0.395	693.666	67.4	67.2	53.3
28	26.03	1.00	-1.324	15.045	-0.658	1.754	226.357	0.432	67.5	72.4	53.3
29	26.82	1.00	-16.060	-2.374	-0.616	257.920	5.636	0.379	73.3	72.5	53.3
30	31.18	1.00	-2.832	-2.517	0.779	8.019	6.333	0.607	73.5	72.75	53.3
			SUMMATIONS			3223.791	3186.667	2337.493			
			TOTAL MASS			4385.169	4384.559	4386.159			

Note:
1. Fixed at elevation 66.5'.
2. The first twelve modes are principally water sloshing in the passive containment system tank.

Table 3.7.2-2

STEEL CONTAINMENT VESSEL LUMPED-MASS STICK MODEL MODAL PROPERTIES

Mode No.	Freq. (CPS)	Genl. Mass	Participation Factors			Modal Masses			Cumulative Mass, %		
			X	Y	Z	X	Y	Z	X	Y	Z
1	2.20	1.00	0.000	-4.007	-0.003	0.000	16.052	0.000	0.0	7.7	0.0
2	4.54	1.00	-0.003	0.000	3.875	0.000	0.000	15.013	0.0	7.7	7.1
3	5.06	1.00	-7.689	0.000	0.037	59.118	0.000	0.001	28.4	7.7	7.1
4	7.42	1.00	0.002	-11.910	-0.128	0.000	141.850	0.016	28.4	75.8	7.1
5	8.02	1.00	-9.919	-0.009	0.158	98.395	0.000	0.025	75.7	75.8	7.1
6	14.70	1.00	0.241	-0.076	0.001	0.058	0.006	0.000	75.7	75.8	7.1
7	18.10	1.00	0.441	-0.129	11.877	0.194	0.017	141.072	75.8	75.9	74.1
8	21.80	1.00	-5.491	0.508	0.441	30.149	0.258	0.195	90.3	76.0	74.2
9	21.94	1.00	0.514	5.466	-0.070	0.264	29.878	0.005	90.4	90.3	74.2
10	29.65	1.00	-0.329	0.087	5.653	0.108	0.008	31.960	90.4	90.3	89.3
11	34.82	1.00	1.687	0.445	0.596	2.847	0.198	0.355	91.8	90.4	89.5
12	34.92	1.00	-0.425	1.671	-0.179	0.181	2.793	0.032	91.9	91.8	89.5
13	42.93	1.00	-0.454	-2.078	0.166	0.206	4.320	0.028	92.0	93.8	89.5
14	43.61	1.00	-1.784	0.826	-0.033	3.181	0.683	0.001	93.5	94.2	89.5
15	45.76	1.00	-1.331	-0.367	0.010	1.771	0.134	0.000	94.4	94.2	89.5
				SUMMATIONS		196.473	196.197	188.704			
				TOTAL MASS		208.206	208.207	210.763			

Note:
1. Fixed at Elevation 100′.
2. The first three modes 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			Modal Masses			Cumulative Mass, %		
			X	Y	Z	X	Y	Z	X	Y	Z
1	12.74	1.00	-4.460	16.161	-0.330	19.887	261.179	0.109	1.5	19.3	0.0
2	14.72	1.00	-15.567	-9.894	0.569	242.321	97.895	0.323	19.3	26.5	0.0
3	16.68	1.00	-13.153	6.314	0.015	173.001	39.863	0.000	32.1	29.4	0.0
4	34.49	1.00	1.594	4.458	7.124	2.541	19.873	50.746	32.3	30.9	3.8
5	37.84	1.00	5.321	-2.266	6.361	28.315	5.133	40.458	34.4	31.3	6.8
6	39.98	1.00	10.471	2.051	-1.860	109.634	4.208	3.461	42.5	31.6	7.0
7	42.11	1.00	0.956	-26.330	0.247	0.915	693.290	0.061	42.5	82.7	7.0
8	42.83	1.00	-23.232	-0.419	0.076	539.727	0.176	0.006	82.3	82.7	7.0
9	45.76	1.00	-2.195	0.594	5.413	4.818	0.353	29.302	82.7	82.8	9.2
10	48.15	1.00	0.158	0.691	-0.882	0.025	0.477	0.778	82.7	82.8	9.2
11	49.66	1.00	1.996	0.607	6.311	3.985	0.368	39.833	83.0	82.8	12.2
12	51.67	1.00	-1.085	1.384	1.070	1.177	1.915	1.144	83.1	83.0	12.3
13	57.94	1.00	2.047	0.089	-19.465	4.190	0.008	378.905	83.4	83.0	40.2
14	58.95	1.00	0.064	0.486	-3.125	0.004	0.236	9.767	83.4	83.0	40.9
15	71.39	1.00	-0.022	-0.016	-0.122	0.000	0.000	0.015	83.4	83.0	40.9
				SUMMATIONS		1130.539	1124.973	554.909			
				TOTAL MASS		1355.830	1355.830	1355.820			

Note:
1. Fixed at Elevation 82.5′.

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			Modal Masses			Cumulative Mass, %		
			X	Y	Z	X	Y	Z	X	Y	Z
1	4.45	1.00	0.004	6.676	-0.025	0.000	44.566	0.001	0.0	3.0	0.0
2	4.58	1.00	0.001	0.000	-0.257	0.000	0.000	0.066	0.0	3.0	0.0
3	6.18	1.00	-5.633	0.459	0.000	31.727	0.210	0.000	2.1	3.0	0.0
4	6.50	1.00	3.845	0.091	-0.046	14.781	0.008	0.002	3.1	3.0	0.0
5	10.56	1.00	2.302	12.546	-0.069	5.298	157.412	0.005	3.5	13.6	0.0
6	11.89	1.00	1.661	-0.287	1.786	2.758	0.082	3.191	3.7	13.6	0.2
7	11.92	1.00	1.226	-0.300	2.155	1.504	0.090	4.645	3.8	13.6	0.5
8	12.11	1.00	1.336	-0.663	-8.176	1.784	0.439	66.851	3.9	13.6	5.0
9	12.34	1.00	-4.276	4.062	-1.198	18.284	16.504	1.435	5.1	14.7	5.1
10	12.86	1.00	-5.752	13.686	-0.344	33.083	187.314	0.118	7.3	27.3	5.1
11	13.59	1.00	-10.061	-0.770	0.079	101.234	0.592	0.006	14.1	27.3	5.1
12	14.28	1.00	10.020	1.801	-0.215	100.405	3.245	0.046	20.8	27.5	5.1
13	14.72	1.00	9.959	8.175	-0.203	99.178	66.836	0.041	27.5	32.0	5.1
14	14.95	1.00	4.718	4.236	-0.555	22.263	17.940	0.308	29.0	33.2	5.1
15	14.98	1.00	-3.009	-2.656	0.233	9.053	7.054	0.054	29.6	33.7	5.1
16	15.47	1.00	1.913	1.141	-0.147	3.660	1.301	0.022	29.8	33.8	5.1
17	16.64	1.00	-1.054	0.466	0.780	1.111	0.218	0.609	29.9	33.8	5.2
18	16.83	1.00	-11.787	5.810	-0.066	138.935	33.751	0.004	39.2	36.0	5.2
19	17.56	1.00	-0.248	-0.186	-0.021	0.062	0.035	0.000	39.2	36.0	5.2
20	18.05	1.00	0.111	-0.091	0.315	0.012	0.008	0.099	39.2	36.0	5.2
21	20.08	1.00	-0.294	0.199	-0.102	0.086	0.039	0.010	39.2	36.1	5.2
22	20.16	1.00	-0.056	-0.894	-0.046	0.003	0.800	0.002	39.2	36.1	5.2
23	20.40	1.00	0.142	0.094	-0.037	0.020	0.009	0.001	39.2	36.1	5.2
24	20.94	1.00	-0.057	0.057	-1.510	0.003	0.003	2.281	39.2	36.1	5.4
25	22.04	1.00	0.183	1.933	-0.088	0.034	3.737	0.008	39.2	36.4	5.4
26	23.47	1.00	-0.037	0.131	-0.098	0.001	0.017	0.010	39.2	36.4	5.4
27	24.03	1.00	-0.503	-0.085	-0.037	0.253	0.007	0.001	39.3	36.4	5.4
28	24.60	1.00	-0.919	0.471	-0.038	0.844	0.221	0.001	39.3	36.4	5.4
29	25.20	1.00	-0.183	-0.189	-0.169	0.034	0.036	0.029	39.3	36.4	5.4
30	31.99	1.00	0.295	0.598	5.066	0.087	0.358	25.662	39.3	36.4	7.1
31	33.71	1.00	1.132	5.424	5.077	1.281	29.422	25.774	39.4	38.4	8.8
32	34.94	1.00	-0.307	0.280	-7.958	0.094	0.078	63.323	39.4	38.4	13.0

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			Modal Masses			Cumulative Mass, %		
			X	Y	Z	X	Y	Z	X	Y	Z
33	35.53	1.00	-0.061	1.334	-0.304	0.004	1.781	0.092	39.4	38.5	13.1
34	36.19	1.00	-5.127	1.020	-6.496	26.291	1.040	42.202	41.2	38.6	15.9
35	36.46	1.00	-1.117	0.194	2.582	1.248	0.038	6.664	41.3	38.6	16.3
36	37.15	1.00	-4.908	0.648	0.424	24.086	0.420	0.180	42.9	38.6	16.3
37	37.44	1.00	-4.137	-0.560	1.546	17.112	0.314	2.390	44.0	38.6	16.5
38	37.60	1.00	3.659	1.011	-2.573	13.389	1.022	6.619	44.9	38.7	16.9
39	37.76	1.00	2.598	-0.540	-0.406	6.750	0.292	0.165	45.4	38.7	17.0
40	39.57	1.00	0.755	3.834	0.061	0.570	14.702	0.004	45.4	39.7	17.0
41	42.30	1.00	-3.351	-24.961	-0.275	11.228	623.055	0.076	46.2	81.5	17.0
42	42.47	1.00	11.107	-6.340	2.925	123.359	40.200	8.555	54.4	84.2	17.5
43	43.01	1.00	-16.324	0.568	0.799	266.482	0.323	0.638	72.3	84.2	17.6
44	43.75	1.00	13.158	-0.477	-1.673	173.139	0.228	2.801	83.9	84.2	17.8
45	46.14	1.00	-1.852	0.659	4.026	3.428	0.435	16.210	84.2	84.2	18.9
46	47.74	1.00	-0.175	-0.933	0.643	0.031	0.870	0.413	84.2	84.3	18.9
47	49.69	1.00	2.082	0.587	6.085	4.333	0.344	37.023	84.4	84.3	21.4
48	51.61	1.00	1.207	-1.408	-1.696	1.456	1.984	2.876	84.5	84.4	21.6
49	55.37	1.00	-0.024	0.003	0.005	0.001	0.000	0.000	84.5	84.4	21.6
50	55.38	1.00	0.016	0.001	0.236	0.000	0.000	0.056	84.5	84.4	21.6
51	57.25	1.00	2.117	-0.126	19.173	4.481	0.016	367.606	84.8	84.5	46.2
52	58.97	1.00	-0.023	0.493	-2.234	0.001	0.243	4.989	84.8	84.5	46.5
				SUMMATIONS		1265.263	1259.640	694.166			
				TOTAL MASS		1491.283	1491.283	1491.273			

Note:
1. Fixed at Elevation 82.5′.

Table 3.7.2-4 (Sheet 1 of 2)

NUCLEAR ISLAND
COMBINED LUMPED-MASS STICK MODEL
MODAL PROPERTIES

Mode No.	Freq. (CPS)	Genl. Mass	Participation Factors			Modal Masses			Cumulative Mass, %		
			X	Y	Z	X	Y	Z	X	Y	Z
1	0.12	1.00	0.069	0.366	0.000	0.005	0.134	0.000	0.0	0.0	0.0
2	0.12	1.00	0.285	-0.066	0.000	0.081	0.004	0.000	0.0	0.0	0.0
3	0.12	1.00	-0.136	-1.508	0.000	0.018	2.273	0.000	0.0	0.0	0.0
4	0.12	1.00	0.843	-0.117	0.000	0.711	0.014	0.000	0.0	0.0	0.0
5	0.12	1.00	0.180	6.407	0.000	0.033	41.056	0.000	0.0	0.7	0.0
6	0.12	1.00	6.070	-0.171	0.000	36.850	0.029	0.000	0.6	0.7	0.0
7	0.12	1.00	0.245	5.329	0.000	0.060	28.398	0.000	0.6	1.2	0.0
8	0.12	1.00	5.753	-0.257	0.000	33.101	0.066	0.000	1.1	1.2	0.0
9	0.12	1.00	0.206	3.124	0.000	0.042	9.757	0.000	1.1	1.3	0.0
10	0.12	1.00	3.293	-0.213	0.000	10.843	0.045	0.000	1.3	1.3	0.0
11	0.12	1.00	0.014	0.046	0.000	0.000	0.002	0.000	1.3	1.3	0.0
12	0.12	1.00	-0.040	0.014	0.000	0.002	0.000	0.000	1.3	1.3	0.0
13	2.20	1.00	0.000	4.080	0.003	0.000	16.648	0.000	1.3	1.6	0.0
14	4.35	1.00	0.283	33.370	-2.009	0.080	1113.573	4.036	1.3	19.6	0.1
15	4.45	1.00	-0.012	5.531	0.053	0.000	30.594	0.003	1.3	20.0	0.1
16	4.54	1.00	-0.117	0.381	3.940	0.014	0.145	15.523	1.3	20.0	0.3
17	4.58	1.00	0.000	-0.002	-0.259	0.000	0.000	0.067	1.3	20.0	0.3
18	4.78	1.00	32.172	-0.667	-1.335	1035.048	0.445	1.784	18.0	20.1	0.3
19	5.06	1.00	5.981	0.027	0.030	35.776	0.001	0.001	18.6	20.1	0.3
20	6.19	1.00	5.326	-0.445	0.009	28.367	0.198	0.000	19.1	20.1	0.3
21	6.50	1.00	3.694	0.078	-0.037	13.644	0.006	0.001	19.3	20.1	0.3
22	6.77	1.00	1.468	2.472	24.673	2.154	6.112	608.762	19.3	20.2	10.2
23	7.37	1.00	0.022	11.550	0.477	0.000	133.412	0.227	19.3	22.3	10.2
24	8.00	1.00	9.840	0.048	-0.116	96.826	0.002	0.013	20.9	22.3	10.2
25	9.10	1.00	2.183	-14.686	-0.073	4.766	215.682	0.005	20.9	25.8	10.2
26	9.26	1.00	13.890	6.919	0.023	192.921	47.879	0.001	24.1	26.6	10.2
27	9.70	1.00	7.971	-6.609	-0.216	63.533	43.682	0.047	25.1	27.3	10.2
28	10.61	1.00	2.072	11.187	-0.020	4.295	125.150	0.000	25.2	29.3	10.2
29	11.89	1.00	1.219	-0.033	1.726	1.486	0.001	2.979	25.2	29.3	10.2
30	11.92	1.00	-1.140	0.348	-2.516	1.300	0.121	6.333	25.2	29.3	10.3
31	12.11	1.00	0.993	-0.376	-8.598	0.986	0.142	73.924	25.2	29.3	11.5
32	12.35	1.00	-3.016	2.922	-1.066	9.095	8.536	1.136	25.4	29.4	11.5
33	12.85	1.00	-1.151	-25.574	-0.532	1.325	654.006	0.283	25.4	40.0	11.5
34	12.99	1.00	-4.943	9.503	-0.237	24.436	90.307	0.056	25.8	41.4	11.5
35	13.56	1.00	-24.820	1.378	0.189	616.039	1.898	0.036	35.7	41.5	11.5
36	13.64	1.00	7.522	-0.760	-0.013	56.579	0.578	0.000	36.6	41.5	11.5
37	14.36	1.00	-6.663	0.502	0.119	44.402	0.252	0.014	37.3	41.5	11.5

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			Modal Masses			Cumulative Mass, %		
			X	Y	Z	X	Y	Z	X	Y	Z
38	14.70	1.00	0.224	-0.038	-0.005	0.050	0.001	0.000	37.3	41.5	11.5
39	14.85	1.00	-5.404	-3.721	-0.251	29.201	13.848	0.063	37.8	41.7	11.5
40	14.97	1.00	1.270	1.122	-0.161	1.612	1.258	0.026	37.8	41.7	11.5
41	15.10	1.00	9.520	8.010	-0.479	90.638	64.159	0.229	39.3	42.8	11.5
42	15.47	1.00	2.667	1.882	-0.189	7.113	3.543	0.036	39.4	42.8	11.5
43	16.15	1.00	3.294	0.340	0.267	10.849	0.116	0.071	39.6	42.8	11.5
44	16.64	1.00	-0.426	0.123	0.930	0.181	0.015	0.864	39.6	42.8	11.6
45	17.13	1.00	11.296	-4.776	-0.354	127.609	22.806	0.125	41.7	43.2	11.6
46	17.56	1.00	-0.365	-0.168	0.000	0.133	0.028	0.000	41.7	43.2	11.6
47	17.90	1.00	-0.837	-0.761	-20.813	0.700	0.578	433.176	41.7	43.2	18.5
48	18.05	1.00	0.134	-0.096	0.226	0.018	0.009	0.051	41.7	43.2	18.5
49	19.34	1.00	-0.299	10.249	20.733	0.089	105.039	429.842	41.7	44.9	25.5
50	19.70	1.00	-1.120	8.840	-17.412	1.255	78.151	303.192	41.7	46.2	30.4
51	20.08	1.00	0.304	-0.160	0.083	0.093	0.026	0.007	41.7	46.2	30.4
52	20.16	1.00	0.039	0.873	-0.066	0.002	0.762	0.004	41.7	46.2	30.4
53	20.37	1.00	15.043	0.634	1.716	226.292	0.402	2.944	45.3	46.2	30.4
54	20.40	1.00	-0.343	0.066	-0.045	0.118	0.004	0.002	45.3	46.2	30.4
55	20.94	1.00	-0.016	0.051	-1.534	0.000	0.003	2.354	45.3	46.2	30.4
56	21.75	1.00	6.740	-1.987	0.788	45.432	3.946	0.622	46.1	46.2	30.5
57	21.88	1.00	1.364	6.208	-6.499	1.860	38.533	42.238	46.1	46.9	31.1
58	22.04	1.00	0.097	1.575	0.234	0.009	2.482	0.055	46.1	46.9	31.1
59	22.38	1.00	-2.106	1.082	-11.473	4.433	1.170	131.628	46.2	46.9	33.3
60	23.18	1.00	-2.361	-1.229	25.422	5.573	1.510	646.287	46.3	46.9	43.7
61	23.32	1.00	-0.236	-1.297	2.118	0.056	1.681	4.487	46.3	47.0	43.8
62	23.91	1.00	-1.754	-0.118	0.407	3.077	0.014	0.166	46.3	47.0	43.8
63	24.33	1.00	0.581	-1.213	-2.539	0.337	1.471	6.446	46.3	47.0	43.9
64	24.64	1.00	-0.255	0.305	-0.296	0.065	0.093	0.087	46.3	47.0	43.9
65	25.23	1.00	-0.105	0.070	0.465	0.011	0.005	0.216	46.3	47.0	43.9
66	28.44	1.00	2.161	-11.286	4.485	4.669	127.377	20.117	46.4	49.0	44.2
67	28.66	1.00	11.364	2.233	-1.151	129.142	4.987	1.325	48.5	49.1	44.2
68	29.52	1.00	-1.003	0.637	7.392	1.007	0.406	54.646	48.5	49.1	45.1
69	31.87	1.00	-1.523	0.486	6.848	2.320	0.236	46.889	48.5	49.1	45.9
70	32.07	1.00	-3.242	-0.312	-1.188	10.512	0.098	1.411	48.7	49.1	45.9
				SUMMATIONS		3019.276	3045.907	2844.838			
				TOTAL MASS		6198.823	6198.213	6201.359			

Note:

1. Fixed at Elevation 66.5′, restrained from horizontal translations at Elevations 82.5′ and 100′.

Table 3.7.2-5 (Sheet 1 of 4)

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	
306.25	1.44		1.47		0.90	
297.08	1.32		1.27		0.90	
284.42	1.20		0.98		0.89	
272.42	1.09		0.94		0.88	
241.00	0.82		0.78		0.55	
220.00	0.73	(0.75)	0.69	(0.73)	0.53	(0.65)
200.00	0.63	(0.64)	0.67	(0.69)	0.49	(0.63)
180.20	0.51	(0.51)	0.60	(0.63)	0.45	(0.59)
161.50	0.44	(0.45)	0.54	(0.56)	0.42	(0.53)
153.50	0.42	(0.43)	0.51	(0.55)	0.40	(0.50)
135.25	0.38	(0.40)	0.41	(0.45)	0.37	(0.45)
117.50	0.34	(0.35)	0.34	(0.37)	0.35	(0.40)
100.00	0.30	(0.30)	0.30	(0.30)	0.32	(0.35)
82.50	0.30	(0.30)	0.30	(0.30)	0.30	(0.32)
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 4)

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	
306.25	1.50		1.42		0.95	
297.08	1.34		1.33		0.95	
284.42	1.11		1.22		0.94	
272.42	1.06		1.10		0.93	
241.00	0.85		0.84		0.50	
220.00	0.72	(0.76)	0.77	(0.79)	0.48	(0.61)
200.00	0.62	(0.65)	0.69	(0.70)	0.46	(0.58)
180.20	0.51	(0.55)	0.58	(0.62)	0.40	(0.53)
161.50	0.42	(0.46)	0.51	(0.53)	0.37	(0.50)
153.50	0.40	(0.43)	0.48	(0.50)	0.36	(0.48)
135.25	0.36	(0.37)	0.40	(0.42)	0.34	(0.45)
117.50	0.34	(0.35)	0.34	(0.36)	0.34	(0.42)
100.00	0.32	(0.32)	0.32	(0.32)	0.33	(0.38)
82.50	0.31	(0.31)	0.31	(0.31)	0.32	(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 4)

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

SOFT-TO-MEDIUM SOIL CONDITION, 1.0 X G_{MAX}

Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g)					
	N-S Direction		E-W Direction		Vertical Direction	
306.25	0.86		1.14		0.88	
297.08	0.83		1.08		0.88	
284.42	0.77		1.00		0.87	
272.42	0.72		0.94		0.87	
241.00	0.68		0.76		0.52	
220.00	0.56	(0.61)	0.64	(0.67)	0.51	(0.61)
200.00	0.51	(0.54)	0.54	(0.56)	0.49	(0.60)
180.20	0.44	(0.48)	0.47	(0.49)	0.45	(0.58)
161.50	0.39	(0.43)	0.44	(0.45)	0.42	(0.56)
153.50	0.37	(0.41)	0.41	(0.43)	0.41	(0.50)
135.25	0.34	(0.38)	0.38	(0.39)	0.40	(0.50)
117.50	0.32	(0.35)	0.34	(0.35)	0.39	(0.48)
100.00	0.30	(0.32)	0.31	(0.31)	0.38	(0.46)
82.50	0.28	(0.30)	0.29	(0.29)	0.38	(0.44)
66.50	0.27		0.28		0.38	

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 4 of 4)

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

SOFT-TO-MEDIUM SOIL CONDITION, 2.0 X G_{MAX}

Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g)					
	N-S Direction		E-W Direction		Vertical Direction	
306.25	1.18		1.38		1.02	
297.08	1.10		1.32		1.02	
284.42	1.04		1.22		1.02	
272.42	0.98		1.14		1.01	
241.00	0.80		0.89		0.54	
220.00	0.68	(0.73)	0.76	(0.78)	0.51	(0.60)
200.00	0.59	(0.64)	0.64	(0.67)	0.48	(0.57)
180.20	0.50	(0.56)	0.55	(0.58)	0.43	(0.53)
161.50	0.45	(0.49)	0.48	(0.49)	0.39	(0.50)
153.50	0.43	(0.47)	0.44	(0.68)	0.38	(0.48)
135.25	0.39	(0.42)	0.38	(0.41)	0.36	(0.45)
117.50	0.36	(0.39)	0.34	(0.36)	0.35	(0.42)
100.00	0.35	(0.35)	0.31	(0.32)	0.33	(0.38)
82.50	0.33	(0.34)	0.30	(0.30)	0.33	(0.36)
66.50	0.32		0.29		0.33	

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 4)

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	0.94		1.21		1.49	
248.33	0.90		1.17		1.20	
240.33	0.87	(0.88)	1.13	(1.14)	1.04	(1.15)
229.52	0.83		1.07		0.84	
218.71	0.78		1.01		0.77	
205.33	0.72	(0.73)	0.93	(0.94)	0.75	(0.85)
205.33 (Polar Crane)	1.82		1.09		1.14	
190.00	0.65		0.82		0.70	
170.00	0.56		0.68		0.64	
162.00	0.51	(0.52)	0.62	(0.63)	0.60	(0.68)
144.50	0.41		0.48		0.53	
138.58	0.38		0.44		0.50	
132.25	0.36		0.39		0.48	
116.86	0.33	(0.33)	0.34	(0.34)	0.41	(0.46)
112.50	0.32		0.33		0.39	
110.50	0.32		0.33		0.36	
104.13	0.31		0.31		0.36	
100.00	0.30		0.30		0.31	

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 2 of 4)

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	0.83		1.06		0.83	
248.33	0.79		1.02		0.69	
240.33	0.75	(0.78)	0.98	(0.99)	0.61	(0.69)
229.52	0.70		0.93		0.51	
218.71	0.64		0.87		0.48	
205.33	0.58	(0.59)	0.79	(0.80)	0.46	(0.55)
205.33 (Polar Crane)	1.27		1.15		1.07	
190.00	0.50		0.69		0.44	
170.00	0.45		0.55		0.41	
162.00	0.43	(0.43)	0.51	(0.52)	0.40	(0.48)
144.50	0.40		0.44		0.38	
138.58	0.38		0.41		0.37	
132.25	0.37		0.39		0.37	
116.86	0.34	(0.35)	0.33	(0.34)	0.36	(0.42)
112.50	0.33		0.32		0.36	
110.50	0.33		0.32		0.35	
104.13	0.32		0.32		0.35	
100.00	0.32		0.32		0.34	

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 3 of 4)

MAXIMUM ABSOLUTE NODAL ACCELERATION (ZPA)
STEEL CONTAINMENT VESSEL

SOFT-TO-MEDIUM SOIL CONDITION, 1.0 XG_{MAX}

Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g)					
	N-S Direction		E-W Direction		Vertical Direction	
256.33	0.47		0.66		0.75	
248.33	0.46		0.64		0.63	
240.33	0.45	(0.45)	0.62	(0.62)	0.56	(0.61)
229.52	0.44		0.59		0.48	
218.71	0.43		0.57		0.47	
205.33	0.41	(0.42)	0.54	(0.55)	0.47	(0.56)
205.33 (Polar Crane)	0.72		1.33		1.26	
190.00	0.39		0.51		0.46	
170.00	0.37		0.46		0.45	
162.00	0.36	(0.36)	0.44	(0.45)	0.45	(0.54)
144.50	0.34		0.40		0.44	
138.58	0.34		0.39		0.44	
132.25	0.33		0.37		0.43	
116.86	0.31	(0.33)	0.34	(0.34)	0.42	(0.49)
112.50	0.31		0.33		0.42	
110.50	0.31		0.32		0.42	
104.13	0.31		0.31		0.42	
100.00	0.31		0.30		0.40	

<u>Note:</u>
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 4 of 4)

MAXIMUM ABSOLUTE NODAL ACCELERATION (ZPA)
STEEL CONTAINMENT VESSEL

SOFT-TO-MEDIUM SOIL CONDITION, 2.0 XG_{MAX}

Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g)					
	N-S Direction		E-W Direction		Vertical Direction	
256.33	0.58		0.86		0.80	
248.33	0.56		0.84		0.59	
240.33	0.54	(0.56)	0.81	(0.82)	0.53	(0.65)
229.52	0.50		0.77		0.45	
218.71	0.49		0.73		0.42	
205.33	0.49	(0.50)	0.69	(0.69)	0.42	(0.51)
205.33 (Polar Crane)	1.03		1.26		1.16	
190.00	0.58		0.63		0.40	
170.00	0.47		0.55		0.38	
162.00	0.45	(0.46)	0.52	(0.52)	0.38	(0.45)
144.50	0.42		0.44		0.37	
138.58	0.41		0.42		0.37	
132.25	0.40		0.39		0.36	
116.86	0.37	(0.38)	0.33	(0.33)	0.35	(0.42)
112.50	0.37		0.33		0.35	
110.50	0.36		0.32		0.35	
104.13	0.35		0.32		0.35	
100.00	0.35		0.31		0.34	

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 3.7.2-7 (Sheet 1 of 4)

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)	0.79		0.65		0.30	
148.00 (SG-West Compartment)	0.73		0.58		0.31	
148.00 (SG-East Compartment)	0.69		0.54		0.32	
135.25	0.61	(0.73)	0.52	(0.71)	0.30	(0.34)
107.17	0.32	(0.32)	0.30	(0.31)	0.30	(0.32)
103.00	0.31		0.30		0.30	
98.10	0.30		0.30		0.30	
87.50	0.30		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 4)

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)	0.43		0.48		0.35	
148.00 (SG-West Compartment)	0.40		0.46		0.35	
148.00 (SG-East Compartment)	0.41		0.46		0.31	
135.25	0.37	(0.42)	0.42	(0.48)	0.34	(0.36)
107.17	0.32	(0.32)	0.32	(0.32)	0.33	(0.36)
103.00	0.31		0.32		0.33	
98.10	0.31		0.32		0.33	
87.50	0.31		0.31		0.33	
82.50	0.31		0.31		0.33	

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 4)

MAXIMUM ABSOLUTE NODAL ACCELERATION (ZPA)
CONTAINMENT INTERNAL STRUCTURE

SOFT-TO-MEDIUM SOIL CONDITION, 1.0 X G_{MAX}

Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g)					
	N-S Direction		E-W Direction		Vertical Direction	
158.00 (PRZ Compartment)	0.34		0.43		0.42	
148.00 (SG-West Compartment)	0.33		0.41		0.42	
148.00 (SG-East Compartment)	0.32		0.40		0.38	
135.25	0.32	(0.33)	0.39	(0.43)	0.41	(0.44)
107.17	0.30	(0.31)	0.31	(0.31)	0.40	(0.43)
103.00	0.29		0.31		0.40	
98.10	0.29		0.30		0.40	
87.50	0.29		0.29		0.39	
82.50	0.28		0.29		0.39	

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 4 of 4)

MAXIMUM ABSOLUTE NODAL ACCELERATION (ZPA)
CONTAINMENT INTERNAL STRUCTURE

SOFT-TO-MEDIUM SOIL CONDITION, 2.0 X G_{MAX}

Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g)					
	N-S Direction		E-W Direction		Vertical Direction	
158.00 (PRZ Compartment)	0.39		0.42		0.37	
148.00 (SG-West Compartment)	0.38		0.39		0.37	
148.00 (SG-East Compartment)	0.37		0.38		0.32	
135.25	0.36	(0.38)	0.36	(0.42)	0.35	(0.39)
107.17	0.34	(0.35)	0.31	(0.31)	0.34	(0.37)
103.00	0.34		0.31		0.34	
98.10	0.34		0.31		0.34	
87.5	0.34		0.30		0.34	
82.50	0.33		0.30		0.34	

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 4)

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

HARD ROCK SITE CONDITION

Elevation (ft)	Maximum Relative Displacement (in.) N-S Direction		E-W Direction		Vertical Direction	
306.25	0.54		0.64		0.19	
297.08	0.50		0.61		0.19	
284.42	0.46		0.56		0.19	
272.42	0.42		0.51		0.19	
241.00	0.33		0.40		0.04	
220.00	0.26	(0.28)	0.32	(0.34)	0.04	(0.15)
200.00	0.19	(0.21)	0.25	(0.27)	0.04	(0.12)
180.20	0.13	(0.15)	0.17	(0.20)	0.02	(0.10)
161.50	0.09	(0.10)	0.12	(0.14)	0.01	(0.08)
153.50	0.07	(0.09)	0.11	(0.12)	0.01	(0.07)
135.25	0.04	(0.05)	0.06	(0.08)	0.01	(0.06)
117.50	0.02	(0.03)	0.03	(0.04)	0.01	(0.04)
100.00	0.	(0.)	0.	(0.)	0.	(0.02)
82.50	0.	(0.)	0.	(0.)	0.	(0.01)
66.50	0.		0.		0.	

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

Table 3.7.2-8 (Sheet 2 of 4)

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

SOFT ROCK SITE CONDITION

Elevation (ft)	Maximum Relative Displacement (in.) N-S Direction		E-W Direction		Vertical Direction	
306.25	0.63		0.82		0.22	
297.08	0.60		0.79		0.23	
284.42	0.57		0.73		0.24	
272.42	0.50		0.67		0.23	
241.00	0.42		0.51		0.06	
220.00	0.32	(0.37)	0.41	(0.44)	0.08	(0.21)
200.00	0.26	(0.28)	0.32	(0.36)	0.05	(0.17)
180.20	0.18	(0.21)	0.23	(0.26)	0.03	(0.15)
161.50	0.13	(0.16)	0.18	(0.20)	0.02	(0.14)
153.50	0.12	(0.12)	0.16	(0.18)	0.04	(0.12)
135.25	0.08	(0.08)	0.12	(0.12)	0.02	(0.08)
117.50	0.05	(0.07)	0.07	(0.08)	0.02	(0.09)
100.00	0.02	(0.05)	0.04	(0.07)	0.03	(0.06)
82.50	0.02	(0.05)	0.01	(0.05)	0.03	(0.09)
66.50	0.		0.		0.	

<u>Note</u>:
1. Enveloped relative displacements at the north, south, east and west edge nodes of the structure are shown in parentheses. This is the maximum value of the relative displacement at any of these edge nodes.

Table 3.7.2-8 (Sheet 3 of 4)

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

SOFT-TO-MEDIUM SOIL CONDITION, 1.0 X G_{MAX}

Elevation (ft)	Maximum Relative Displacement (in.) N-S Direction		E-W Direction		Vertical Direction	
306.25	0.51		0.89		0.24	
297.08	0.50		0.85		0.23	
284.42	0.46		0.78		0.24	
272.42	0.44		0.72		0.21	
241.00	0.34		0.58		0.09	
220.00	0.29	(0.34)	0.49	(0.51)	0.07	(0.25)
200.00	0.24	(0.25)	0.39	(0.42)	0.05	(0.25)
180.20	0.20	(0.25)	0.30	(0.33)	0.07	(0.19)
161.50	0.13	(0.17)	0.23	(0.25)	0.07	(0.19)
153.50	0.11	(0.14)	0.21	(0.23)	0.03	(0.18)
135.25	0.11	(0.12)	0.15	(0.16)	0.04	(0.16)
117.50	0.06	(0.10)	0.10	(0.11)	0.03	(0.14)
100.00	0.08	(0.07)	0.05	(0.07)	0.02	(0.13)
82.50	0.04	(0.04)	0.02	(0.04)	0.07	(0.11)
66.50	0.		0.		0.	

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

Table 3.7.2-8 (Sheet 4 of 4)

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

SOFT-TO-MEDIUM SOIL CONDITION, 2.0 X G_{MAX}

Elevation (ft)	Maximum Relative Displacement (in.) N-S Direction		E-W Direction		Vertical Direction	
306.25	0.64		0.95		0.24	
297.08	0.61		0.91		0.26	
284.42	0.54		0.85		0.23	
272.42	0.49		0.76		0.24	
241.00	0.38		0.63		0.06	
220.00	0.35	(0.35)	0.49	(0.51)	0.06	(0.24)
200.00	0.26	(0.26)	0.38	(0.42)	0.06	(0.22)
180.20	0.20	(0.21)	0.28	(0.33)	0.03	(0.19)
161.50	0.14	(0.17)	0.22	(0.25)	0.05	(0.16)
153.50	0.13	(0.15)	0.19	(0.25)	0.03	(0.16)
135.25	0.09	(0.11)	0.13	(0.15)	0.05	(0.14)
117.50	0.07	(0.07)	0.10	(0.10)	0.02	(0.12)
100.00	0.05	(0.04)	0.05	(0.05)	0.03	(0.10)
82.50	0.02	(0.06)	0.03	(0.05)	0.01	(0.07)
66.50	0.		0.		0.	

Note:

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

Table 3.7.2-9 (Sheet 1 of 4)

MAXIMUM DISPLACEMENT RELATIVE TO TOP OF BASEMAT
STEEL CONTAINMENT VESSEL

HARD ROCK SITE CONDITION

Elevation (ft)	Maximum Relative Displacement (in.) N-S Direction		E-W Direction		Vertical Direction	
256.33	0.21		0.22		0.05	
248.33	0.20		0.22		0.04	
240.33	0.19	(0.19)	0.21	(0.21)	0.04	(0.06)
229.52	0.18		0.20		0.03	
218.71	0.17		0.18		0.03	
205.33	0.15	(0.15)	0.17	(0.17)	0.02	(0.05)
205.33 (Polar Crane)	0.59		2.20		0.54	
190.00	0.13		0.14		0.02	
170.00	0.10		0.11		0.02	
162.00	0.09	(0.09)	0.10	(0.10)	0.02	(0.05)
144.50	0.06		0.07		0.01	
138.58	0.05		0.06		0.04	
132.25	0.04		0.05		0.01	
116.86	0.02	(0.02)	0.02	(0.02)	0.01	(0.03)
112.50	0.02		0.02		0.01	
110.50	0.01		0.01		0.02	
104.13	0.01		0.01		0.01	
100.00	0.		0.		0.01	

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

Table 3.7.2-9 (Sheet 2 of 4)

MAXIMUM DISPLACEMENT RELATIVE TO TOP OF BASEMAT
STEEL CONTAINMENT VESSEL

SOFT ROCK SITE CONDITION

Elevation (ft)	Maximum Relative Displacement (in.)					
	N-S Direction		E-W Direction		Vertical Direction	
256.33	0.19		0.29		0.04	
248.33	0.23		0.27		0.06	
240.33	0.18	(0.23)	0.28	(0.28)	0.03	(0.11)
229.52	0.19		0.25		0.03	
218.71	0.19		0.22		0.02	
205.33	0.18	(0.19)	0.20	(0.22)	0.03	(0.09)
205.33 (Polar Crane)	0.54		2.38		0.52	
190.00	0.15		0.20		0.03	
170.00	0.11		0.14		0.03	
162.00	0.11	(0.14)	0.13	(0.15)	0.04	(0.09)
144.50	0.08		0.11		0.03	
138.58	0.08		0.09		0.04	
132.25	0.07		0.09		0.06	
116.86	0.06	(0.06)	0.06	(0.06)	0.02	(0.06)
112.50	0.05		0.05		0.05	
110.50	0.05		0.05		0.07	
104.13	0.05		0.03		0.02	
100.00	0.02		0.03		0.01	

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

Table 3.7.2-9 (Sheet 3 of 4)

MAXIMUM DISPLACEMENT RELATIVE TO TOP OF BASEMAT STEEL CONTAINMENT VESSEL

SOFT-TO-MEDIUM SOIL CONDITION, 1.0 X G_{MAX}

Elevation (ft)	Maximum Relative Displacement (in.)					
	N-S Direction		E-W Direction		Vertical Direction	
256.33	0.23		0.39		0.08	
248.33	0.22		0.37		0.08	
240.33	0.23	(0.23)	0.36	(0.36)	0.03	(0.15)
229.52	0.21		0.34		0.14	
218.71	0.19		0.32		0.04	
205.33	0.19	(0.19)	0.29	(0.30)	0.04	(0.14)
205.33 (Polar Crane)	0.38		2.53		0.61	
190.00	0.15		0.26		0.04	
170.00	0.14		0.21		0.12	
162.00	0.14	(0.14)	0.19	(0.19)	0.06	(0.14)
144.50	0.11		0.15		0.09	
138.58	0.10		0.13		0.06	
132.25	0.09		0.12		0.03	
116.86	0.05	(0.08)	0.08	(0.09)	0.05	(0.12)
112.50	0.07		0.07		0.05	
110.50	0.05		0.07		0.05	
104.13	0.07		0.06		0.10	
100.00	0.05		0.05		0.09	

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

Table 3.7.2-9 (Sheet 4 of 4)

MAXIMUM DISPLACEMENT RELATIVE TO TOP OF BASEMAT
STEEL CONTAINMENT VESSEL

SOFT-TO-MEDIUM SOIL CONDITION, 2.0 X G_{MAX}

Elevation (ft)	**Maximum Relative Displacement (in.)** N-S Direction		E-W Direction		Vertical Direction	
256.33	0.21		0.36		0.06	
248.33	0.22		0.32		0.08	
240.33	0.22	(0.24)	0.31	(0.31)	0.04	(0.13)
229.52	0.21		0.28		0.02	
218.71	0.20		0.26		0.03	
205.33	0.21	(0.21)	0.25	(0.26)	0.05	(0.13)
205.33 (Polar Crane)	0.50		2.55		0.56	
190.00	0.27		0.22		0.02	
170.00	0.13		0.18		0.02	
162.00	0.12	(0.13)	0.18	(0.18)	0.02	(0.11)
144.50	0.09		0.14		0.02	
138.58	0.10		0.11		0.05	
132.25	0.07		0.10		0.02	
116.86	0.06	(0.07)	0.07	(0.07)	0.06	(0.10)
112.50	0.05		0.06		0.05	
110.50	0.07		0.06		0.03	
104.13	0.03		0.04		0.03	
100.00	0.03		0.05		0.06	

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

Table 3.7.2-10 (Sheet 1 of 4)

MAXIMUM DISPLACEMENT RELATIVE TO TOP OF BASEMAT CONTAINMENT INTERNAL STRUCTURE

HARD ROCK SITE CONDITION

Elevation (ft)	Maximum Relative Displacement (in.)					
	N-S Direction		E-W Direction		Vertical Direction	
158.00 (PRZ Compartment)	0.04		0.05		0.01	
148.00 (SG-West Compartment	0.04		0.04		0.01	
148.00 (SG-East Compartment)	0.02		0.04		0.	
135.25	0.03	(0.04)	0.03	(0.05)	0.	(0.01)
107.17	0.	0.	0.	0.	0.	(0.01)
103.00	0.		0.		0.	
98.10	0.		0.		0.	
87.50	0.		0.		0.	
82.50	0.		0.		0.	

<u>Note</u>:
1. Enveloped relative displacements at the north, south, east and west edge nodes of the structure are shown in parentheses. This is the maximum value of the relative displacement at any of these edge nodes.

Table 3.7.2-10 (Sheet 2 of 4)

MAXIMUM DISPLACEMENT RELATIVE TO TOP OF BASEMAT CONTAINMENT INTERNAL STRUCTURE

SOFT ROCK SITE CONDITION

Elevation (ft)	Maximum Relative Displacement (in.)					
	N-S Direction		E-W Direction		Vertical Direction	
158.00 (PRZ Compartment)	0.06		0.09		0.03	
148.00 (SG-West Compartment)	0.06		0.09		0.02	
148.00 (SG-East Compartment)	0.06		0.09		0.03	
135.25	0.04	(0.06)	0.07	(0.09)	0.02	(0.04)
107.17	0.05	(0.05)	0.04	(0.07)	0.04	(0.09)
103.00	0.02		0.03		0.01	
98.10	0.04		0.03		0.01	
87.50	0.02		0.02		0.06	
82.50	0.04		0.04		0.02	

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

Table 3.7.2-10 (Sheet 3 of 4)

MAXIMUM DISPLACEMENT RELATIVE TO TOP OF BASEMAT CONTAINMENT INTERNAL STRUCTURE

SOFT-TO-MEDIUM SOIL CONDITION, 1.0 X G_{MAX}

Elevation (ft)	Maximum Relative Displacement (in.) N-S Direction		E-W Direction		Vertical Direction	
158.00 (PRZ Compartment)	0.15		0.15		0.06	
148.00 (SG-West Compartment)	0.10		0.13		0.06	
148.00 (SG-East Compartment)	0.09		0.13		0.08	
135.25	0.07	(0.11)	0.12	(0.12)	0.05	(0.10)
107.17	0.06	(0.07)	0.06	(0.06)	0.03	(0.10)
103.00	0.04		0.06		0.09	
98.10	0.08		0.04		0.05	
87.5	0.04		0.03		0.05	
82.50	0.04		0.04		0.02	

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

Table 3.7.2-10 (Sheet 4 of 4)

MAXIMUM DISPLACEMENT RELATIVE TO TOP OF BASEMAT CONTAINMENT INTERNAL STRUCTURE

SOFT-TO-MEDIUM SOIL CONDITION, 2.0 X G_{MAX}

Elevation (ft)	Maximum Relative Displacement (in.)					
	N-S Direction		E-W Direction		Vertical Direction	
158.00 (PRZ Compartment)	0.08		0.12		0.04	
148.00 (SG-West Compartment)	0.07		0.12		0.04	
148.00 (SG-East Compartment)	0.05		0.10		0.04	
135.25	0.06	(0.06)	0.10	(0.10)	0.06	(0.07)
107.17	0.03	(0.04)	0.06	(0.06)	0.05	(0.06)
103.00	0.06		0.04		0.02	
98.10	0.02		0.04		0.05	
87.5	0.03		0.02		0.04	
82.50	0.02		0.02		0.04	

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

Table 3.7.2-11 (Sheet 1 of 4)

MAXIMUM MEMBER FORCES AND MOMENTS
COUPLED AUXILIARY & SHIELD BUILDINGS

HARD ROCK SITE CONDITION

Elevation (ft)	Maximum Forces (x10³ Kips)				Maximum Moment (x10³ K-ft)	
	Axial	N-S Shear	E-W Shear	Torque	about N-S Axis	about E-W Axis
306.25					18.20	18.20
	1.45	2.46	2.43	3.88		
297.08					59.80	59.10
	3.40	4.47	4.36	9.17		
284.42					181.90	178.70
	7.65	8.30	7.67	25.50		
272.42					274.00	266.00
	11.54	12.52	10.57	46.08		
241.00					747.50	746.00
	15.44	16.43	15.68	81.10		
220.00					1072.00	1109.00
	18.05	18.72	18.32	109.50		
200.00					1402.00	1488.00
	20.43	20.68	20.32	134.50		
180.20					1835.00	2140.00
	23.40	23.28	23.03	923.80		
161.50					2243.00	2483.00
	25.45	25.51	25.17	911.40		
153.50					2389.00	2482.00
	28.14	28.82	28.40	716.10		
135.25					2896.00	2972.00
	31.92	34.03	33.57	1157.00		
117.50					3539.00	3417.00
	34.96	37.54	37.59	1396.00		
100.00					4188.00	4045.00

Table 3.7.2-11 (Sheet 2 of 4)

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
306.25					13.40	15.30
	1.59	2.53	2.37	3.50		
297.08					46.00	55.00
	3.71	4.54	4.41	8.26		
284.42					142.00	170.00
	8.34	8.14	8.57	23.00		
272.42					258.00	303.00
	12.60	12.00	13.10	41.40		
241.00					724.00	771.00
	15.40	17.00	18.20	75.50		
220.00					1150.00	1130.00
	17.10	20.10	20.60	98.40		
200.00					1590.00	1500.00
	18.80	22.60	22.30	126.00		
180.20					2130.00	2040.00
	21.30	25.80	25.30	971.00		
161.50					2650.00	2360.00
	23.30	28.00	27.60	910.00		
153.50					2870.00	2650.00
	26.10	30.80	31.00	754.00		
135.25					3430.00	3290.00
	31.20	35.70	36.70	1160.00		
117.50					3950.00	4000.00
	36.80	40.00	42.20	1330.00		
100.00					4460.00	4680.00

Table 3.7.2-11 (Sheet 3 of 4)

MAXIMUM MEMBER FORCES AND MOMENTS
COUPLED AUXILIARY & SHIELD BUILDINGS

SOFT-TO-MEDIUM SOIL CONDITION, 1.0 X G_{MAX}

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
306.25					6.74	5.81
	1.47	1.44	1.89	1.88		
297.08					28.60	23.90
	3.44	2.70	3.57	4.45		
284.42					92.90	76.40
	7.73	5.32	6.98	12.40		
272.42					174.00	137.00
	11.60	8.27	10.90	22.60		
241.00					574.00	433.00
	15.00	12.10	15.60	41.10		
220.00					932.00	698.00
	17.00	14.50	18.30	56.20		
200.00					1320.00	999.00
	19.00	16.40	20.50	74.80		
180.20					1720.00	1840.00
	21.60	19.10	23.10	873.00		
161.50					2110.00	2190.00
	23.80	21.10	25.00	662.00		
153.50					2330.00	2050.00
	27.00	24.00	27.70	463.00		
135.25					2850.00	2750.00
	32.90	29.00	31.20	841.00		
117.50					3470.00	3420.00
	39.40	34.50	37.30	1140.00		
100.00					4120.00	4020.00

Table 3.7.2-11 (Sheet 4 of 4)

MAXIMUM MEMBER FORCES AND MOMENTS
COUPLED AUXILIARY & SHIELD BUILDINGS

SOFT-TO-MEDIUM SOIL CONDITION, 2.0 X G_{MAX}

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
306.25					10.2	10.60
	1.71	1.99	2.33	2.37		
297.08					36.00	39.50
	3.99	3.60	4.35	5.58		
284.42					119.00	123.00
	8.79	7.06	8.44	15.50		
272.42					213.00	223.00
	13.60	11.30	13.10	28.30		
241.00					713.00	610.00
	17.10	16.10	18.50	52.20		
220.00					1140.00	920.00
	19.20	18.90	21.80	73.50		
200.00					1600.00	1320.00
	21.00	20.90	23.90	96.80		
180.20					2170.00	2120.00
	23.60	23.60	26.80	1010.00		
161.50					2770.00	2300.00
	25.60	25.90	29.00	841.00		
153.50					3030.00	2530.00
	28.40	29.20	32.00	695.00		
135.25					3670.00	3280.00
	33.90	34.90	36.60	1100.00		
117.50					4300.00	4030.00
	40.00	41.20	41.60	1330.00		
100.00					4980.00	4780.00

Table 3.7.2-12 (Sheet 1 of 4)

MAXIMUM MEMBER FORCES AND MOMENTS
STEEL CONTAINMENT VESSEL

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**
256.33					0.00	0.00
	0.28	0.17	0.22	0.00		
248.33					2.73	2.34
	0.61	0.46	0.59	0.16		
240.33					9.36	7.79
	0.99	0.77	0.99	0.47		
229.52					22.70	18.44
	1.33	1.06	1.37	0.90		
218.71					40.57	32.52
	1.66	1.36	1.76	1.37		
205.33					70.98	58.59
	2.79	2.60	2.78	10.26		
190.00					118.10	101.30
	3.23	3.00	3.29	10.59		
170.00					187.40	162.80
	3.58	3.29	3.67	10.81		
162.00					219.60	191.10
	3.91	3.52	3.96	10.97		
144.50					291.40	254.30
	4.19	3.72	4.22	11.99		
138.58					316.40	276.40
	4.21	3.73	4.24	11.43		
132.25					345.30	302.10
	4.41	3.86	4.40	11.52		
116.86					413.10	362.40
	4.49	3.89	4.44	11.55		
112.50					433.10	380.00
	4.55	3.92	4.47	11.36		
110.50					442.30	387.90
	4.57	3.92	4.48	11.22		
104.13					471.20	413.80
	4.60	3.93	4.49	11.23		
100					489.70	429.50

Table 3.7.2-12 (Sheet 2 of 4)

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	0.15	0.19	0.00		
248.33					2.48	2.12
	0.37	0.40	0.51	0.18		
240.33					8.42	7.07
	0.58	0.66	0.85	0.52		
229.52					20.10	16.70
	0.77	0.91	1.19	1.00		
218.71					35.70	29.30
	0.95	1.15	1.52	1.55		
205.33					62.50	51.80
	1.89	2.04	2.43	8.96		
190.00					103.00	86.40
	2.16	2.35	2.87	9.51		
170.00					164.00	136.00
	2.39	2.58	3.21	9.77		
162.00					193.00	158.00
	2.62	2.71	3.49	10.10		
144.50					257.00	209.00
	2.85	2.91	3.80	11.20		
138.58					280.00	226.00
	2.86	2.93	3.82	11.00		
132.25					307.00	247.00
	3.05	3.13	4.23	11.00		
116.86					369.00	294.00
	3.14	3.22	4.08	11.30		
112.50					388.00	310.00
	3.23	3.32	3.89	11.20		
110.50					398.00	316.00
	3.23	3.34	3.80	11.90		
104.13					426.00	338.00
	3.29	3.40	3.92	11.70		
100.00					443.00	394.00

Table 3.7.2-12 (Sheet 3 of 4)

MAXIMUM MEMBER FORCES AND MOMENTS
STEEL CONTAINMENT VESSEL

SOFT-TO-MEDIUM SOIL CONDITION, 1.0 X G_{MAX}

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
	0.13	0.09	0.12	0.00		
248.33					1.65	0.93
	0.34	0.24	0.33	0.21		
240.33					5.55	3.27
	0.53	0.40	0.55	0.62		
229.52					13.10	8.19
	0.71	0.56	0.77	1.19		
218.71					23.30	14.90
	0.88	0.73	0.99	1.83		
205.33					40.70	27.30
	1.82	1.84	1.78	5.98		
190.00					69.00	50.30
	2.07	1.82	2.06	6.35		
170.00					111.00	85.10
	2.29	2.03	2.31	6.80		
162.00					130.00	120.00
	2.35	2.23	2.53	7.22		
144.50					176.00	145.00
	2.54	2.43	2.76	8.78		
138.58					194.00	162.00
	2.56	2.45	2.78	8.83		
132.25					214.00	178.00
	2.75	2.64	2.98	8.96		
116.86					260.00	218.00
	2.85	2.72	3.08	8.98		
112.50					274.00	230.00
	2.95	2.79	3.17	9.05		
110.50					282.00	236.00
	2.96	2.81	3.18	9.81		
104.13					303.00	254.00
	3.02	2.86	3.24	9.84		
100.00					316.00	265.00

Table 3.7.2-12 (Sheet 4 of 4)

MAXIMUM MEMBER FORCES AND MOMENTS
STEEL CONTAINMENT VESSEL

SOFT-TO-MEDIUM SOIL CONDITION, 2.0 X G_{MAX}

Elevation (ft)	Maximum Forces ($x10^3$ Kips) Axial	N-S Shear	E-W Shear	Maximum Moment ($x10^3$ K-ft) Torque	about N-S Axis	about E-W Axis
256.33					0.00	0.00
	0.14	0.10	0.16	0.00		
248.33					2.16	1.44
	0.33	0.29	0.43	0.18		
240.33					7.26	4.84
	0.51	0.48	0.71	0.53		
229.52					17.10	11.40
	0.69	0.68	0.99	1.02		
218.71					30.20	20.30
	0.85	0.88	1.27	1.56		
205.33					52.7	36.70
	1.71	1.93	2.01	7.70		
190.00					84.10	64.60
	1.96	2.26	2.40	8.23		
170.00					132.00	111.00
	2.17	2.51	2.70	8.58		
162.00					155.00	132.00
	2.38	2.72	2.95	8.85		
144.50					209.00	181.00
	2.59	2.98	3.20	10.00		
138.58					229.00	198.00
	2.60	3.00	3.22	10.30		
132.25					250.00	221.00
	2.79	3.21	3.43	10.40		
116.86					305.00	270.00
	2.88	3.29	3.52	10.40		
112.50					322.00	285.00
	2.97	3.37	3.59	10.70		
110.50					330.00	292.00
	2.99	3.37	3.60	11.70		
104.13					353.00	313.00
	3.04	3.43	3.64	11.10		
100.00					369.00	327.00

Table 3.7.2-13 (Sheet 1 of 4)

MAXIMUM MEMBER FORCES AND MOMENTS
CONTAINMENT INTERNAL STRUCTURES

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
Above Elevation 135.25′, West SG Compartment						
158.00					0.07	0.07
	0.05	0.16	0.15	0.25		
153.56					0.71	0.74
	0.05	0.29	0.28	0.25		
148.00					2.65	3.05
	0.24	0.81	0.76	6.69		
135.25					12.25	13.23
Above Elevation 135.25′, East SG Compartment						
148.00					0.56	0.16
	0.13	0.31	0.27	2.20		
135.25					3.79	4.10
Below Elevation 135.25′						
135.25					40.40	35.70
	1.99	5.73	5.98	245.70		
121.50					117.40	108.60
	1.99	5.83	6.07	247.50		
107.17					219.60	196.10
	4.07	7.02	6.90	321.90		
103.00					244.30	225.90

Table 3.7.2-13 (Sheet 2 of 4)

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	Maximum Moment ($x10^3$ K-ft) Torque	about N-S Axis	about E-W Axis
Above Elevation 135.25′, West SG Compartment						
158.00					0.06	0.06
	0.60	0.08	0.10	0.12		
153.56					0.45	0.35
	0.60	0.14	0.15	0.10		
148.00					1.29	1.54
	0.30	0.42	0.47	3.35		
135.25					7.05	6.94
Above Elevation 135.25′, East SG Compartment						
148.00					0.42	0.05
	0.13	0.17	0.18	0.76		
135.25					2.74	2.21
Below Elevation 135.25′						
135.25					29.70	21.60
	2.46	4.23	5.54	76.30		
121.50					83.10	74.00
	2.46	4.27	5.60	77.80		
107.17					169.00	135.00
	5.08	7.49	8.19	195.00		
103.00					209.00	157.00
	7.95	8.96	9.33	86.90		
100.00					237.00	183.00

Table 3.7.2-13 (Sheet 3 of 4)

MAXIMUM MEMBER FORCES AND MOMENTS
CONTAINMENT INTERNAL STRUCTURES

SOFT-TO-MEDIUM SOIL CONDITION, 1.0 X G_{MAX}

Elevation (ft)	Maximum Forces (x103 Kips) Axial	N-S Shear	E-W Shear	Torque	Maximum Moment (x103 K-ft) about N-S Axis	about E-W Axis
Above Elevation 135.25′, West SG Compartment						
158.00					0.08	0.08
	0.08	0.06	0.08	0.09		
153.56					0.38	0.29
	0.08	0.11	0.14	0.07		
148.00					1.29	1.68
	0.37	0.34	0.43	2.55		
135.25					6.75	5.97
Above Elevation 135.25′, East SG Compartment						
148.00					0.48	0.05
	0.15	0.13	0.17	0.55		
135.25					2.33	1.64
Below Elevation 135.25′						
135.25					34.60	24.60
	2.99	3.85	4.61	57.80		
121.50					84.30	76.20
	2.99	3.89	4.67	59.50		
107.17					175.00	133.00
	6.15	6.91	8.52	174.00		
103.00					214.00	160.00
	9.64	8.21	10.90	51.80		
100.00					244.00	183.00

Table 3.7.2-13 (Sheet 4 of 4)

MAXIMUM MEMBER FORCES AND MOMENTS
CONTAINMENT INTERNAL STRUCTURES

SOFT-TO-MEDIUM SOIL CONDITION, 2.0 X GMAX

Elevation (ft)	Maximum Forces (x103 Kips) Axial	N-S Shear	E-W Shear	Torque	Maximum Moment (x103 K-ft) about N-S Axis	about E-W Axis
Above Elevation 135.25', West SG Compartment						
158.00					0.06	0.07
	0.07	0.07	0.08	0.10		
153.56					0.39	0.31
	0.07	0.13	0.13	0.08		
148.00					1.18	1.62
	0.32	0.39	0.40	2.76		
135.25					5.99	6.40
Above Elevation 135.25', East SG Compartment						
148.00					0.43	0.05
	0.13	0.15	0.16	0.69		
135.25					2.38	1.89
Below Elevation 135.25'						
135.25					30.30	22.40
	2.59	4.58	4.91	53.70		
121.50					80.20	83.60
	2.59	4.63	4.97	55.00		
107.17					171.00	149.00
	5.30	8.06	7.63	178.00		
103.00					195.00	173.00
	8.26	9.56	9.01	55.80		
100.00					218.00	202.00

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 (Appendices 2A, 2B). To develop lateral earth pressures for wall design.
2D lumped mass stick, fixed base models	Mode superposition time history analysis	BSAP	To identify governing site properties and design soil profiles (Appendix 2A).
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, soft-to-medium soil, and upper bound 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).

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, fixed base models, coupled aux/shield buildings, with stick models of containment internal structures and steel containment vessel	Response spectrum analysis	BSAP	Performed for the hard rock profile with ground response spectra input at basemat. To obtain the in-plane forces[1] for the design of floors and walls of the auxiliary and shield building.
3D finite element, flexible base model, coupled aux/shield buildings, with stick models of containment internal structures and steel containment vessel	Response spectrum analysis	BSAP	Performed for soil profile with ground response spectra input at base of soil springs. To obtain the in-plane forces for the design of floors and walls of the auxiliary and shield building.
3D finite element, fixed base model of containment internal structures	Response spectrum analysis	BSAP	Performed for the hard rock profile with ground response spectra input at elevation 82′-6″. To obtain the in-plane forces[1] for the design of floors and walls of the containment internal structures.
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 stresses 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

Note:

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	Three 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	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	Double Sum
3D finite element, flexible base model, coupled Aux/Shield buildings	Response spectrum analysis	BSAP	SRSS	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 3.7.2-17 (Sheet 1 of 3)

COMPARISON OF MAXIMUM ABSOLUTE NODAL ACCELERATION (ZPA) DUE TO TIME HISTORY (TH) AND RESPONSE SPECTRUM (RSA) ANALYSES(1)(2)

COUPLED AUXILIARY & SHIELD BUILDINGS HARD ROCK CONDITION

Maximum Absolute Nodal Acceleration, ZPA (g)

Elevation	**N-S Direction**		**E-W Direction**		**Vertical Direction**	
(ft)	**TH**	**RSA**	**TH**	**RSA**	**TH**	**RSA**
306.25	1.44	1.43	1.47	1.49	0.9	0.88
297.08	1.32	1.31	1.27	1.39	0.9	0.88
284.42	1.2	1.16	0.98	1.25	0.89	0.87
272.42	1.09	1.09	0.94	1.16	0.88	0.86
241.00	0.82	0.85	0.78	0.97	0.55	0.75
220.00	0.73 (0.75)	0.75 (0.95)	0.69 (0.73)	0.89 (1.06)	0.53 (0.65)	0.70 (1.26)
200.00	0.63 (0.64)	0.69 (0.84)	0.67 (.069)	0.77 (0.93)	0.49 (0.63)	0.62 (1.10)
180.20	0.51 (0.51)	0.59 (0.71)	0.60 (0.63)	0.61 (0.72)	0.45 (0.59)	0.47 (0.93)
161.50	0.44 (0.45)	0.48 (0.58)	0.54 (0.56)	0.56 (0.62)	0.42 (0.53)	0.37 (0.75)
153.50	0.42 (0.43)	0.44 (0.53)	0.51 (0.55)	0.54 (0.67)	0.40 (0.50)	0.33 (0.67)
135.25	0.38 (0.40)	0.33 (0.41)	0.41 (0.45)	0.45 (0.57)	0.37 (0.45)	0.30 (0.51)
117.50	0.34 (0.35)	0.30 (0.30)	0.34 (0.37)	0.30 (0.37)	0.35 (0.40)	0.30 (0.36)
100.00	0.30 (0.30)	0.30 (0.30)	0.30 (0.30)	0.30 (0.30)	0.32 (0.35)	0.30 (0.30)
82.50	0.30 (0.30)	0.30 (0.30)	0.30 (0.30)	0.30 (0.30)	0.30 (0.32)	0.30 (0.30)
66.50	0.30	0.30	0.30	0.30	0.30	0.30

Notes:

1. Time history analyses consider vibration modes up to 33 Hertz. Response spectrum analyses combine vibration modes up to 33 Hertz by double sum method. (See subsection 3.7.2.7.)

2. 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 3.7.2-17 (Sheet 2 of 3)

COMPARISON OF MAXIMUM ABSOLUTE NODAL ACCELERATION (ZPA) DUE TO TIME HISTORY (TH) AND RESPONSE SPECTRUM (RSA) ANALYSES(1)(2)

STEEL CONTAINMENT VESSEL
HARD ROCK CONDITION

Maximum Absolute Nodal Acceleration, ZPA (g)

Elevation	N-S Direction		E-W Direction		Vertical Direction	
(ft)	TH	RSA	TH	RSA	TH	RSA
256.33	0.94	1.13	1.21	1.33	1.49	2.91
248.33	0.90	1.05	1.17	1.23	1.20	2.03
240.33	0.87 (0.88)	0.98 (0.99)	1.13 (1.14)	1.14 (1.16)	1.04 (1.15)	1.62 (1.96)
229.52	0.83	0.88	1.07	1.03	0.84	1.17
218.71	0.78	0.80	1.01	0.93	0.77	1.00
205.33	0.72 (0.73)	0.73 (0.74)	0.93 (0.94)	0.86 (0.87)	0.75 (0.85)	0.95 (1.30)
205.33 (Polar Crane)	1.82	1.54	1.09	0.74	1.14	1.04
190.00	0.65	0.69	0.82	0.82	0.70	0.85
170.00	0.56	0.66	0.68	0.79	0.64	0.72
162.00	0.51 (0.52)	0.64 (0.65)	0.62 (0.63)	0.76 (0.77)	0.60 (0.68)	0.66 (0.87)
144.50	0.41	0.54	0.48	0.64	0.53	0.66
138.58	0.38	0.49	0.44	0.58	0.50	0.6
132.25	0.36	0.42	0.39	0.51	0.48	0.4
116.86	0.33 (0.33)	0.30 (0.30)	0.34 (0.34)	0.30 (0.30)	0.41 (0.46)	0.30 (0.34)
112.50	0.32	0.30	0.33	0.30	0.39	0.30
110.50	0.32	0.30	0.33	0.30	0.36	0.30
104.13	0.31	0.30	0.31	0.30	0.36	0.30
100.00	0.30	0.30	0.30	0.30	0.31	0.30

Notes:
1. Time history analyses consider vibration modes up to 33 Hertz. Response spectrum analyses combine vibration modes up to 33 Hertz by double sum method. (See subsection 3.7.2.7.)

2. 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 3.7.2-17 (Sheet 3 of 3)

COMPARISON OF MAXIMUM ABSOLUTE NODAL ACCELERATION (ZPA) DUE TO TIME HISTORY (TH) AND RESPONSE SPECTRUM (RSA) ANALYSES(1)(2)

CONTAINMENT INTERNAL STRUCTURES HARD ROCK CONDITION

Maximum Absolute Nodal Acceleration, ZPA (g)

Elevation	**N-S Direction**		**E-W Direction**		**Vertical Direction**	
(ft)	**TH**	**RSA**	**TH**	**RSA**	**TH**	**RSA**
158.00 (PRZ Compartment)	0.79	0.84	0.65	0.82	0.30	0.30
148.00 (SG-West Compartment)	0.73	0.76	0.58	0.70	0.31	0.30
148.00 (SG-East Compartment)	0.69	0.77	0.54	0.67	0.32	0.30
135.25	0.61 (0.73)	0.53 (0.94)	0.52 (0.71)	0.57 (0.93)	0.30 (0.34)	0.30
107.17	0.32 (0.32)	0.30	0.30 (0.31)	0.30	0.30 (0.32)	0.30
103.00	0.31	0.30	0.30	0.30	0.30	0.30
98.10	0.30	0.30	0.30	0.30	0.30	0.30
87.5	0.30	0.30	0.30	0.30	0.30	0.30
82.50	0.30	0.30	0.30	0.30	0.30	0.30

Notes:

1. Time history analyses consider vibration modes up to 33 Hertz. Response spectrum analyses combine vibration modes up to 33 Hertz by double sum method. (See subsection 3.7.2.7.)

2. 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 3.7.2-18 (Sheet 1 of 4)

COMPARISON OF MAXIMUM MEMBER FORCES
DUE TO TIME HISTORY (TH) AND RESPONSE SPECTRUM (RSA) ANALYSES

COUPLED AUXILIARY & SHIELD BUILDINGS
HARD ROCK CONDITION
Maximum Forces (x103 Kips)

Elevation	Axial		N-S Shear		E-W Shear	
(ft)	TH	RSA	TH	RSA	TH	RSA
306.25						
	1.45	1.48	2.46	2.40	2.43	2.51
297.08						
	3.40	3.46	4.47	4.40	4.36	4.63
284.42						
	7.65	7.76	8.30	8.26	7.67	8.80
272.42						
	11.54	11.66	12.52	12.39	10.57	13.22
241.00						
	15.44	12.78	16.43	16.41	15.68	17.03
220.00						
	18.05	14.24	18.72	18.64	18.32	19.44
200.00						
	20.43	15.77	20.68	20.21	20.32	21.32
180.20						
	23.40	17.62	23.28	22.11	23.03	23.18
161.50						
	25.45	18.90	25.51	23.32	25.17	24.48
153.50						
	28.14	20.80	28.82	25.11	28.40	26.57
135.25						
	31.92	23.54	34.03	27.82	33.57	29.96
117.50						
	34.96	26.04	37.54	29.79	37.59	32.85
100.00						

Note:
Time history analyses consider vibration modes up to 33 Hertz. Response spectrum analyses combine vibration modes up to 33 Hertz by double sum method and add high frequency effects. (See subsection 3.7.2.7.)

Table 3.7.2-18 (Sheet 2 of 4)

COMPARISON OF MAXIMUM MEMBER FORCES DUE TO TIME HISTORY (TH) AND RESPONSE SPECTRUM (RSA) ANALYSES

STEEL CONTAINMENT VESSEL HARD ROCK CONDITION

Maximum Forces (x103 Kips)

Elevation	Axial		N-S Shear		E-W Shear	
(ft)	TH	RSA	TH	RSA	TH	RSA
256.33						
	0.28	0.53	0.17	0.20	0.22	0.24
248.33						
	0.61	1.18	0.46	0.55	0.59	0.64
240.33						
	0.99	1.74	0.77	0.89	0.99	1.04
229.52						
	1.33	2.15	1.06	1.20	1.37	1.40
218.71						
	1.66	2.51	1.36	1.49	1.76	1.74
205.33						
	2.79	3.52	2.60	2.21	2.78	2.58
190.00						
	3.23	4.04	3.00	2.54	3.29	2.99
170.00						
	3.58	4.43	3.29	2.79	3.67	3.31
162.00						
	3.91	4.80	3.52	3.00	3.96	3.58
144.50						
	4.19	5.09	3.72	3.21	4.22	3.83
138.58						
	4.21	5.12	3.73	3.22	4.24	3.85
132.25						
	4.41	5.32	3.86	3.37	4.40	4.03
116.86						
	4.49	5.39	3.89	3.41	4.44	4.08
112.50						
	4.55	5.44	3.92	3.44	4.47	4.12

Table 3.7.2-18 (Sheet 3 of 4)

COMPARISON OF MAXIMUM MEMBER FORCES
DUE TO TIME HISTORY (TH) AND RESPONSE SPECTRUM (RSA) ANALYSES

STEEL CONTAINMENT VESSEL (cont.)
HARD ROCK CONDITION

Elevation	Axial		N-S Shear		E-W Shear	
(ft)	TH	RSA	TH	RSA	TH	RSA
110.50						
	4.57	5.45	3.92	3.45	4.48	4.12
104.13						
	4.60	5.47	3.93	3.46	4.49	4.13
100.00						

Note:
Time history analyses consider vibration modes up to 33 Hertz. Response spectrum analyses combine vibration modes up to 33 Hertz by double sum method and add high frequency effects. (See subsection 3.7.2.7.)

Table 3.7.2-18 (Sheet 4 of 4)

COMPARISON OF MAXIMUM MEMBER FORCES DUE TO TIME HISTORY (TH) AND RESPONSE SPECTRUM (RSA) ANALYSES

CONTAINMENT INTERNAL STRUCTURES
HARD ROCK CONDITION
Maximum Forces (x103 Kips)

Elevation	Axial		N-S Shear		E-W Shear	
(ft)	TH	RSA	TH	RSA	TH	RSA
	Above Elevation 135.25′, West SG Compartment					
158.00						
	0.02	0.05	0.16	0.16	0.13	0.15
153.56						
	0.02	0.05	0.28	0.29	0.22	0.28
148.00						
	0.08	0.24	0.81	0.81	0.65	0.76
135.25						
	Above Elevation 135.25′, East SG Compartment					
148.00						
	0.03	0.13	0.31	0.31	0.24	0.27
135.25						
	Below Elevation 135.25′					
135.25						
	0.32	1.99	6.14	5.73	6.09	5.98
121.50						
	0.32	1.99	6.24	5.83	6.16	6.07
107.17						
	0.67	4.07	7.30	7.02	6.34	6.90
103.00						
	0.86	6.55	7.35	7.65	6.37	7.54
100.00						

Note:
Time history analyses consider vibration modes up to 33 Hertz. Response spectrum analyses combine vibration modes up to 33 Hertz by double sum method and add high frequency effects. (See subsection 3.7.2.7.)

Table 3.7.2-19 (Sheet 1 of 4)

COMPARISON OF MAXIMUM MEMBER MOMENTS
DUE TO TIME HISTORY (TH) AND RESPONSE SPECTRUM ANALYSES (RSA)

COUPLED AUXILIARY & SHIELD BUILDINGS
HARD ROCK CONDITION
Maximum Moments (x103 K-Ft)

Elevation	Torque		N-S Moment		E-W Moment	
(ft)	TH	RSA	TH	RSA	TH	RSA
306.25			18.2	24.3	18.7	23.0
	3.9	20.5				
297.08			59.8	63.5	59.1	61.5
	9.2	48.0				
284.42			181.9	186.0	178.7	181.0
	25.5	128.5				
272.42			274.0	260.6	266.0	255.1
	46.1	220.1				
241.00			747.5	685.7	746.0	668.1
	81.1	339.6				
220.00			1072.0	1110.8	1109.0	1068.7
	109.5	442.6				
200.00			1402.0	1501.6	1488.0	1437.1
	134.5	523.0				
180.20			1835.0	1966.6	2140.0	1975.4
	923.8	1197.0				
161.50			2243.0	2431.5	2483.0	2330.6
	911.4	1142.2				
153.50			2389.0	2635.4	2482.0	2505.9
	716.1	1108.1				
135.25			2896.0	3127.6	2972.0	3073.8
	1157.0	1471.4				
117.50			3539.0	3607.1	3417.0	3635.1
	1396.0	1719.3				
100.00			4188.0	4090.4	4045.0	4070.4

<u>Note</u>:
Time history analyses consider vibration modes up to 33 Hertz. Response spectrum analyses combine vibration modes up to 33 Hertz by double sum method and add high frequency effects. (See subsection 3.7.2.7.)

Table 3.7.2-19 (Sheet 2 of 4)

COMPARISON OF MAXIMUM MEMBER MOMENTS
DUE TO TIME HISTORY (TH) AND RESPONSE SPECTRUM ANALYSES (RSA)

STEEL CONTAINMENT VESSEL
HARD ROCK CONDITION

Maximum Moments (x103 K-Ft)

Elevation	Torque		N-S Moment		E-W Moment	
(ft)	TH	RSA	TH	RSA	TH	RSA
256.33			0.0	0.0	0.0	0.0
	0.00	0.00				
248.33			2.7	4.7	2.3	3.9
	0.16	0.36				
240.33			9.4	15.0	7.8	12.4
	0.47	1.04				
229.52			22.7	32.8	18.4	27.4
	0.90	1.97				
218.71			40.6	55.0	32.5	46.0
	1.37	3.00				
205.33			70.0	93.8	58.6	84.1
	10.26	10.65				
190.00			118.1	138.6	101.3	120.2
	10.59	11.59				
170.00			187.4	199.0	162.8	170.1
	10.81	12.25				
162.00			219.6	227.8	191.1	194.1
	10.97	12.75				
144.50			291.4	288.6	254.3	243.6
	11.99	13.18				
138.58			316.4	308.4	276.4	260.1
	11.43	13.17				
132.25			345.3	309.3	302.1	260.2
	11.52	13.43				
116.86			413.1	390.2	362.4	327.8
	11.55	13.51				

Table 3.7.2-19 (Sheet 3 of 4)

COMPARISON OF MAXIMUM MEMBER MOMENTS
DUE TO TIME HISTORY (TH) AND RESPONSE SPECTRUM ANALYSES (RSA)

STEEL CONTAINMENT VESSEL (cont.)
HARD ROCK CONDITION

Maximum Moments (x103 K-Ft)

Elevation	Torque		N-S Moment		E-W Moment	
(ft)	TH	RSA	TH	RSA	TH	RSA
112.50			433.1	407.1	380.0	341.9
	11.36	13.56				
110.5			442.3	414.9	387.9	348.13
	11.22	13.7				
104.13			471.2	439.1	413.3	368.4
	11.23	13.71				
100			489.7	454.7	429.5	381.5

Note:
Time history analyses consider vibration modes up to 33 Hertz. Response spectrum analyses combine vibration modes up to 33 Hertz by double sum method and add high frequency effects. (See subsection 3.7.2.7.)

Table 3.7.2-19 (Sheet 4 of 4)

COMPARISON OF MAXIMUM MEMBER MOMENTS
DUE TO TIME HISTORY (TH) AND RESPONSE SPECTRUM ANALYSES (RSA)

CONTAINMENT INTERNAL STRUCTURES
HARD ROCK CONDITION
Maximum Moments (x103 K-Ft)

Elevation	Torque		N-S Moment		E-W Moment	
(ft)	TH	RSA	TH	RSA	TH	RSA
	Above Elevation 135.25′, West SG Compartment					
158.00			0.02	0.07	0.04	0.07
	0.18	0.25				
153.56			0.61	0.71	0.76	0.74
	0.16	0.25				
148.00			2.19	2.65	2.94	3.05
	4.98	6.69				
135.25			10.46	12.25	13.29	13.23
	Above Elevation 135.25′, East SG Compartment					
148.00			0.24	0.56	0.15	0.16
	1.33	2.2				
135.25			3.22	3.79	4.04	4.1
	Below Elevation 135.25′					
135.25			31.2	40.4	33	35.7
	124.7	245.7				
121.50			114.9	117.4	117.2	108.6
	126.5	247.5				
107.17			216.8	219.6	210.4	196.1
	218.2	321.9				
103.00			243.3	244.3	247.1	225.9
	116.1	280.5				
100.00			258	258.9	269.1	244.9

Note:
Time history analyses consider vibration modes up to 33 Hertz. Response spectrum analyses combine vibration modes up to 33 Hertz by double sum method and add high frequency effects. (See subsection 3.7.2.7.)

Table 3.7.2-20

COMPARISON OF FREQUENCIES IN SEISMIC MODELS

COUPLED AUXILIARY AND SHIELD BUILDINGS

Mode/Direction	Model A (hertz)	Model B (hertz)
1 st. Horizontal, E/W	4.35	4.23
1 st. Horizontal, N/S	4.78	4.65
2 nd. Horizontal, E/W	9.10	8.94
2 nd. Horizontal, N/S	9.26	9.10
1 st. Vertical	6.77	6.60
2 nd. Vertical	19.34	19.10

Note:
See subsection 3.7.2.2.1 for description of the seismic models.

Table 3.7.2-21

**COMPARISON OF MAXIMUM ABSOLUTE NODAL ACCELERATION (ZPA)
NUCLEAR ISLAND STRUCTURES
SOFT-TO-MEDIUM SOIL CONDITIONS, 2 X G_{MAX}**

	Elevation (ft)	Direction	Model A	Model B
Aux./shield building	66.50	N-S	0.32	0.32
		E-W	0.29	0.29
		V	0.33	0.33
Aux./shield building	100.00	N-S	0.35	0.35
		E-W	0.31	0.30
		V	0.33	0.33
Aux./shield building	117.50	N-S	0.36	0.37
		E-W	0.34	0.34
		V	0.35	0.34
Aux./shield building	180.00	N-S	0.50	0.52
		E-W	0.55	0.52
		V	0.43	0.42
Aux./shield building	307.00	N-S	1.18	1.17
		E-W	1.38	1.46
		V	1.02	1.03
Steel Containment	205.33	N-S	0.49	0.52
Vessel		E-W	0.69	0.69
		V	0.42	0.42
Steel Containment	256.33	N-S	0.58	0.59
Vessel		E-W	0.86	0.88
		V	0.80	0.78
Containment Internal	135.25	N-S	0.36	0.37
Structures		E-W	0.36	0.35
		V	0.35	0.35

Note:
See subsection 3.7.2.2.1 for description of the seismic models.

Table 3.7.2-22

COMPARISON OF MAXIMUM MEMBER FORCES AND MOMENTS AT ELEVATION 100' CONTAINMENT INTERNAL STRUCTURES & STEEL CONTAINMENT VESSEL SOFT-TO-MEDIUM SOIL CONDITIONS, 2 X G_{MAX}

		Maximum Forces (x 10^3 Kips)			Maximum Moments (x 10^3 Kips)		
Structure	**Model**	**Vertical**	**N-S Shear**	**E-W Shear**	**Torque**	**about N-S Axis**	**about E-W Axis**
Containment Internal structures							
	A	8.26	9.56	9.01	55.8	218.0	202.0
	B	8.45	9.82	9.45	60.8	227.0	208.0
Steel Containment Vessel							
	A	3.04	3.43	3.64	11.1	369.0	327.0
	B	3.02	3.47	3.54	11.3	385.0	324.0

Note:
See subsection 3.7.2.2.1 for description of the seismic models

Table 3.7.2-23 (Sheet 1 of 2)

COMPARISON OF MAXIMUM MEMBER FORCES
COUPLED AUXILIARY & SHIELD BUILDINGS
SOFT-T0-MEDIUM SOIL CONDITIONS, 2 X G_{MAX}
Maximum Forces (x 1000 Kips)

Elevation	**Axial**		**N-S Shear**		**E-W Shear**	
(ft)	**Model A**	**Model B**	**Model A**	**Model B**	**Model A**	**Model B**
306.25						
	1.71	1.74	1.99	1.97	2.33	2.42
297.08						
	3.99	4.97	3.60	4.15	4.35	5.06
284.42						
	8.97	9.96	7.06	7.88	8.44	9.33
272.42						
	17.10	18.10	16.10	17.10	18.50	19.90
241.00						
220.00						
	19.20	20.10	18.90	19.70	21.80	23.00
200.00						
	21.00	22.10	20.90	22.20	23.90	25.30
180.20						
	23.60	24.80	23.60	25.50	26.80	28.10
161.50						
	25.60	26.80	25.90	27.90	29.00	30.00
153.50						
	28.40	29.80	29.20	31.40	32.00	32.80
135.25						
	33.90	35.50	34.90	37.60	36.60	37.30
117.50						
	40.00	41.40	41.20	44.10	41.60	42.50
100.00						

Note:
See subsection 3.7.2.2.1 for description of the seismic models

Table 3.7.2-23 (Sheet 2 of 2)

COMPARISON OF MAXIMUM MEMBER MOMENTS
COUPLED AUXILIARY & SHIELD BUILDINGS
SOFT-TO-MEDIUM SOIL CONDITIONS, 2 X G_{MAX}
Maximum Moments (x 1000 K-Ft)

Elevation	Torque		N-S Moment		E-W Moment	
(ft)	Model A	Model B	Model A	Model B	Model A	Model B
306.25			10.2	10.3	10.6	10.3
	2.4	2.1				
297.08			36.8	40.3	39.5	41.1
	5.6	6.3				
284.42			119.0	132.0	123.0	127.0
	15.5	15.9				
272.42			213.0	244.0	223.0	225.0
	28.3	27.9				
241.00			713.0	779.0	610.0	627.0
	52.2	51.8				
220.00			1140.0	1240.0	920.0	1010.0
	73.5	71.5				
200.00			1600.0	1720.0	1320.0	1430.0
	96.8	94.3				
180.20			2170.0	2350.0	2120.0	2350.0
	1010.0	1070.0				
161.50			2770.0	2960.0	2590.0	2830.0
	841.0	877.0				
153.50			3030.0	3230.0	2530.0	2770.0
	695.0	713.0				
135.25			3670.0	3910.0	3280.0	3610.0
	1100.0	1130.0				
117.50			4300.0	4590.0	4030.0	4440.0
	1330.0	1380.0				
100.00			4980.0	5280.0	4780.0	5220.0

Note:
See subsection 3.7.2.2.1 for description of the seismic models

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	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	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
12203	Division C dc equipment room	dc equipment
12204	Division B battery room 2	Batteries
12205	Division D dc equipment room	dc equipment
12207	Division B dc equipment room	dc equipment
12211	Corridor	Divisional cables
12212	Division B RCP trip switchgear room	RCP trip switchgear
12244	Lower annulus valve area	CVS/WLS containment isolation valves
12251	Demineralizer/filter access area	CVS/DWS isolation valves
12254	SFS penetration room	SFS containment isolation valve
12256	Containment isolation valve room	RNS containment isolation valves
12259	Pipe chase	RNS piping
12262	Piping/Valve room	RNS pressure boundary, SFS piping
12265	Waste monitor tank room C	SFS piping
12269	Pipe chase	RNS pressure boundary
12300	Corridor	Divisional cable
12301	Division A I&C room	Divisional I&C
12302	Division C I&C room	Divisional I&C

Table 3.7.3-1 (Sheet 2 of 3)

SEISMIC CATEGORY I EQUIPMENT OUTSIDE CONTAINMENT BY ROOM NUMBER

Room No.	Room Name	Equipment Description
12303	Remote shutdown workstation	Remote shutdown workstation
12304	Division B I&C/penetration room	Divisional I&C/electrical penetrations
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	Division C RCP trip switchgear room	RCP trip switchgear
12313	Division C I&C/penetration room	Divisional I&C/electrical penetrations
12321	Non-1E equipment/penetration room	Divisional cabling
12341	Middle annulus	Class 1E electrical penetrations Various mechanical piping penetrations
12351	Maintenance floor staging area	Divisional cabling (ceiling)
12352	Personnel hatch	Personnel airlock (interlocks)
12354	Middle annulus access room	PSS/SFS containment isolation valves
12362	RNS HX room	RNS pressure boundary
12365	Waste monitor tank room B	SFS piping
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	Lower 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 penetration

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	Non 1E equipment/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	VFS/SFS/PSS penetration room	SFS/PSS/VFS containment isolation valves, RNS pressure boundary
12462	Cask washdown pit	SFS piping
12504	Upper MSIV compartment B	SGS CIVs, instrumentation and controls
12506	Upper MSIV compartment A	SGS CIVs, instrumentation and controls
12541	Upper annulus	PCS piping and cabling PCS air baffle
12553	Personnel access area	Personnel airlock (interlocks)
12555	Operating deck staging area/VES air storage	VES high pressure air bottles
12562	Fuel handling area	Spent fuel storage racks
12701	PCS valve room	PCS isolation valves/instrumentation
12703	PCS water storage tank	PCS piping, 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

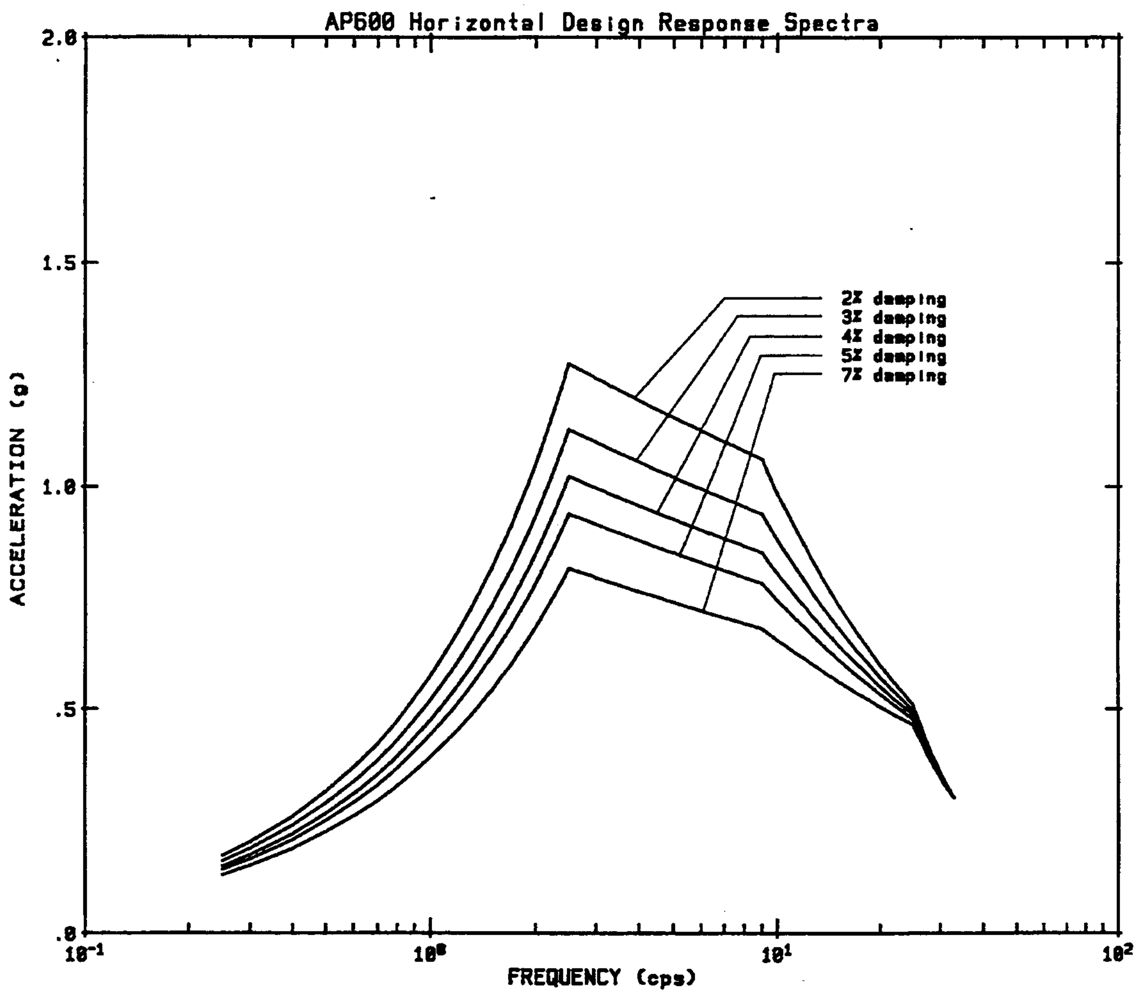


Figure 3.7.1-1

Horizontal Design Response Spectra
Safe Shutdown Earthquake

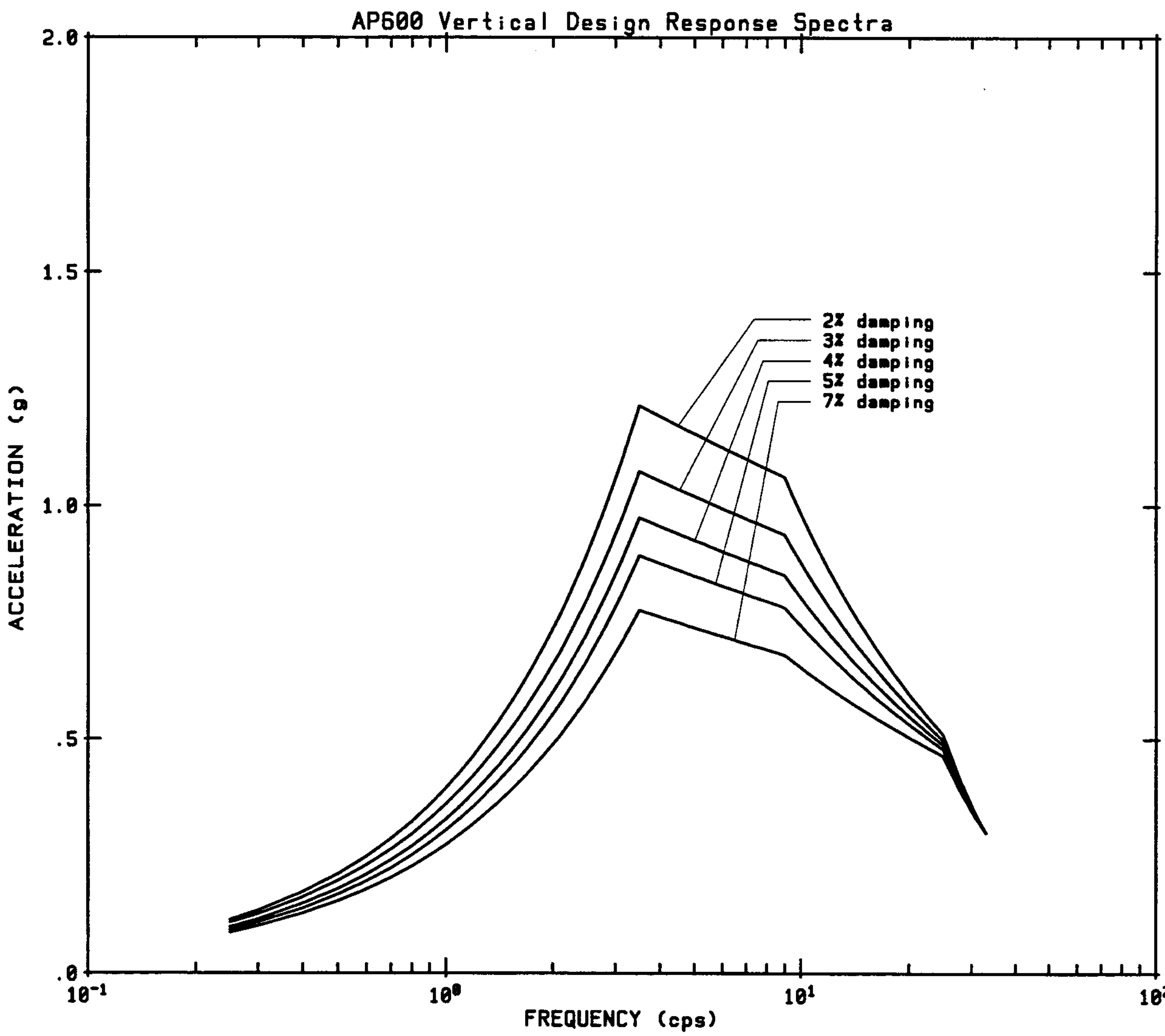


Figure 3.7.1-2

Vertical Design Response Spectra
Safe Shutdown Earthquake

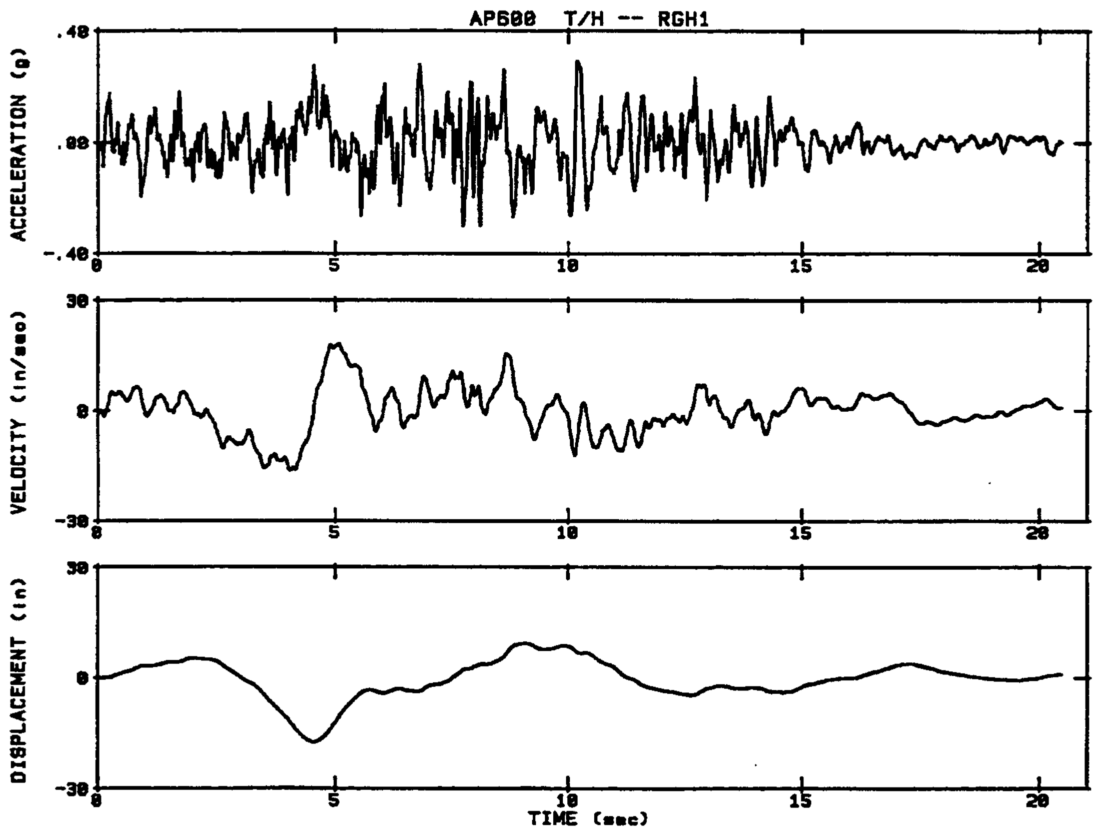


Figure 3.7.1-3

Design Horizontal Time History, "H1"
Acceleration, Velocity & Displacement Plots

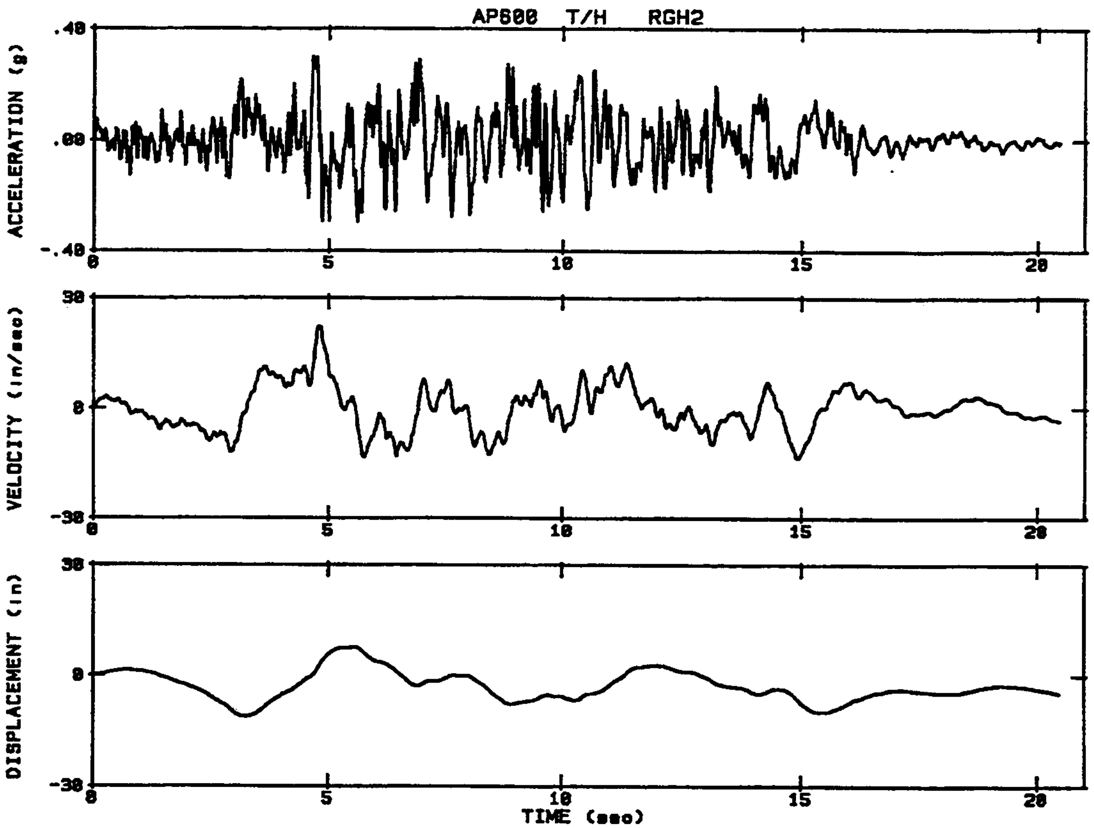


Figure 3.7.1-4

Design Horizontal Time History, "H2"
Acceleration, Velocity & Displacement Plots

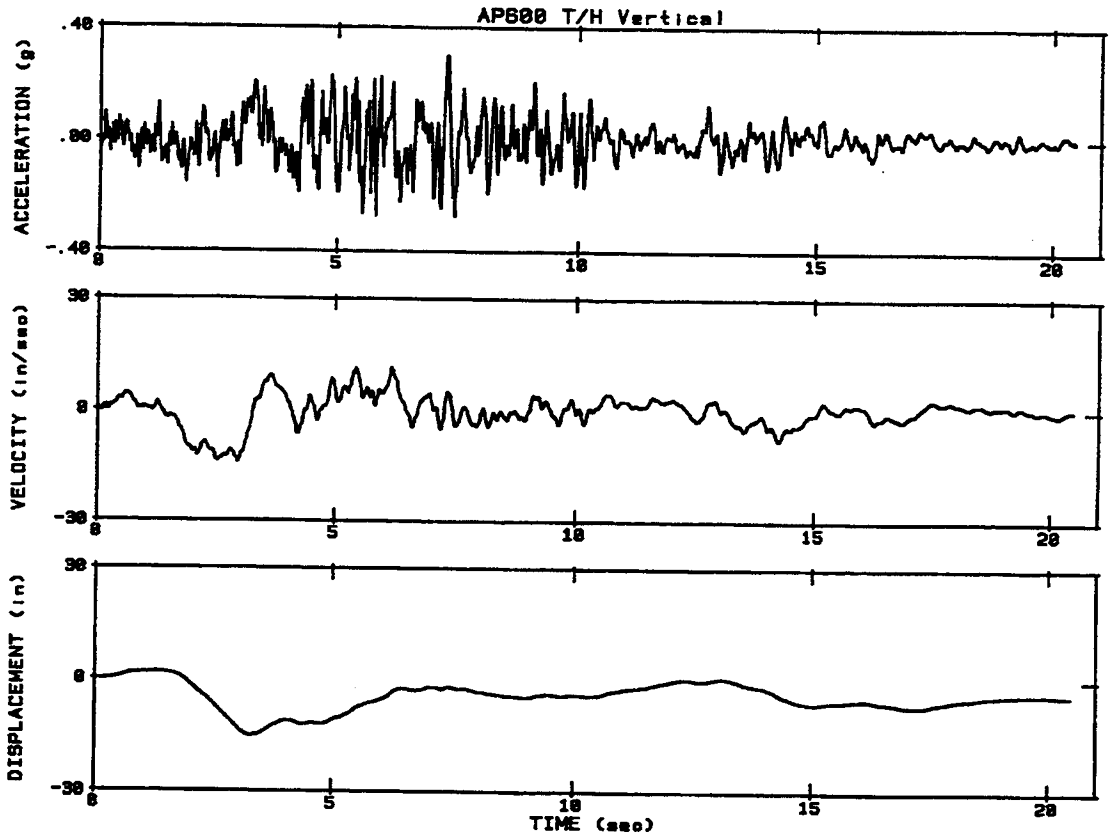


Figure 3.7.1-5

Design Vertical Time History
Acceleration, Velocity & Displacement Plots

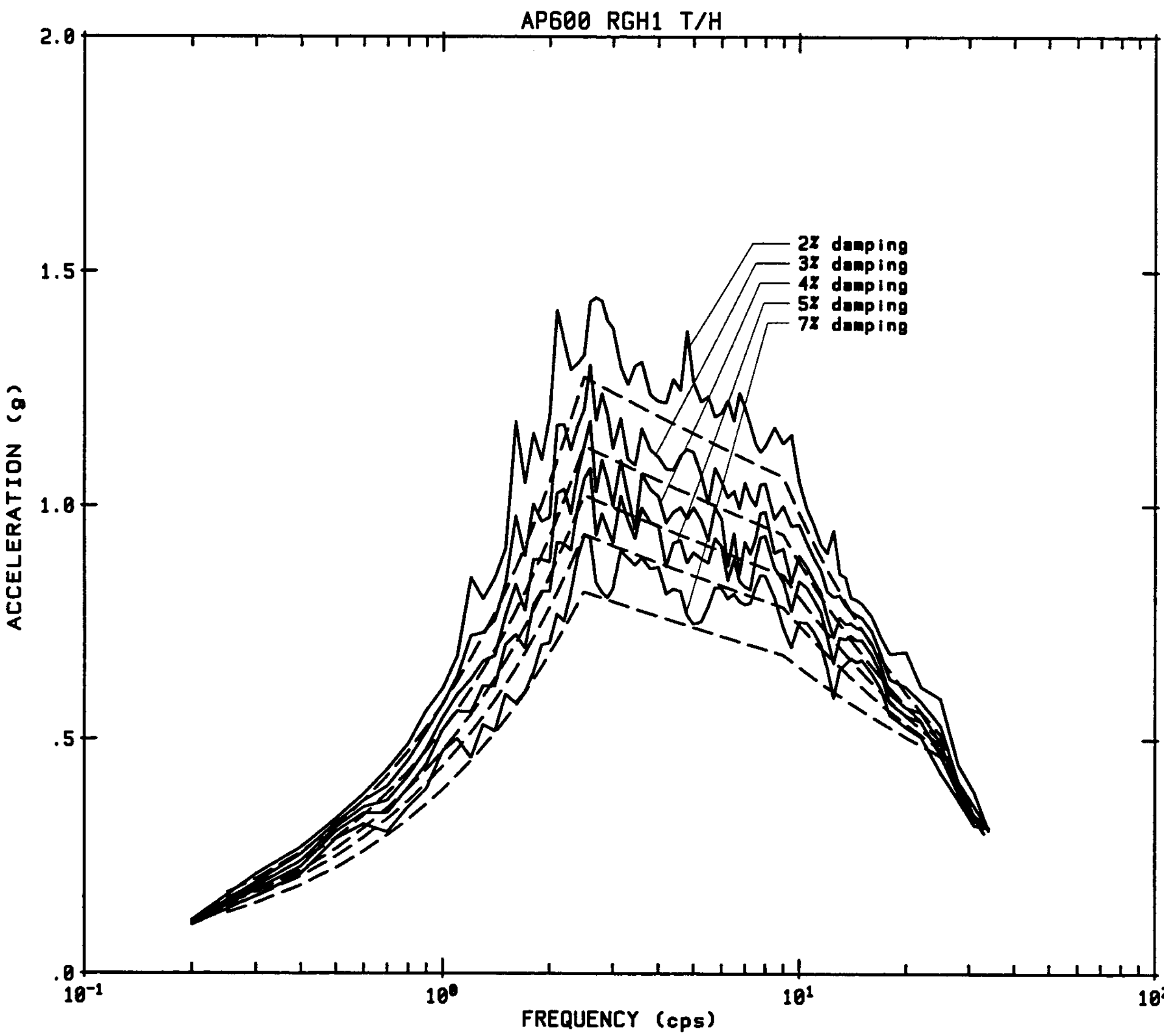


Figure 3.7.1-6

Acceleration Response Spectra of Design Horizontal Time History, "H1"

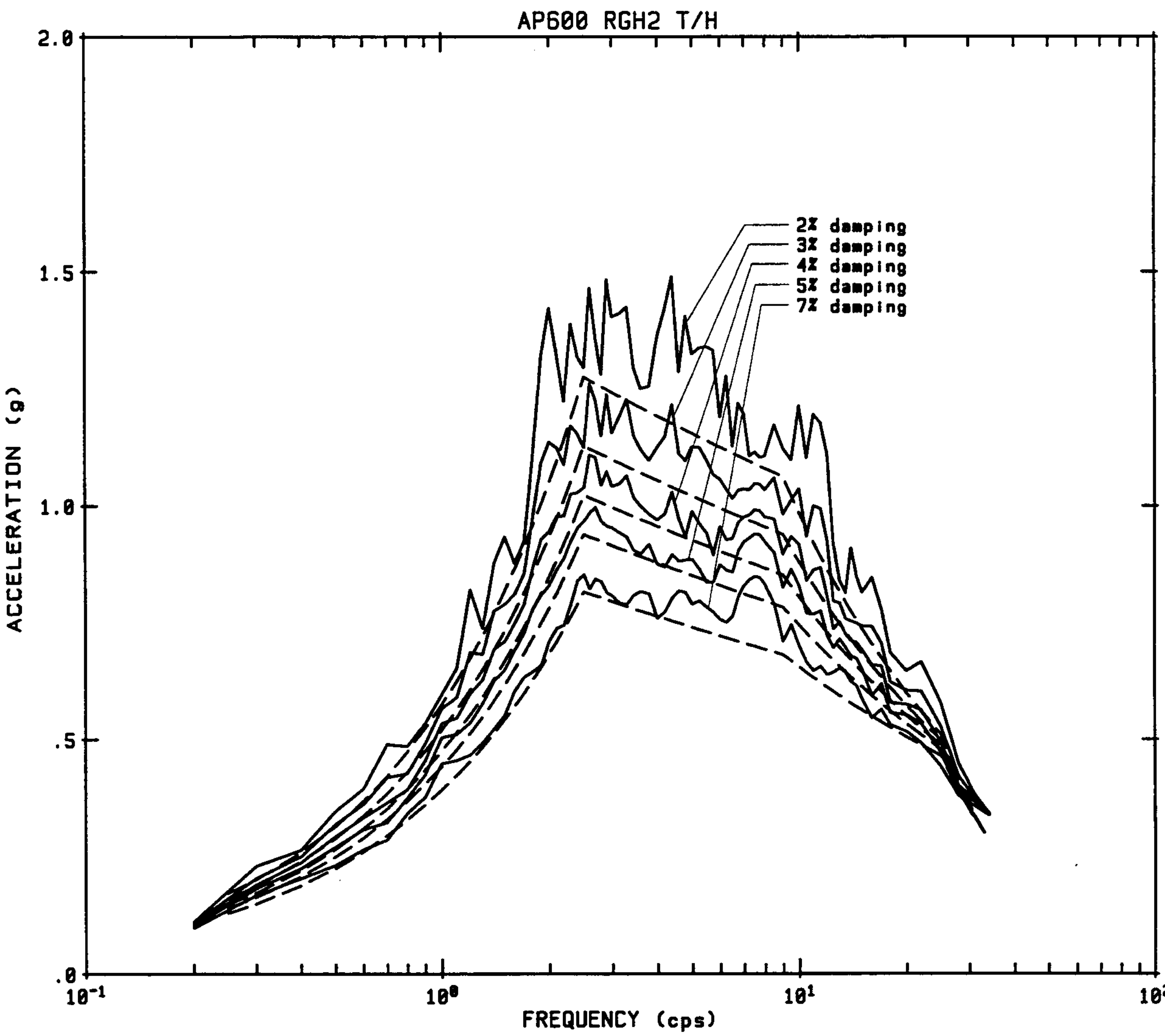


Figure 3.7.1-7

Acceleration Response Spectra of Design Horizontal Time History, "H2"

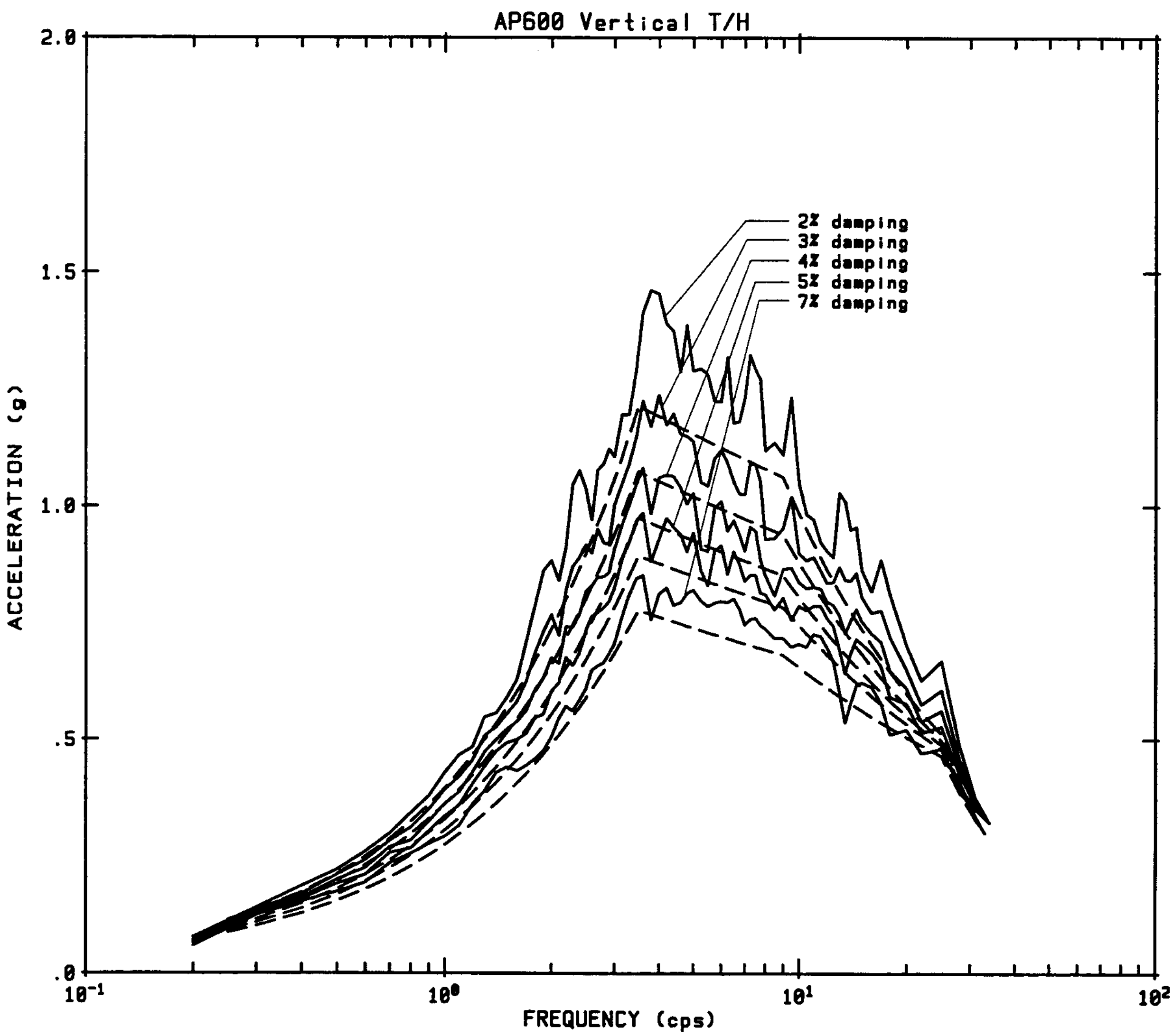


Figure 3.7.1-8

Acceleration Response Spectra of Design Vertical Time History

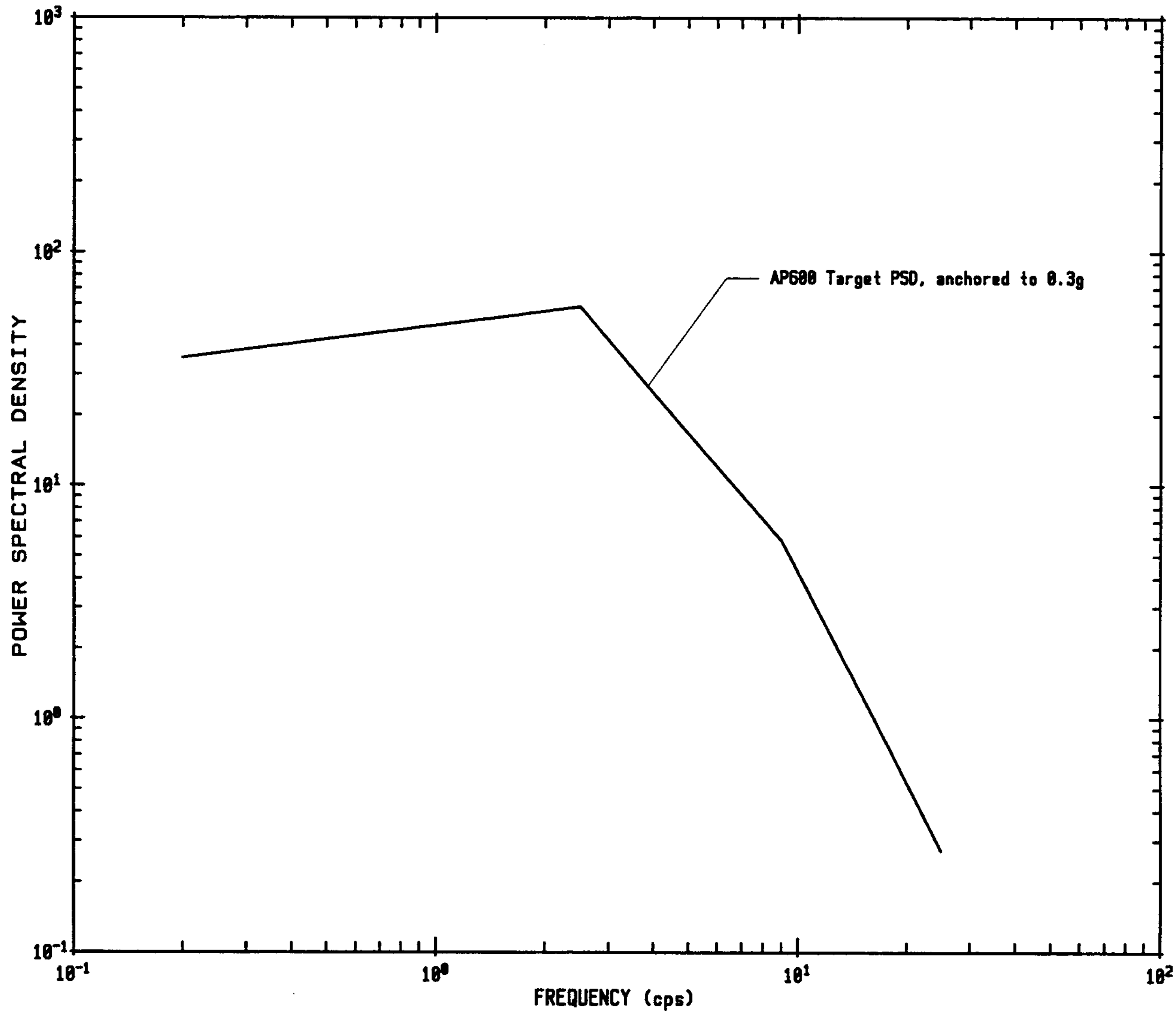


Figure 3.7.1-9

Minimum Power Spectral Density Curve (Normalized to 0.3g)

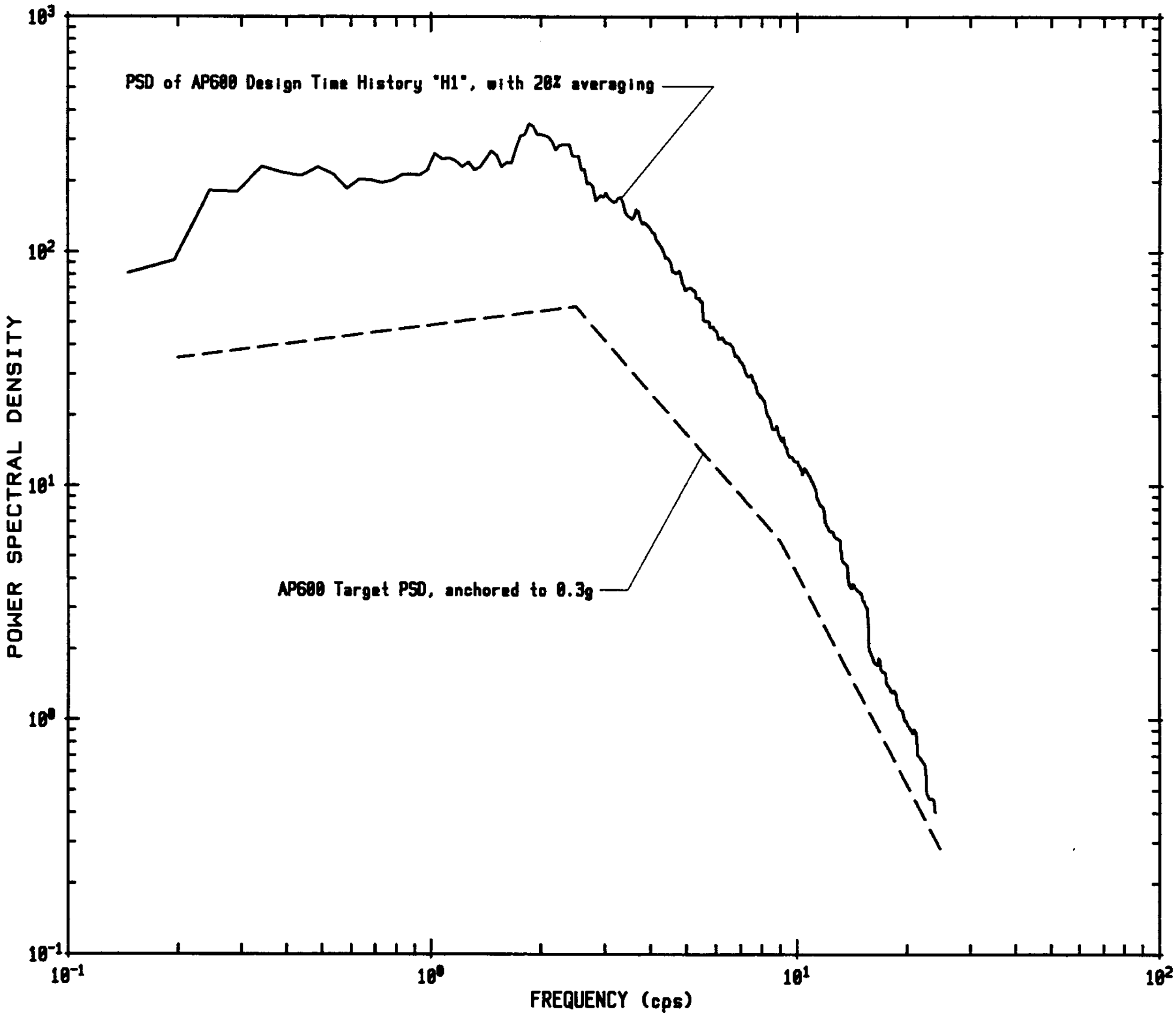


Figure 3.7.1-10

Power Spectral Density of Design Horizontal Time History, "H1"

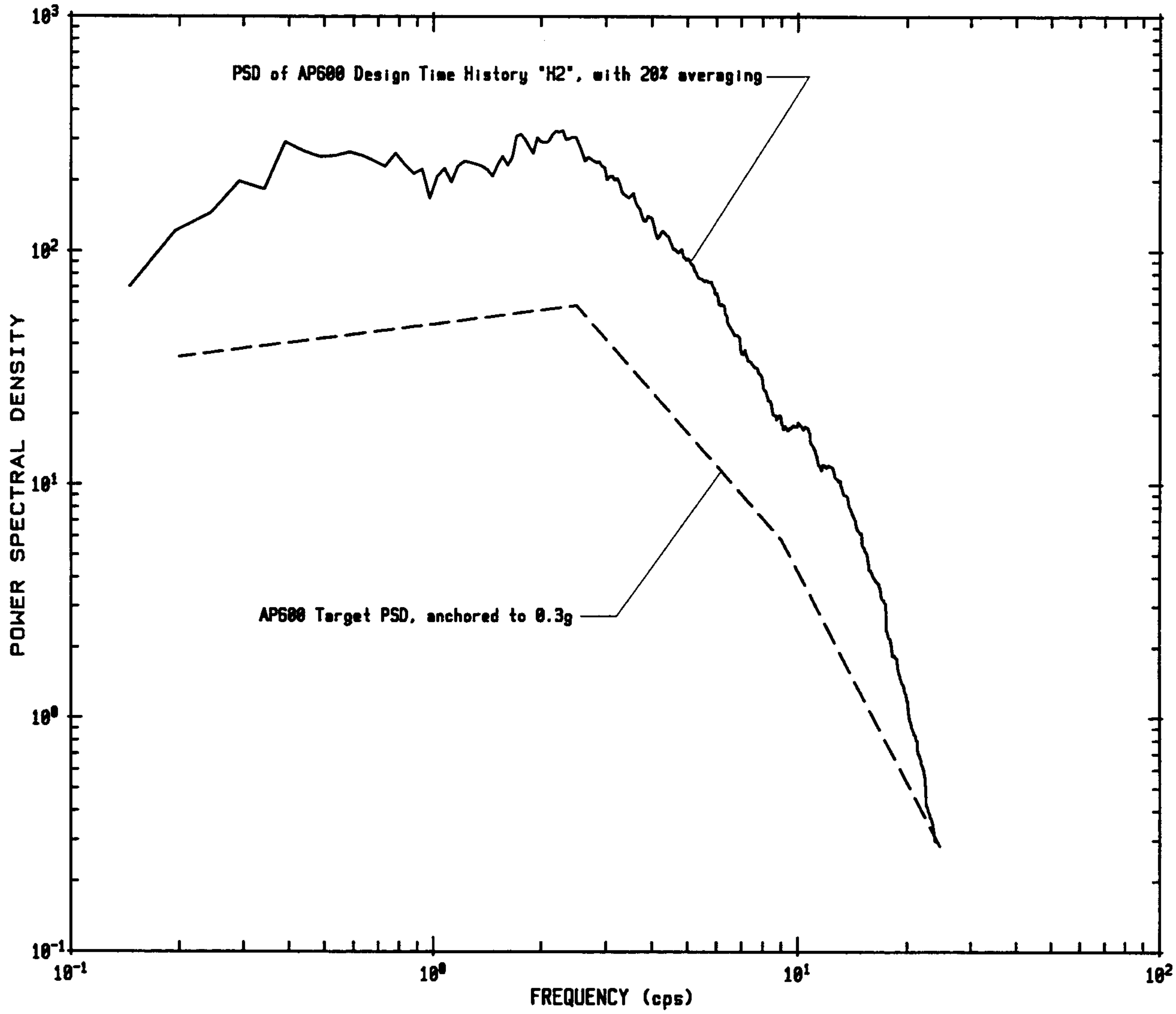


Figure 3.7.1-11

Power Spectral Density of Design Horizontal Time History, "H2"

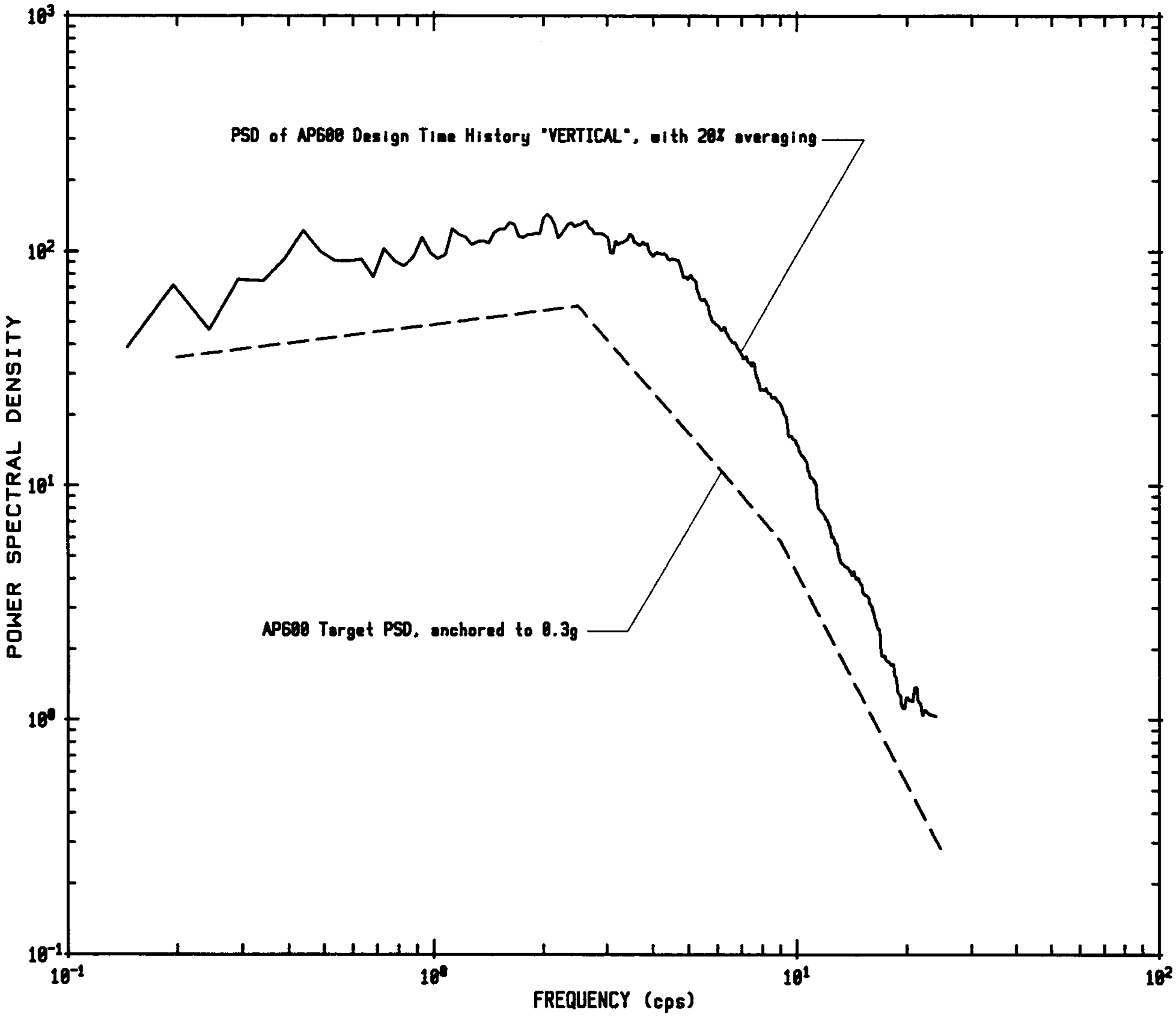
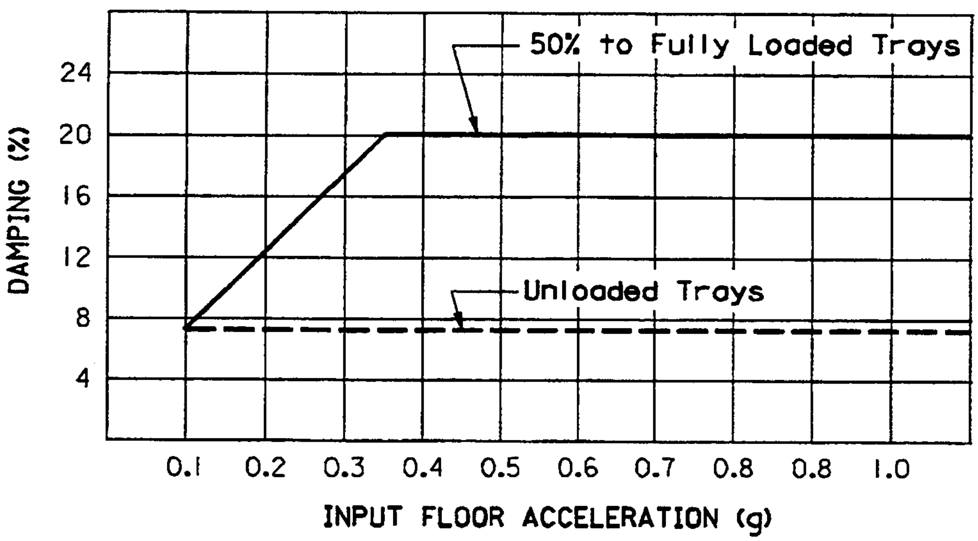


Figure 3.7.1-12

Power Spectral Density of Design Vertical Time History



Notes:

- The damping value curve shown is applicable for 50% to fully loaded cable trays.
- For cable trays loaded to less than 50%, linear interpolated damping values shall be used.
- For unloaded cable trays, damping value equal to 7% of critical shall be used for all floor acceleration values.

Figure 3.7.1-13

Damping Values for Cable Trays & Supports

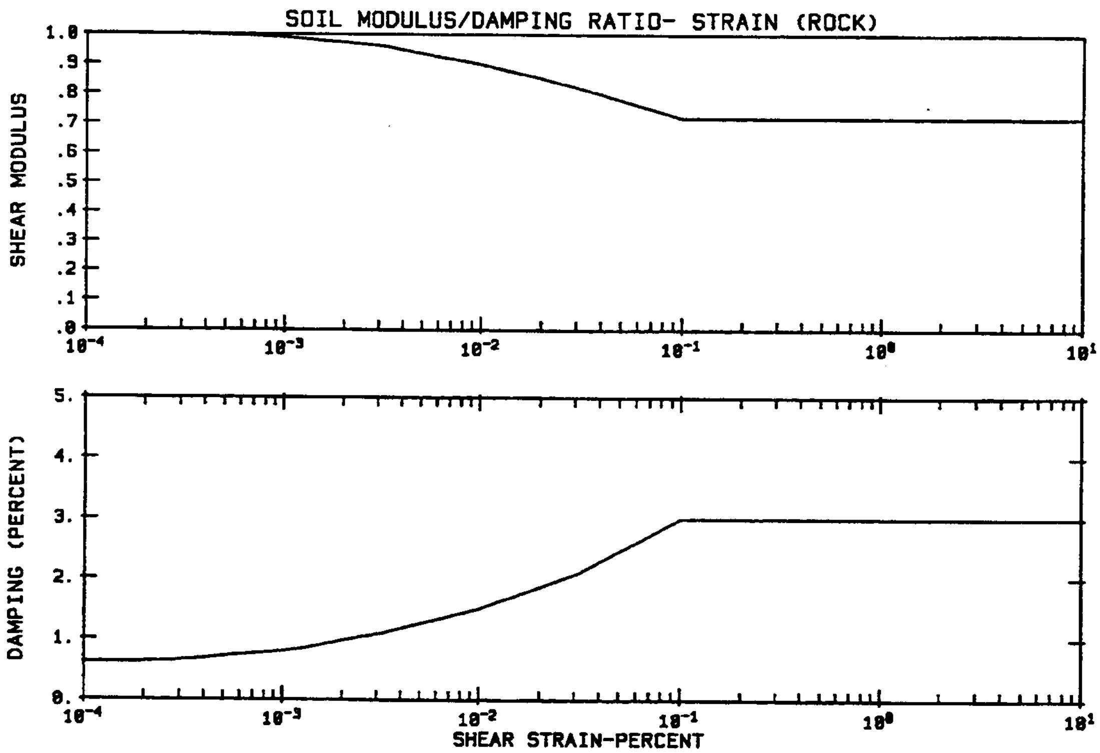


Figure 3.7.1-14

Strain Dependent Properties of Rock Material

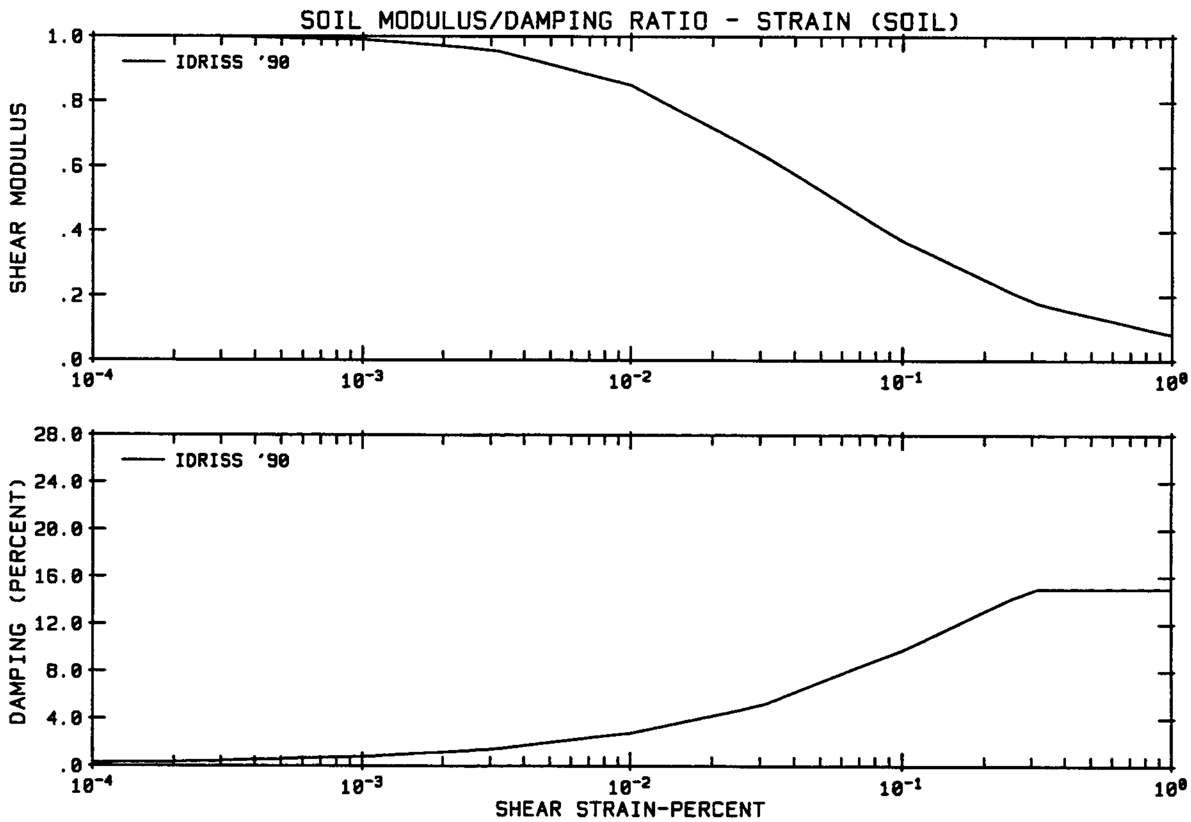


Figure 3.7.1-15

Strain Dependent Properties of Soil Material

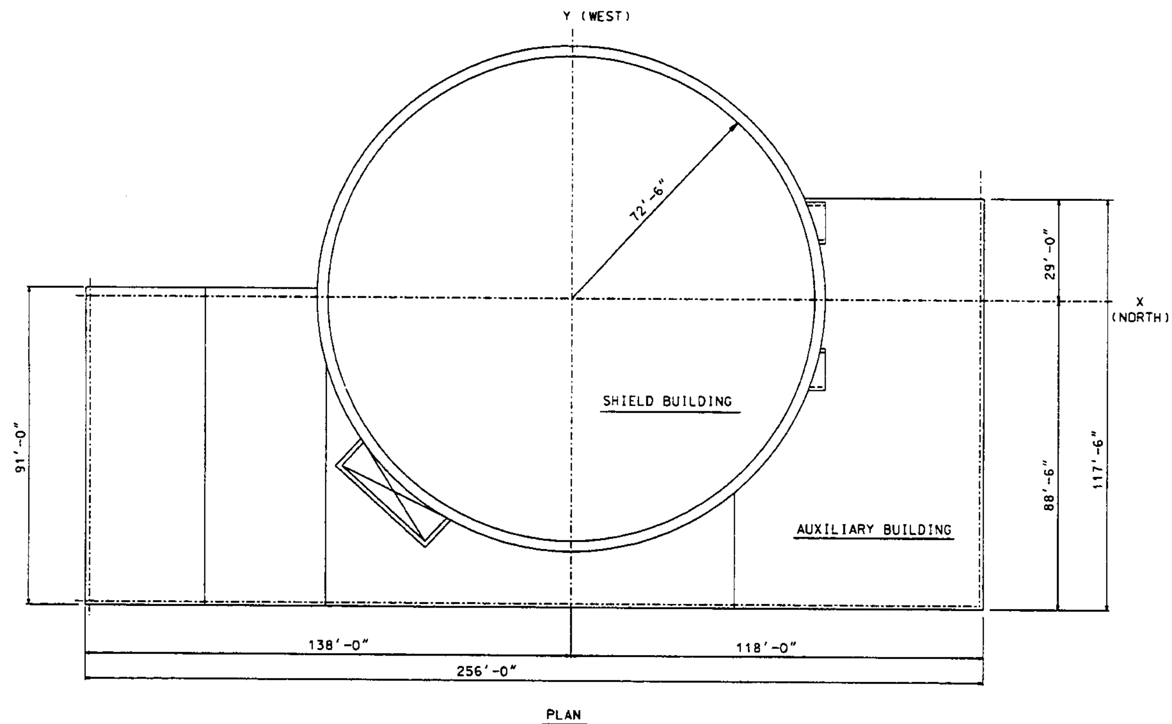
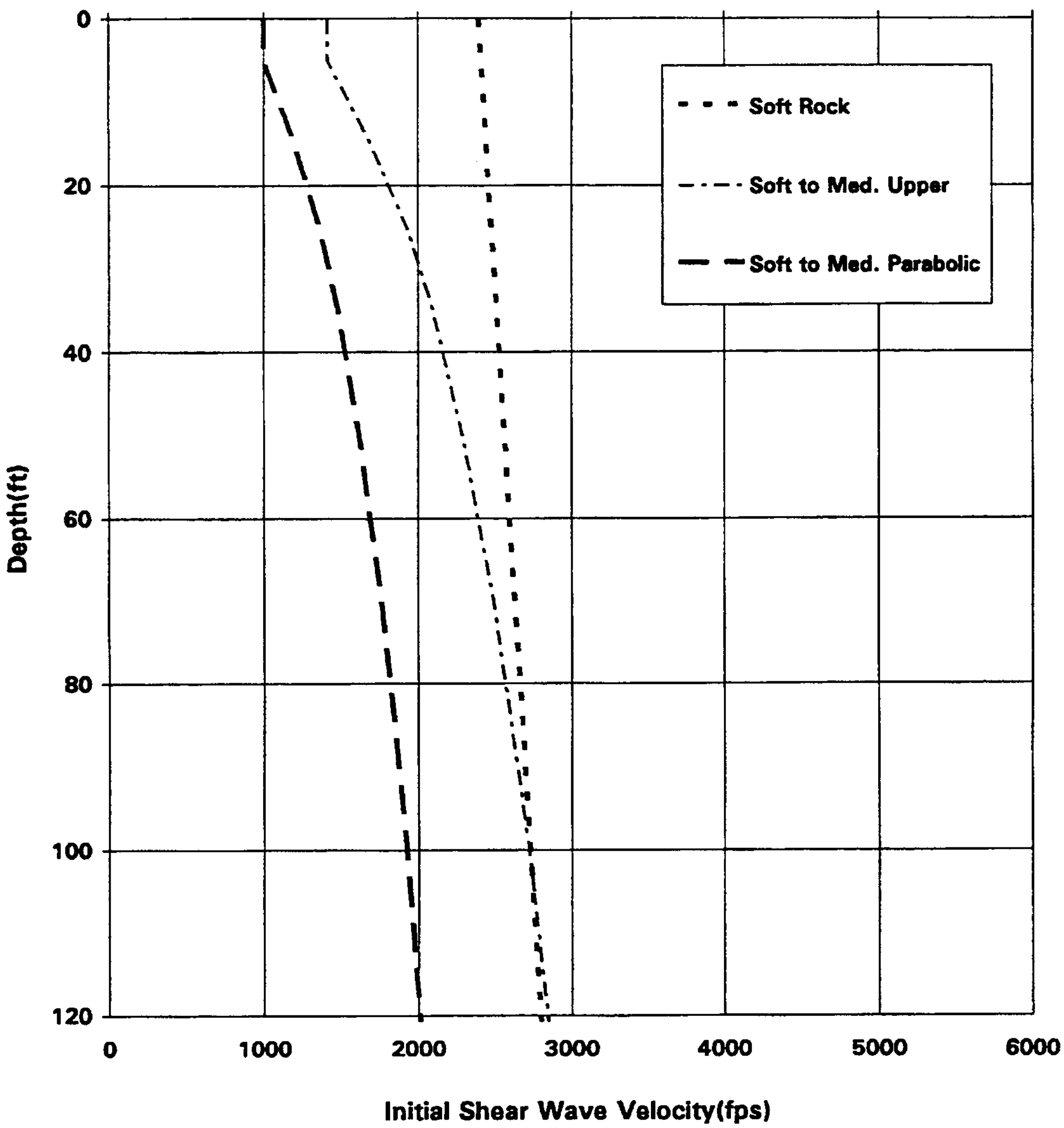


Figure 3.7.1-16

*[Nuclear Island Structures Dimensions]**

* NRC Staff approval is required prior to implementing a change in this material; see DCD Section 3.5.



**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

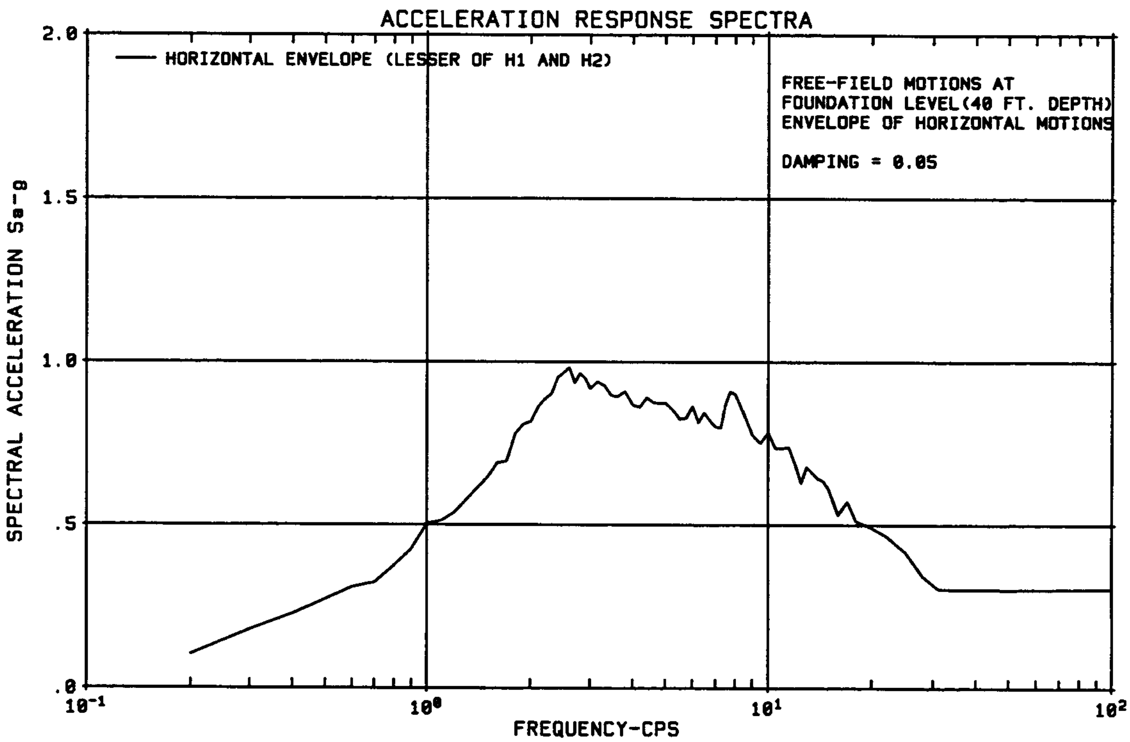


Figure 3.7.1-18

Free-Field Motions at Foundation Level (40 ft Depth)
Envelope of Horizontal Motions

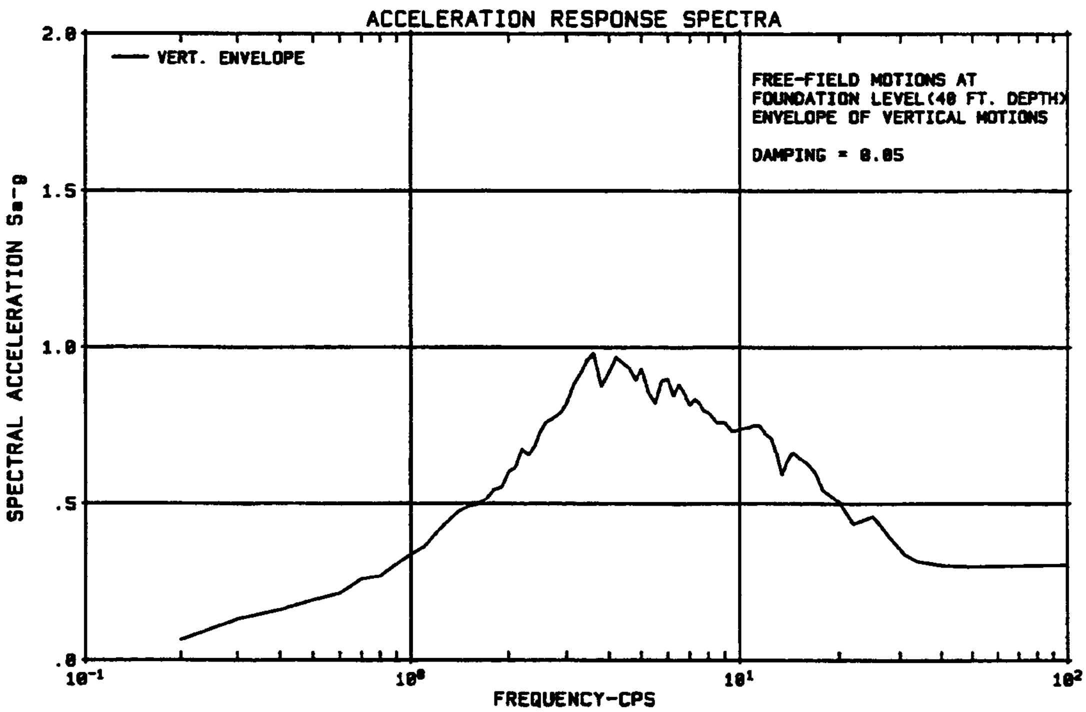


Figure 3.7.1-19

Free-Field Motions at Foundation Level (40 ft Depth)
Envelope of Vertical Motions

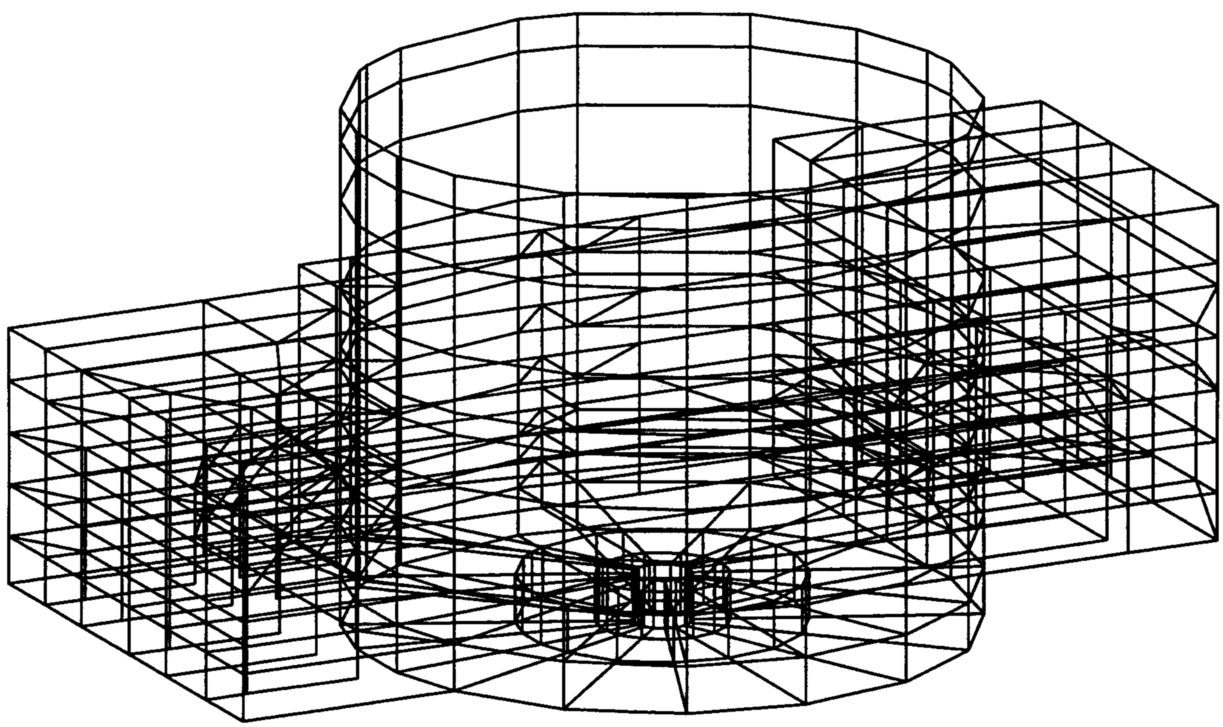


Figure 3.7.2-1

3-D Finite Element Model of Coupled Shield & Auxiliary Building

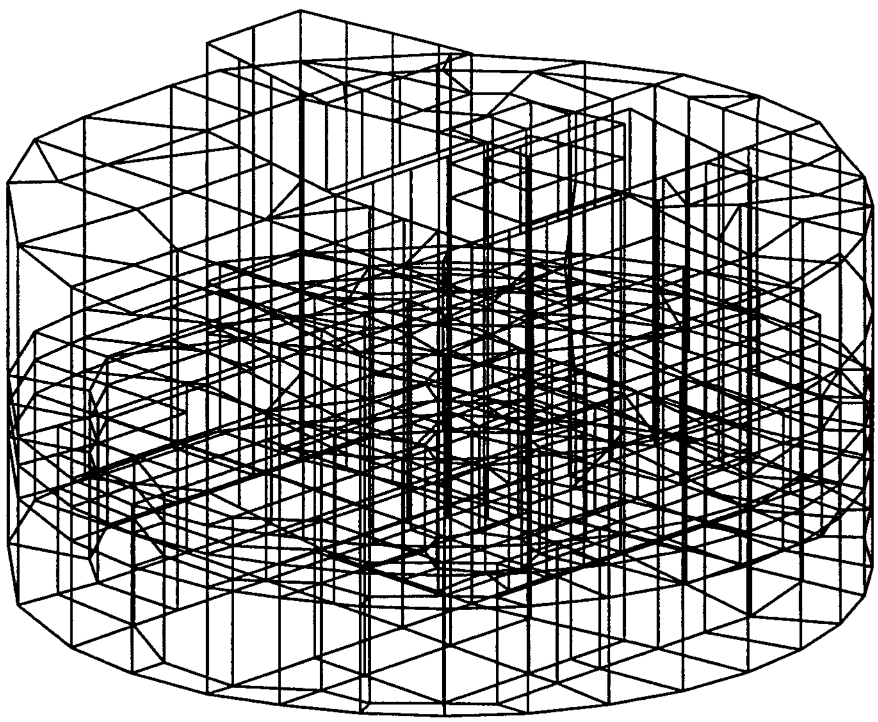
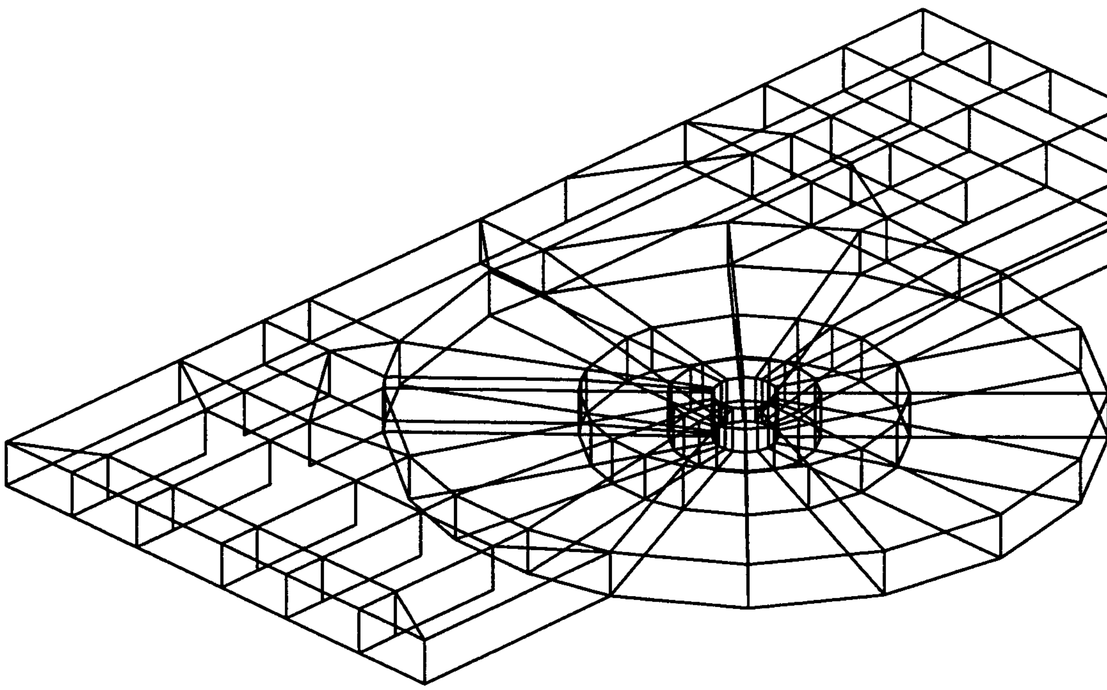


Figure 3.7.2-2

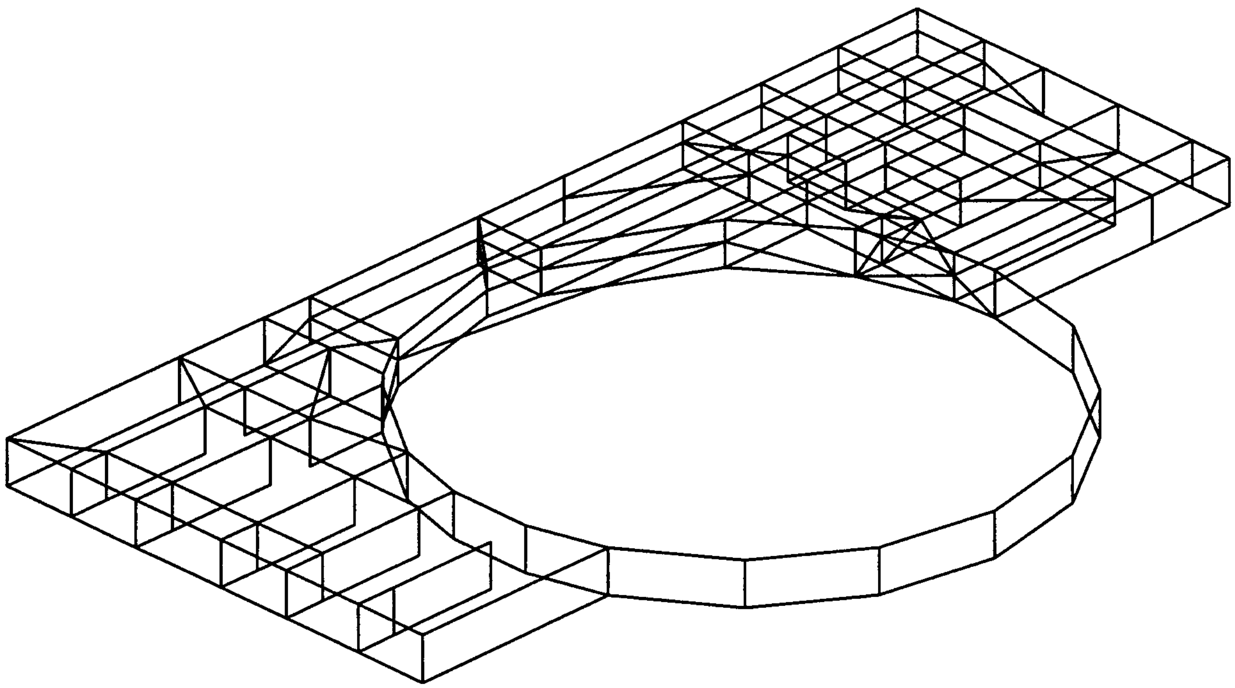
3-D Finite Element Model of Containment Internal Structures



Typical Section, El. 66.5' to 82.5'

Figure 3.7.2-3 (Sheet 1 of 2)

Coupled Shield & Auxiliary Building
Finite Element Model



Typical Section, El. 82.5' to 100.0'

Figure 3.7.2-3 (Sheet 2 of 2)

Coupled Shield & Auxiliary Building Finite Element Model

The top are at elevations 307.25' and 297.58'
in the modified model described in subsection 3.7.2.2.1

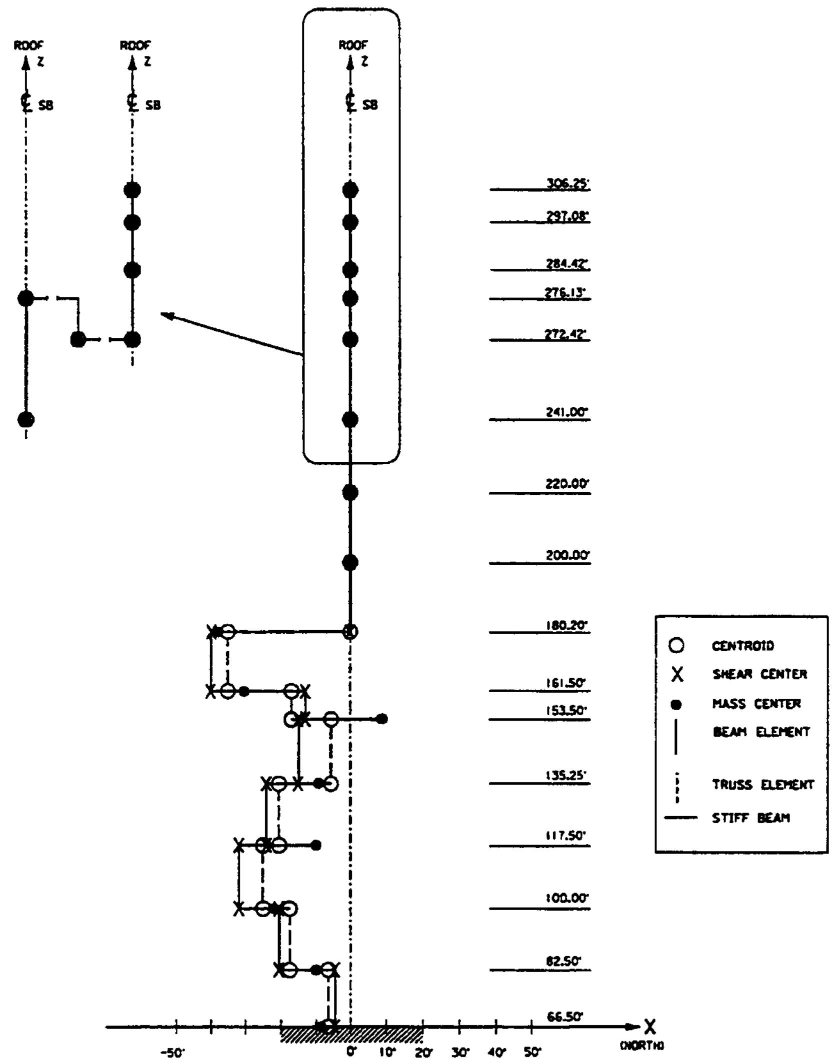


Figure 3.7.2-4 (Sheet 1 of 2)

Coupled Shield & Auxiliary Building
Lumped Mass Stick Model (North -South)

The top nodes are at elevations 307.25' and 297.58'
in the modified model described in Subsection 3.7.2.2.1

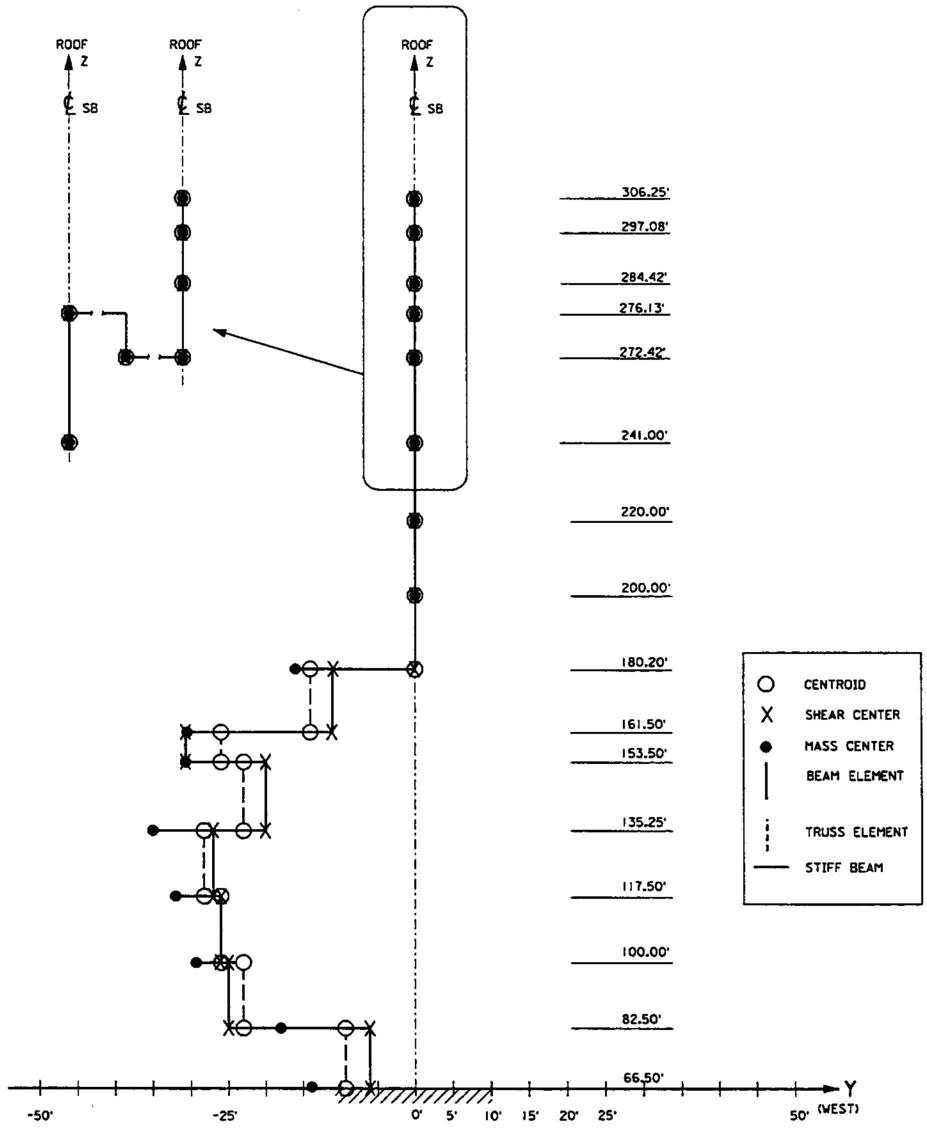


Figure 3.7.2-4 (Sheet 2 of 2)

Coupled Shield & Auxiliary Building
Lumped Mass Stick Model (East-West)

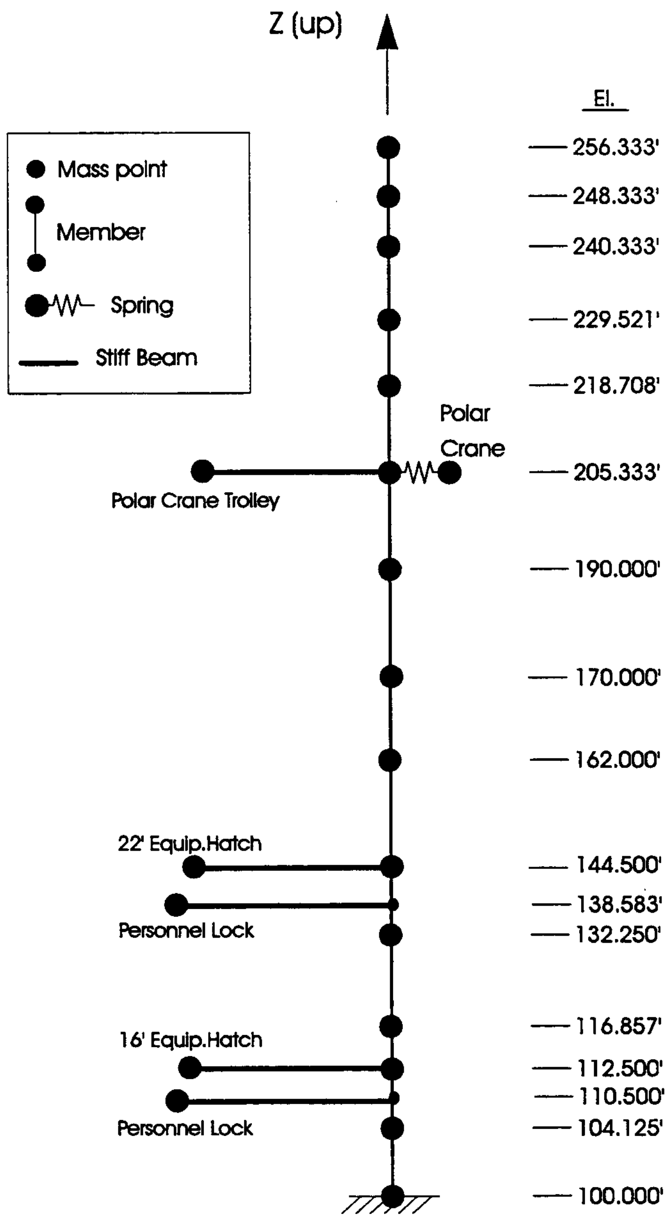


Figure 3.7.2-5

Steel Containment Vessel Lumped Mass Stick Model

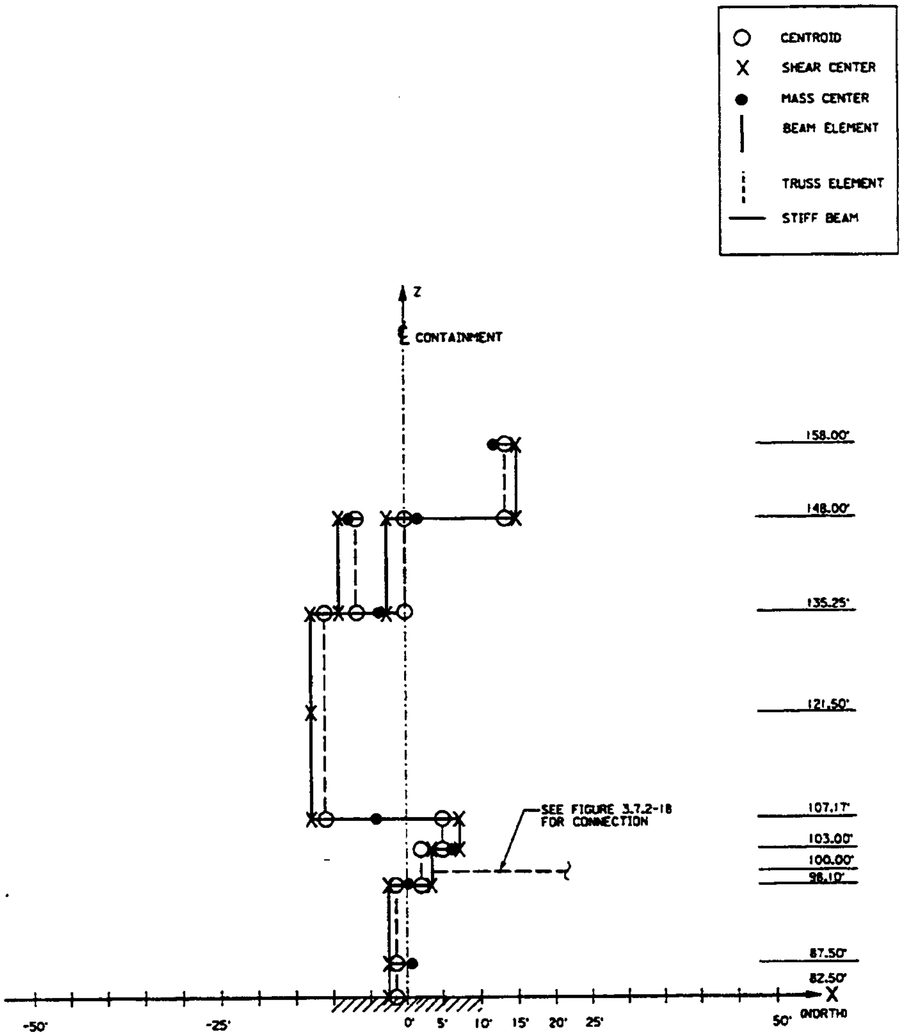


Figure 3.7.2-6 (Sheet 1 of 2)

Containment Internal Structure
Mass Stick Model (North-South)

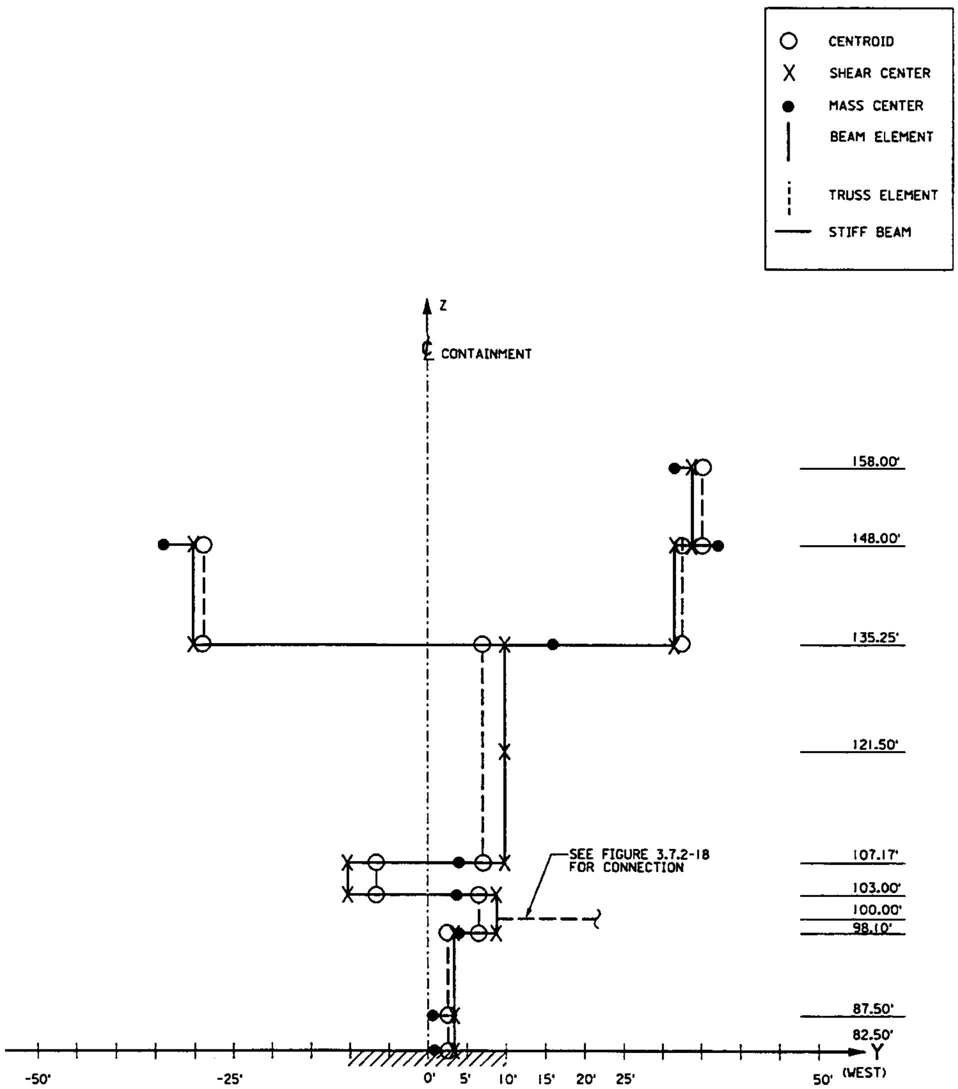


Figure 3.7.2-6 (Sheet 2 of 2)

Containment Internal Structure
Mass Stick Model (East-West)

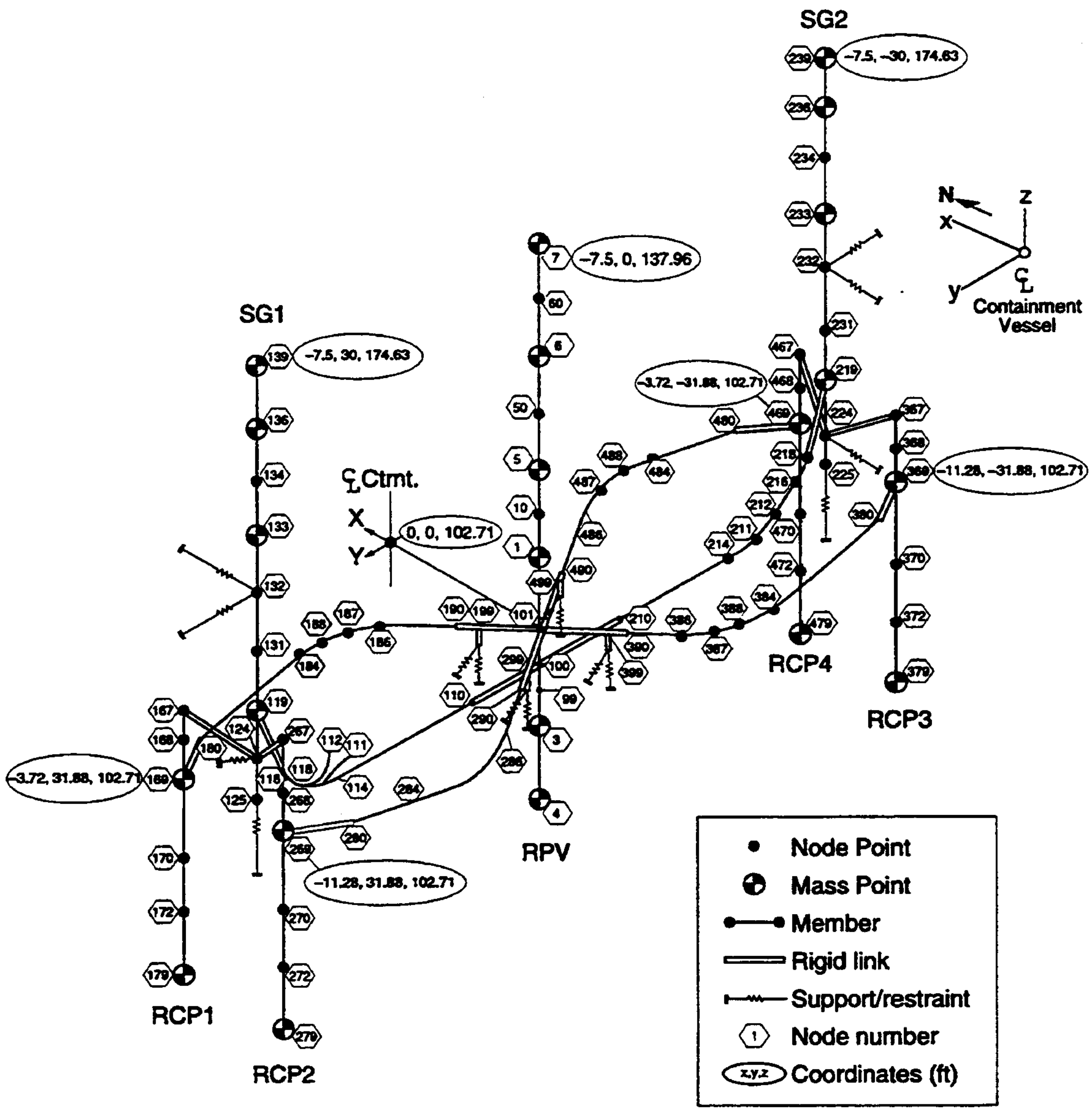


Figure 3.7.2-7

Reactor Coolant Loop Lumped Mass Stick Model

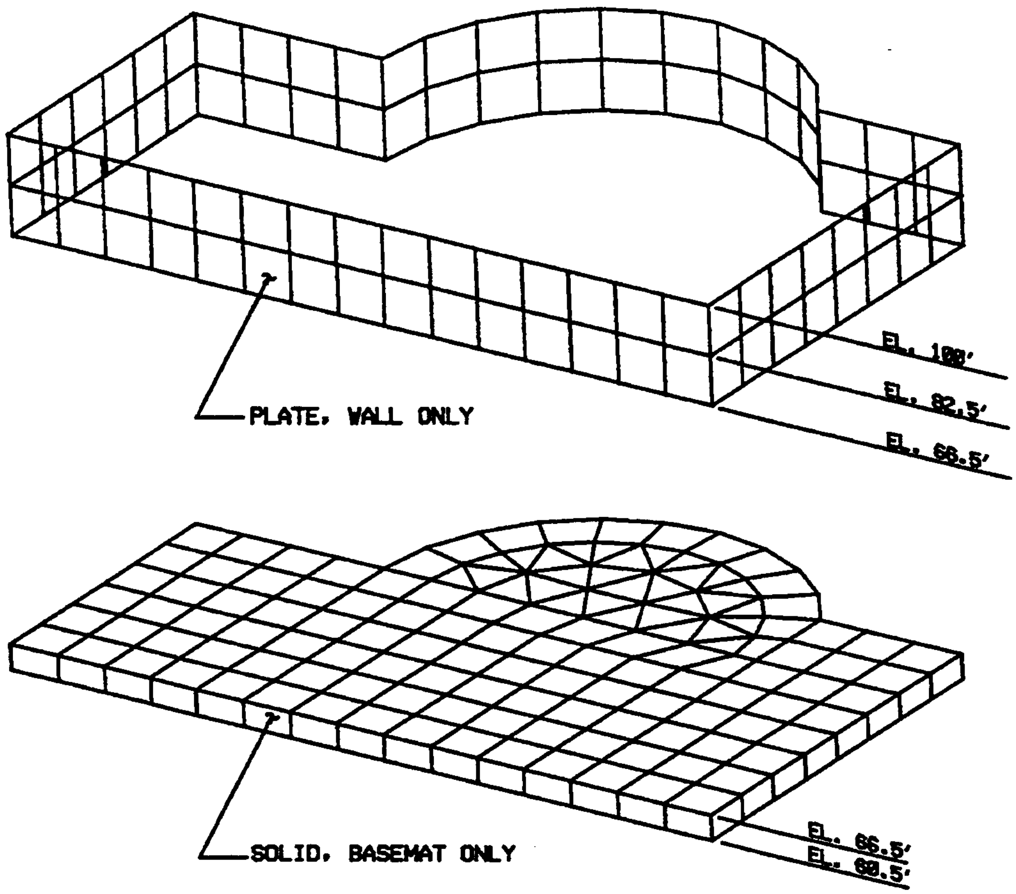
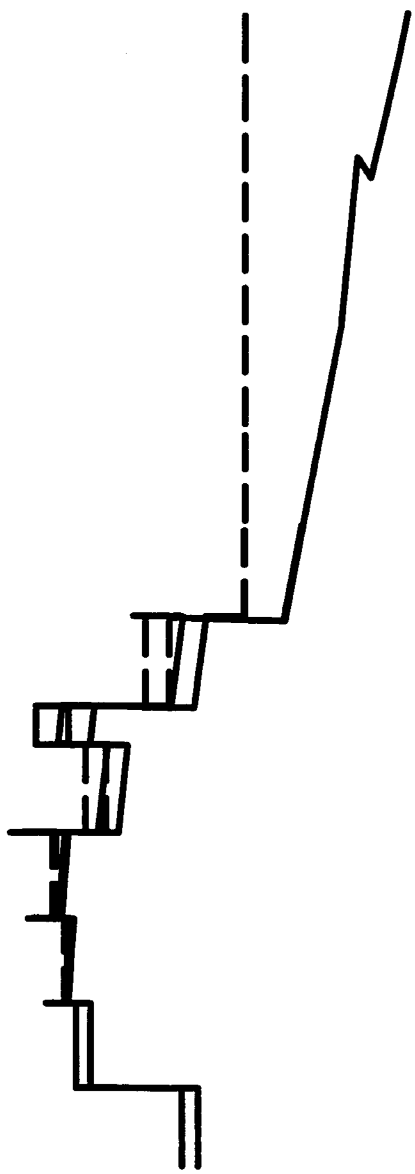


Figure 3.7.2-8

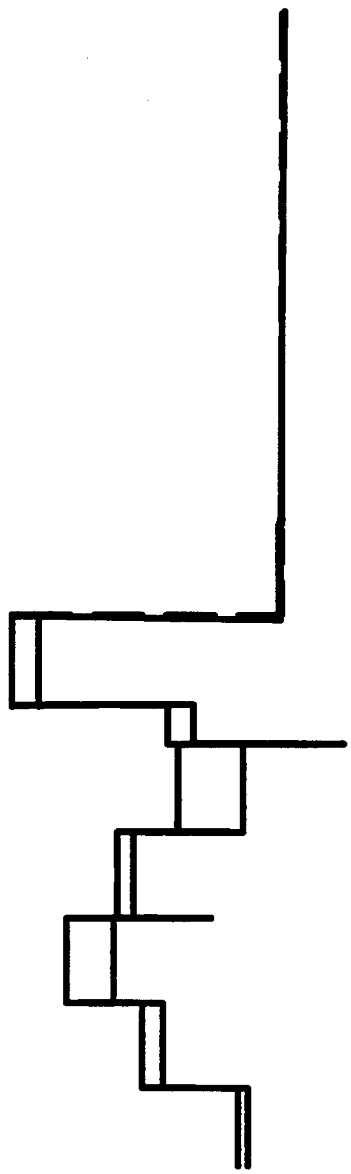
Foundation of the Seismic Analysis Model for the Nuclear Island



Mode Number: 14
Frequency: 4.35 Hz
Description: EW Translation

Figure 3.7.2-9 (Sheet 1 of 16)

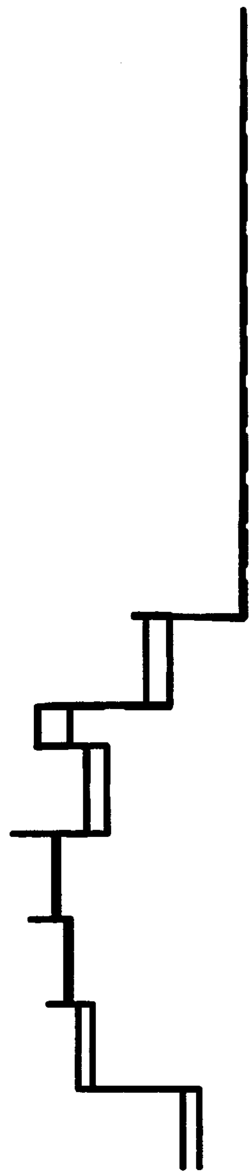
Coupled Shield and Auxiliary Buildings Modeshape Plots



Mode Number: 14
Frequency: 4.35 Hz
Description: NS Translation

Figure 3.7.2-9 (Sheet 2 of 16)

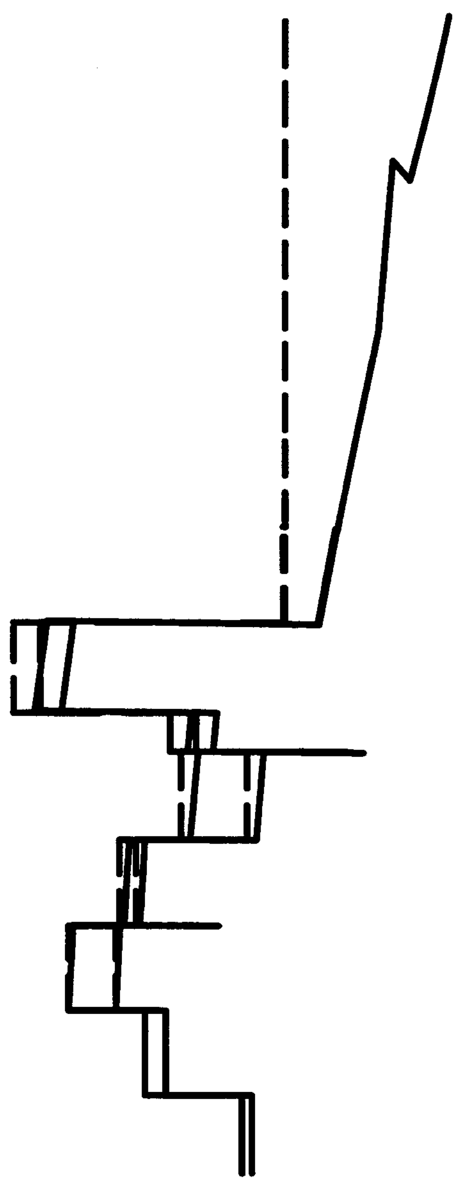
Coupled Shield and Auxiliary Buildings Modeshape Plots



Mode Number: 18
Frequency: 4.78 Hz
Description: EW Translation

Figure 3.7.2-9 (Sheet 3 of 16)

Coupled Shield and Auxiliary Buildings Modeshape Plots



Mode Number: 18
Frequency: 4.78 Hz
Description: NS Translation

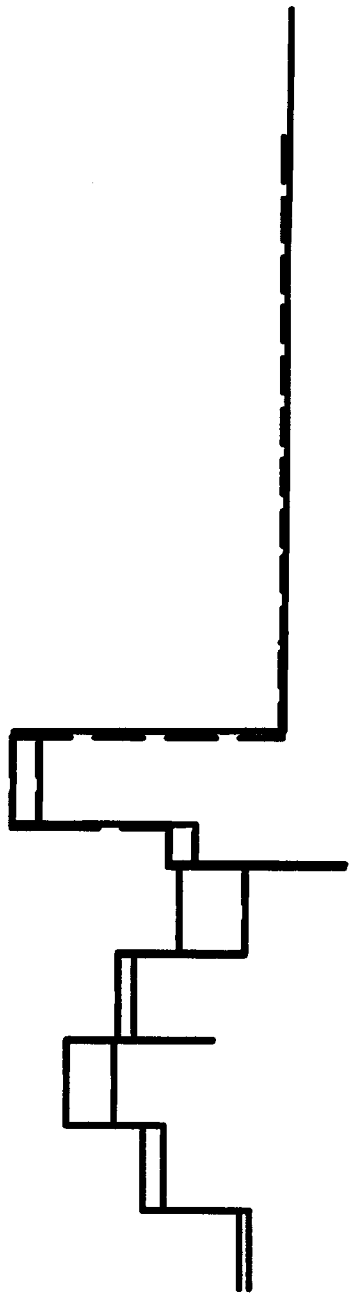
Figure 3.7.2-9 (Sheet 4 of 16)

Coupled Shield and Auxiliary Buildings Modeshape Plots

Mode Number: 22
Frequency: 6.77 Hz
Description: Vert. Translation (EW-Direction)

Figure 3.7.2-9 (Sheet 5 of 16)

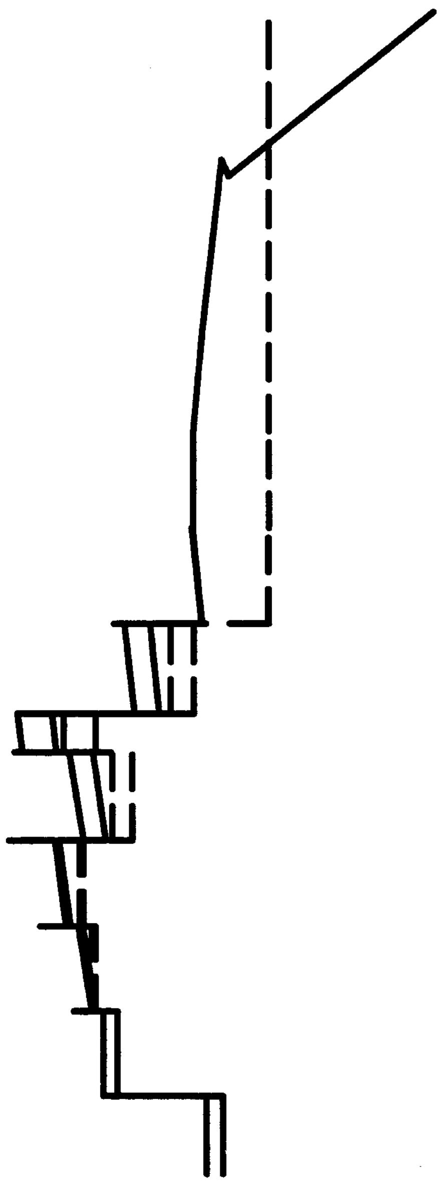
Coupled Shield and Auxiliary Buildings Modeshape Plots



Mode Number: 22
Frequency: 6.77 Hz
Description: Vert. Translation (NS-Direction)

Figure 3.7.2-9 (Sheet 6 of 16)

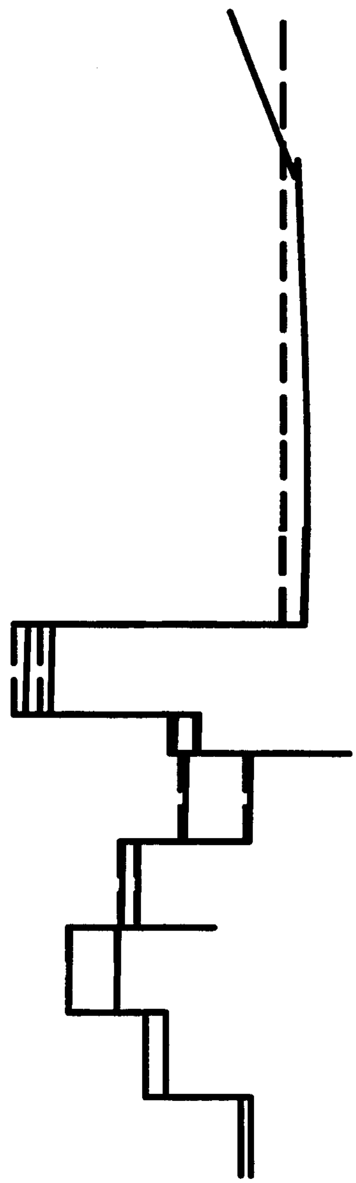
Coupled Shield and Auxiliary Buildings Modeshape Plots



Mode Number: 25
Frequency: 9.10 Hz
Description: EW Translation

Figure 3.7.2-9 (Sheet 7 of 16)

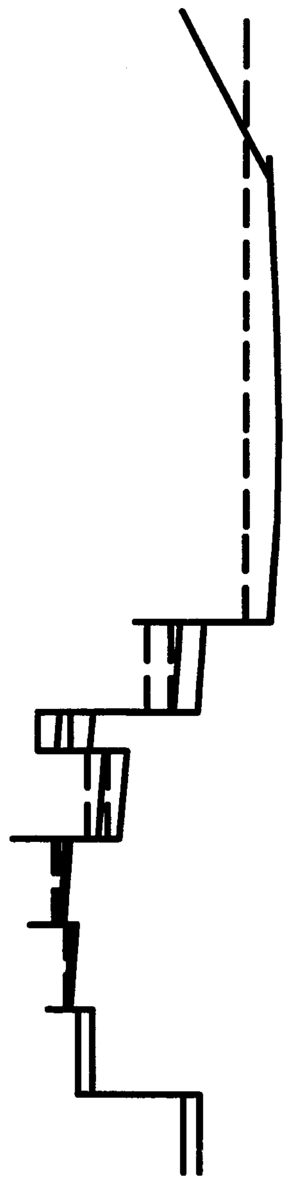
Coupled Shield and Auxiliary Buildings Modeshape Plots



Mode Number: 25
Frequency: 9.10 Hz
Description: NS Translation

Figure 3.7.2-9 (Sheet 8 of 16)

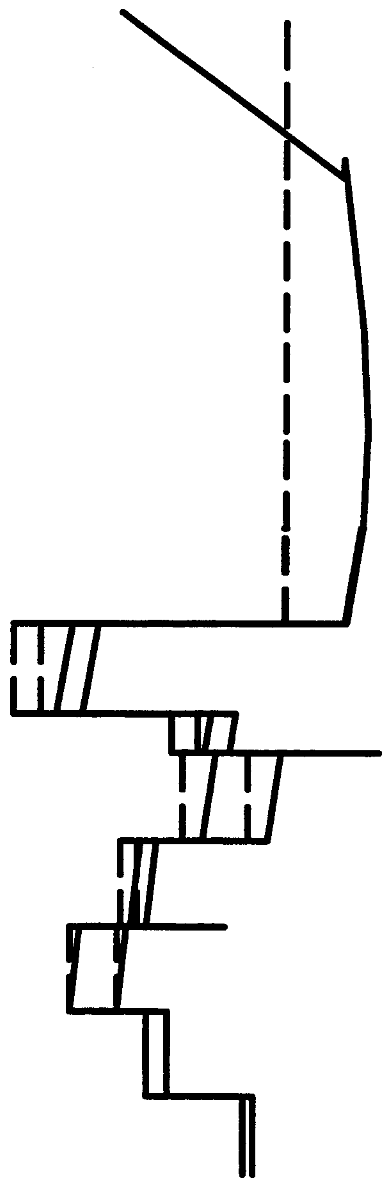
Coupled Shield and Auxiliary Buildings Modeshape Plots



Mode Number: 26
Frequency: 9.26 Hz
Description: EW Translation

Figure 3.7.2-9 (Sheet 9 of 16)

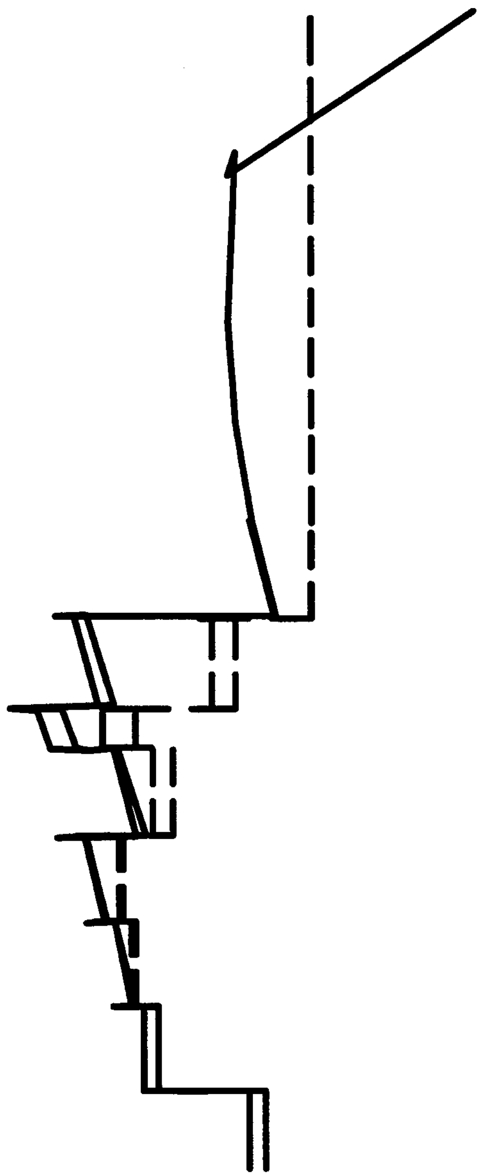
Coupled Shield and Auxiliary Buildings Modeshape Plots



Mode Number: 26
Frequency: 9.26 Hz
Description: NS Translation

Figure 3.7.2-9 (Sheet 10 of 16)

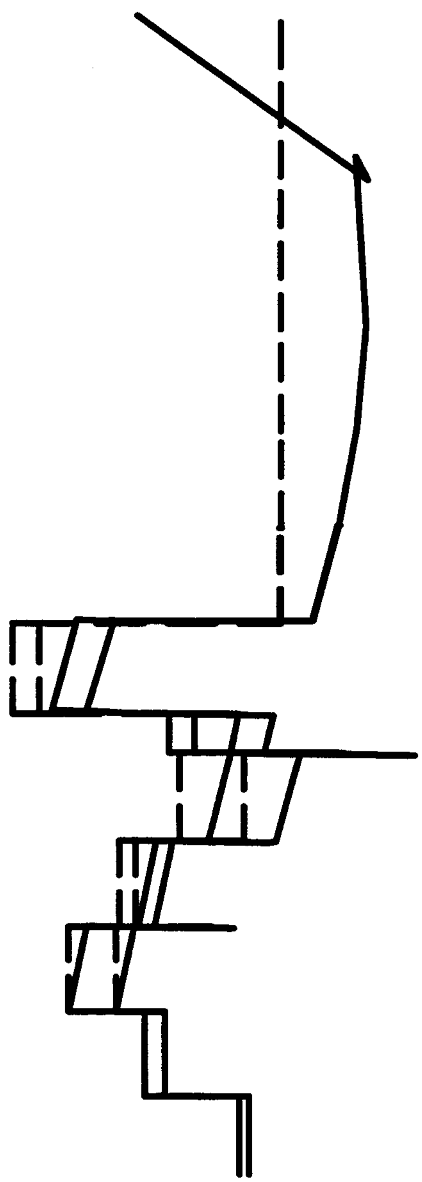
Coupled Shield and Auxiliary Buildings Modeshape Plots



Mode Number: 27
Frequency: 9.70 Hz
Description: EW Translation

Figure 3.7.2-9 (Sheet 11 of 16)

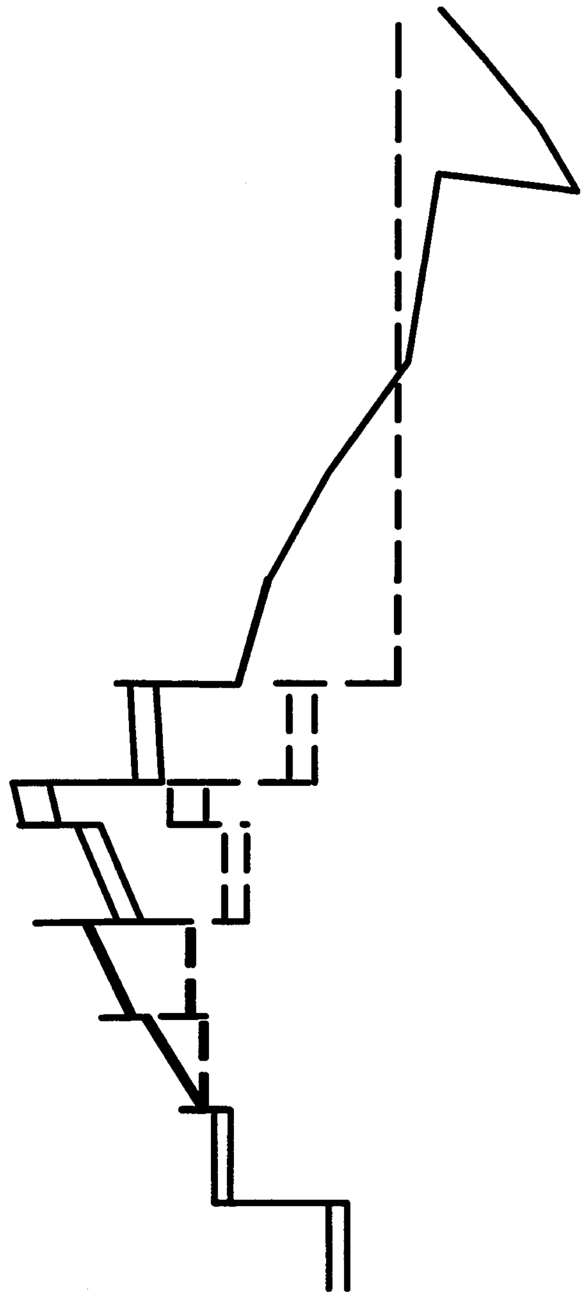
Coupled Shield and Auxiliary Buildings Modeshape Plots



Mode Number: 27
Frequency: 9.70 Hz
Description: NS Translation

Figure 3.7.2-9 (Sheet 12 of 16)

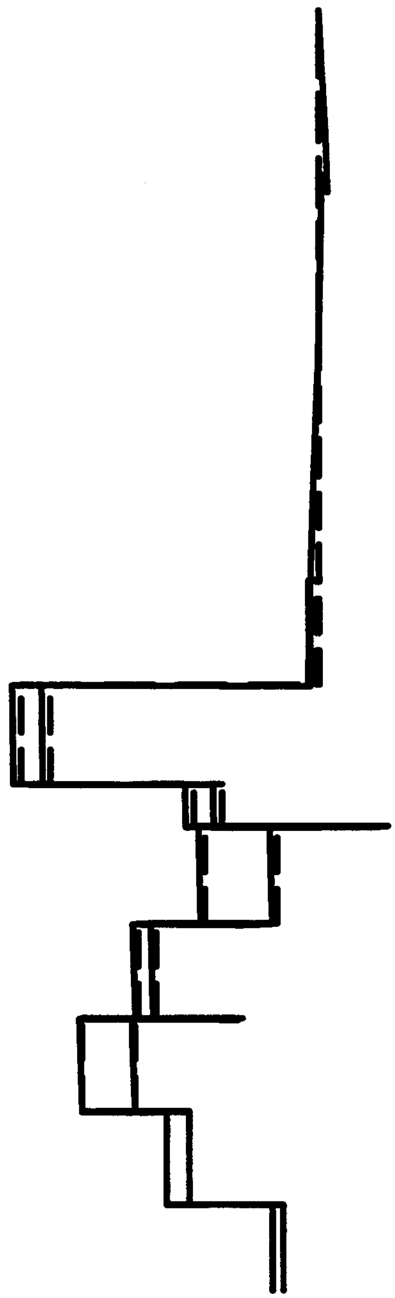
Coupled Shield and Auxiliary Buildings Modeshape Plots



Mode Number: 33
Frequency: 12.85 Hz
Description: EW Translation

Figure 3.7.2-9 (Sheet 13 of 16)

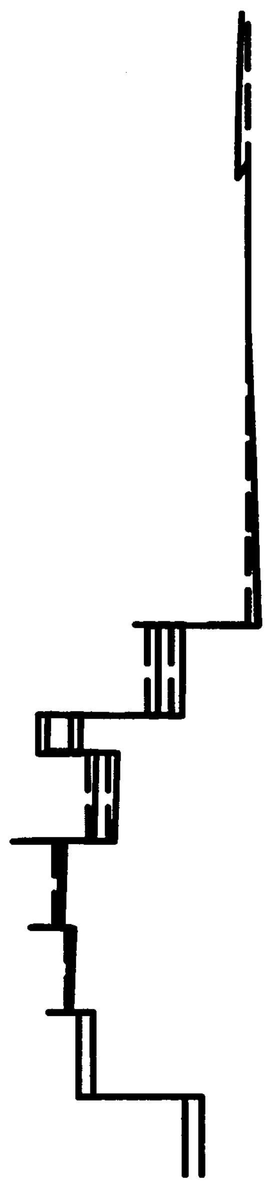
Coupled Shield and Auxiliary Buildings Modeshape Plots



Mode Number: 33
Frequency: 12.85 Hz
Description: NS Translation

Figure 3.7.2-9 (Sheet 14 of 16)

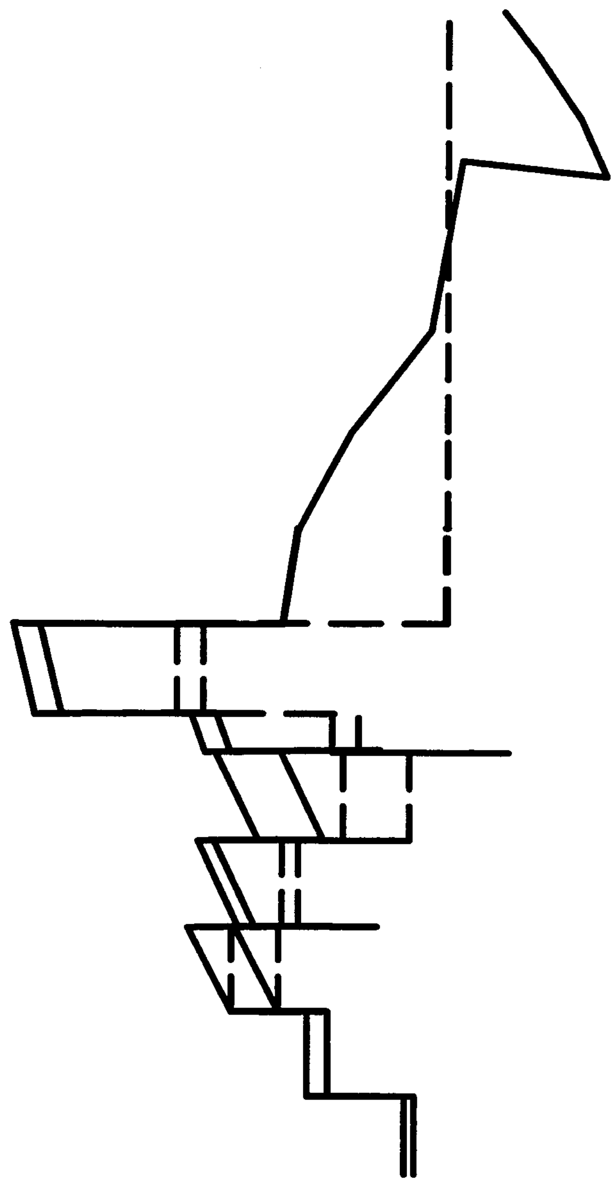
Coupled Shield and Auxiliary Buildings Modeshape Plots



Mode Number: 35
Frequency: 13.56 Hz
Description: EW Translation

Figure 3.7.2-9 (Sheet 15 of 16)

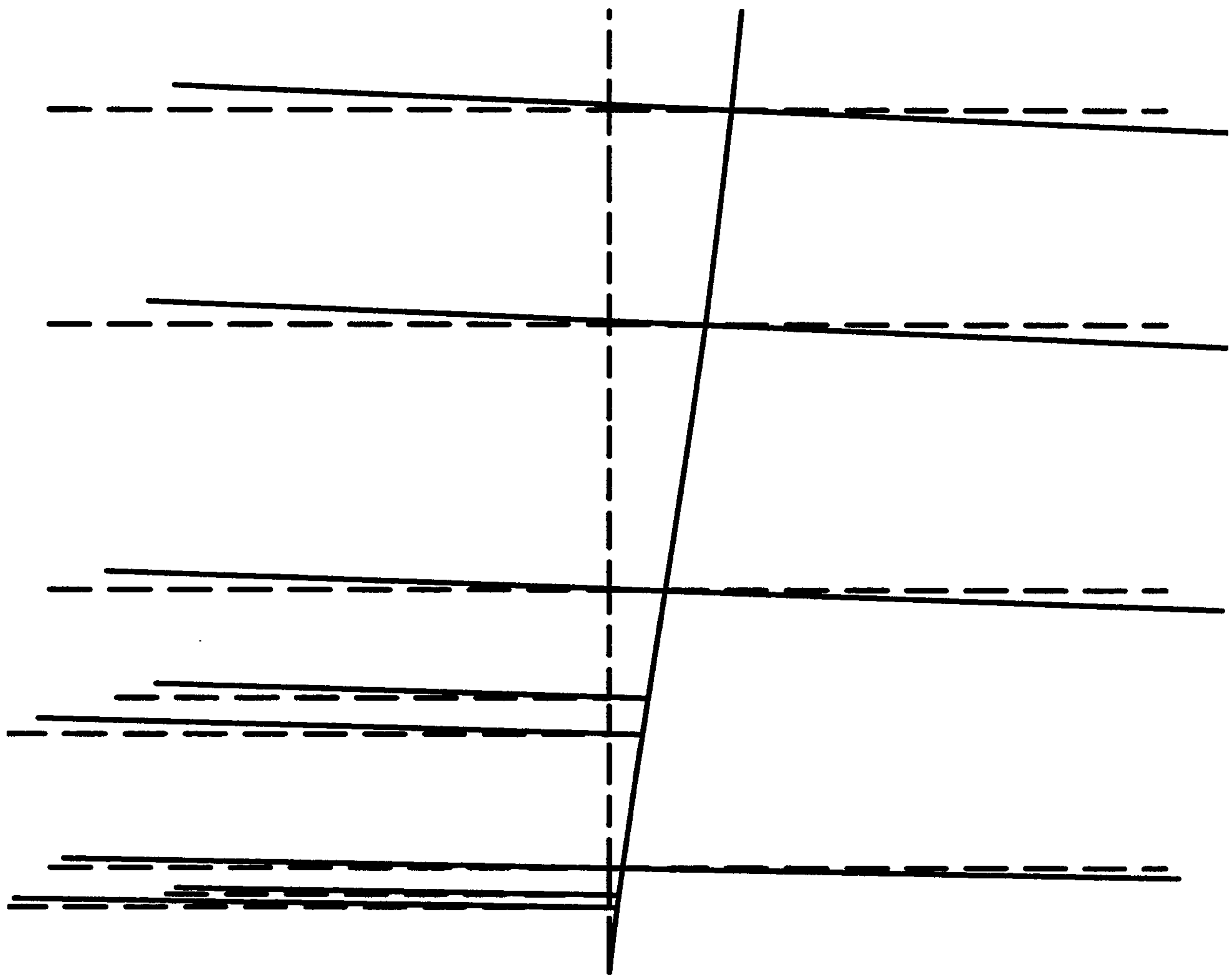
Coupled Shield and Auxiliary Buildings Modeshape Plots



Mode Number: 35
Frequency: 13.56 Hz
Description: NS Translation

Figure 3.7.2-9 (Sheet 16 of 16)

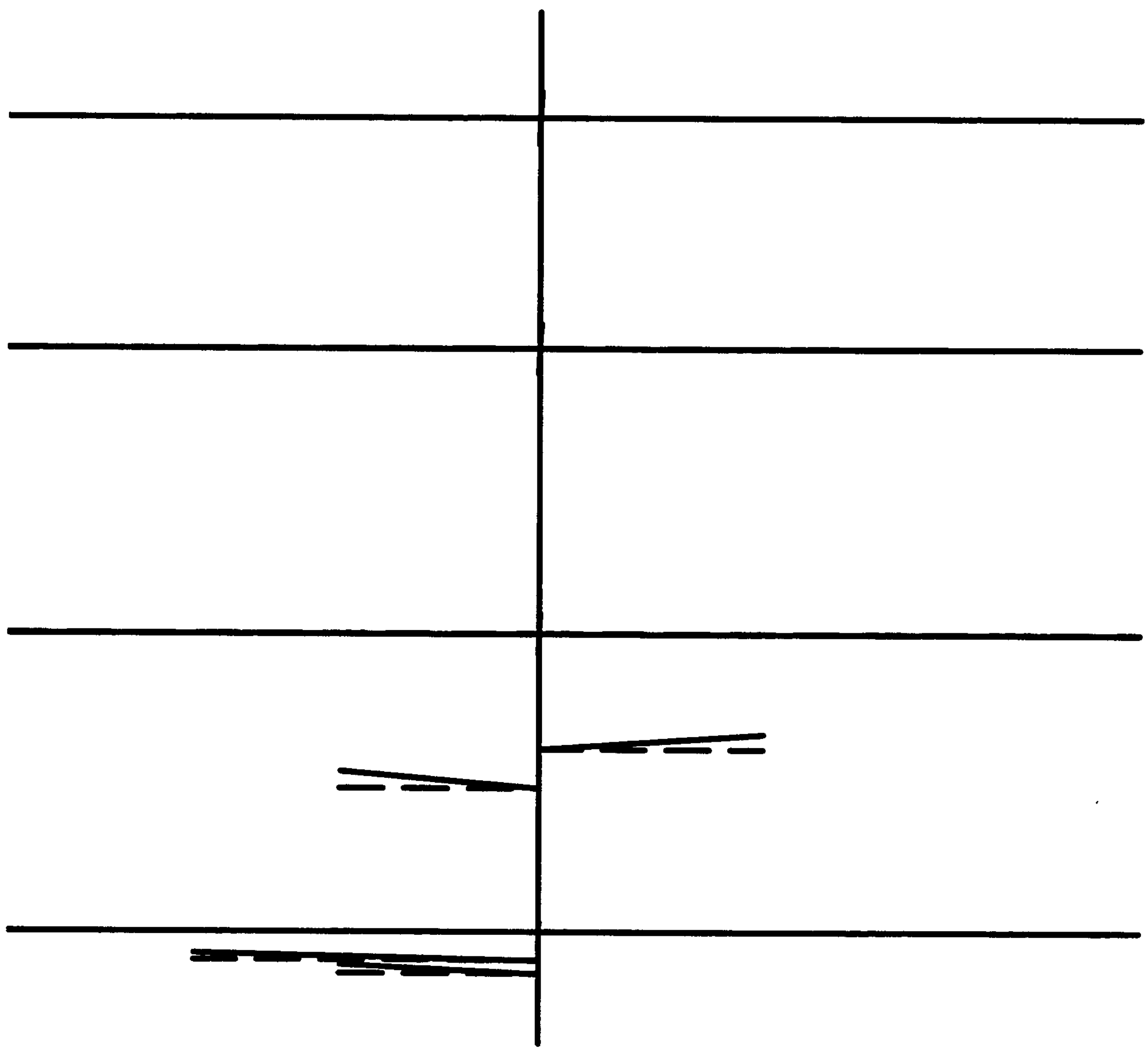
Coupled Shield and Auxiliary Buildings Modeshape Plots



Mode Number: 23
Frequency: 7.37 Hz
Description: EW Translation

Figure 3.7.2-10 (Sheet 1 of 4)

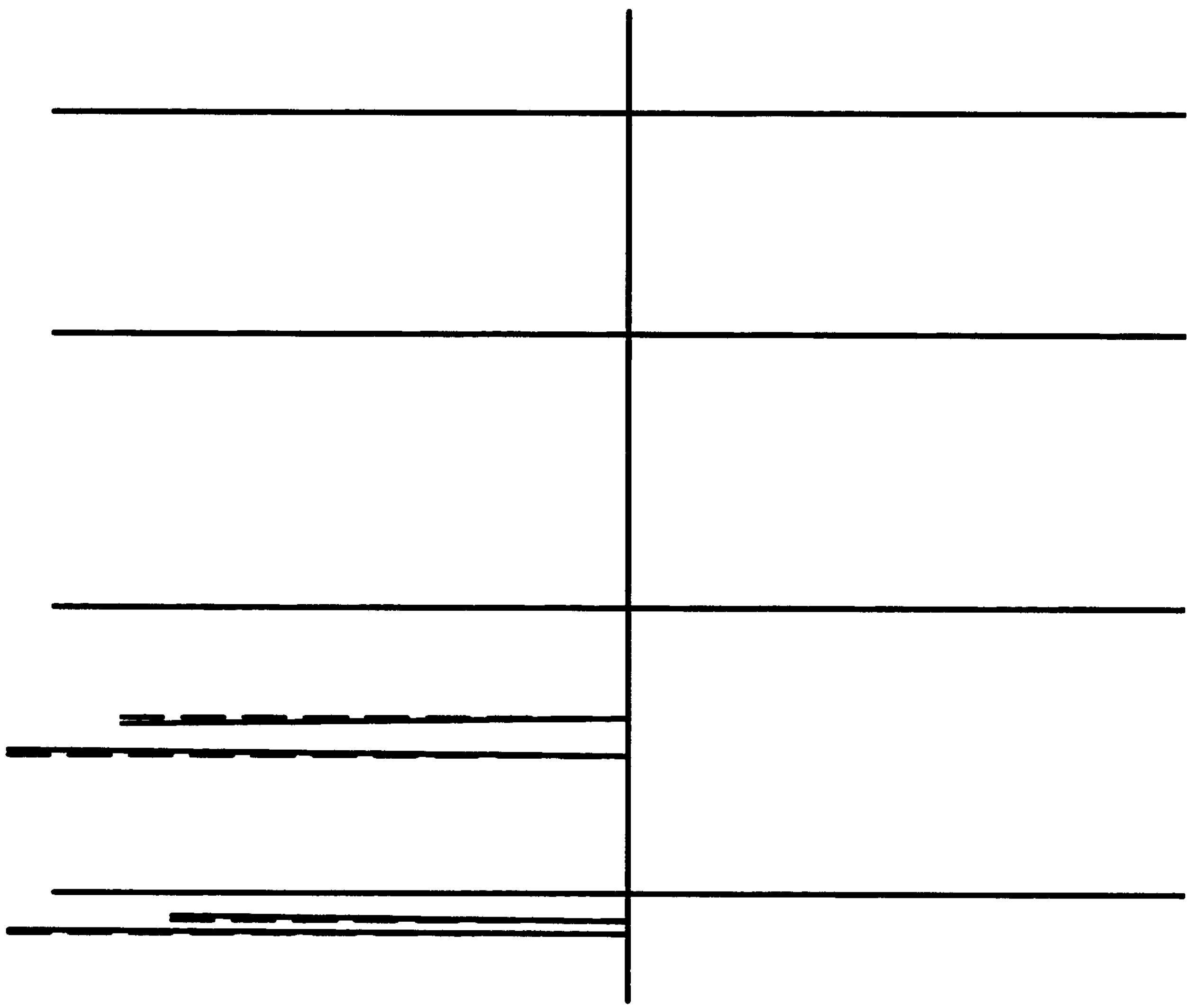
Steel Containment Vessel Modeshape Plots



Mode Number: 23
Frequency: 7.37 Hz
Description: NS Translation

Figure 3.7.2-10 (Sheet 2 of 4)

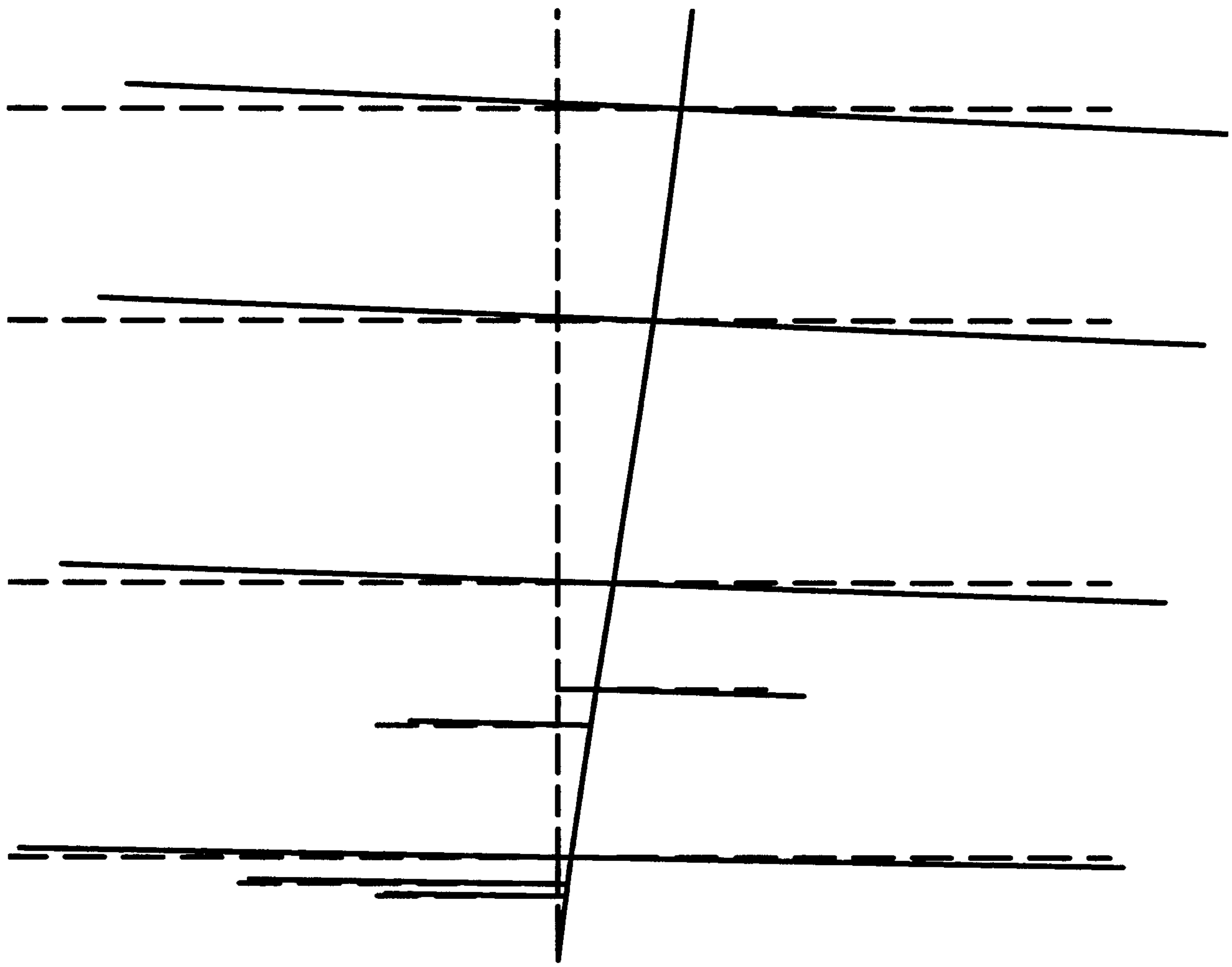
Steel Containment Vessel Modeshape Plots



Mode Number: 24
Frequency: 8.00 Hz
Description: EW Translation

Figure 3.7.2-10 (Sheet 3 of 4)

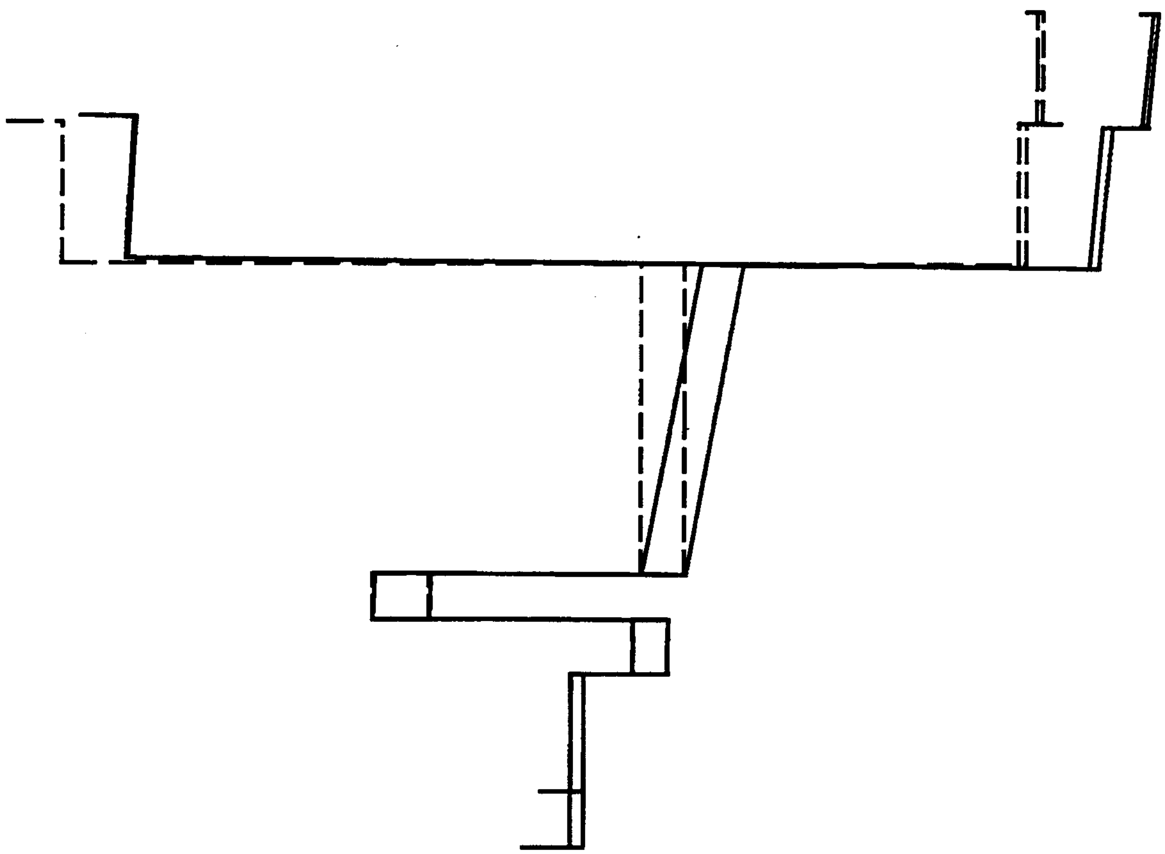
Steel Containment Vessel Modeshape Plots



Mode Number: 24
Frequency: 8.00 Hz
Description: NS Translation

Figure 3.7.2-10 (Sheet 4 of 4)

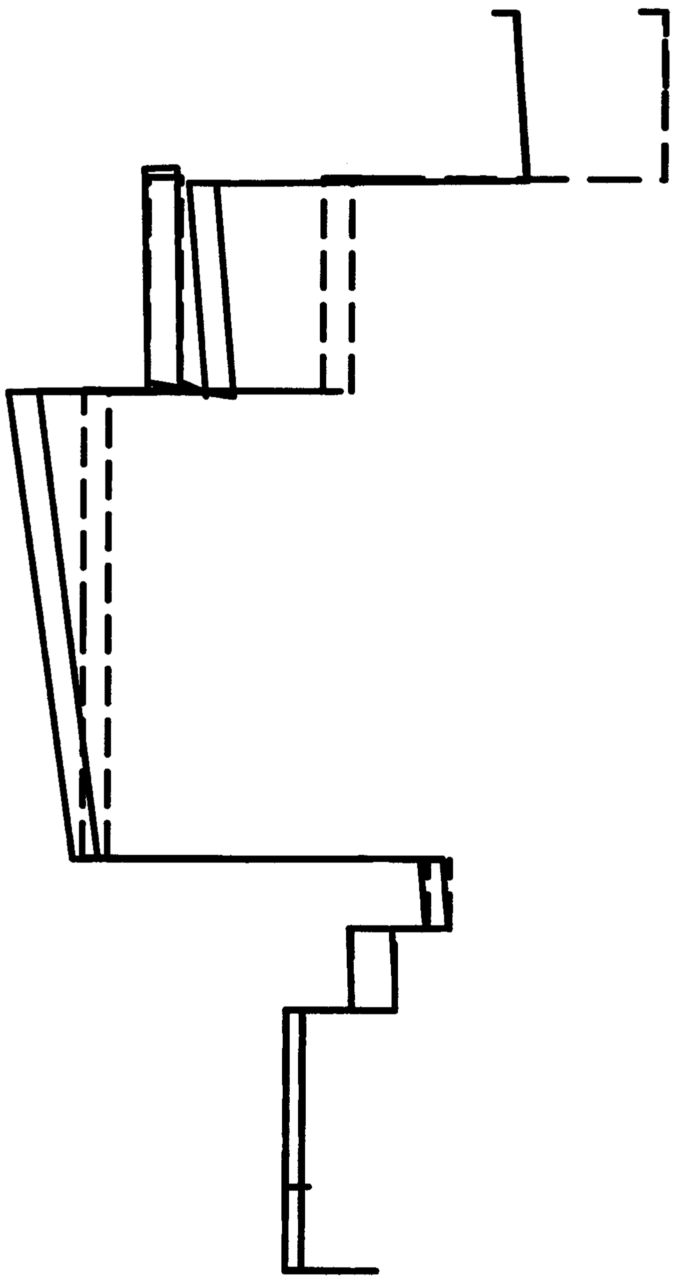
Steel Containment Vessel Modeshape Plots



Mode Number: 32
Frequency: 12.35 Hz
Description: EW Translation

Figure 3.7.2-11 (Sheet 1 of 12)

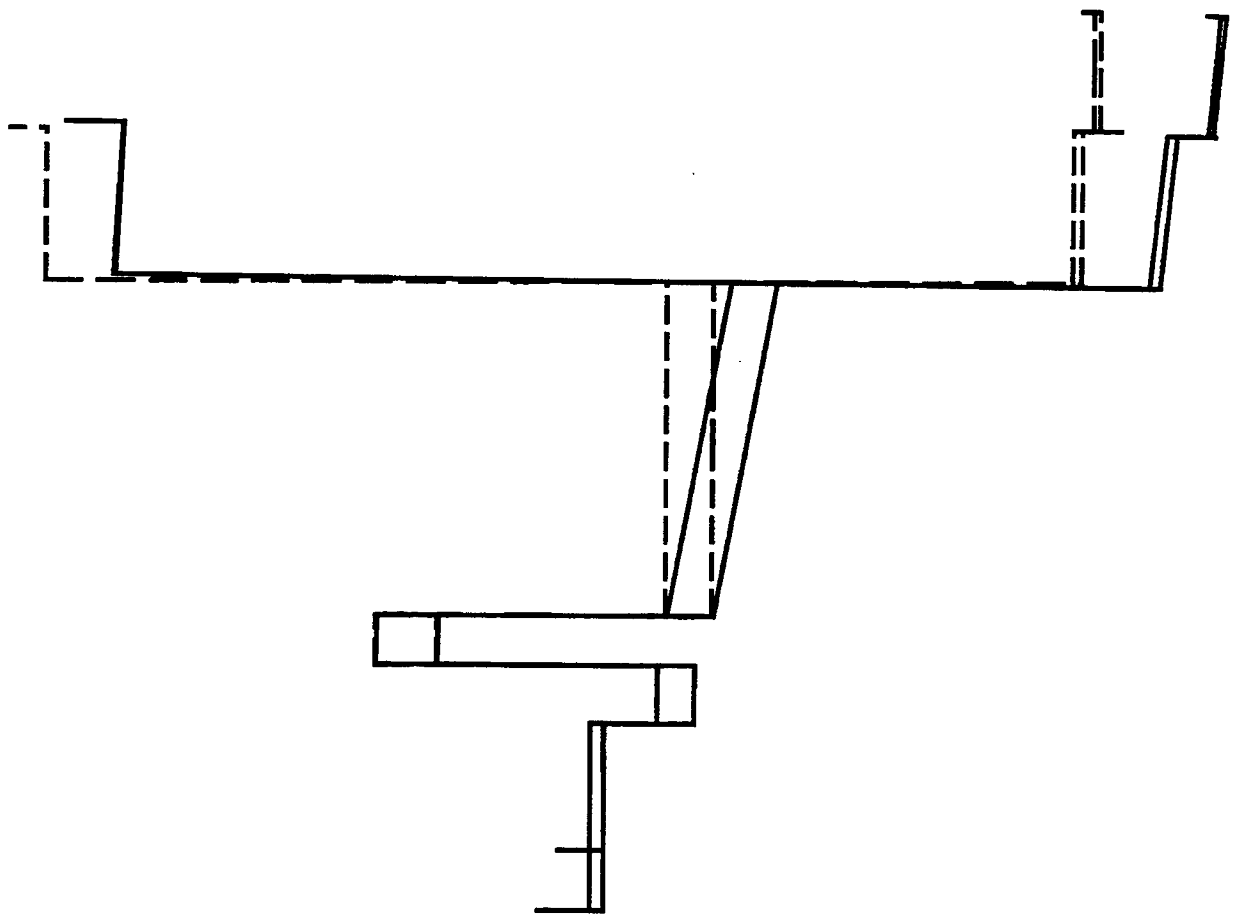
Containment Internal Structures Without RCL Modeshape Plots



Mode Number: 32
Frequency: 12.35 Hz
Description: NS Translation

Figure 3.7.2-11 (Sheet 2 of 12)

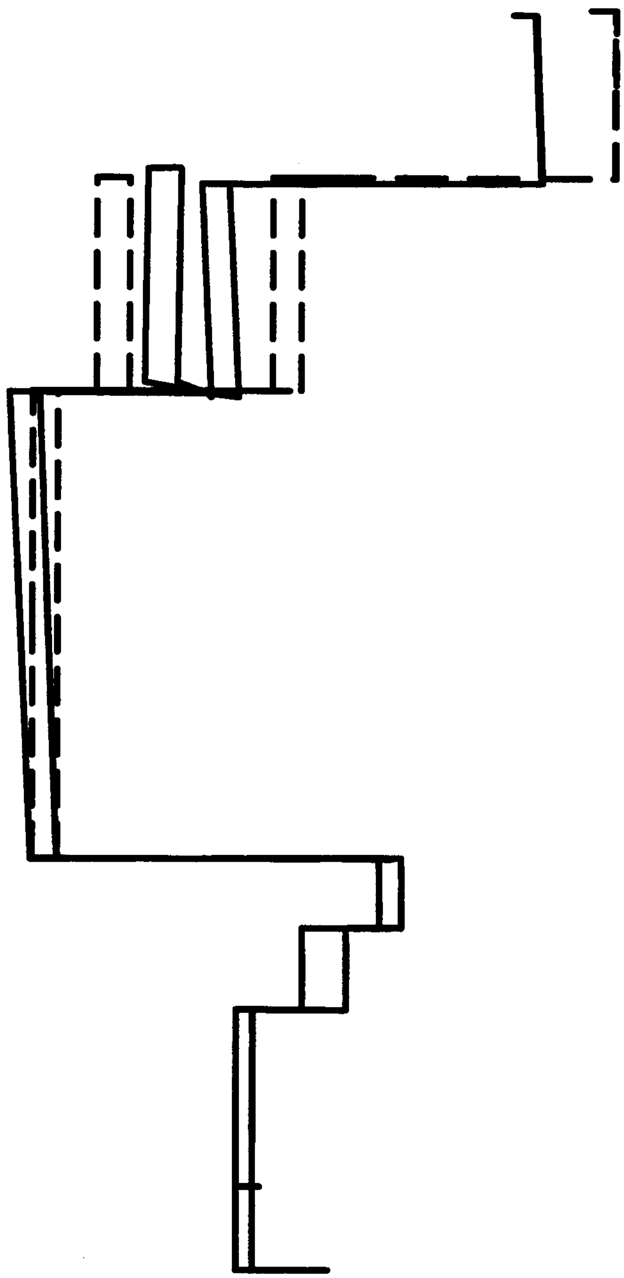
Containment Internal Structures Without RCL Modeshape Plots



Mode Number: 34
Frequency: 12.99 Hz
Description: EW Translation

Figure 3.7.2-11 (Sheet 3 of 12)

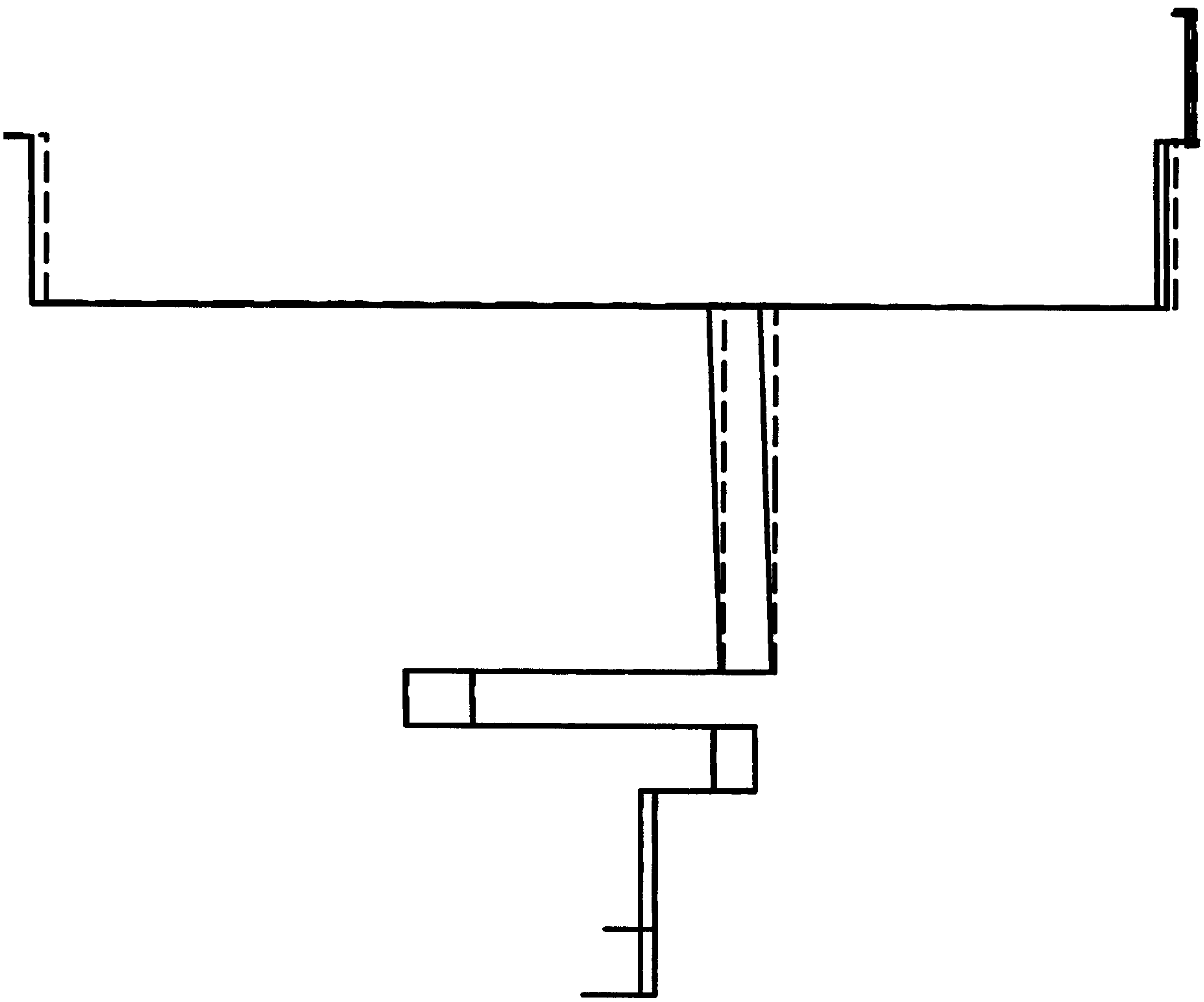
Containment Internal Structures Without RCL Modeshape Plots



Mode Number: 34
Frequency: 12.99 Hz
Description: NS Translation

Figure 3.7.2-11 (Sheet 4 of 12)

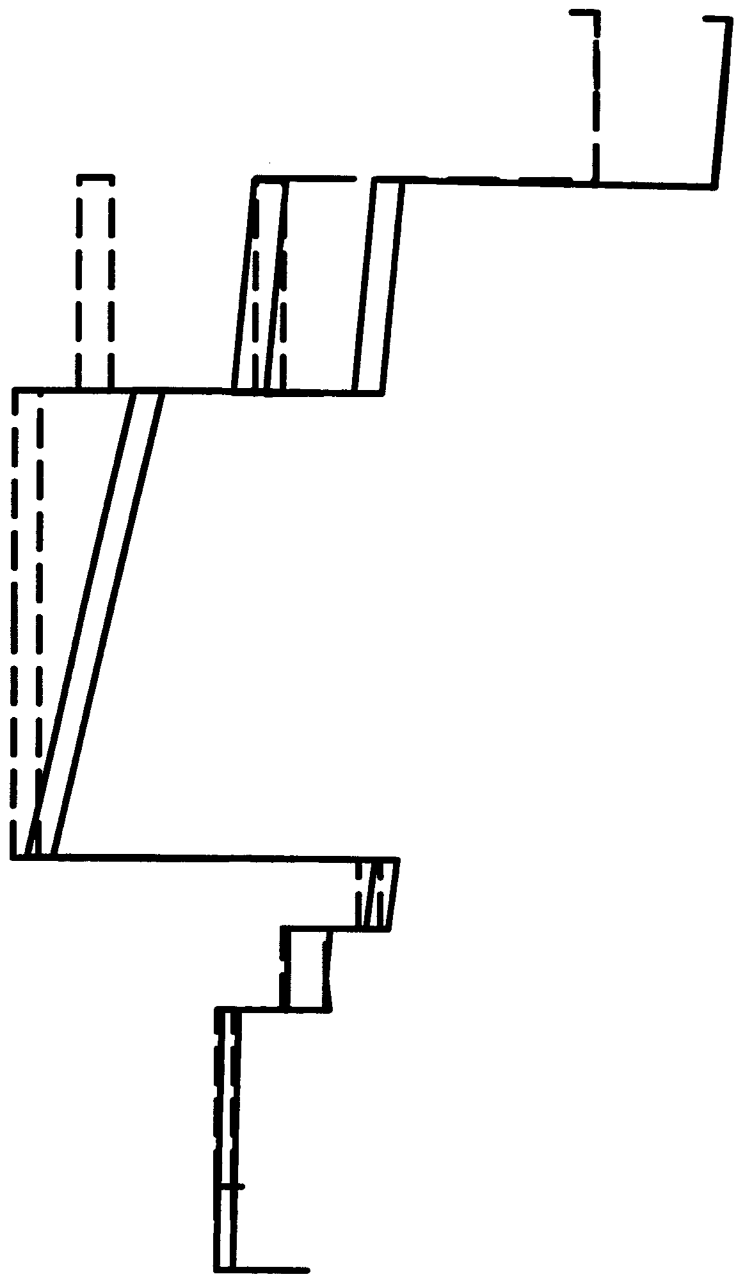
Containment Internal Structures Without RCL Modeshape Plots



Mode Number: 36
Frequency: 13.64 Hz
Description: EW Translation

Figure 3.7.2-11 (Sheet 5 of 12)

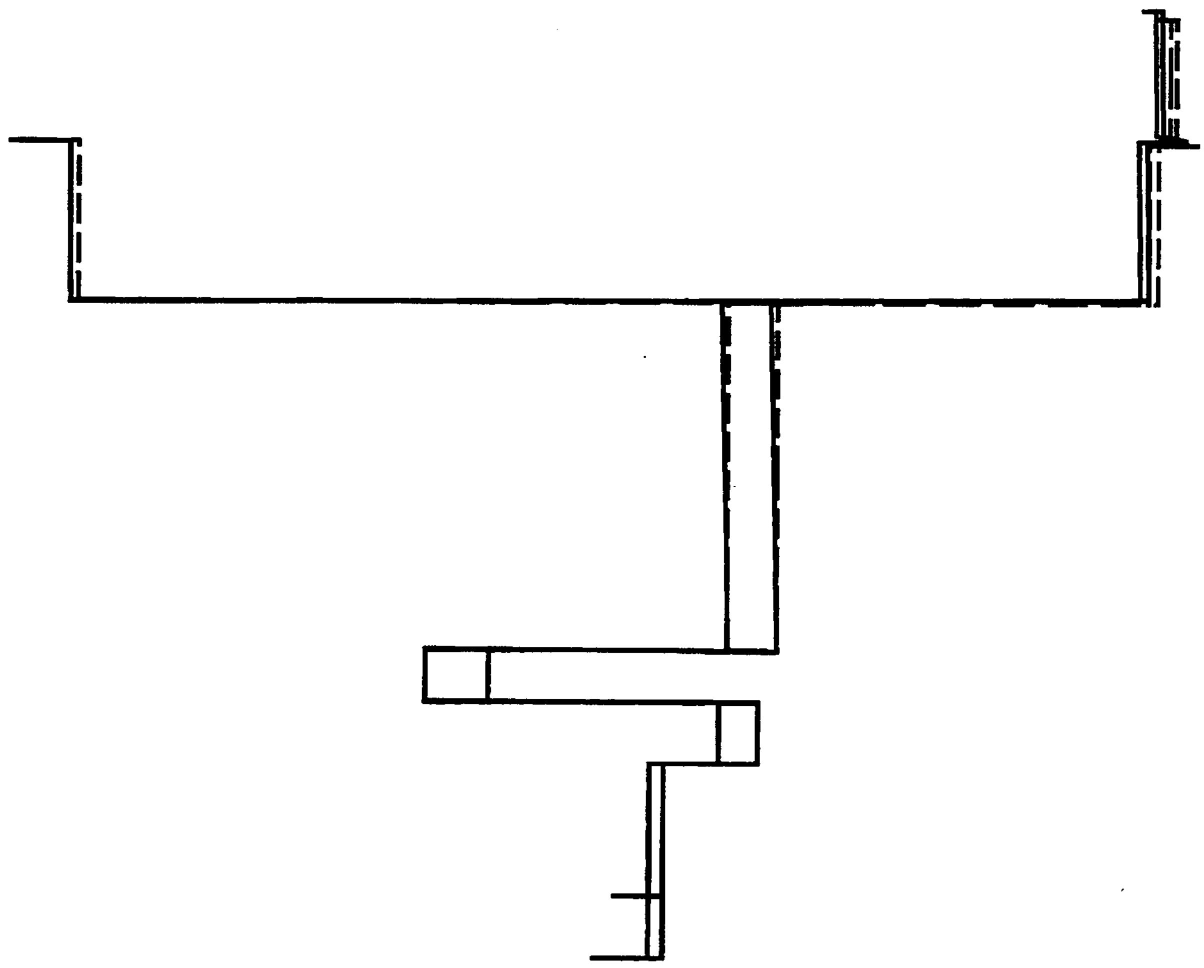
Containment Internal Structures Without RCL Modeshape Plots



Mode Number: 36
Frequency: 13.64 Hz
Description: NS Translation

Figure 3.7.2-11 (Sheet 6 of 12)

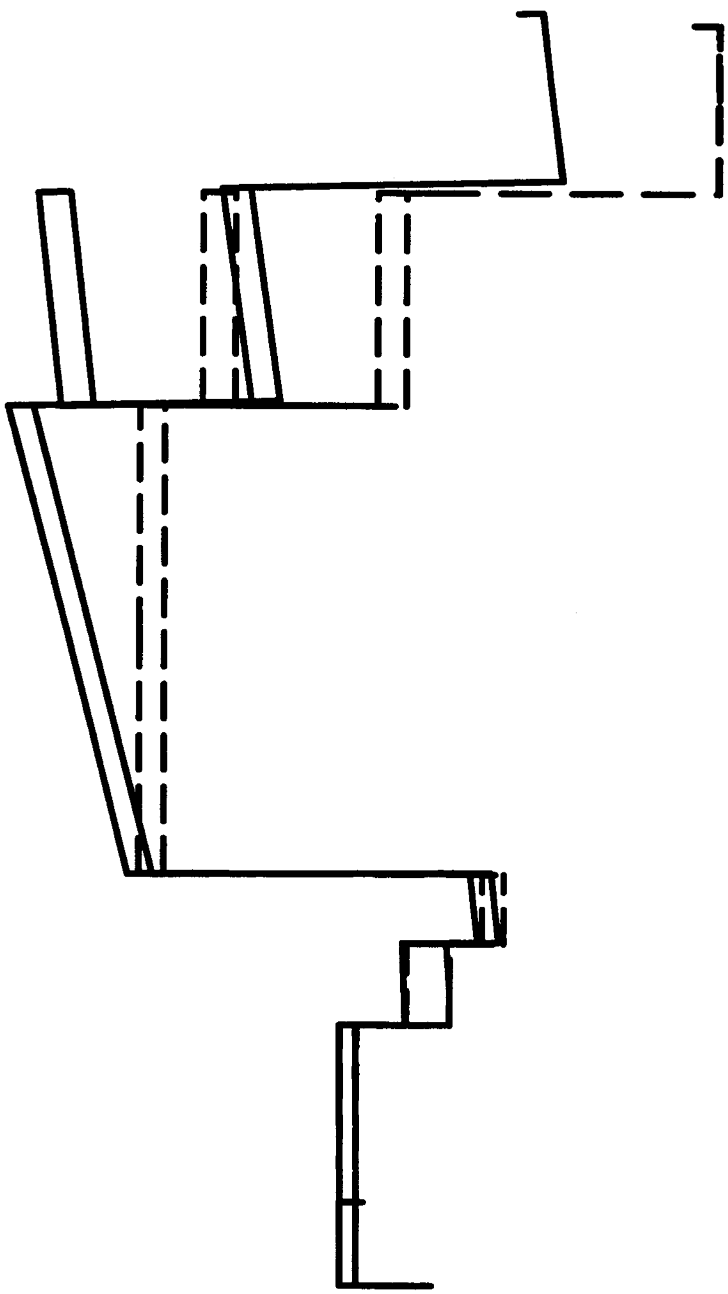
Containment Internal Structures Without RCL Modeshape Plots



Mode Number: 37
Frequency: 14.36 Hz
Description: EW Translation

Figure 3.7.2-11 (Sheet 7 of 12)

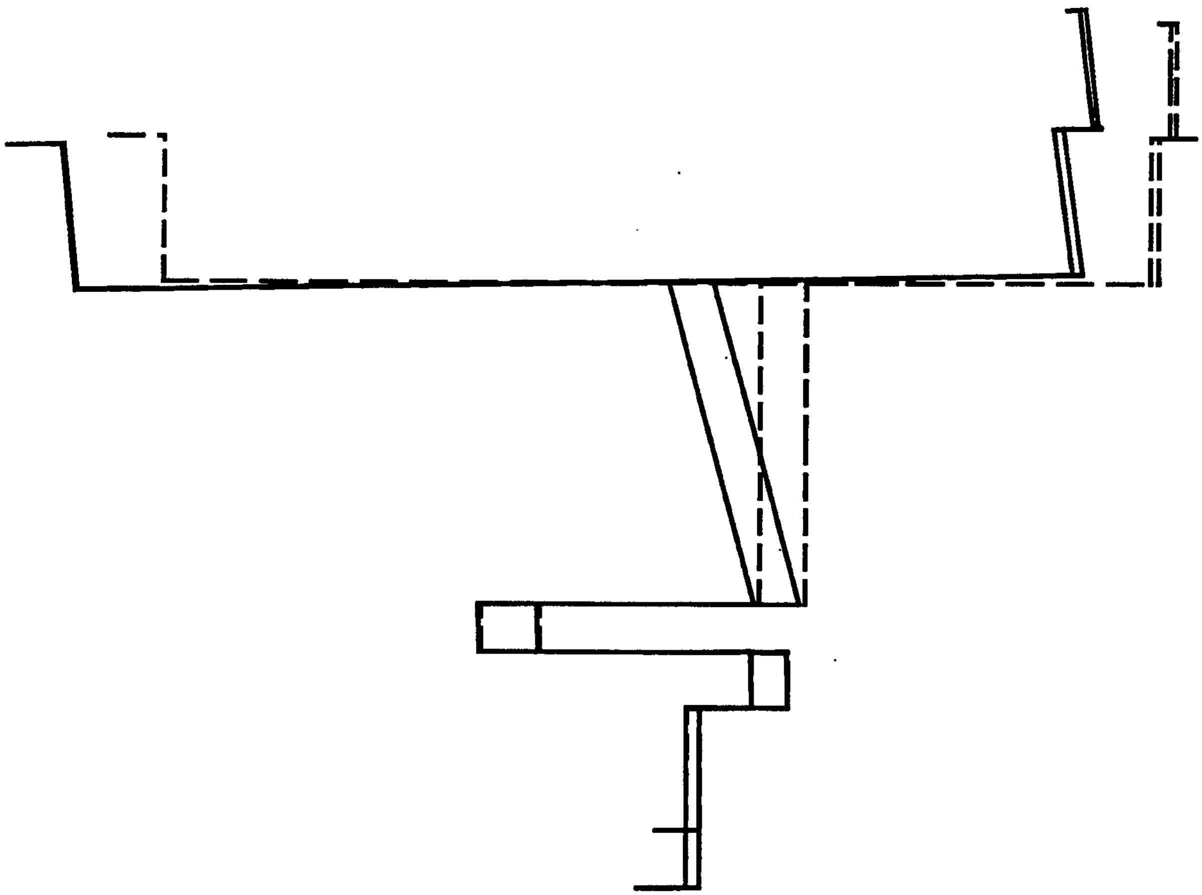
Containment Internal Structures Without RCL Modeshape Plots



Mode Number: 37
Frequency: 14.36 Hz
Description: NS Translation

Figure 3.7.2-11 (Sheet 8 of 12)

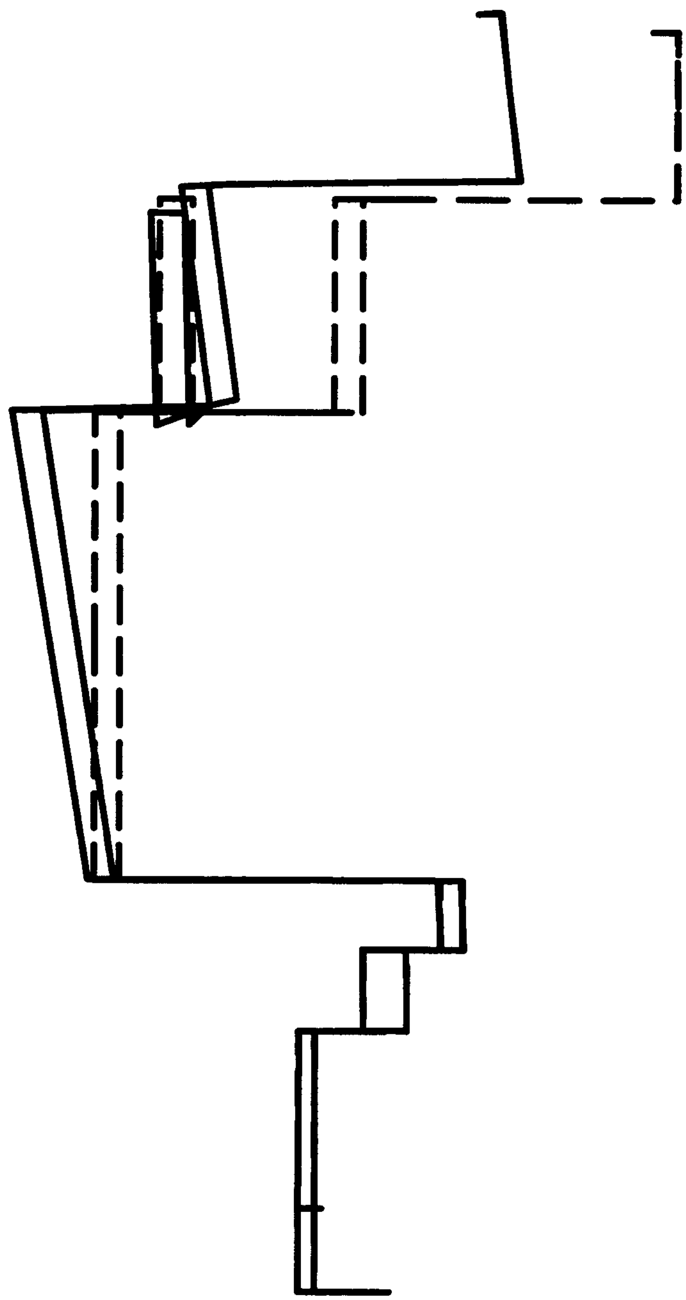
Containment Internal Structures Without RCL Modeshape Plots



Mode Number: 39
Frequency: 14.85 Hz
Description: EW Translation

Figure 3.7.2-11 (Sheet 9 of 12)

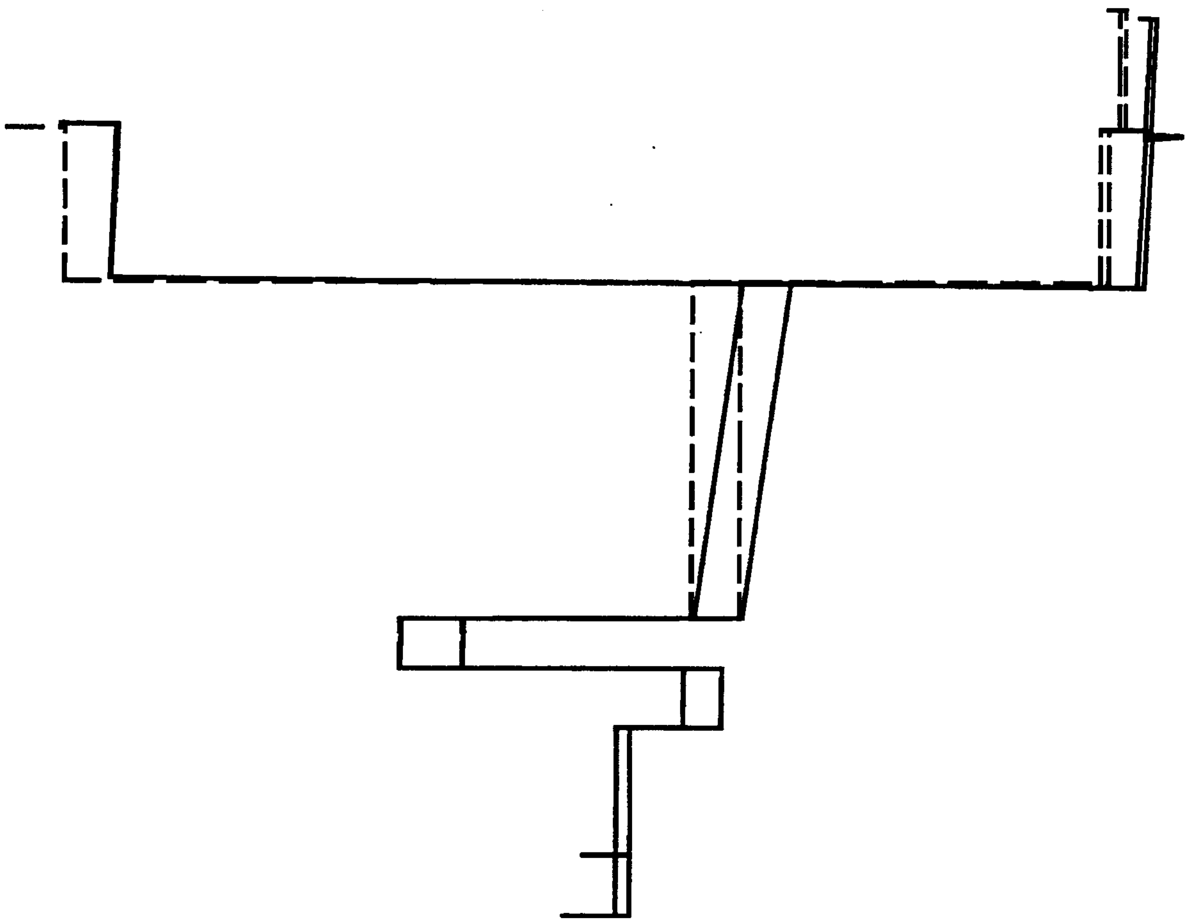
Containment Internal Structures Without RCL Modeshape Plots



Mode Number: 39
Frequency: 14.85 Hz
Description: NS Translation

Figure 3.7.2-11 (Sheet 10 of 12)

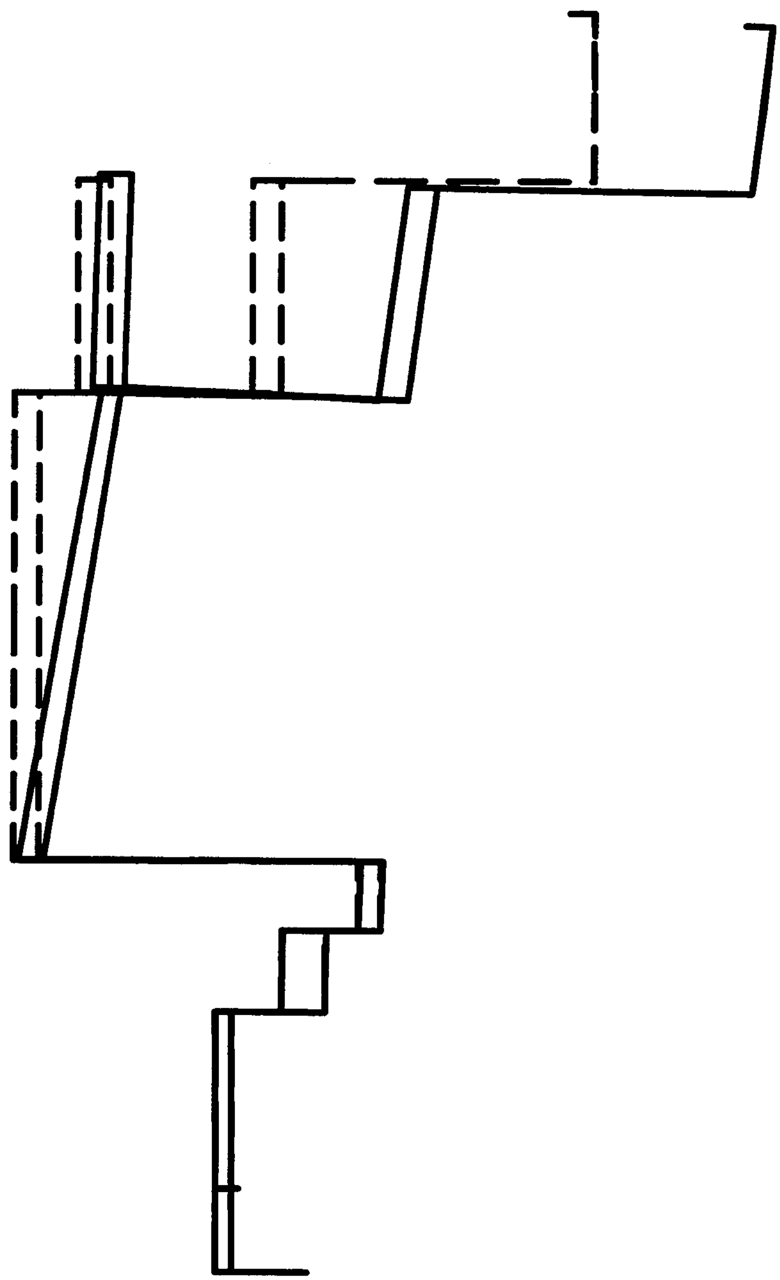
Containment Internal Structures Without RCL Modeshape Plots



Mode Number: 40
Frequency: 14.97 Hz
Description: EW Translation

Figure 3.7.2-11 (Sheet 11 of 12)

Containment Internal Structures Without RCL Modeshape Plots



Mode Number: 40
Frequency: 14.97 Hz
Description: NS Translation

Figure 3.7.2-11 (Sheet 12 of 12)

Containment Internal Structures Without RCL Modeshape Plots

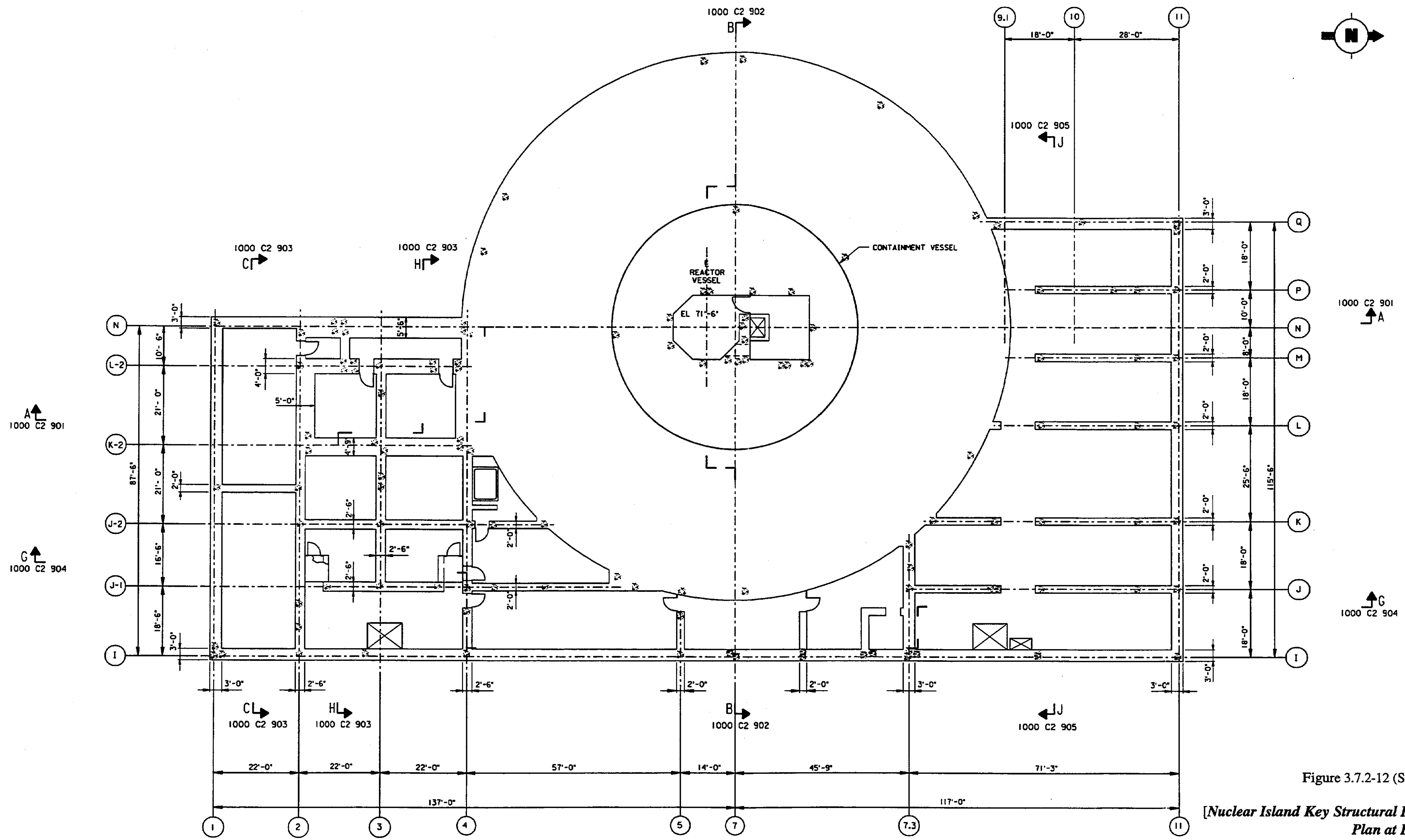


Figure 3.7.2-12 (Sheet 1 of 12)

*[Nuclear Island Key Structural Dimensions Plan at El. 66'-6"]**

*NRC Staff approval is required prior to implementing a change in this information; see DCD Section 3.5.

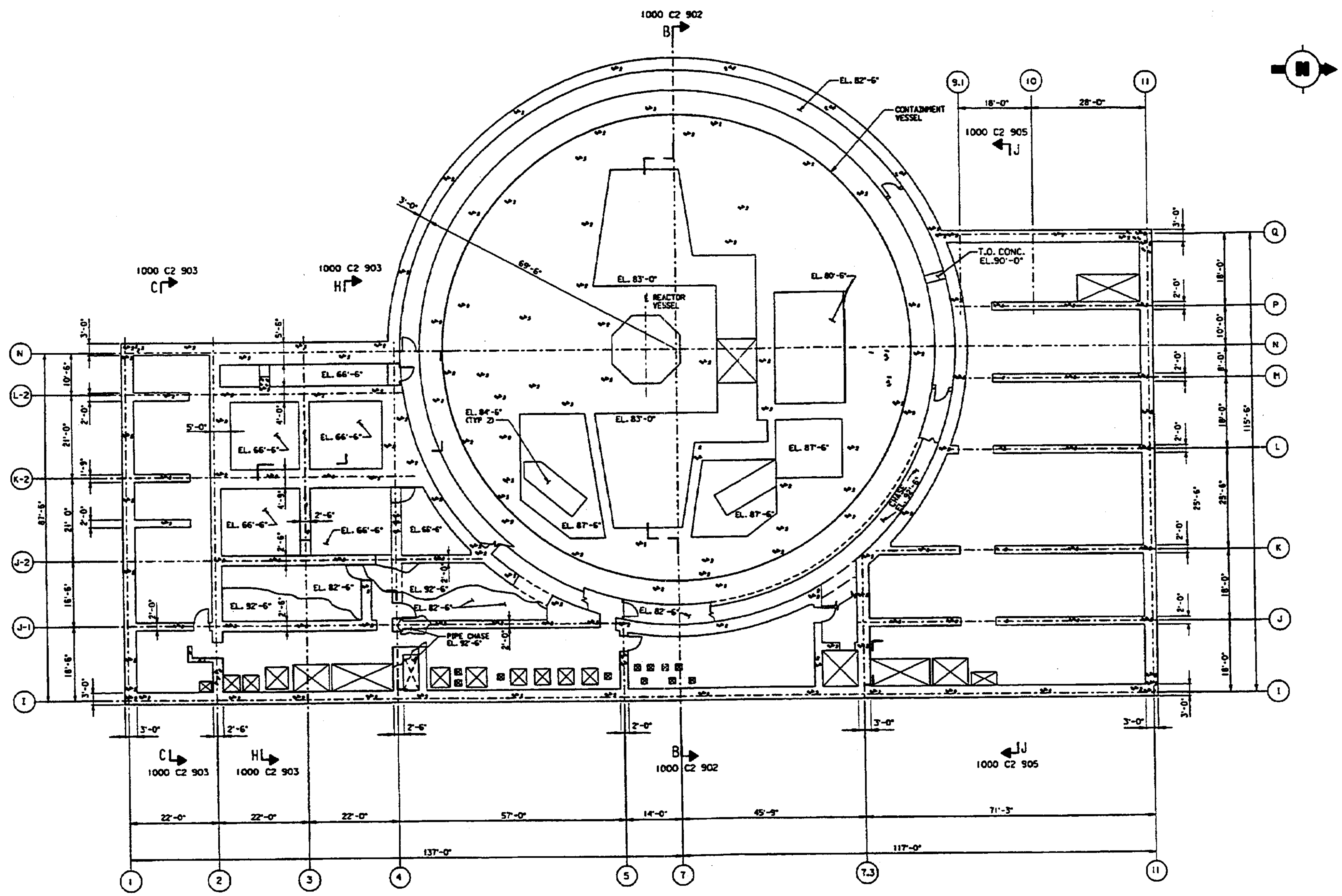


Figure 3.7.2-12 (Sheet 2 of 12)

*[Nuclear Island Key Structural Dimensions Plan at El.82'-6"]**

*NRC Staff approval is required prior to implementing a change in this information; see DCD Section 3.5.

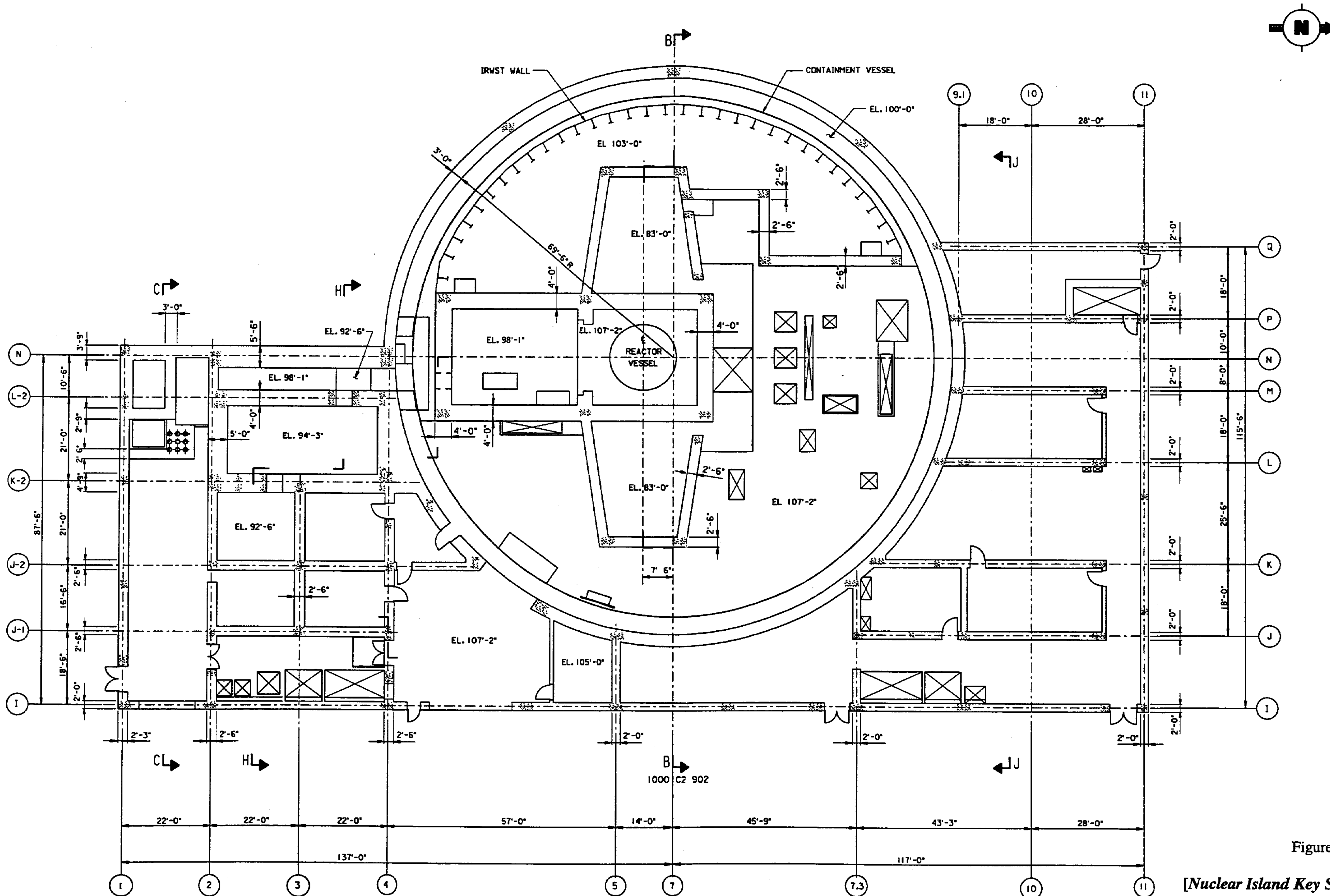


Figure 3.7.2-12 (Sheet 3 of 12)

*[Nuclear Island Key Structural Dimensions Plan at Elevation 100'-0" & 107'-2"]**

*NRC Staff approval is required prior to implementing a change in this information; see DCD Section 3.5.

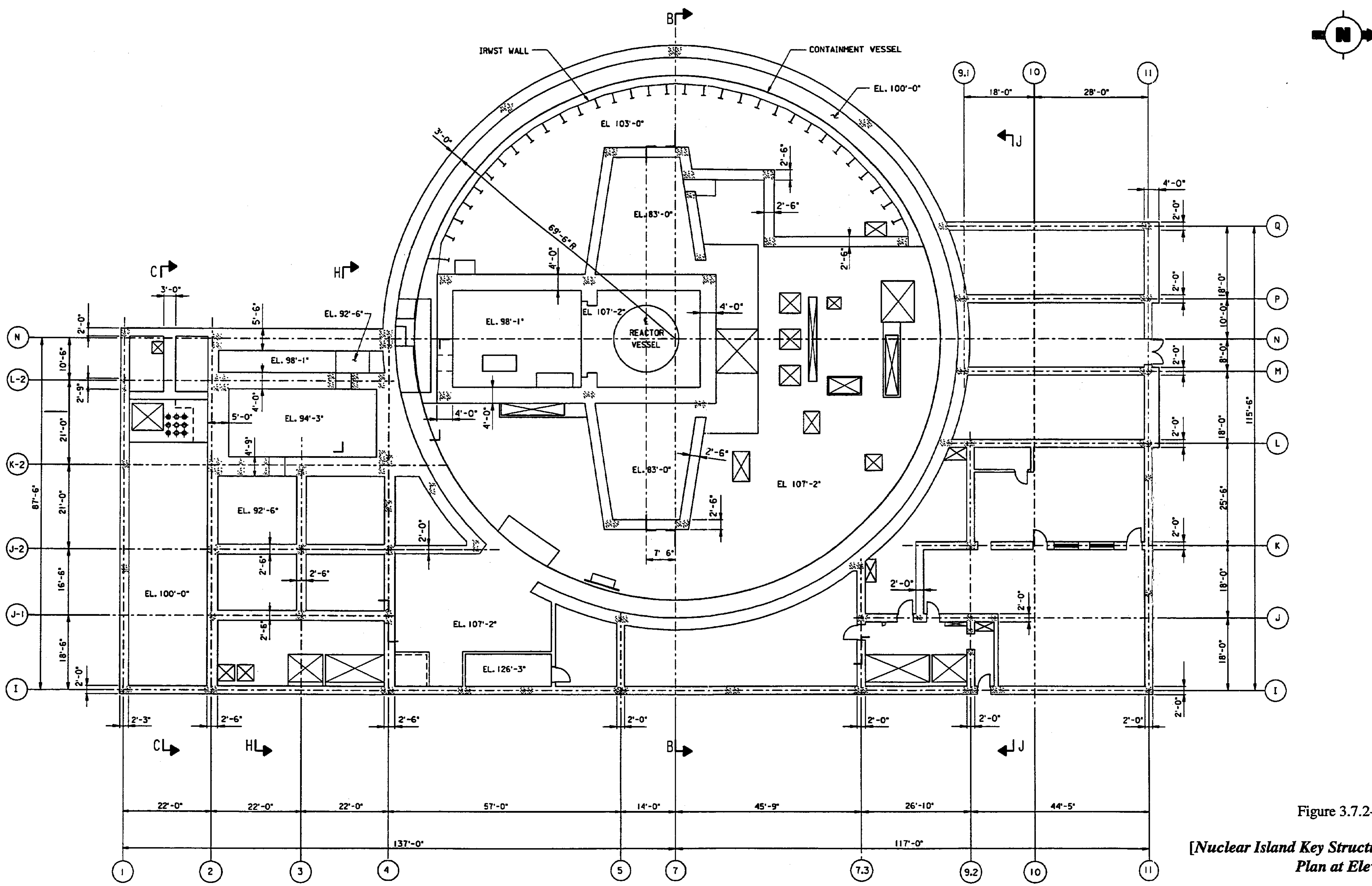


Figure 3.7.2-12 (Sheet 4 of 12)

*[Nuclear Island Key Structural Dimensions Plan at Elevation 117'-6"]**

*NRC Staff approval is required prior to implementing a change in this information; see DCD Section 3.5.

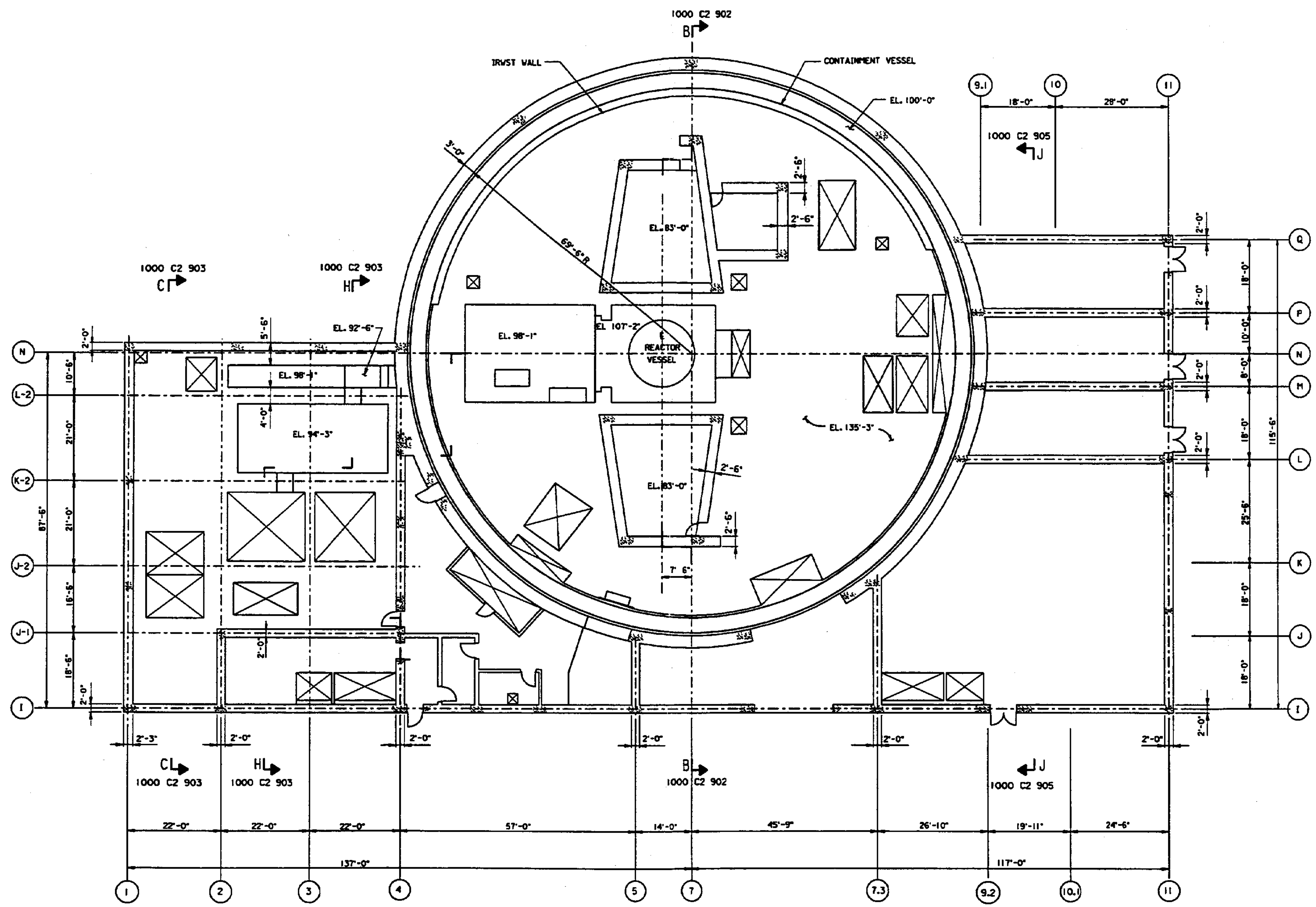


Figure 3.7.2-12 (Sheet 5 of 12)

[*Nuclear Island Key Structural Dimensions Plan at El. 135'-3"*]*

*NRC Staff approval is required prior to implementing a change in this information; see DCD Section 3.5.

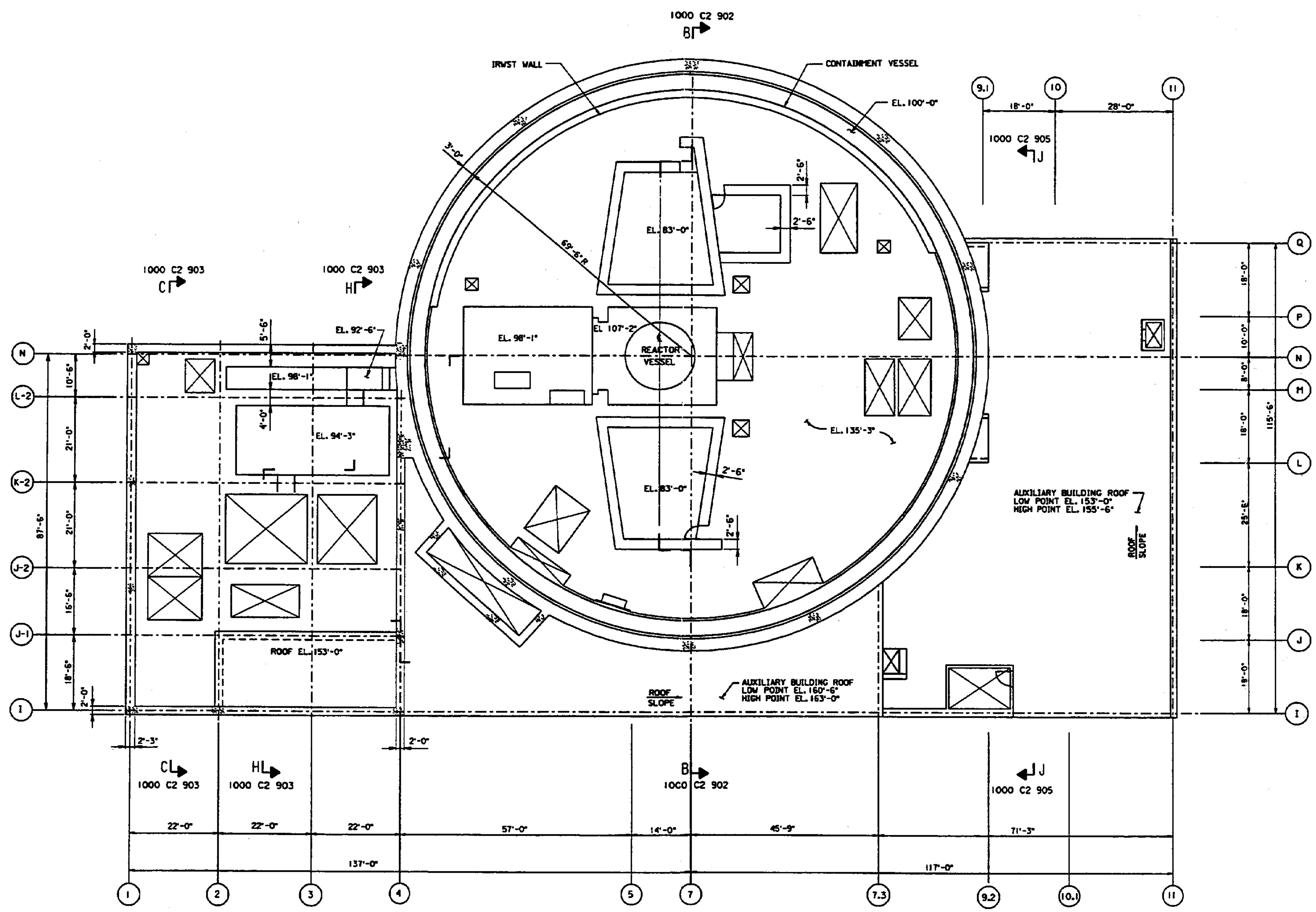


Figure 3.7.2-12 (Sheet 6 of 12)

*[Nuclear Island Key Structural Dimensions Plan at EL 153'-0" & 160'-6"]**

*NRC Staff approval is required prior to implementing a change in this information; see DCD Section 3.5.

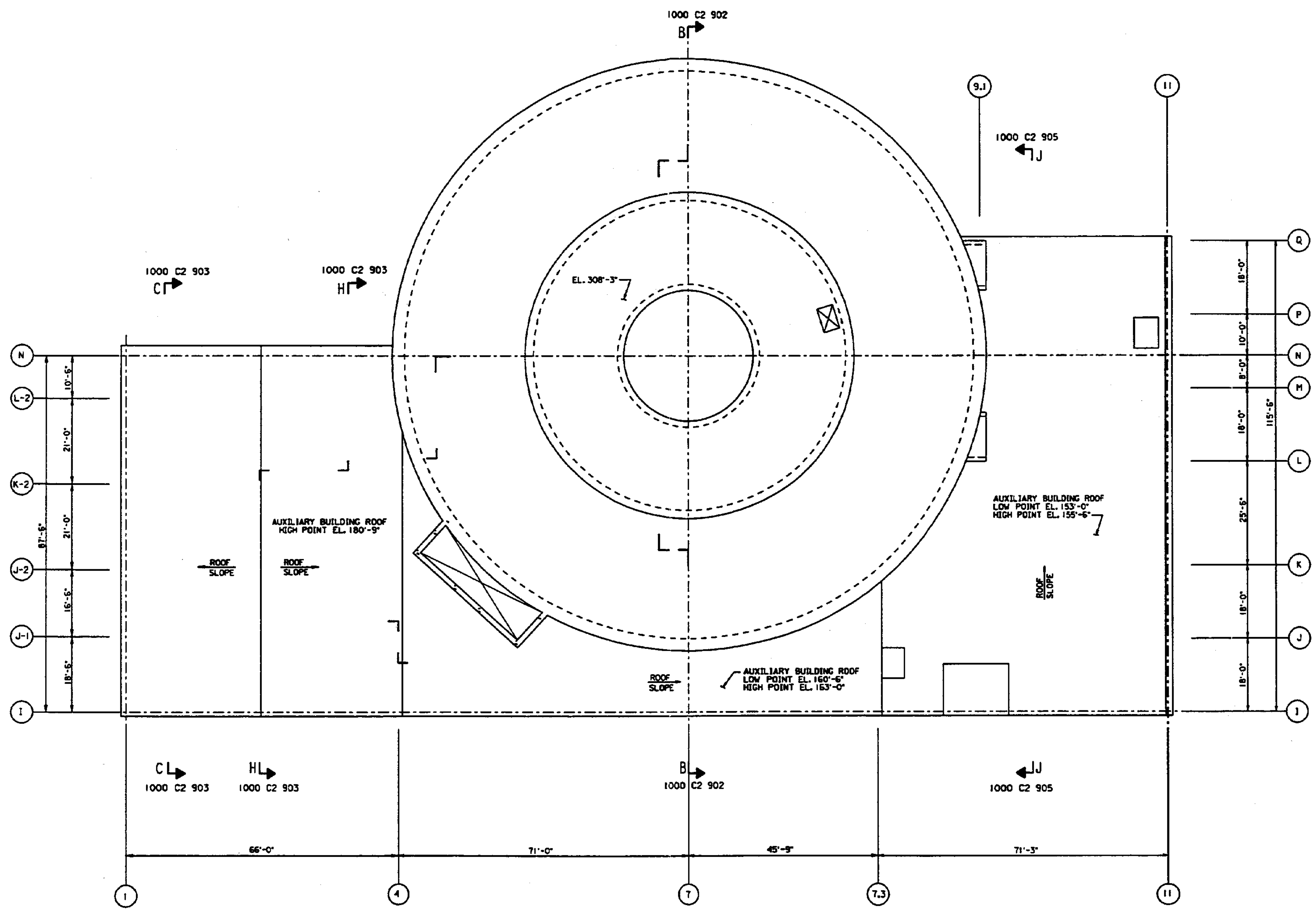


Figure 3.7.2-12 (Sheet 7 of 12)

[Key Structural Dimensions
*Plan at El. 160'-6" & 180'-0"]**

*NRC Staff approval is required prior to implementing a change in this information; see DCD Section 3.5.

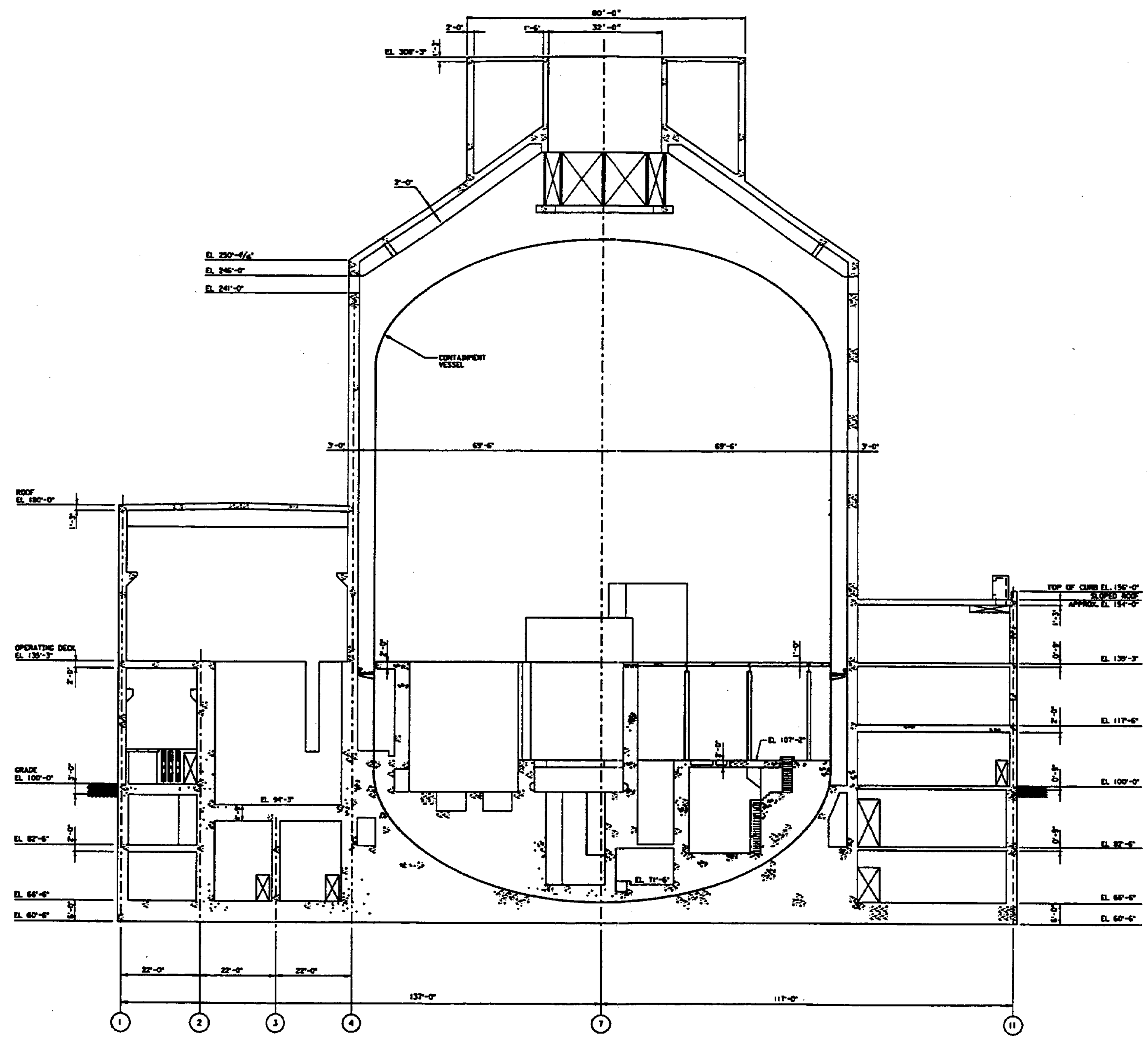
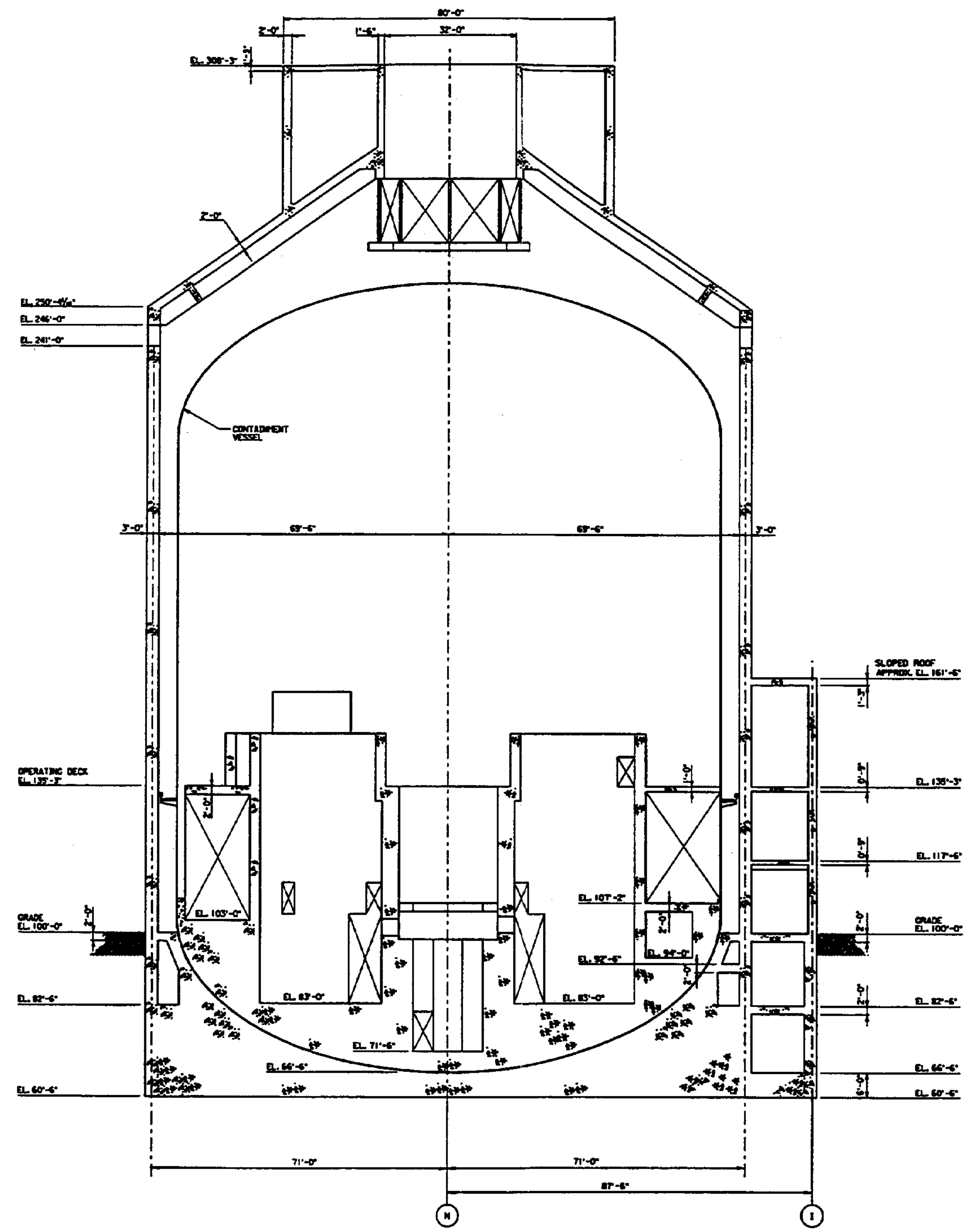


Figure 3.7.2-12 (Sheet 8 of 12)

*[Nuclear Island Key Structural Dimensions Section A-A]**

*NRC Staff approval is required prior to implementing a change in this information; see DCD Section 3.5.

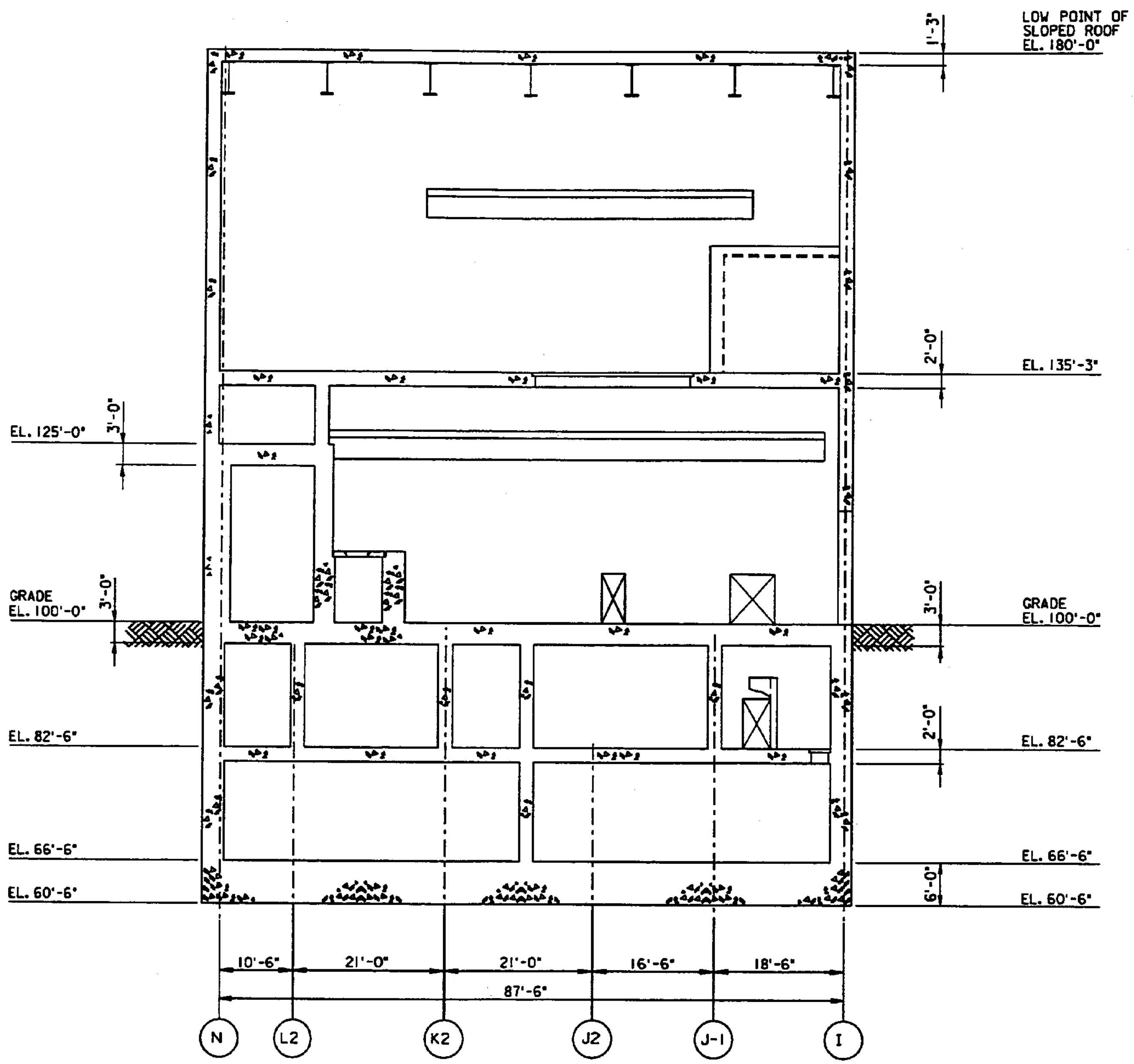


SECTION B-B

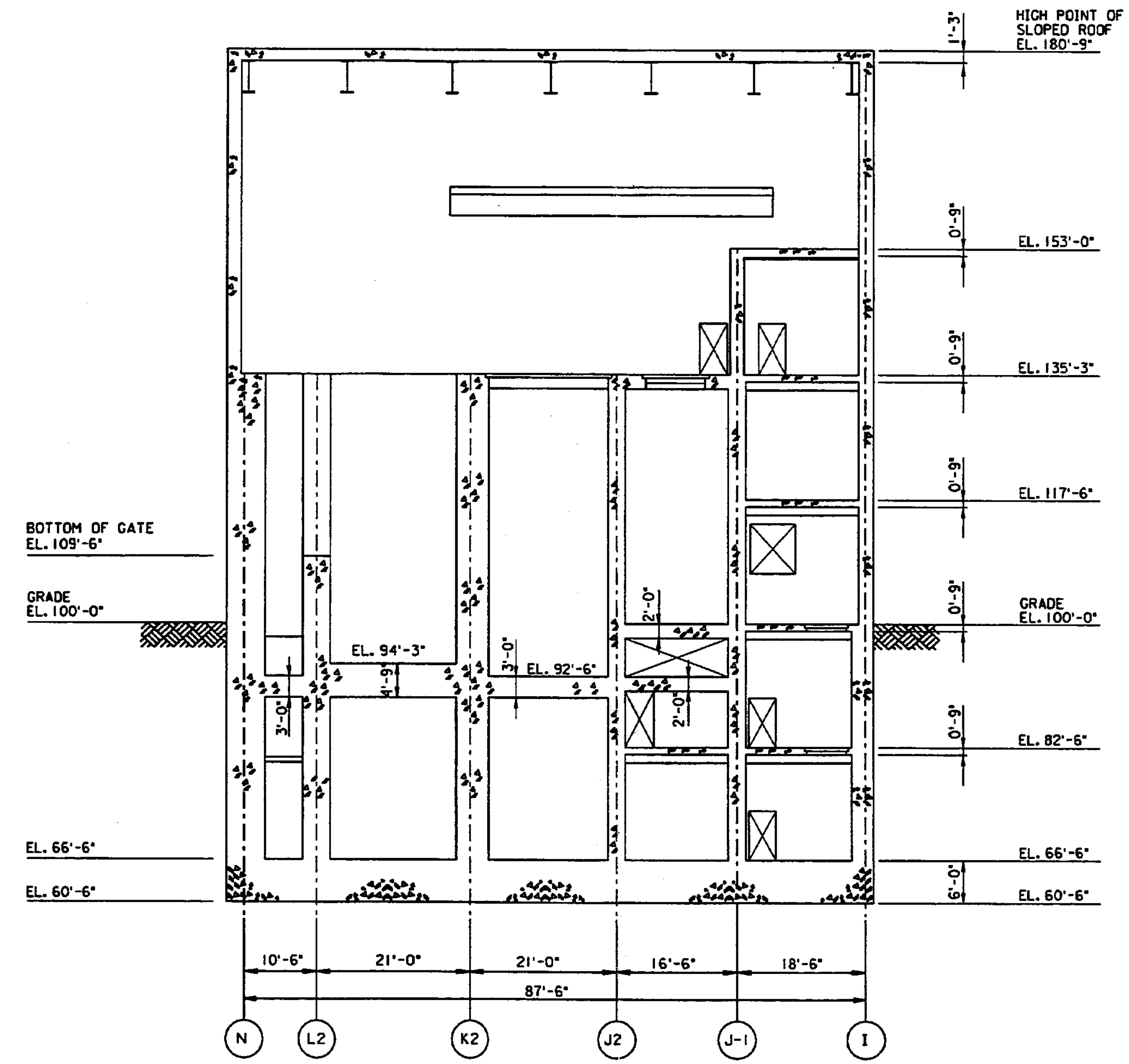
Figure 3.7.2-12 (Sheet 9 of 12)

*[Nuclear Island Key Structural Dimensions Section B-B]**

*NRC Staff approval is required prior to implementing a change in this information; see DCD Section 3.5.



SECTION C-C

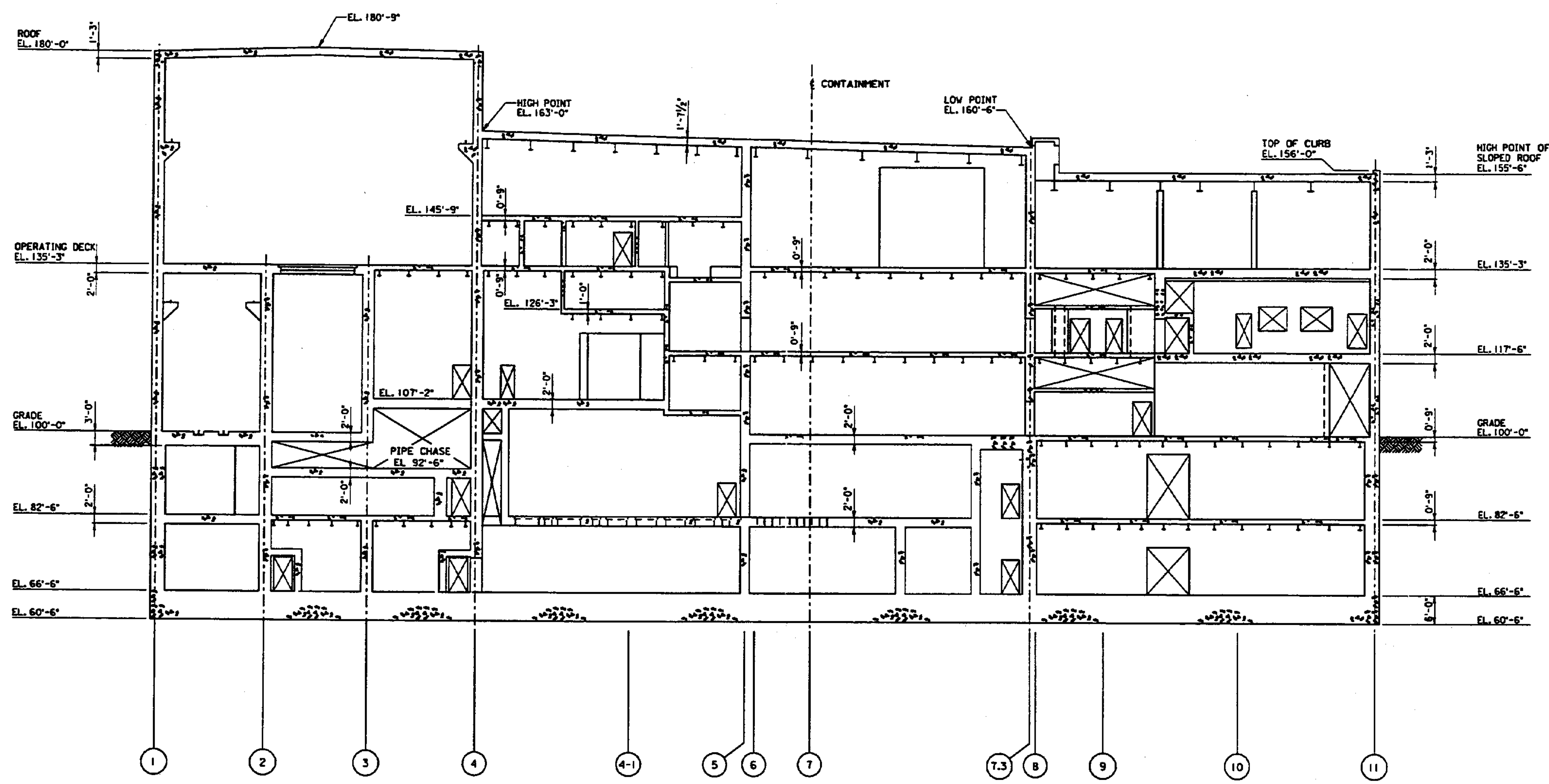


SECTION H-H

Figure 3.7.2-12 (Sheet 10 of 12)

*[Nuclear Island Key Structural Dimensions Section C-C And H-H]**

*NRC Staff approval is required prior to implementing a change in this information; see DCD Section 3.5.

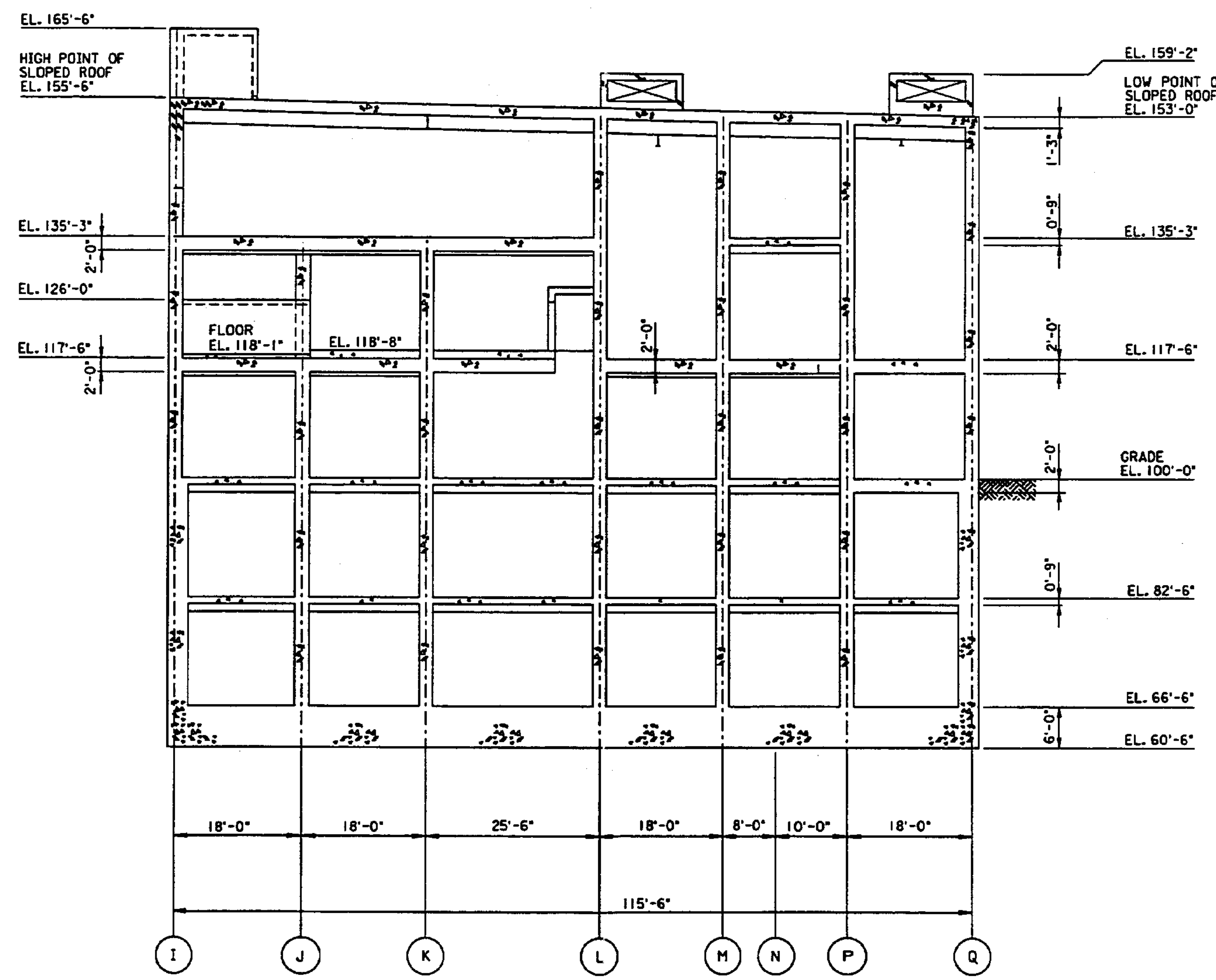


SECTION G-G

Figure 3.7.2-12 (Sheet 11 of 12)

*[Nuclear Island Key Structural Dimensions Section G-G]**

*NRC Staff approval is required prior to implementing a change in this information; see DCD Section 3.5.

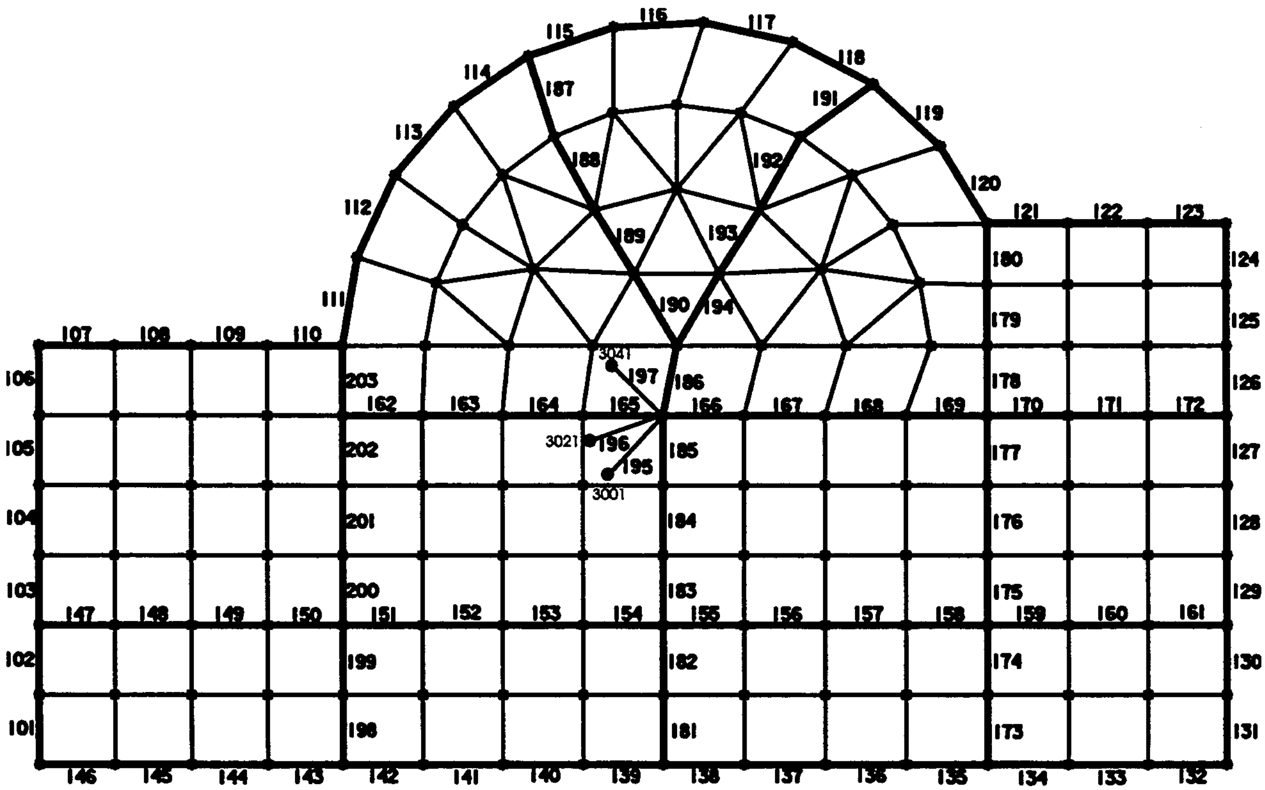


SECTION J-J

Figure 3.7.2-12 (Sheet 12 of 12)

[*Nuclear Island Key Structural Dimensions Section J-J*]*

*NRC Staff approval is required prior to implementing a change in this information; see DCD Section 3.5.



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

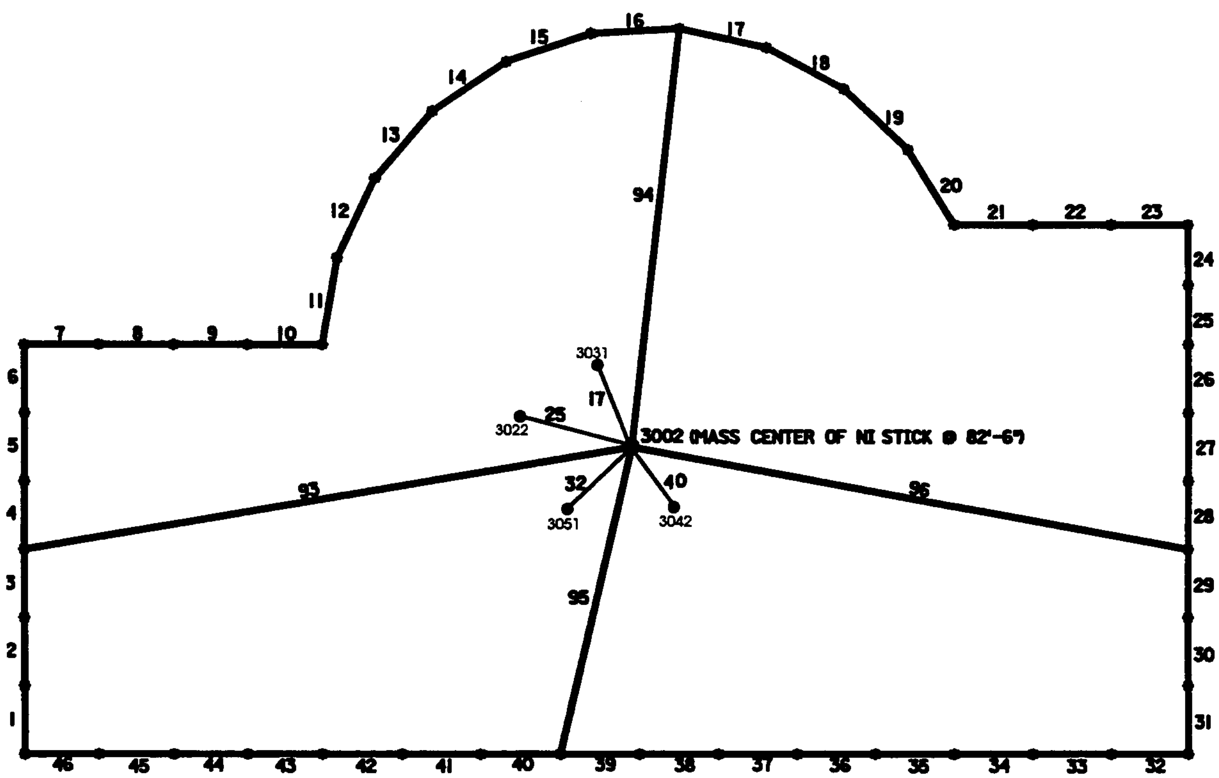
NOTE:
NUMBERED LINES INDICATE RIGID BEAM ELEMENTS

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′.
(Locations are not to scale)

Figure 3.7.2-13 (Sheet 1 of 3)

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



LOCATION OF RIGID BEAM ELEMENTS @ EL. 82'-6"

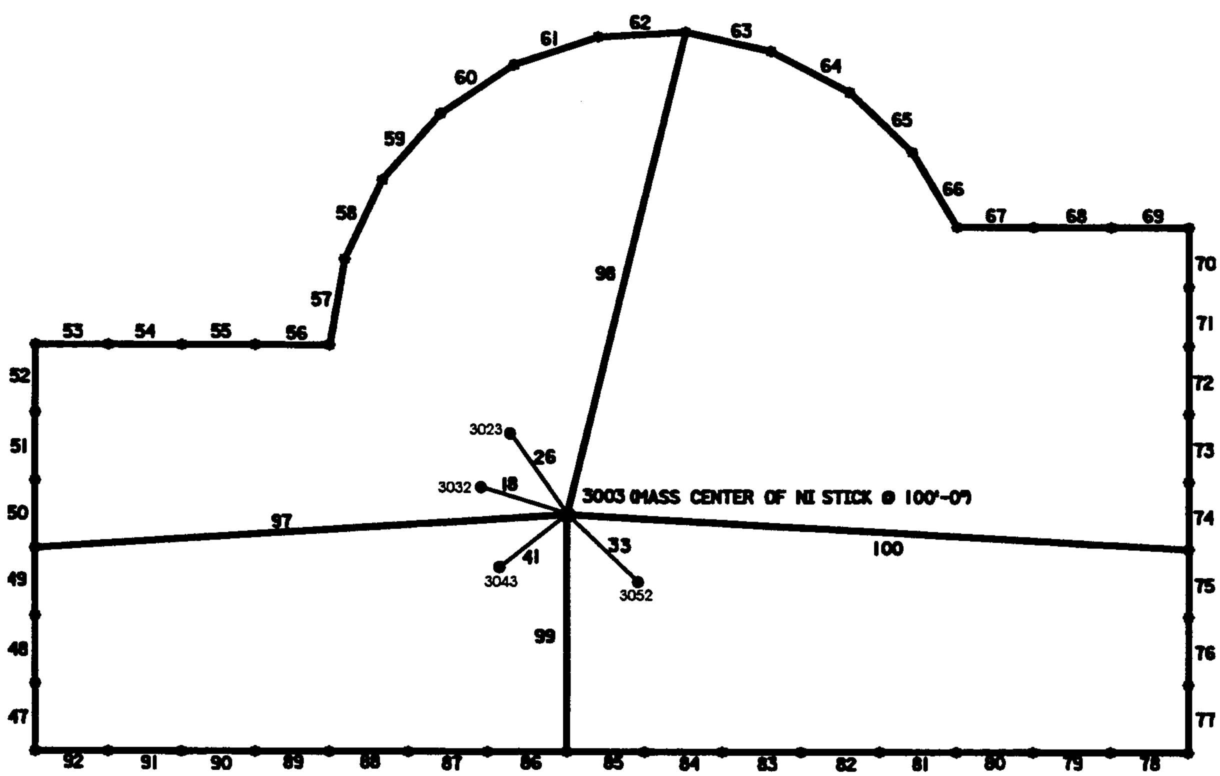
NOTE:
NUMBERED LINES INDICATE RIGID BEAM ELEMENTS

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 100.0′.

(Locations are not to scale)

Figure 3.7.2-13 (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

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′.
(Locations are not to scale)

Figure 3.7.2-13 (Sheet 3 of 3)

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

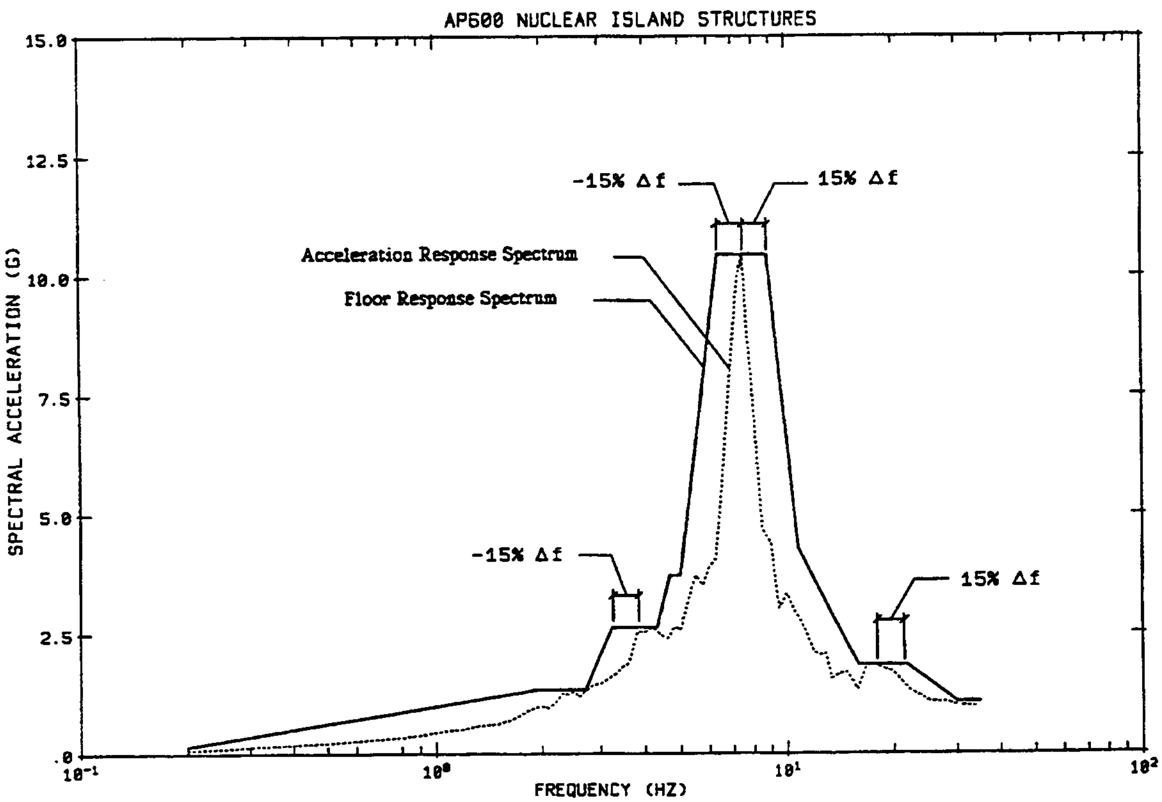


Figure 3.7.2-14

Typical Design Floor Response Spectrum

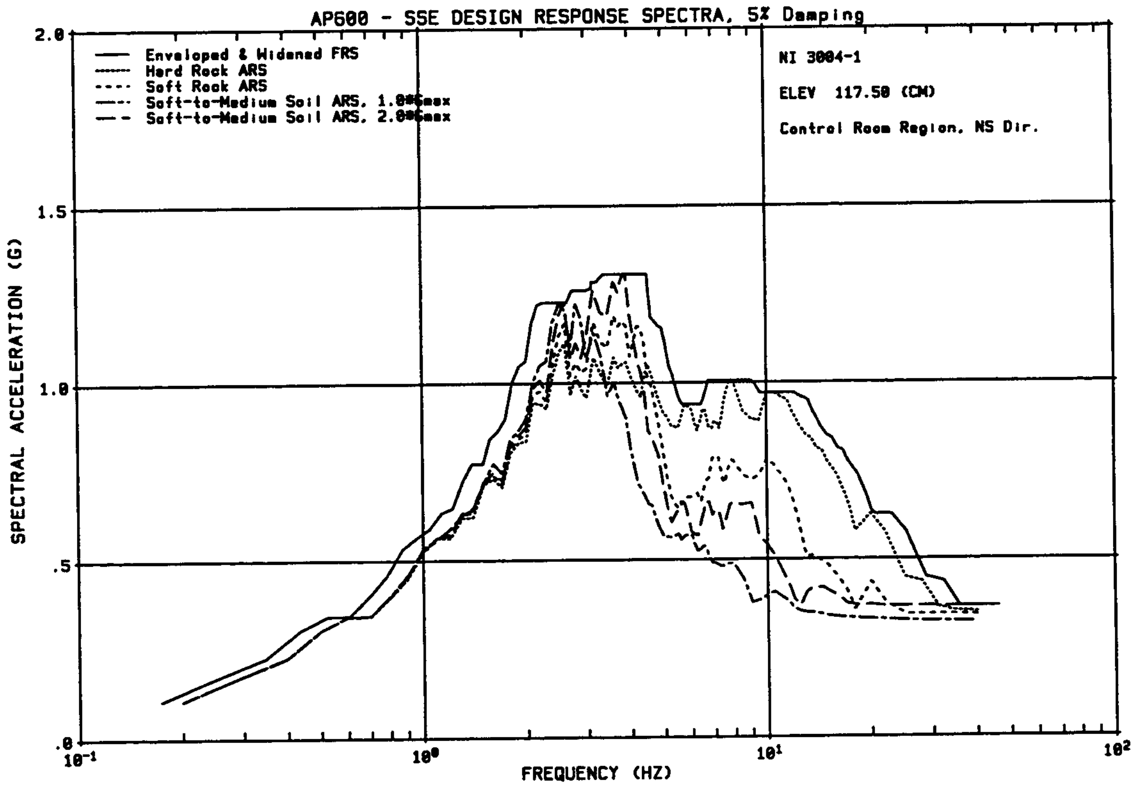


Figure 3.7.2-15 (Sheet 1 of 12)

Coupled Shield & Auxiliary Buildings SSE Floor Response Spectra

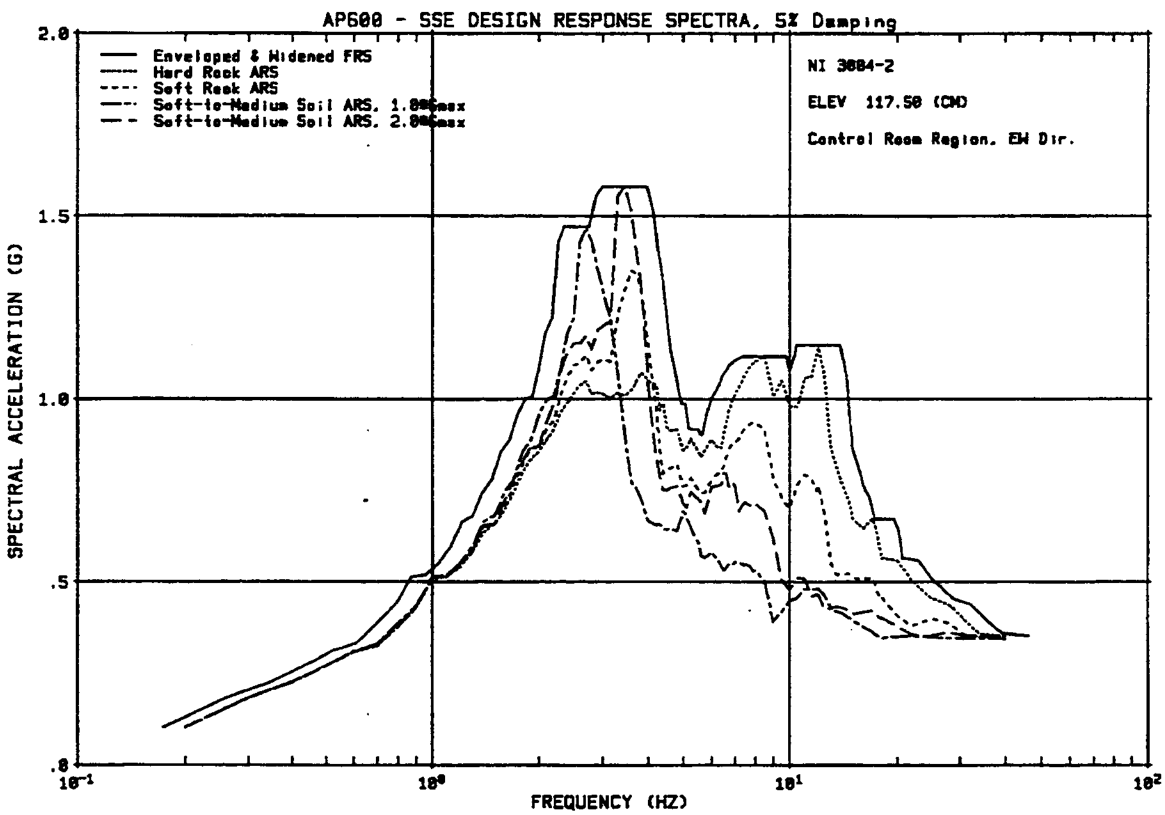


Figure 3.7.2-15 (Sheet 2 of 12)

Coupled Shield & Auxiliary Buildings SSE Floor Response Spectra

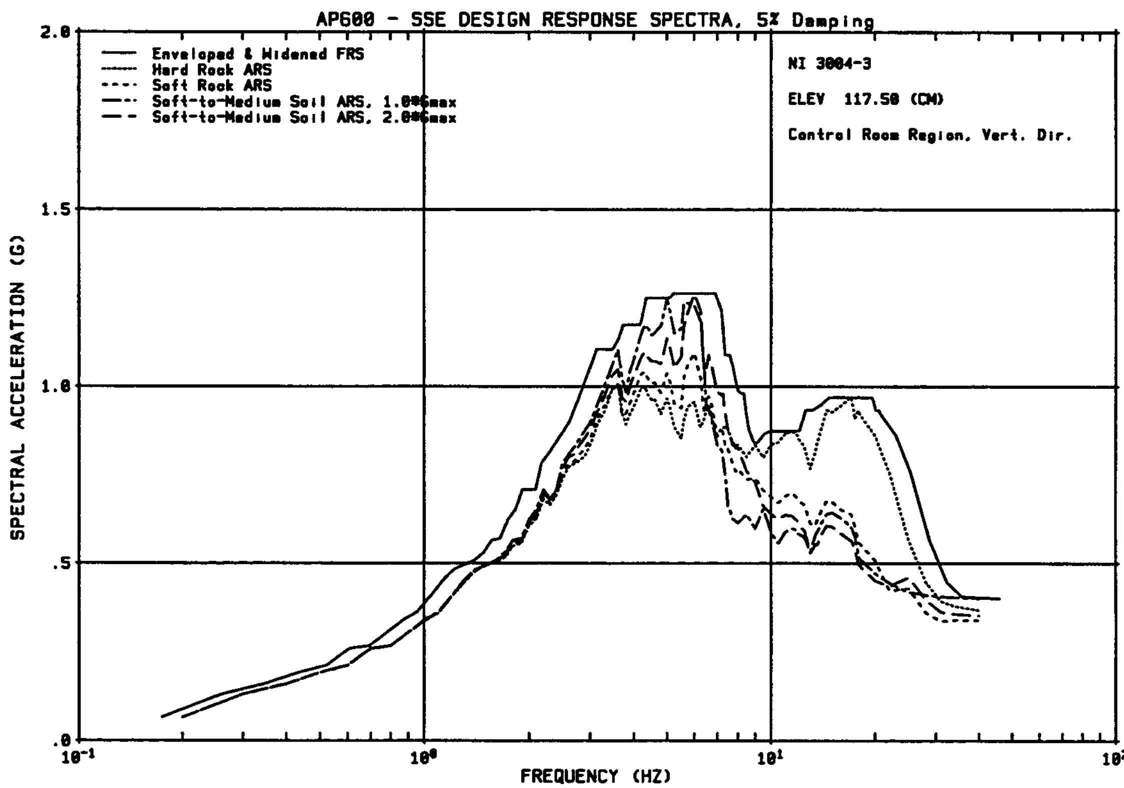


Figure 3.7.2-15 (Sheet 3 of 12)

Coupled Shield & Auxiliary Buildings SSE Floor Response Spectra

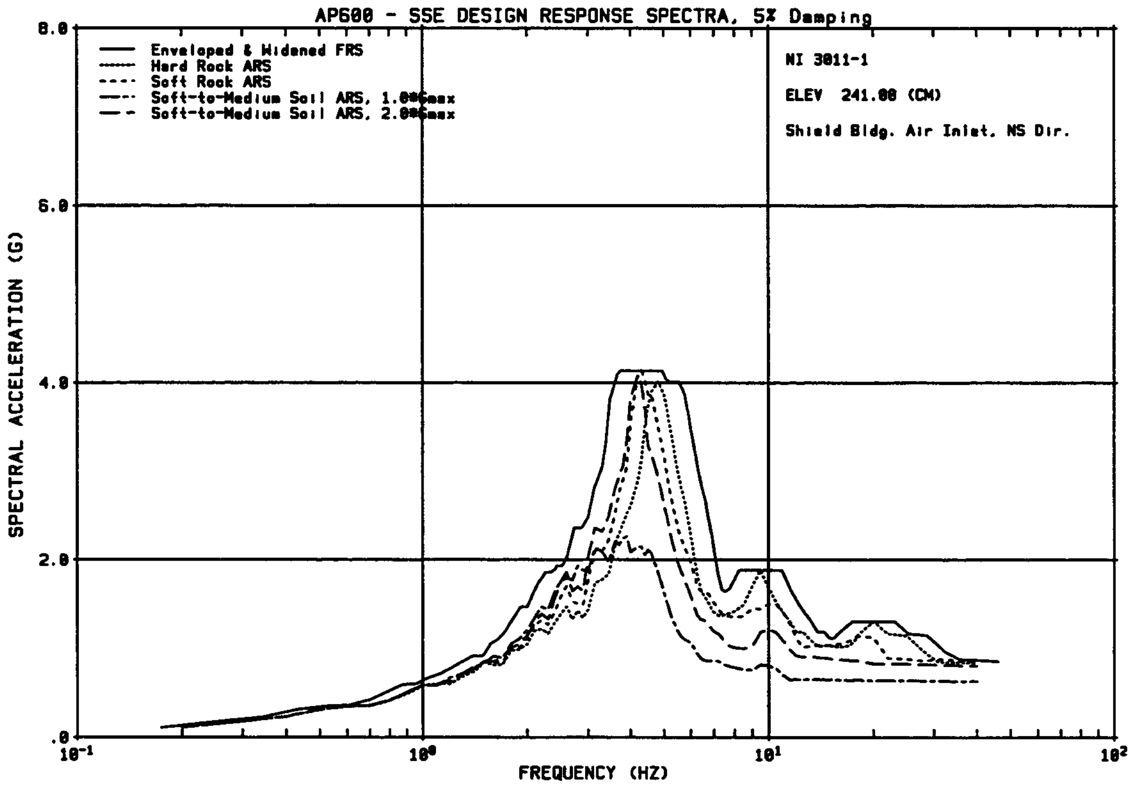


Figure 3.7.2-15 (Sheet 4 of 12)

Coupled Shield & Auxiliary Buildings SSE Floor Response Spectra

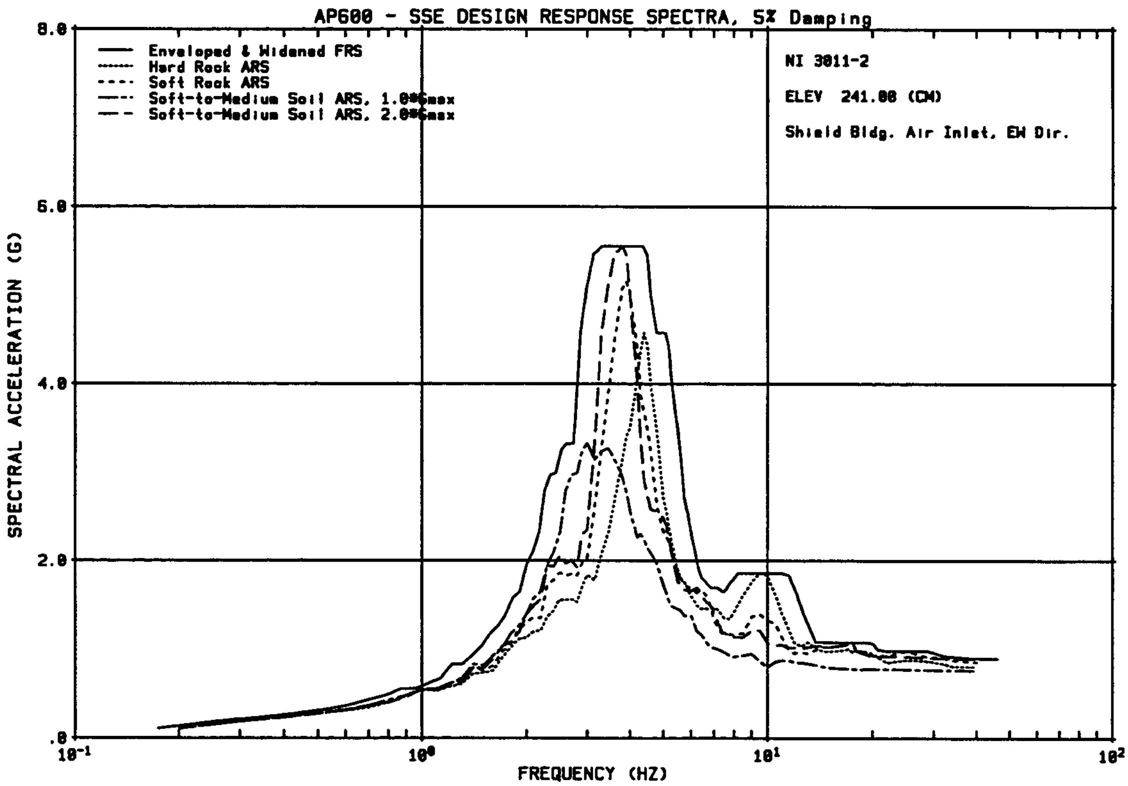


Figure 3.7.2-15 (Sheet 5 of 12)

Coupled Shield & Auxiliary Buildings SSE Floor Response Spectra

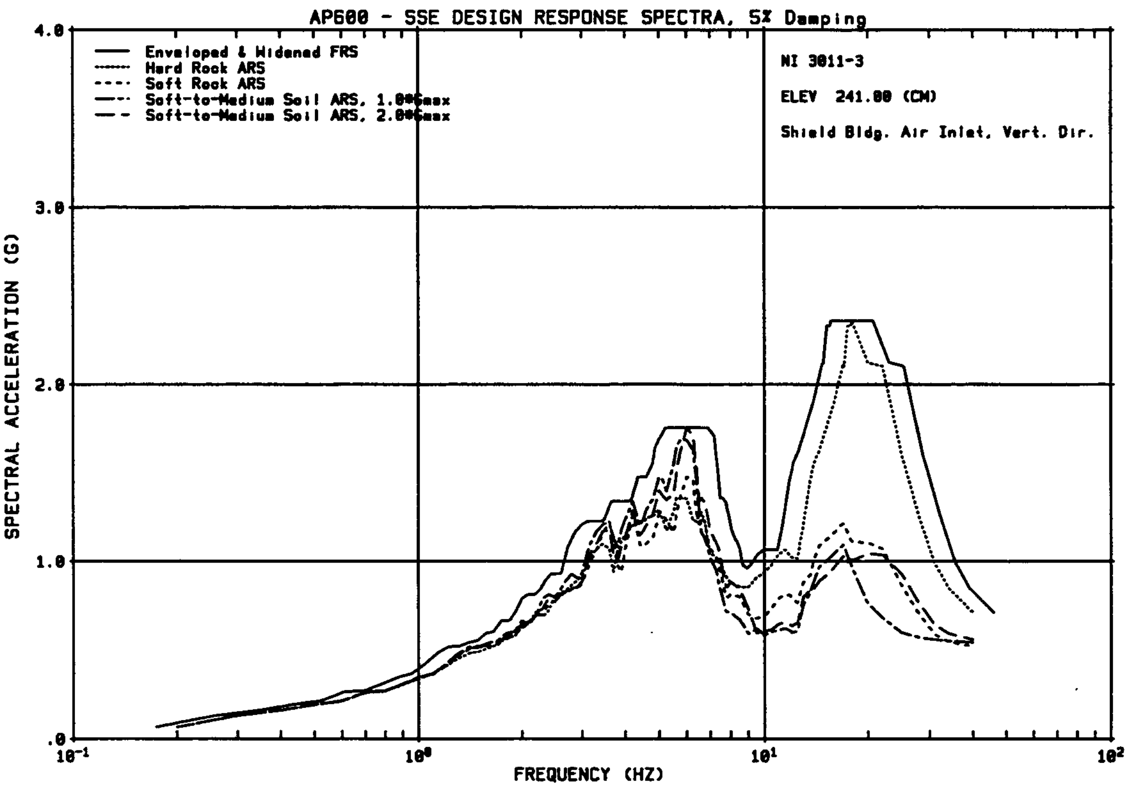


Figure 3.7.2-15 (Sheet 6 of 12)

Coupled Shield & Auxiliary Buildings SSE Floor Response Spectra

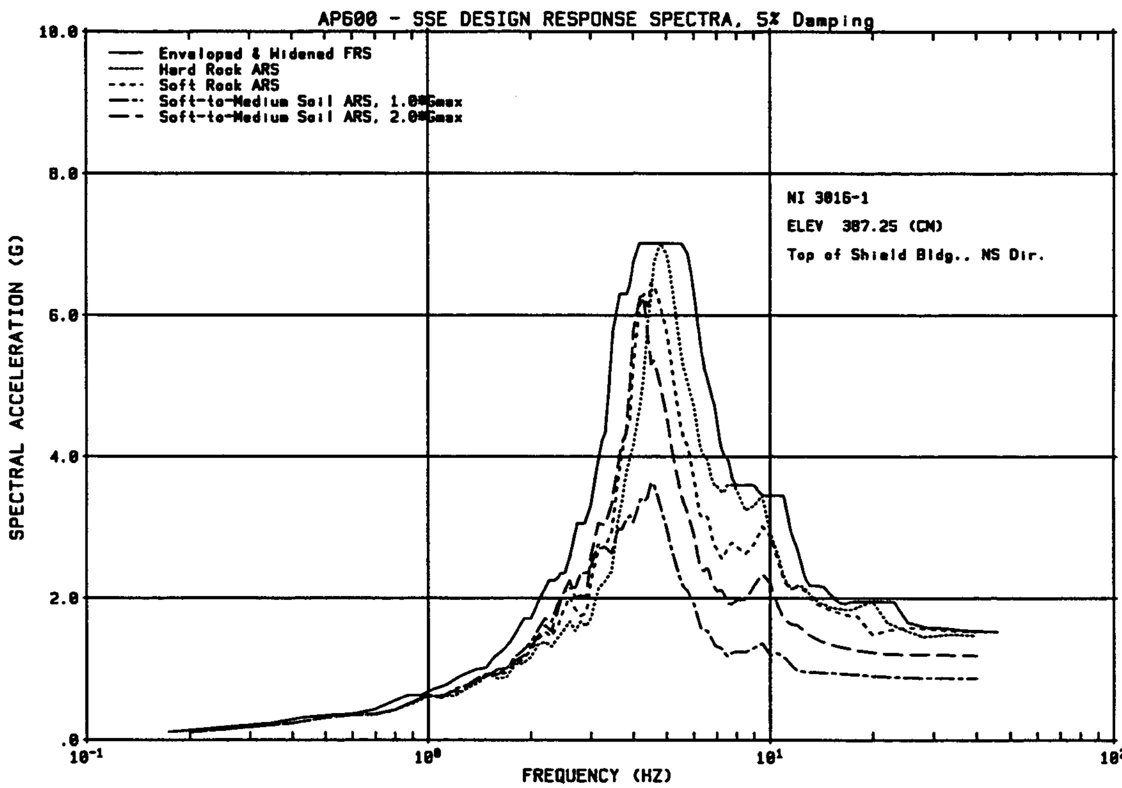


Figure 3.7.2-15 (Sheet 7 of 12)

Coupled Shield & Auxiliary Buildings SSE Floor Response Spectra

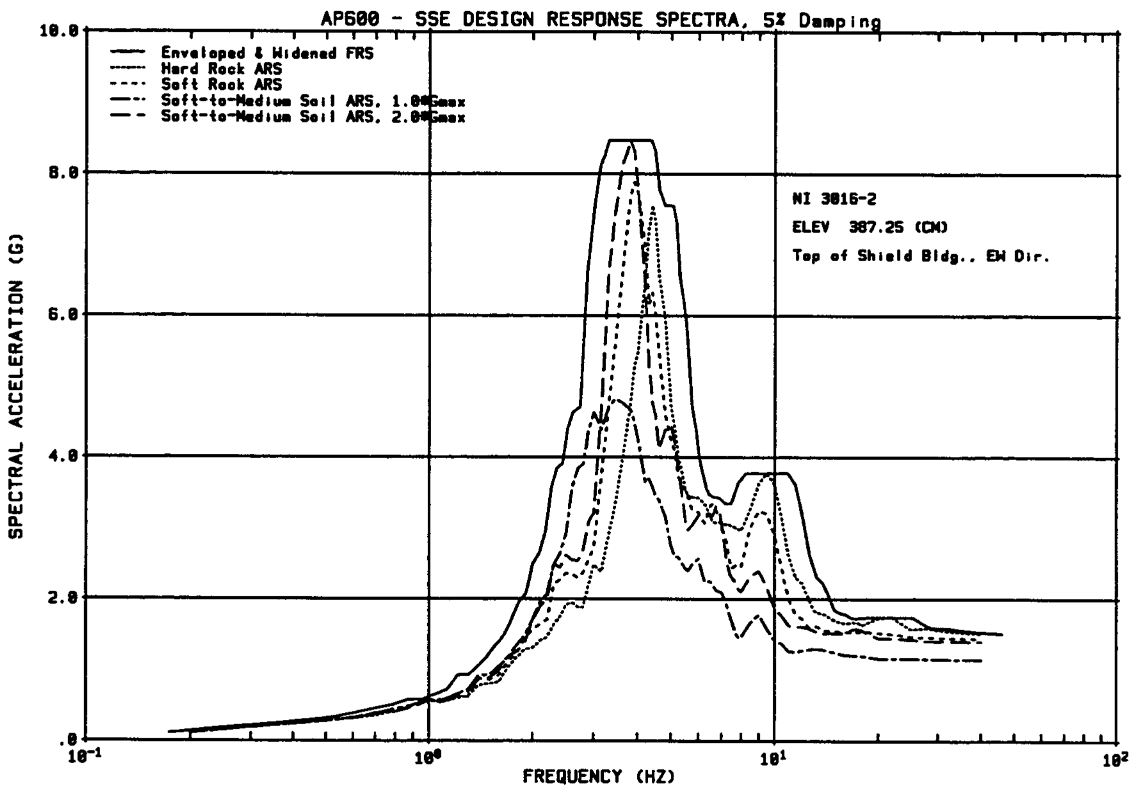


Figure 3.7.2-15 (Sheet 8 of 12)

Coupled Shield & Auxiliary Buildings SSE Floor Response Spectra

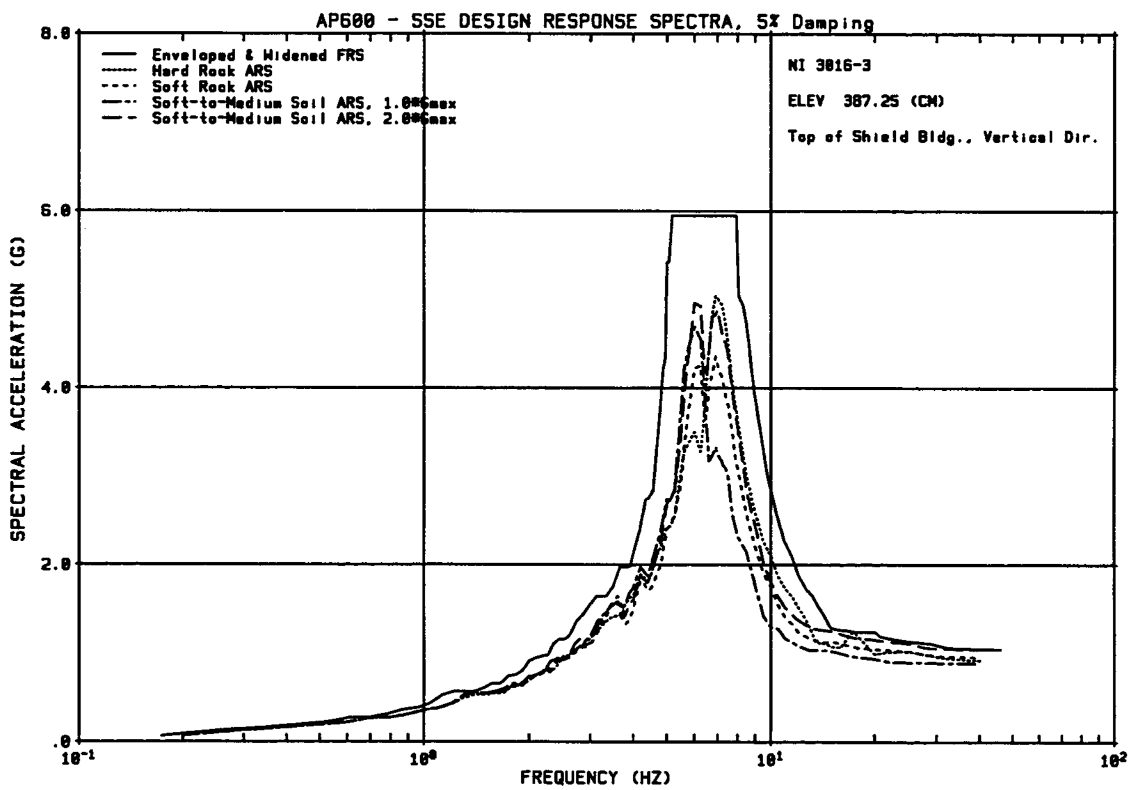


Figure 3.7.2-15 (Sheet 9 of 12)

Coupled Shield & Auxiliary Buildings SSE Floor Response Spectra

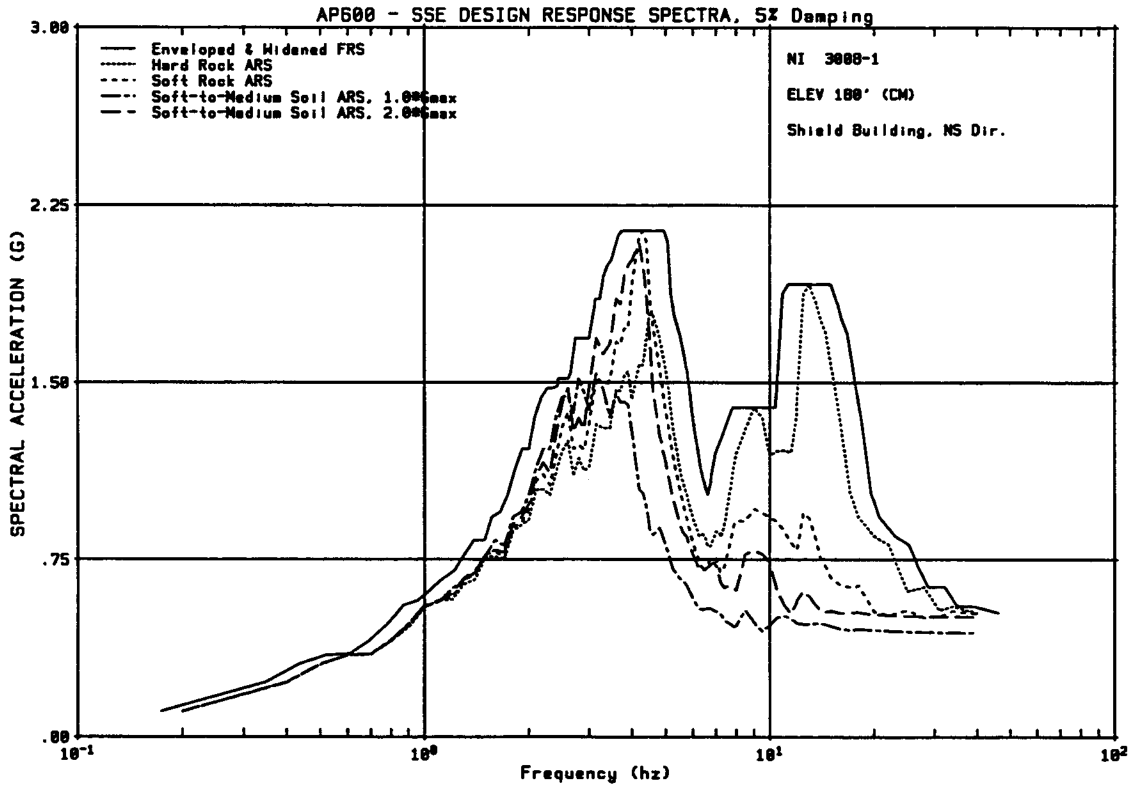


Figure 3.7.2-15 (Sheet 10 of 12)

Coupled Shield & Auxiliary Buildings SSE Floor Response Spectra

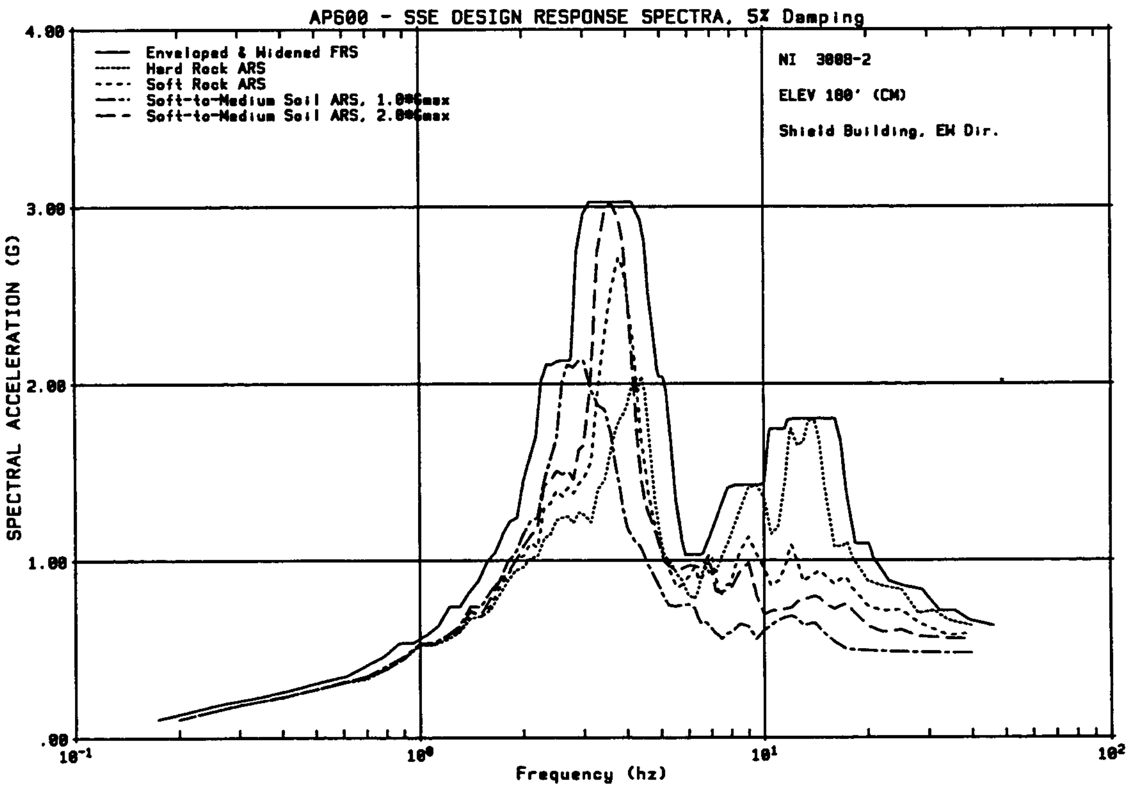


Figure 3.7.2-15 (Sheet 11 of 12)

Coupled Shield & Auxiliary Buildings SSE Floor Response Spectra

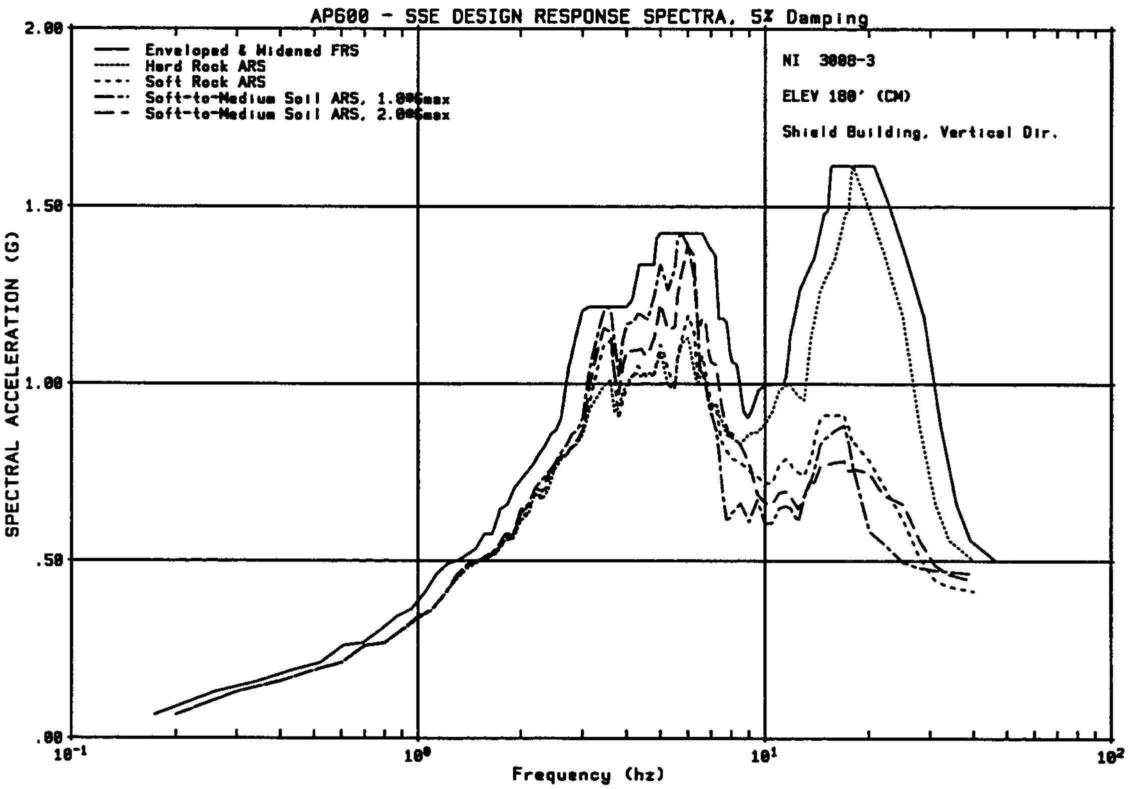


Figure 3.7.2-15 (Sheet 12 of 12)

Coupled Shield & Auxiliary Buildings SSE Floor Response Spectra

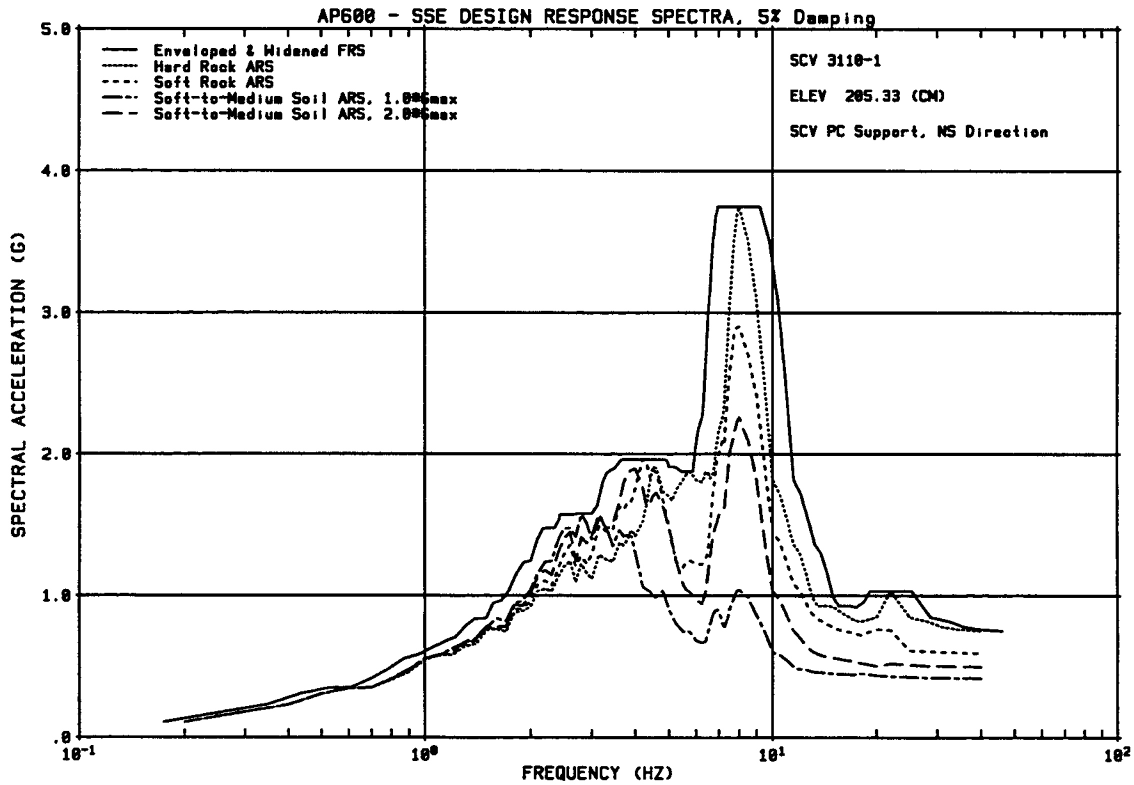


Figure 3.7.2-16 (Sheet 1 of 6)

Steel Containment Vessel SSE Floor Response Spectra

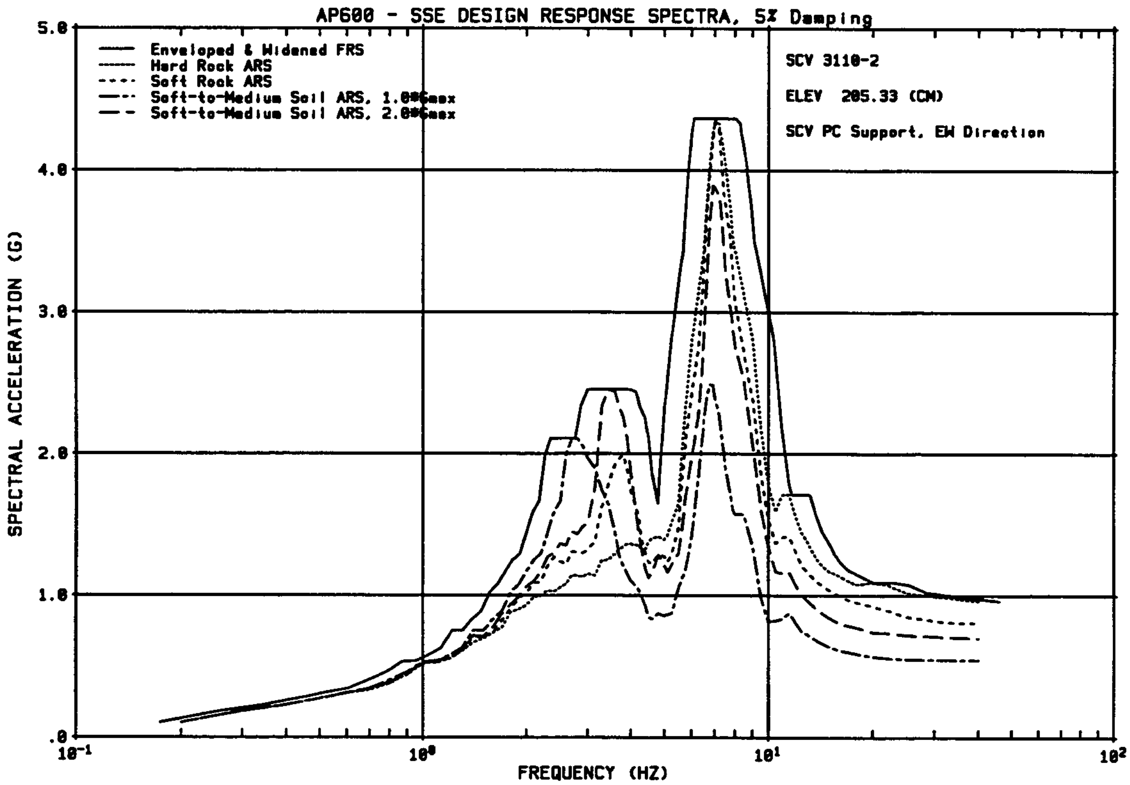


Figure 3.7.2-16 (Sheet 2 of 6)

Steel Containment Vessel SSE Floor Response Spectra

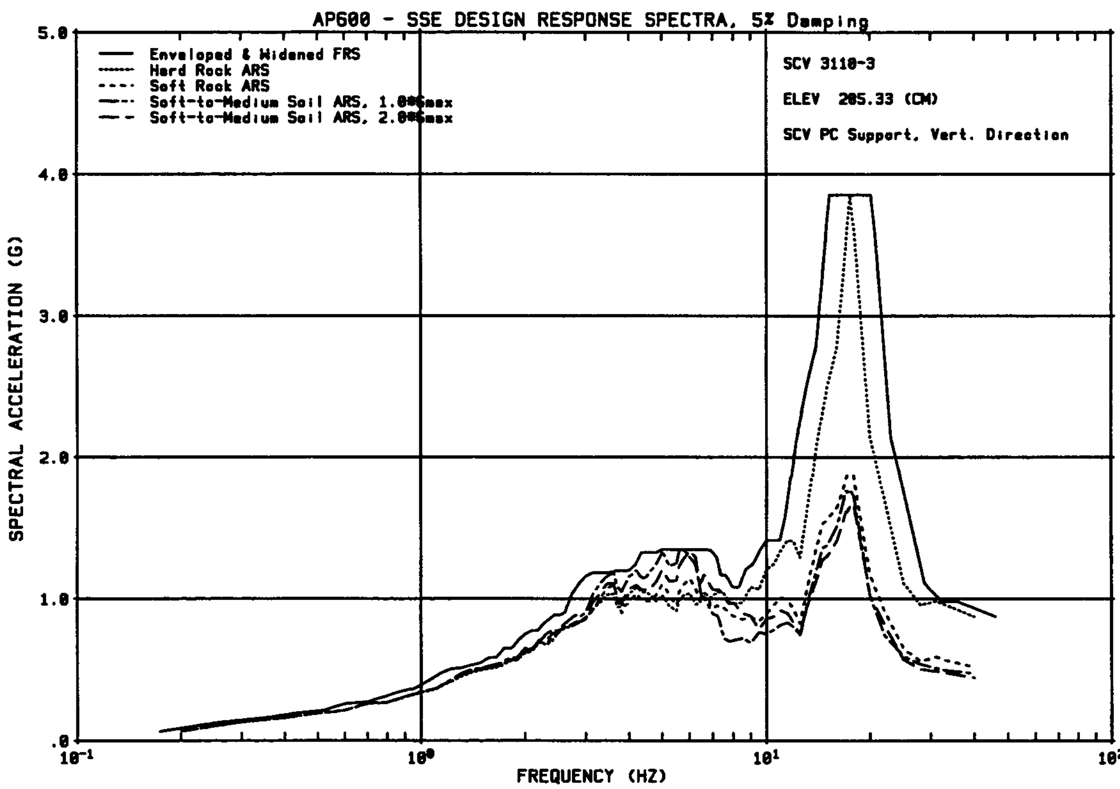


Figure 3.7.2-16 (Sheet 3 of 6)

Steel Containment Vessel SSE Floor Response Spectra

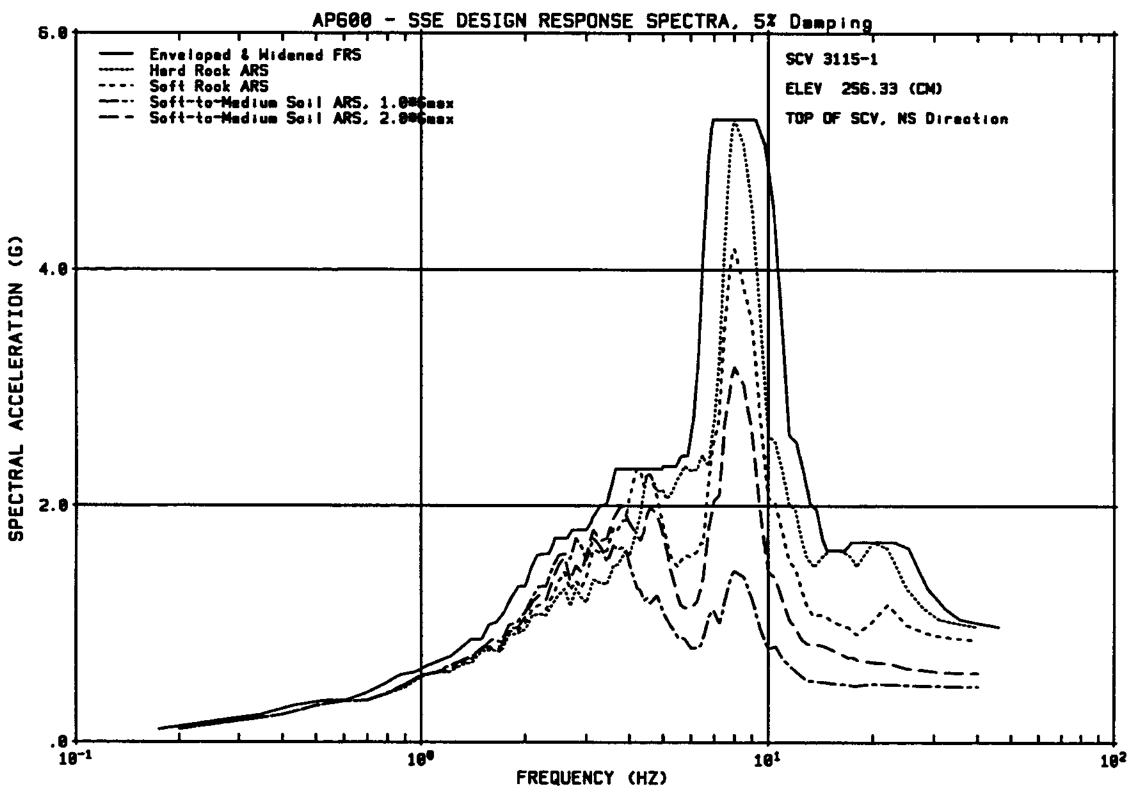


Figure 3.7.2-16 (Sheet 4 of 6)

Steel Containment Vessel SSE Floor Response Spectra

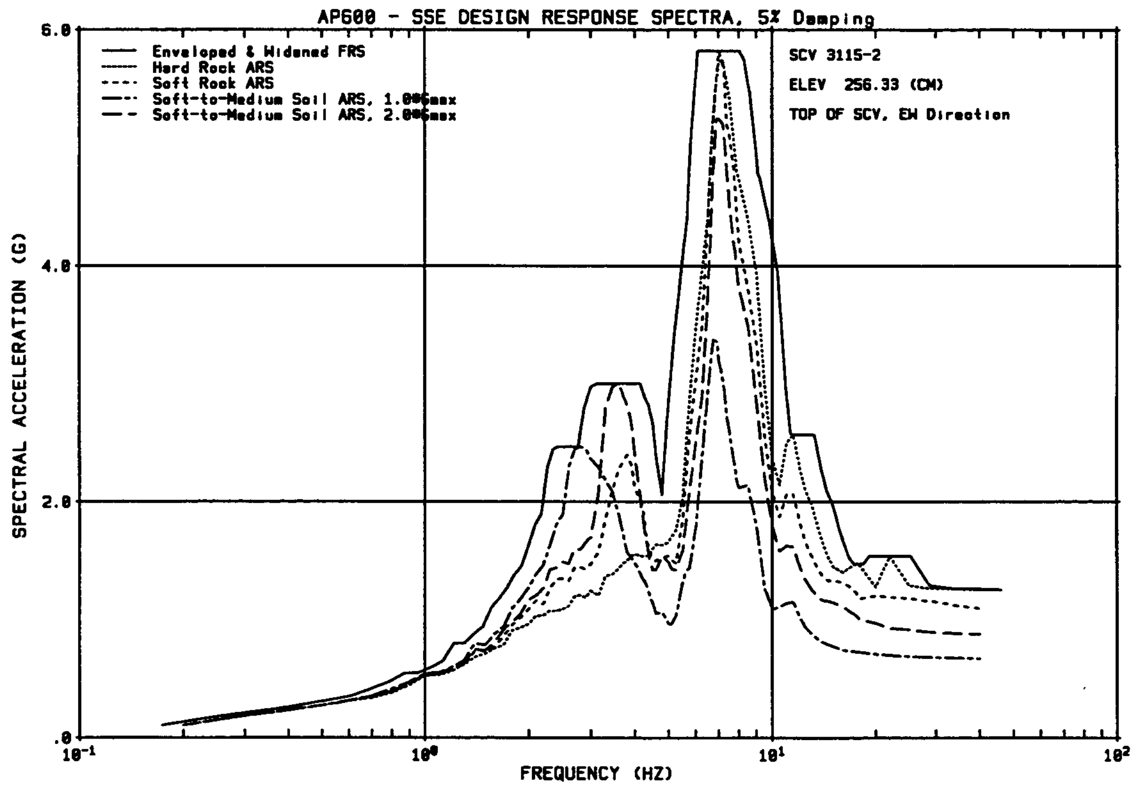


Figure 3.7.2-16 (Sheet 5 of 6)

Steel Containment Vessel SSE Floor Response Spectra

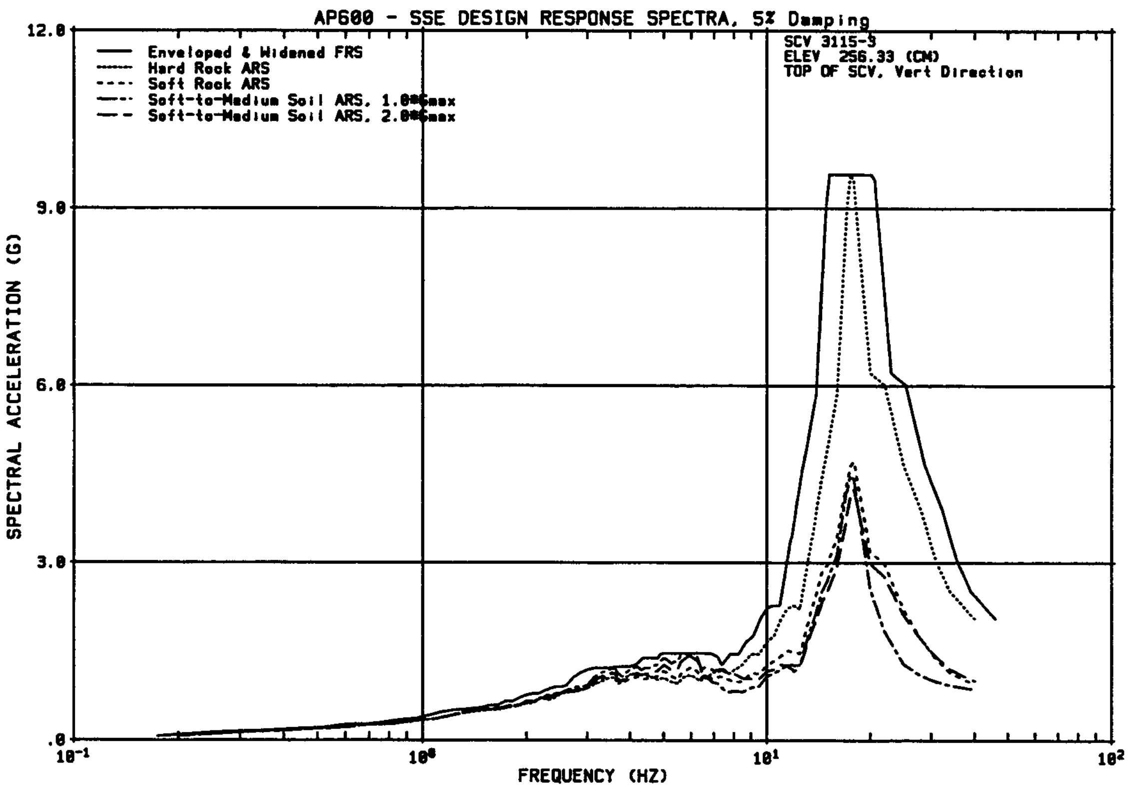


Figure 3.7.2-16 (Sheet 6 of 6)

Steel Containment Vessel SSE Floor Response Spectra

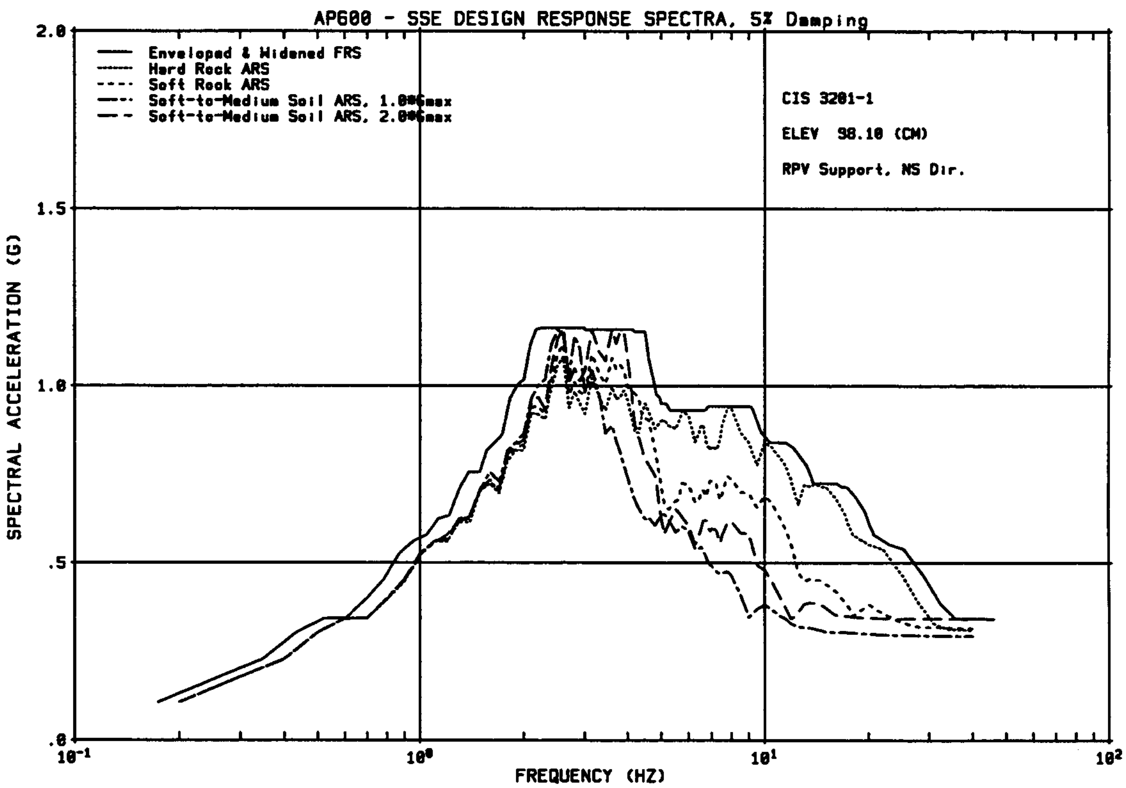


Figure 3.7.2-17 (Sheet 1 of 9)

Containment Internal Structures SSE Floor Response Spectra

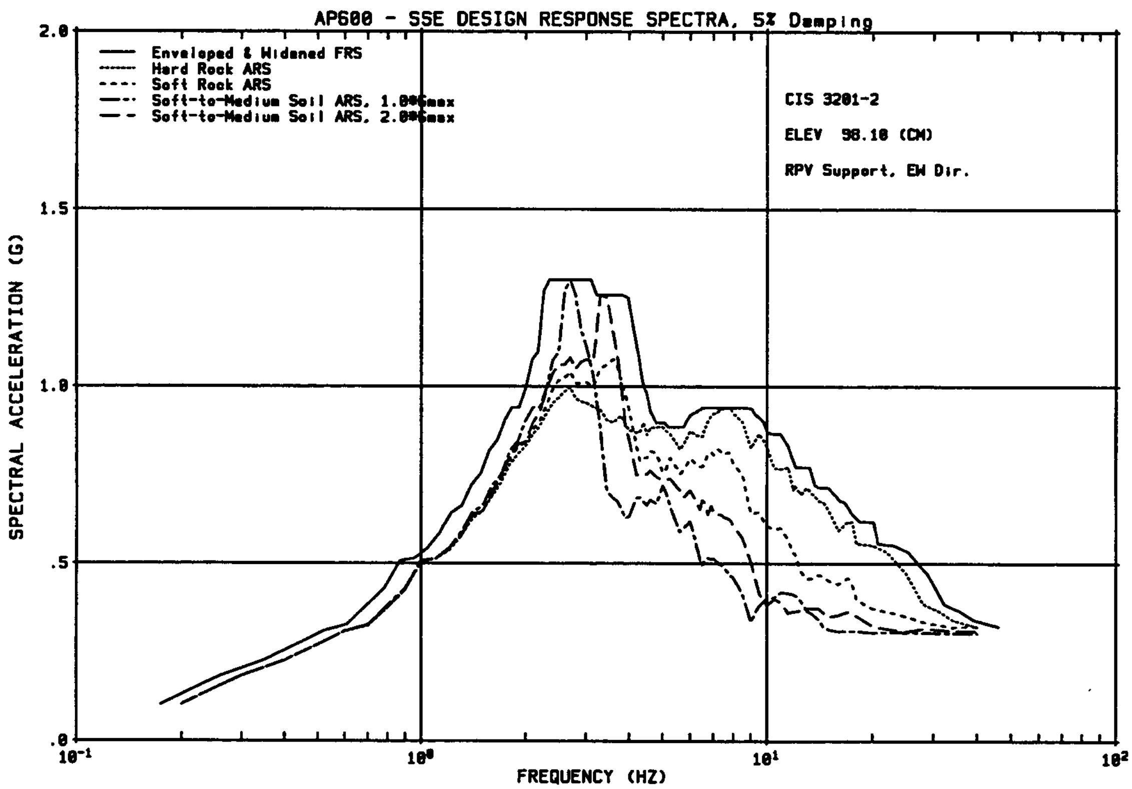


Figure 3.7.2-17 (Sheet 2 of 9)

Containment Internal Structures SSE Floor Response Spectra

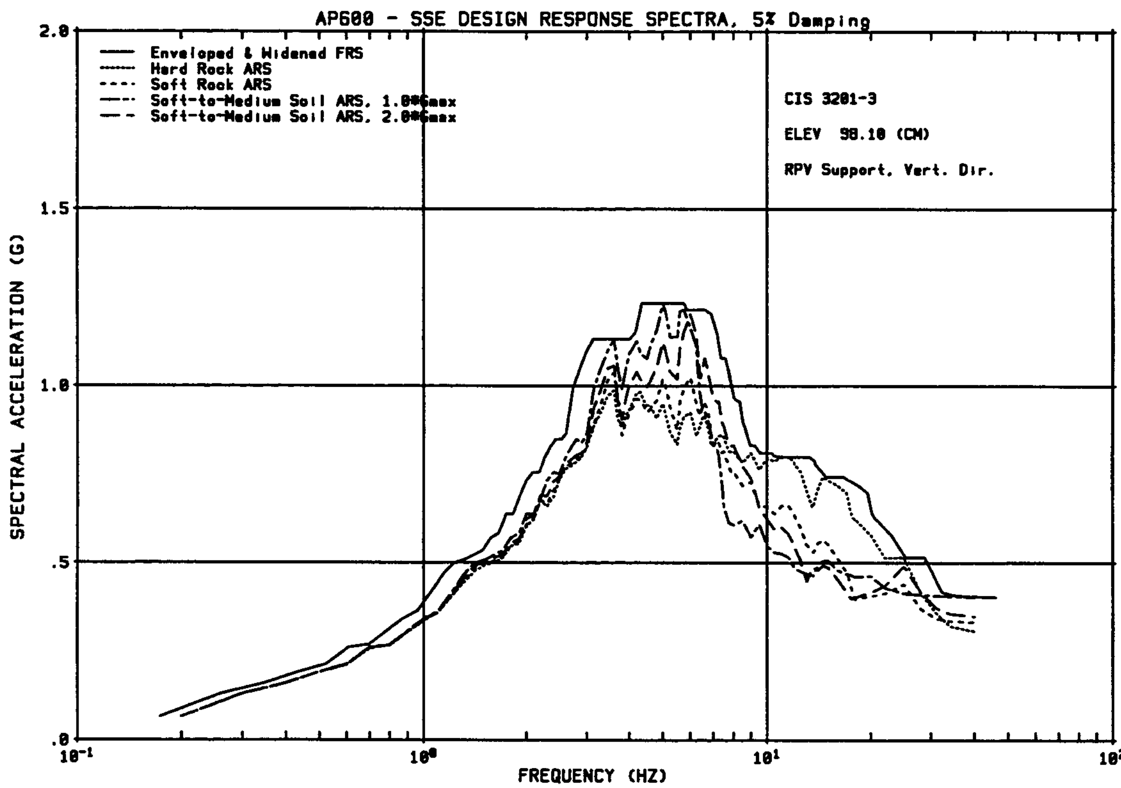


Figure 3.7.2-17 (Sheet 3 of 9)

Containment Internal Structures SSE Floor Response Spectra

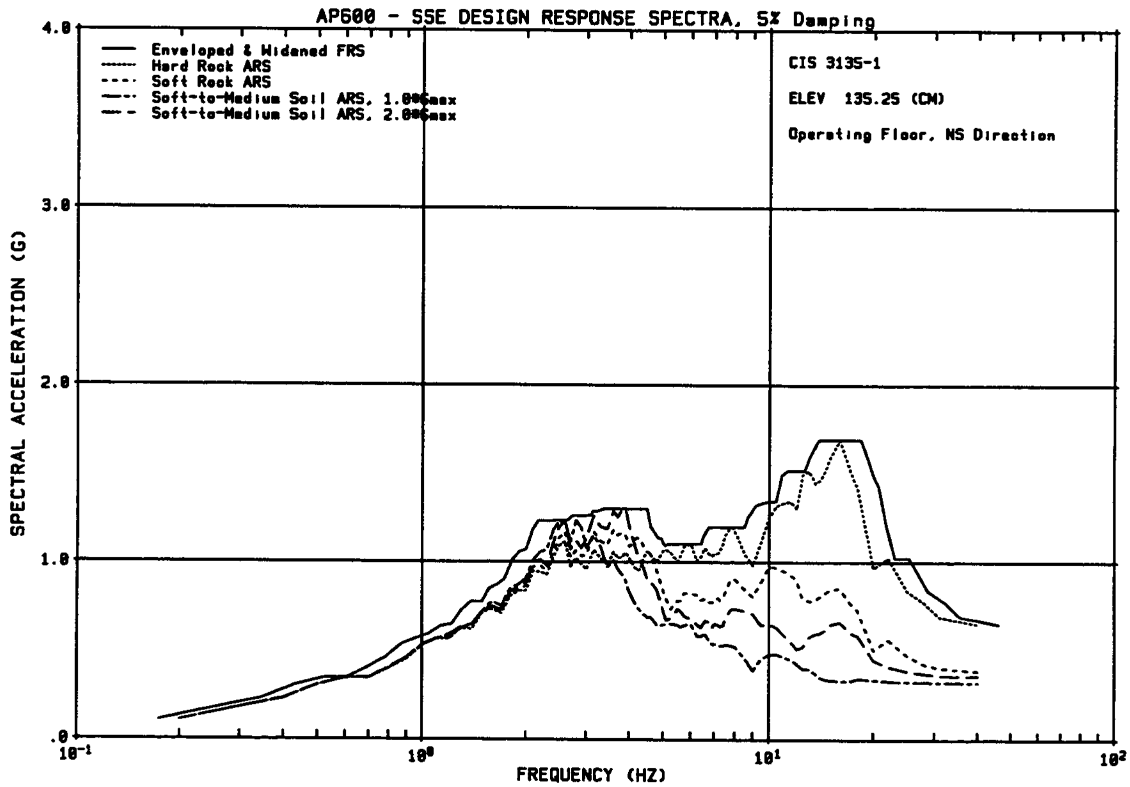


Figure 3.7.2-17 (Sheet 4 of 9)

Containment Internal Structures SSE Floor Response Spectra

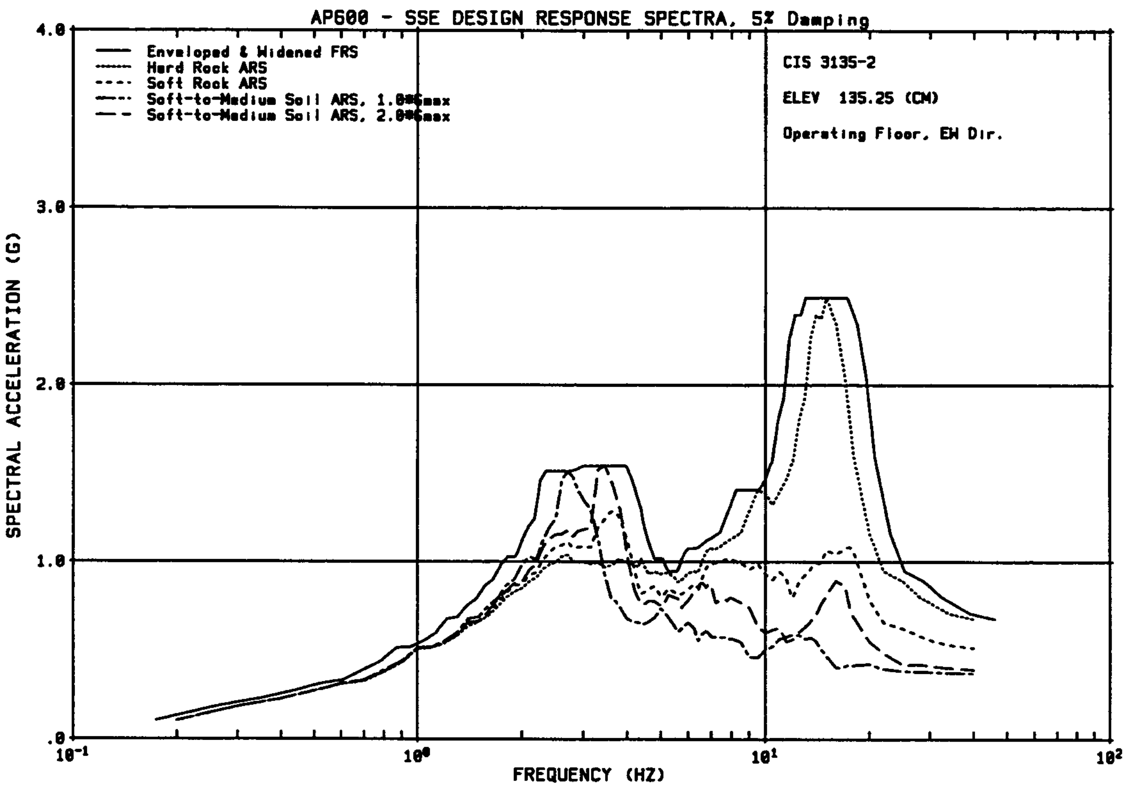


Figure 3.7.2-17 (Sheet 5 of 9)

Containment Internal Structures SSE Floor Response Spectra

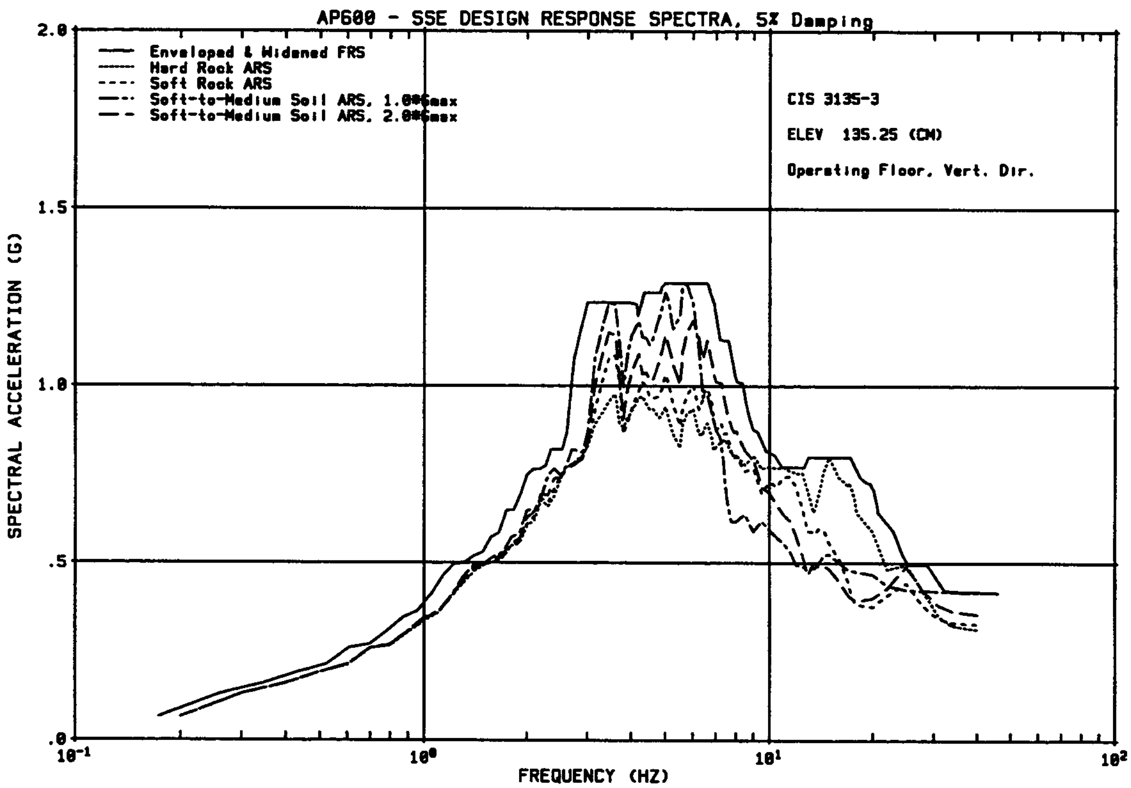


Figure 3.7.2-17 (Sheet 6 of 9)

Containment Internal Structures SSE Floor Response Spectra

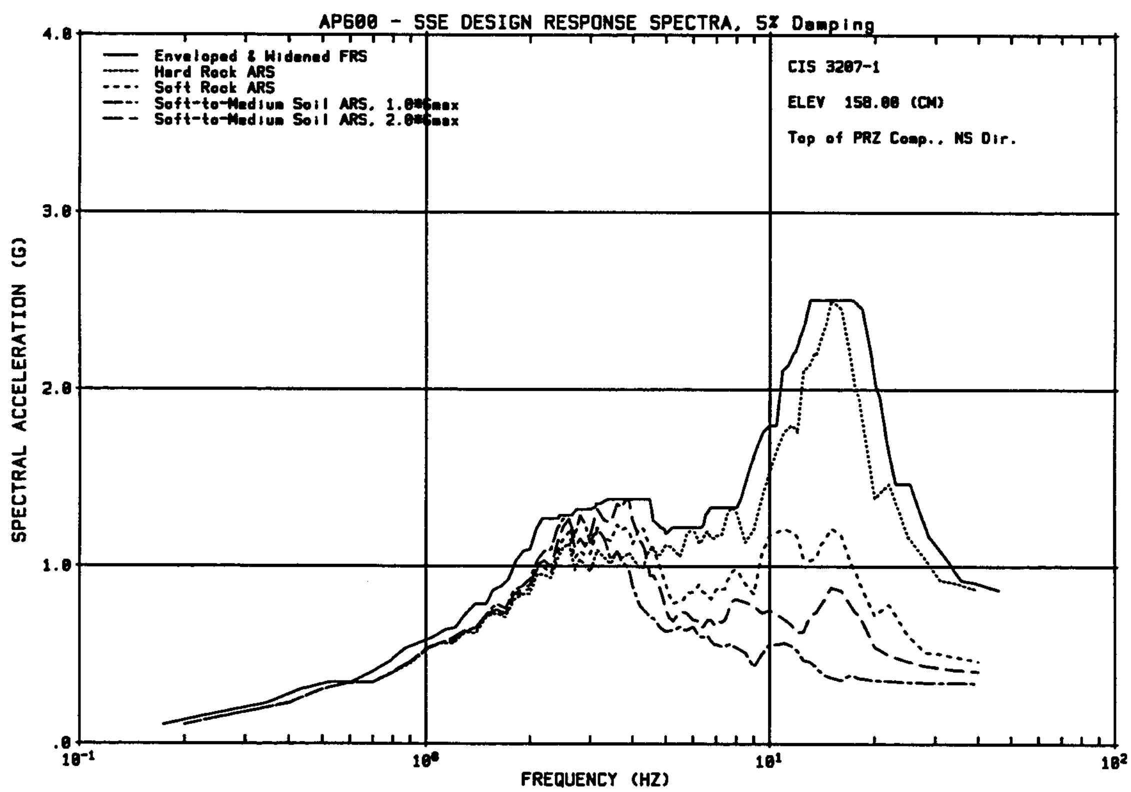


Figure 3.7.2-17 (Sheet 7 of 9)

Containment Internal Structures SSE Floor Response Spectra

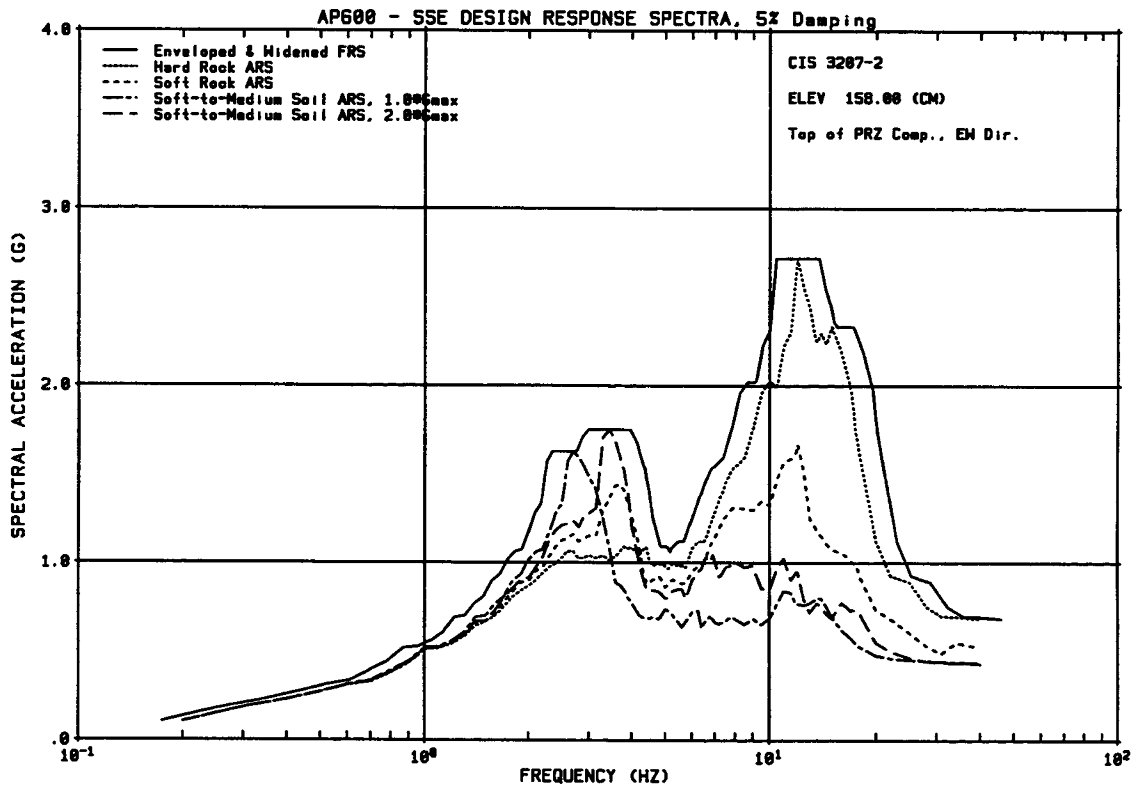


Figure 3.7.2-17 (Sheet 8 of 9)

Containment Internal Structures SSE Floor Response Spectra

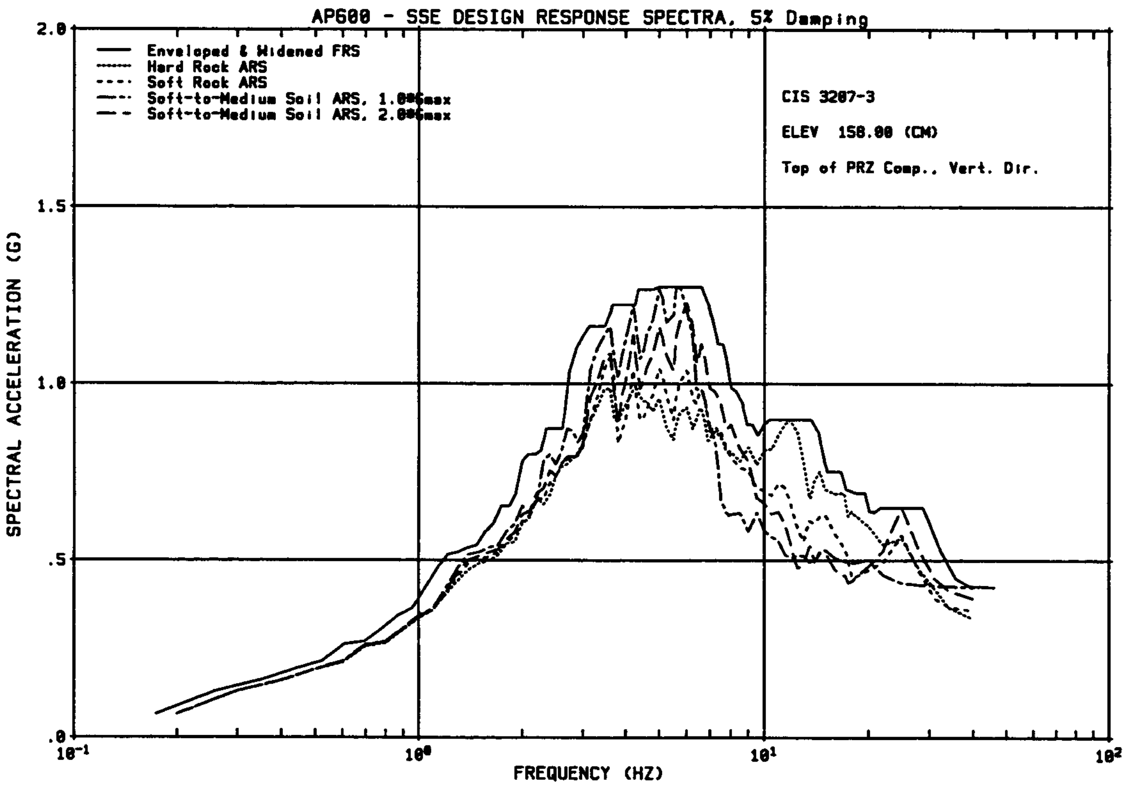


Figure 3.7.2-17 (Sheet 9 of 9)

Containment Internal Structures SSE Floor Response Spectra

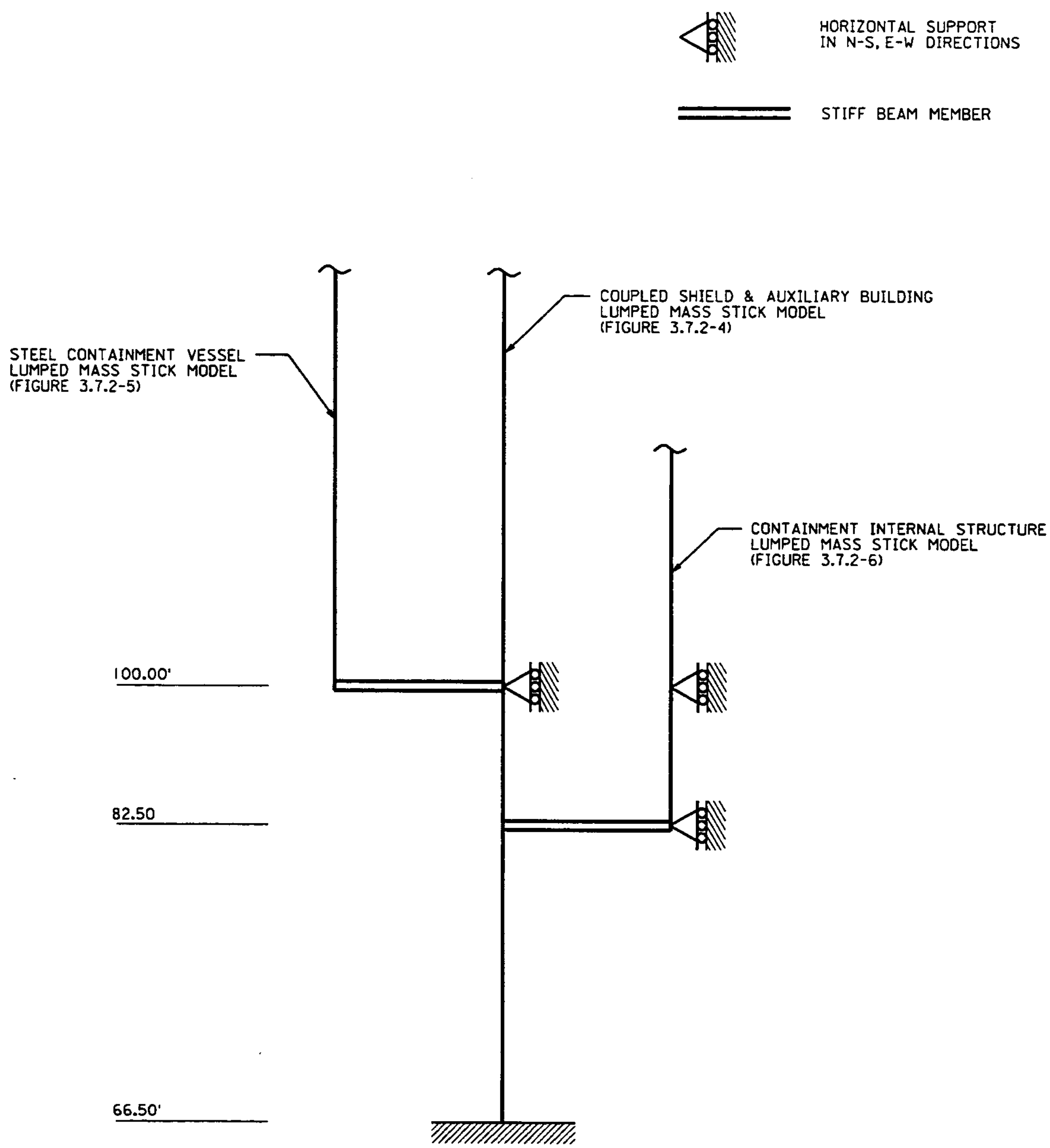


Figure 3.7.2-18 (Sheet 1 of 2)

Connection Between Lumped Mass Stick Model - Fixed Base Analysis

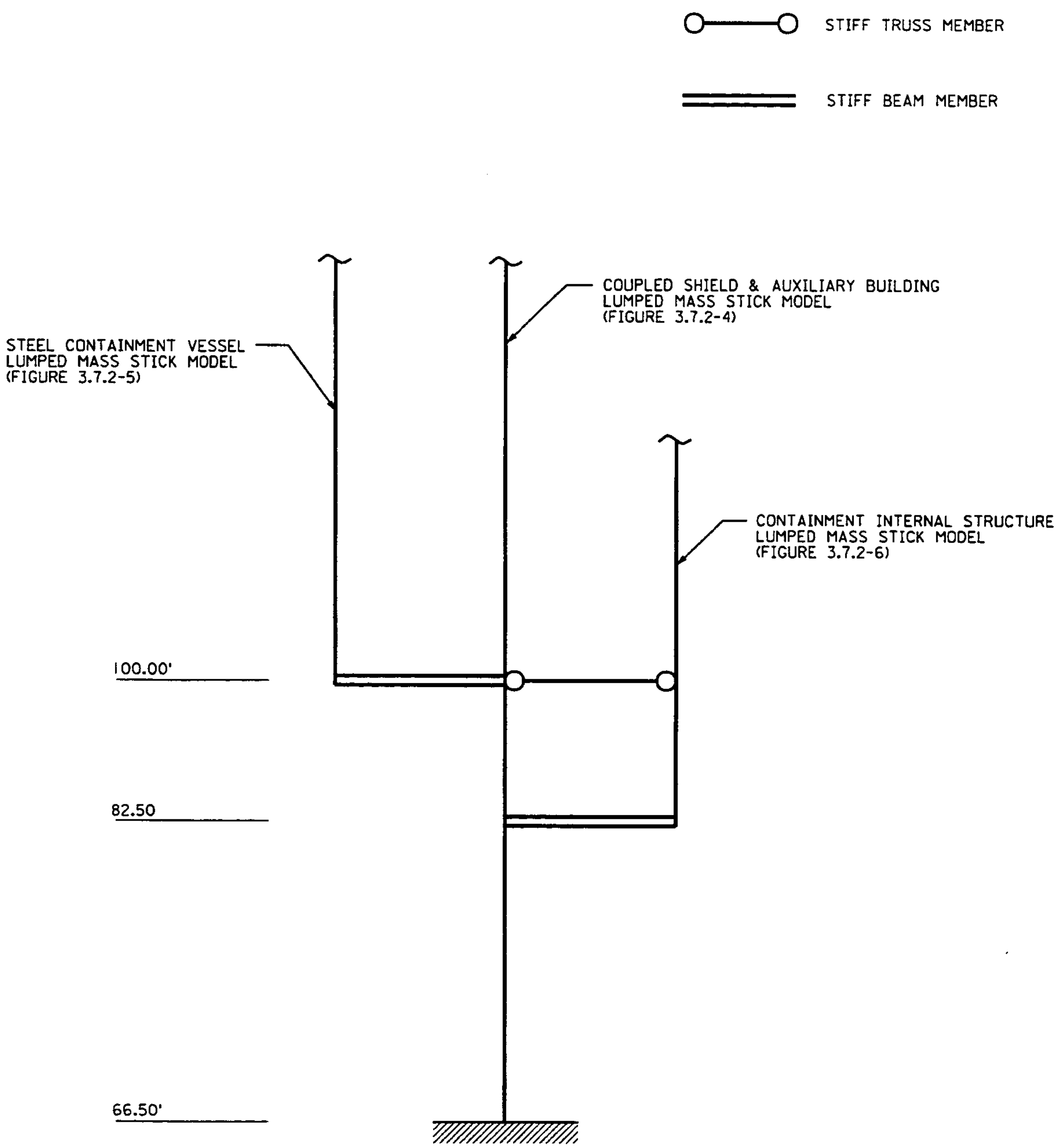


Figure 3.7.2-18 (Sheet 2 of 2)

Connection Between Lumped Mass Stick Model - Sassi Analysis

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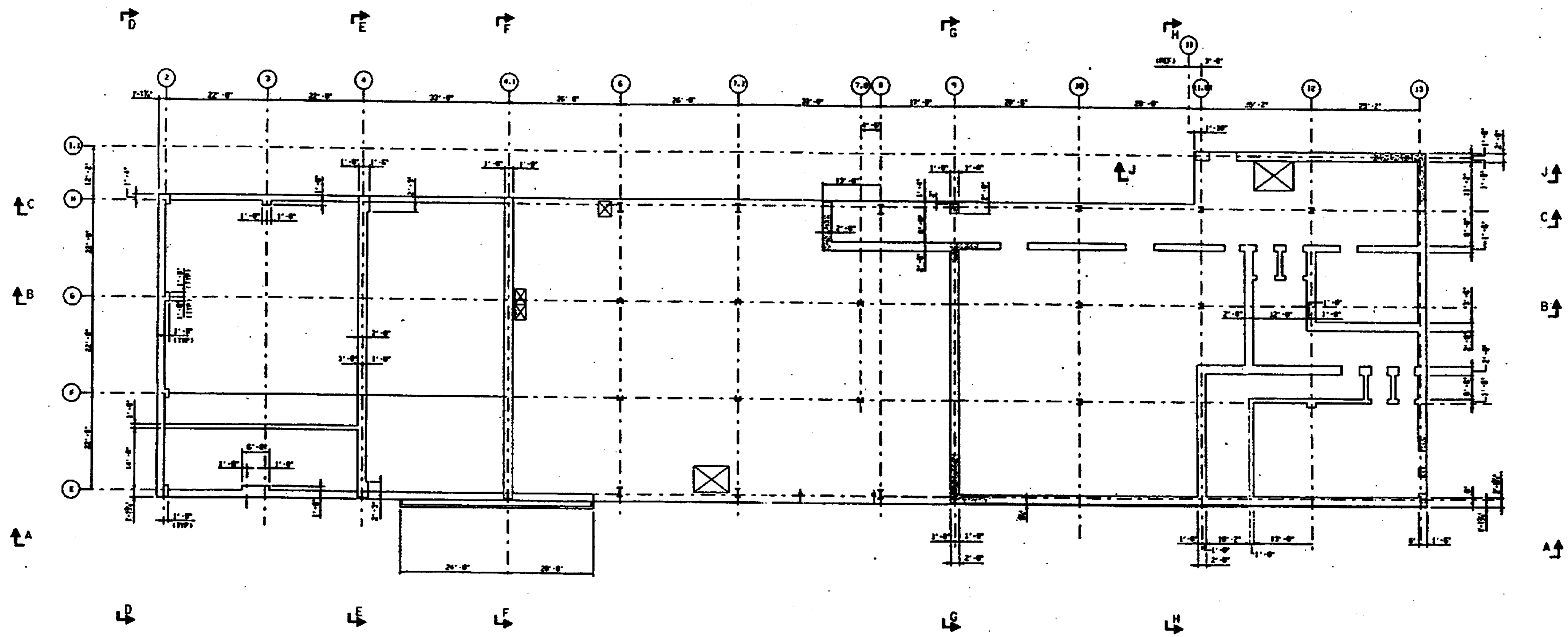


Figure 3.7.2-19 (Sheet 1 of 10)

Annex Building Key Structural Dimensions
Plan at Elevation 100'-0"

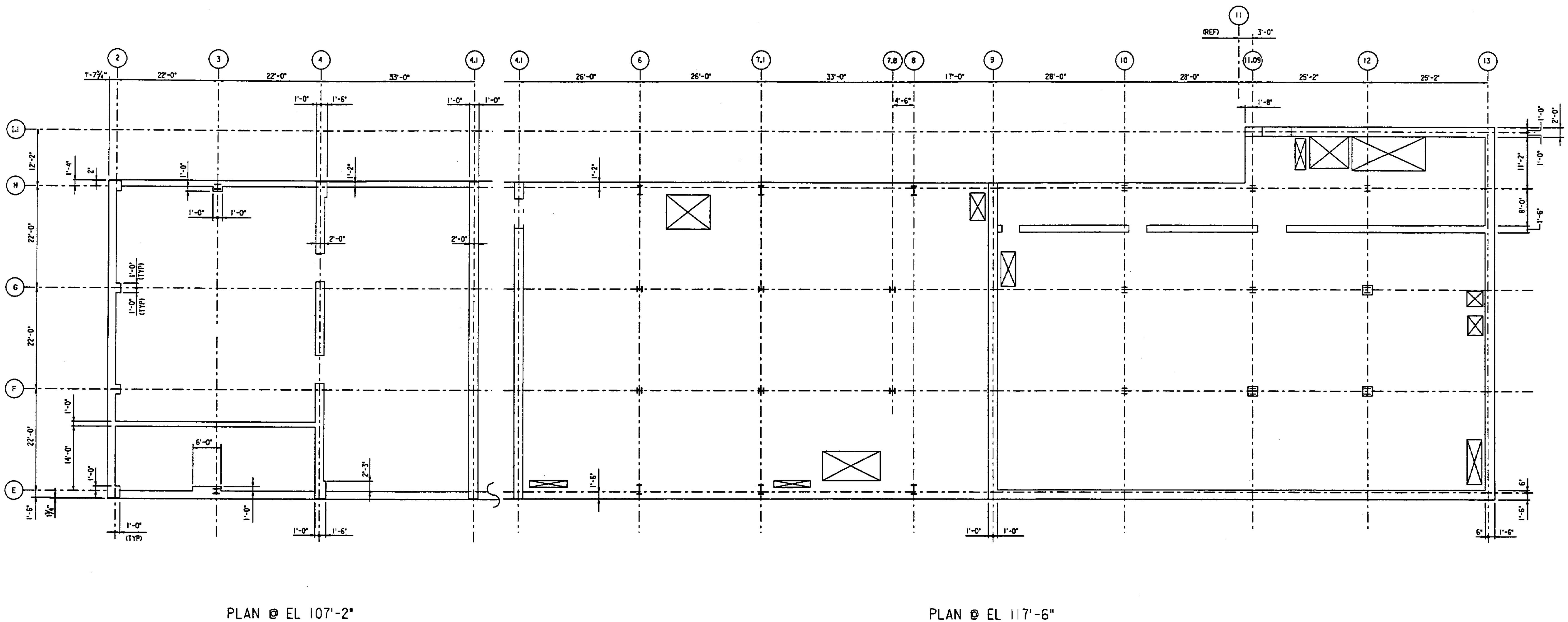


Figure 3.7.2-19 (Sheet 2 of 10)

Annex Building Key Structural Dimensions
Plan At Elevation 107′-2″ and 117′-6″

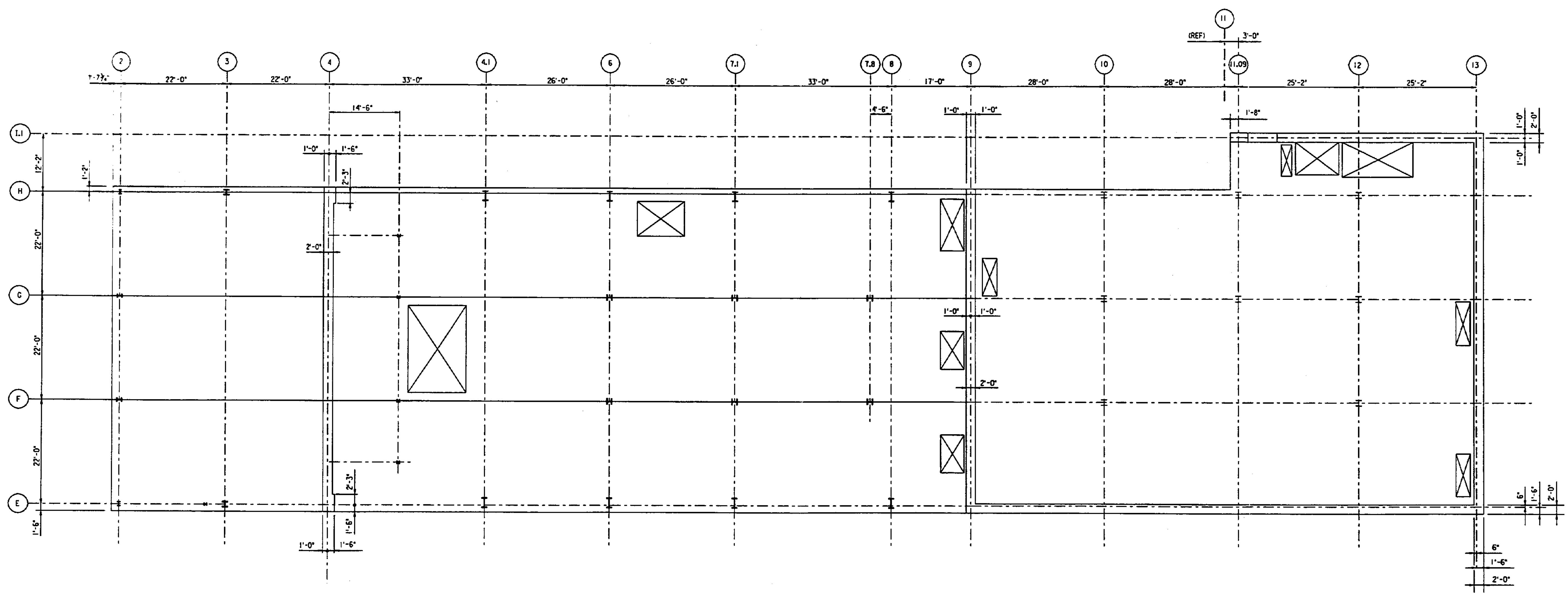


Figure 3.7.2-19 (Sheet 3 of 10)

Annex Building Key Structural Dimensions
Plan At Elevation 135'-3"

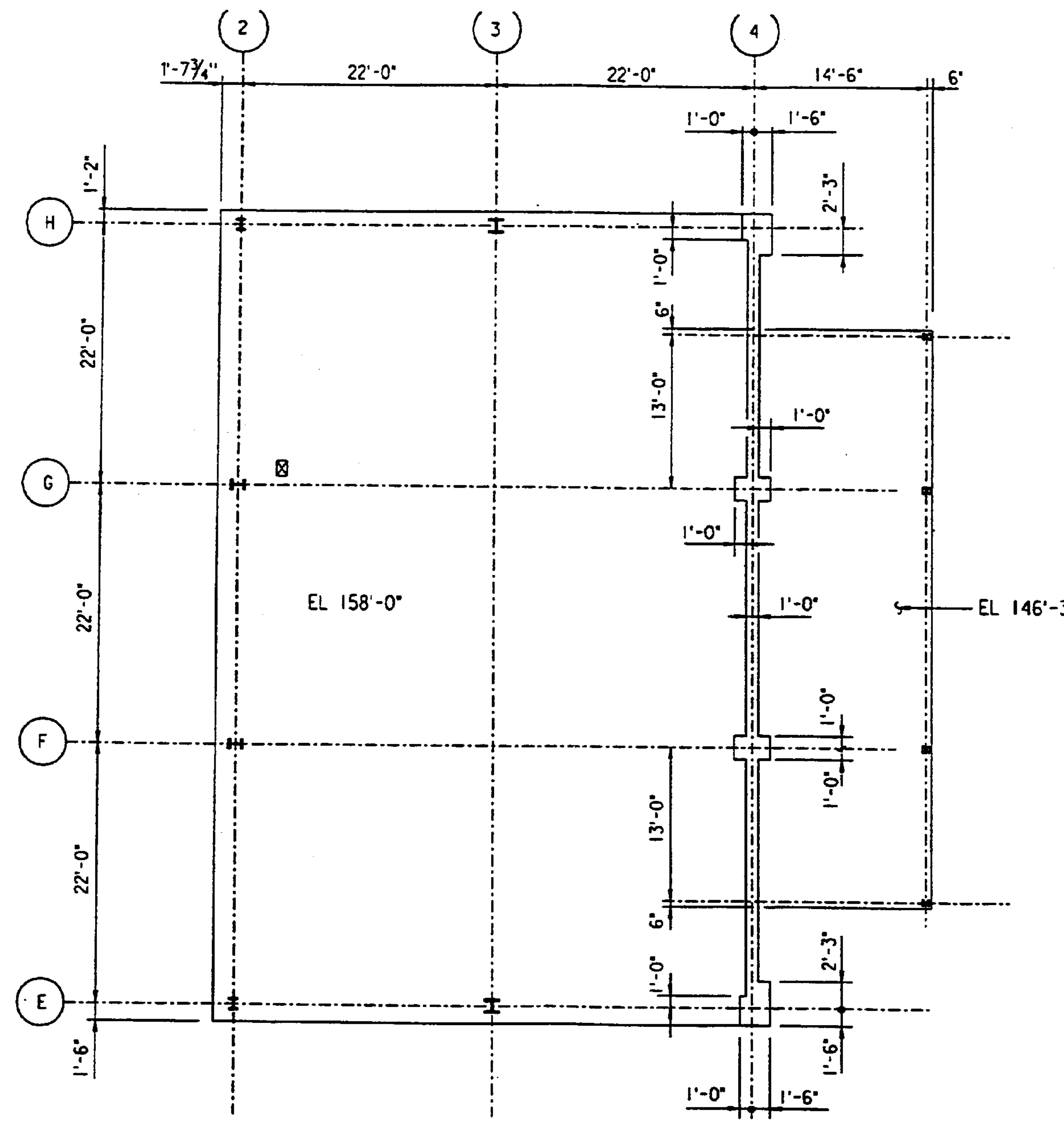


Figure 3.7.2-19 (Sheet 4 of 10)

Nuclear Island Annex Building Key Structural Dimensions Plan At Elevation 158'-0" And 146'-3"

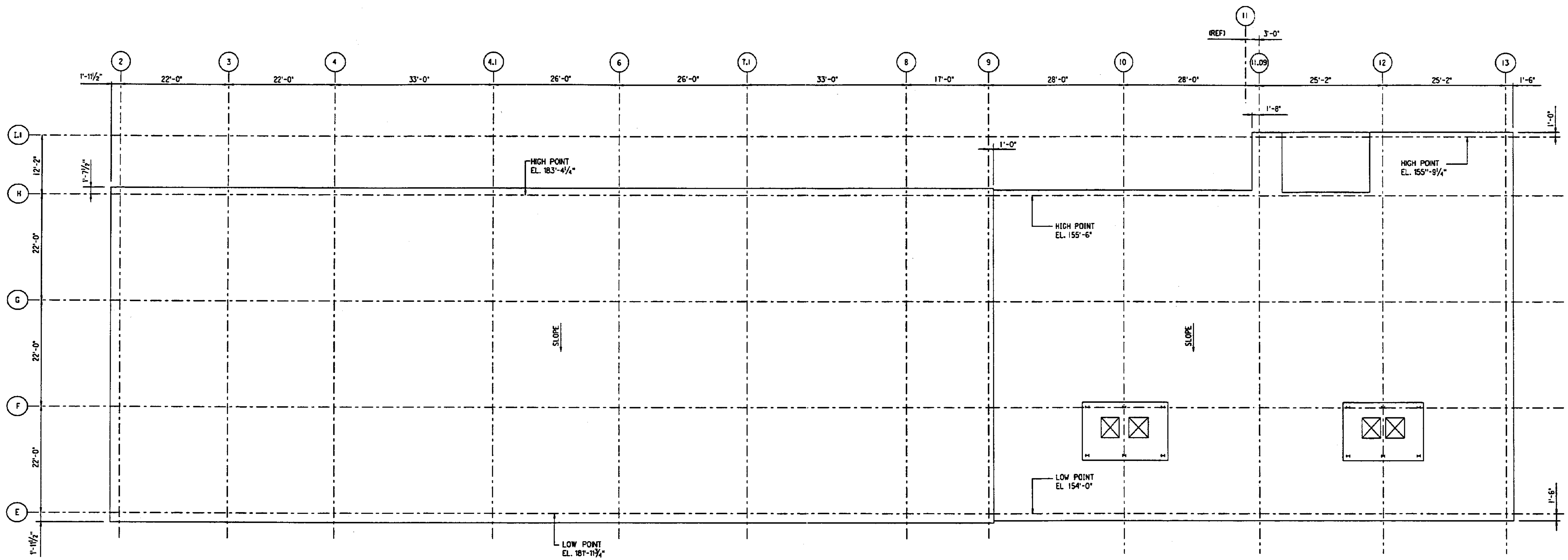
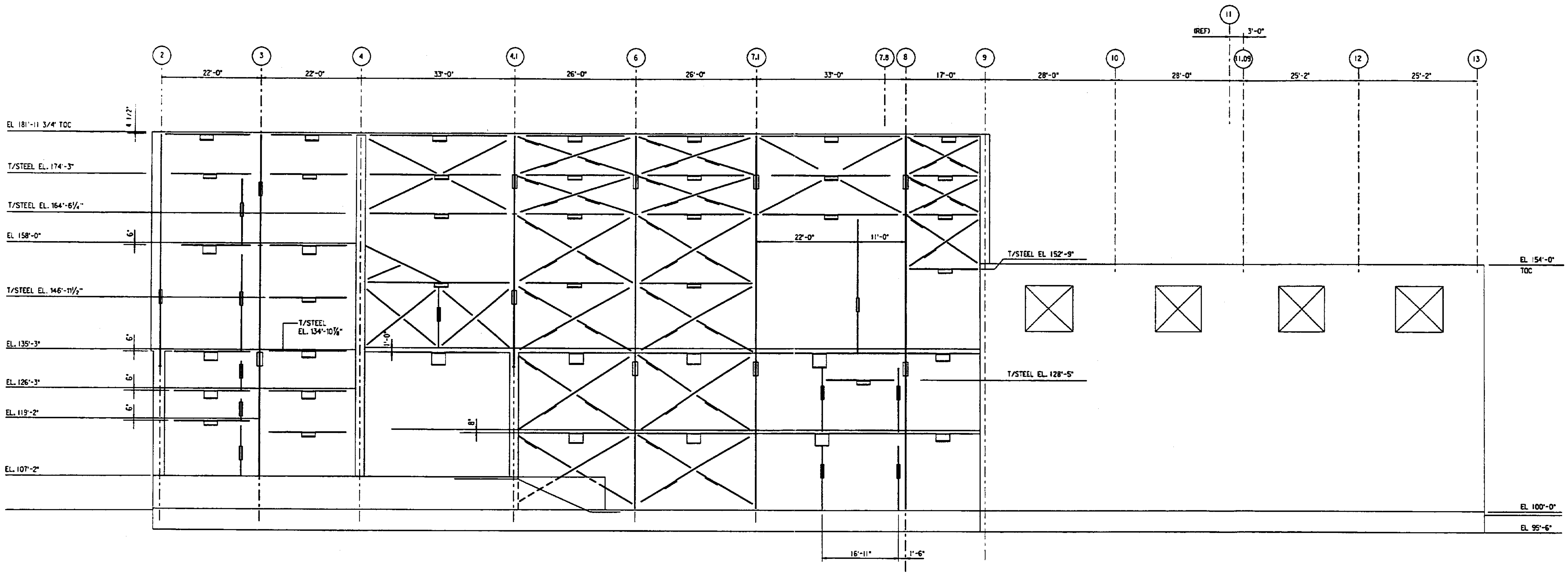


Figure 3.7.2-19 (Sheet 5 of 10)

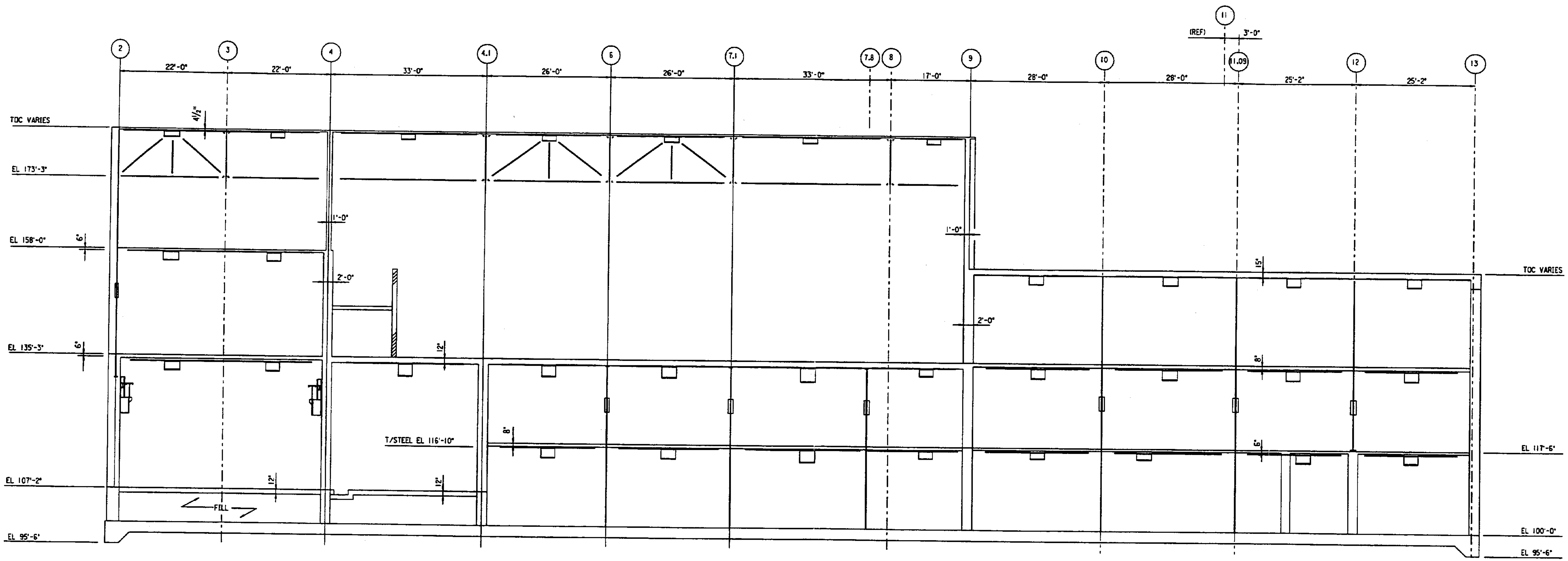
Annex Building Key Structural Dimensions
Roof Plan At Elevation 154'-0" & 181'-11 3/4"



SECTION A-A

Figure 3.7.2-19 (Sheet 6 of 10)

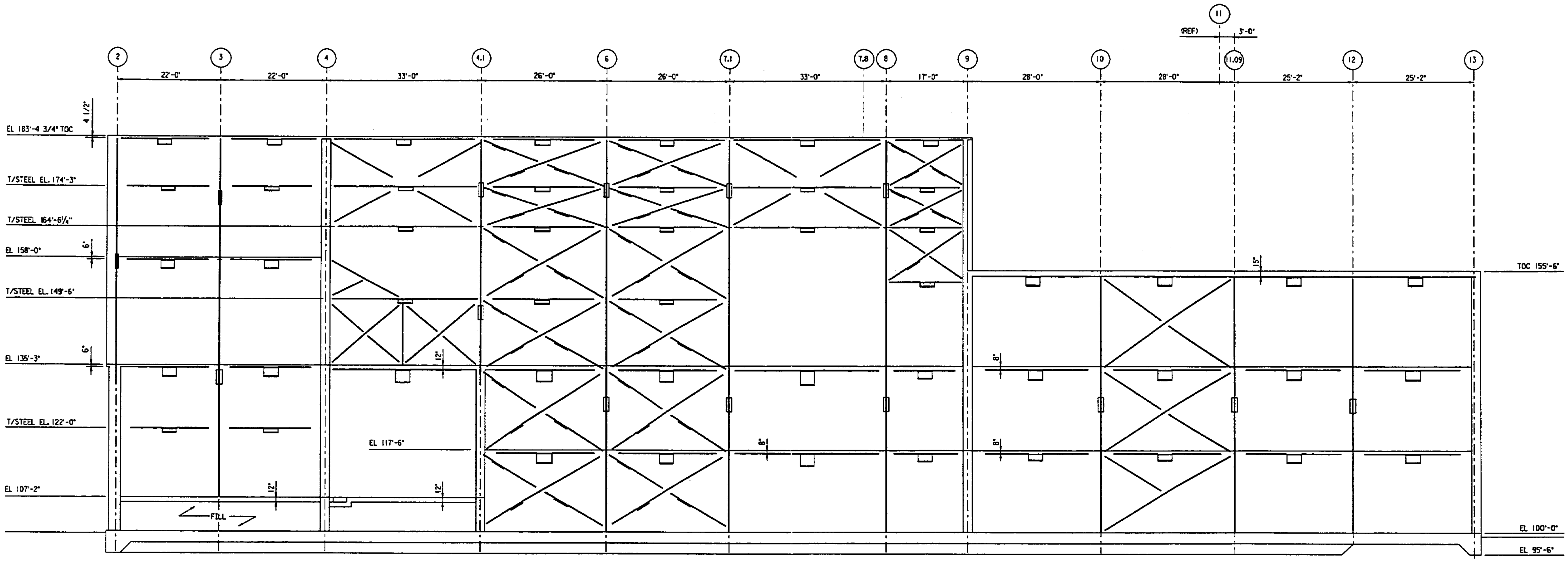
Annex Building Key Structural Dimensions
Section A - A



SECTION B-B

Figure 3.7.2-19 (Sheet 7 of 10)

Annex Building Key Structural Dimensions
Section B - B



SECTION C-C

Figure 3.7.2-19 (Sheet 8 of 10)

Annex Building Key Structural Dimensions
Section C - C

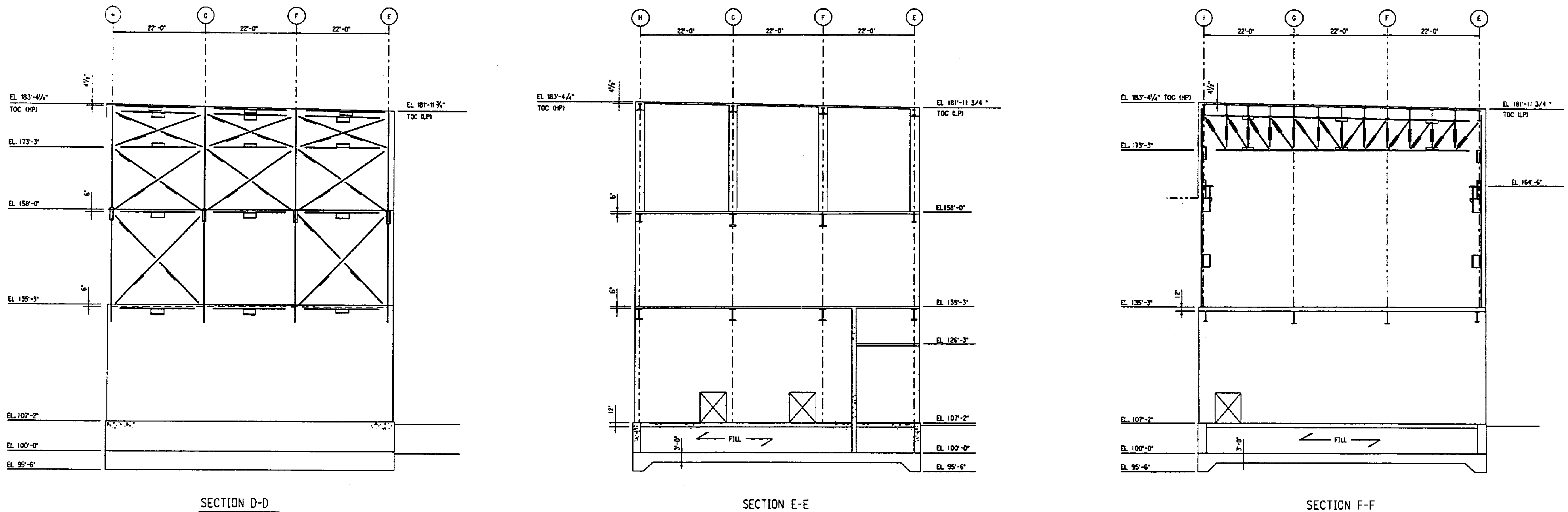
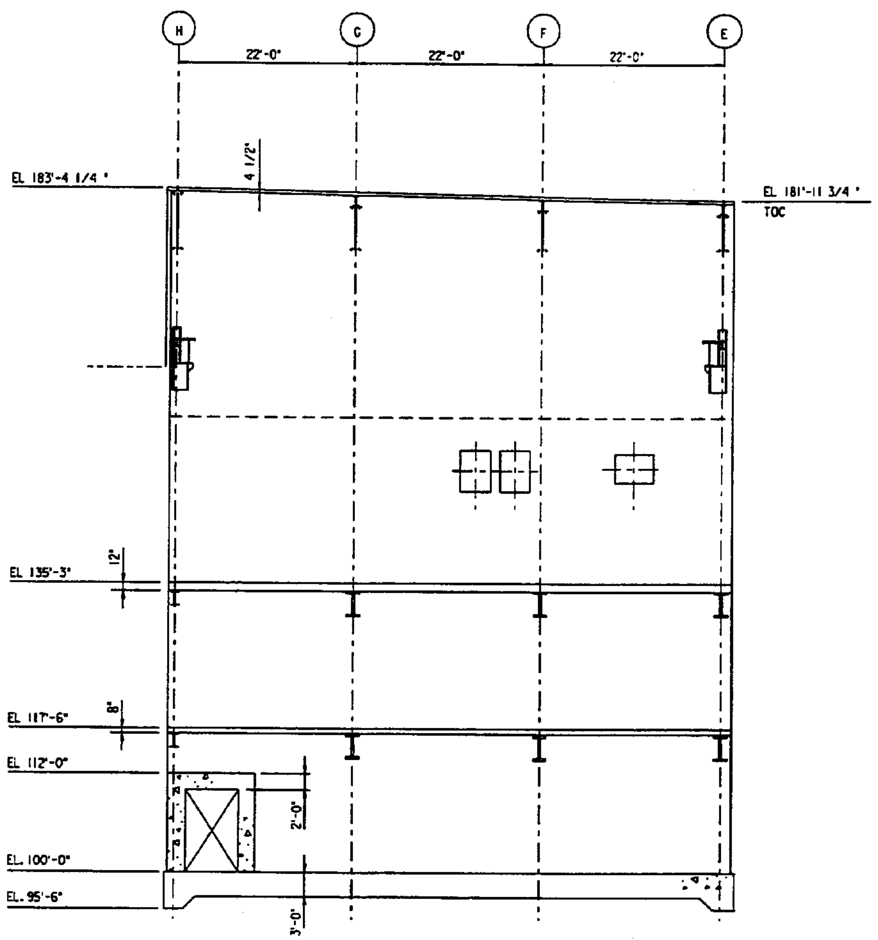
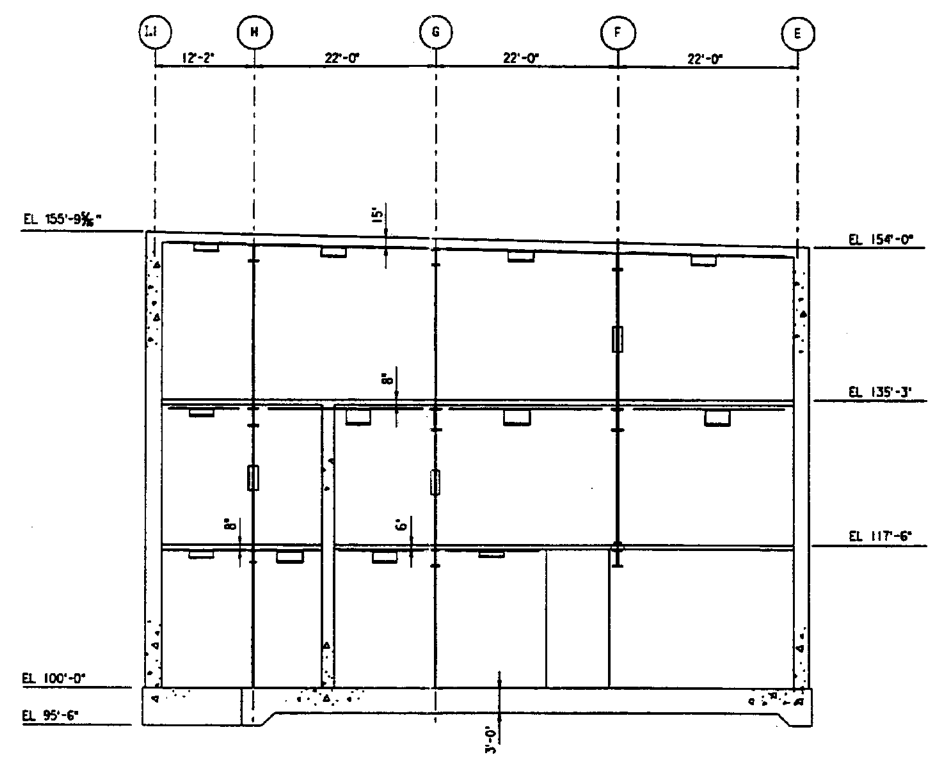


Figure 3.7.2-19 (Sheet 9 of 10)

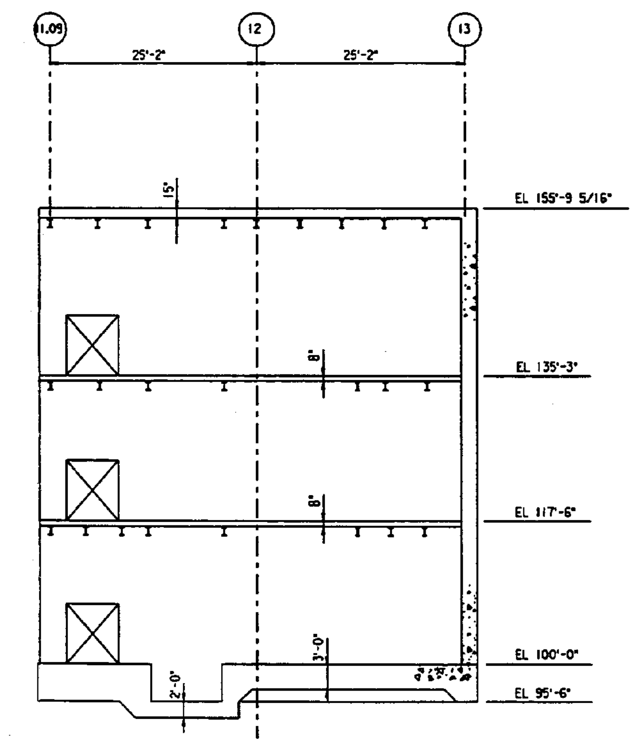
Annex Building Key Structural Dimensions
Sections D - D, E - E, & F - F



SECTION G-G



SECTION H-H



SECTION J-J

Figure 3.7.2-19 (Sheet 10 of 10)

Annex Building Key Structural Dimensions
Sections G - G, H - H, & J - J

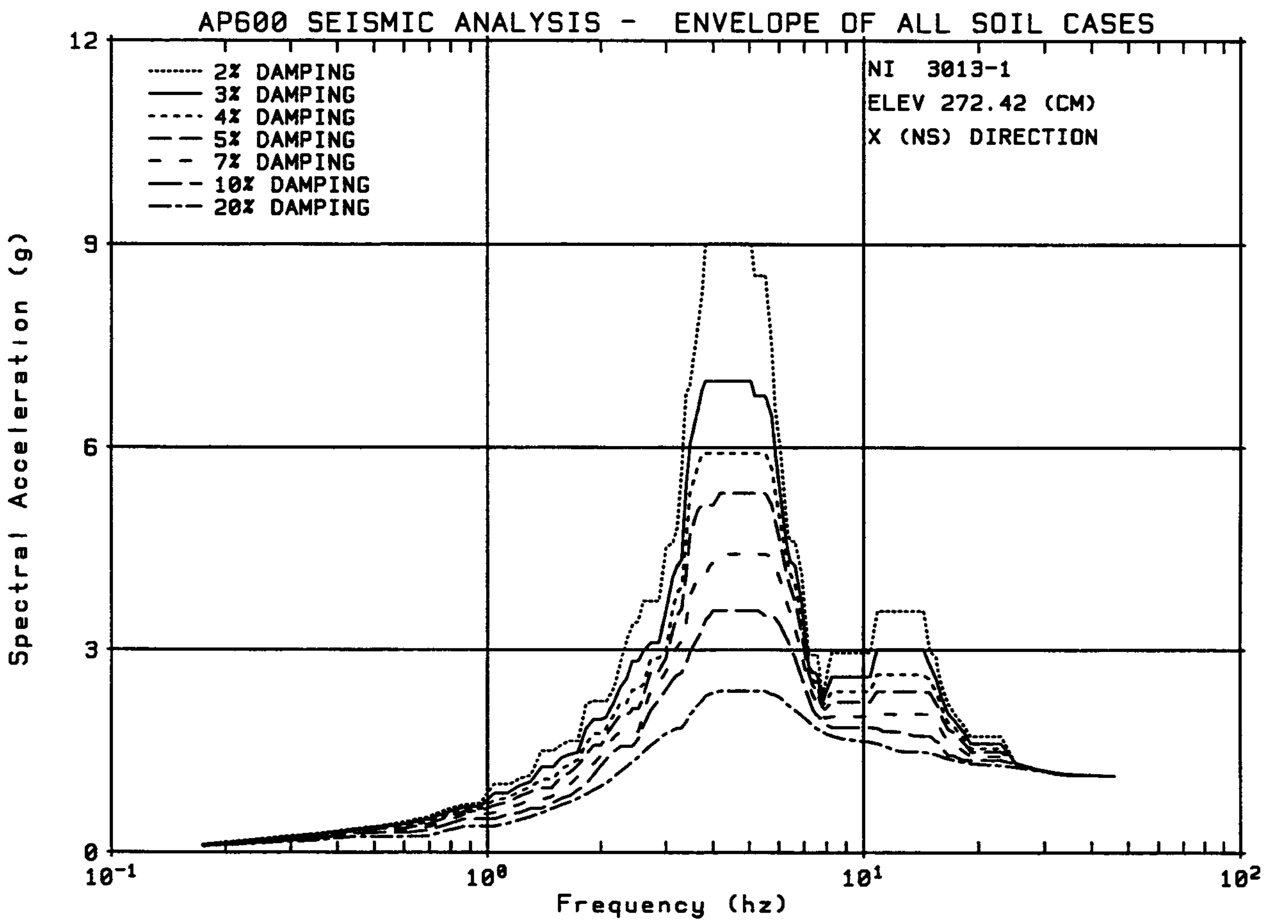


Figure 3.7.2-20 (Sheet 1 of 3)

Coupled Shield & Auxiliary Building SSE Floor Response Spectra

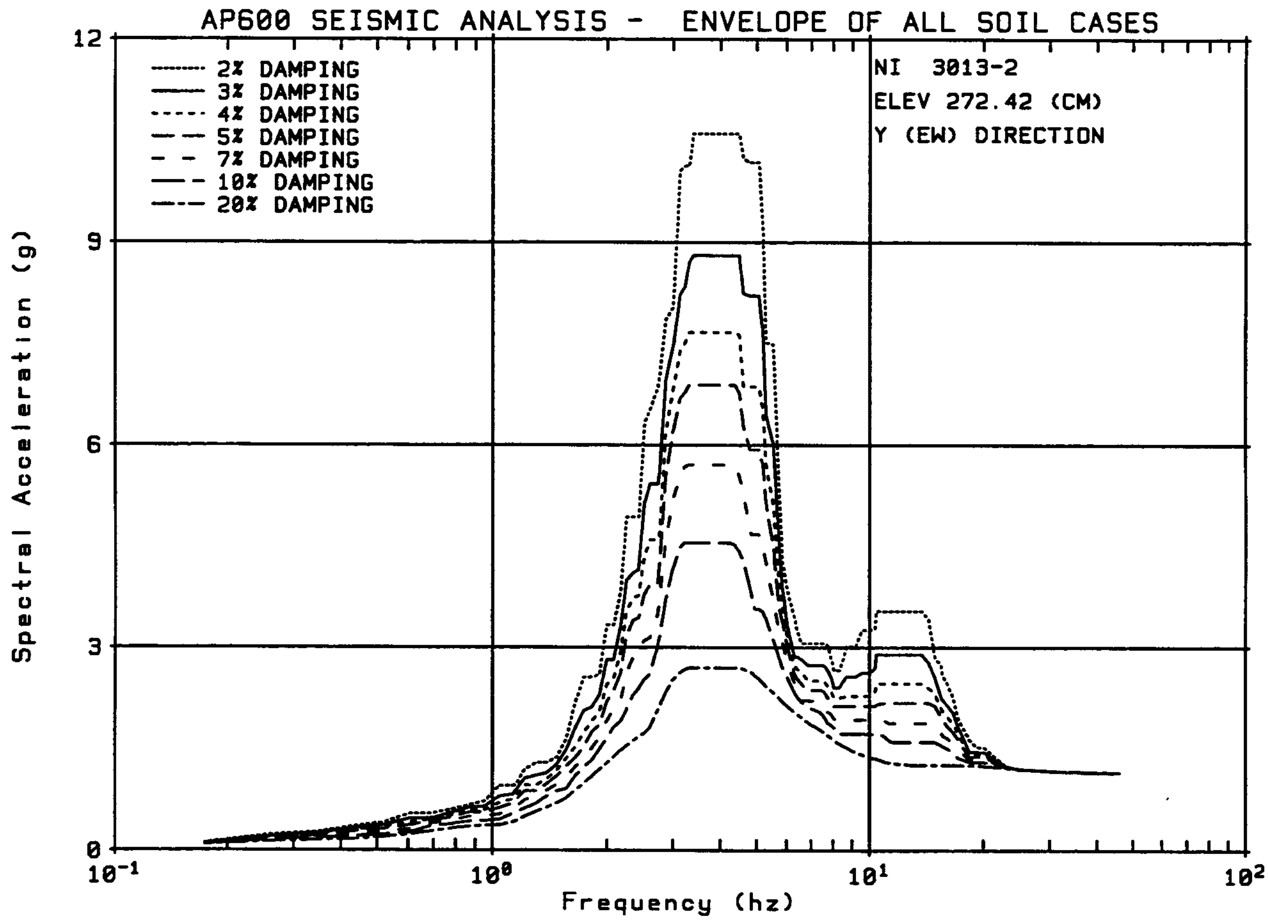


Figure 3.7.2-20 (Sheet 2 of 3)

Coupled Shield & Auxiliary Building SSE Floor Rsponse Spectra

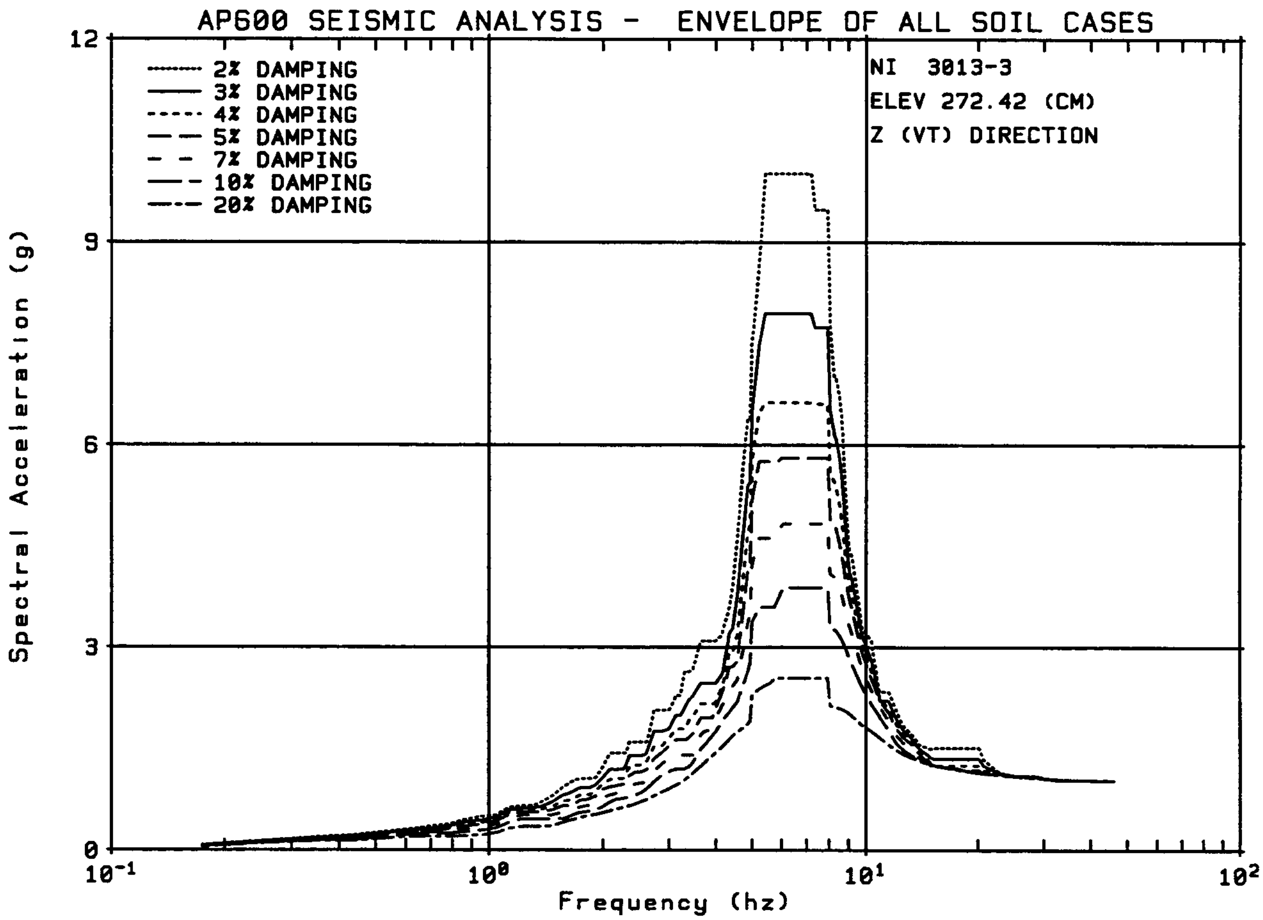


Figure 3.7.2-20 (Sheet 3 of 3)

Coupled Shield & Auxiliary Building SSE Floor Response Spectra

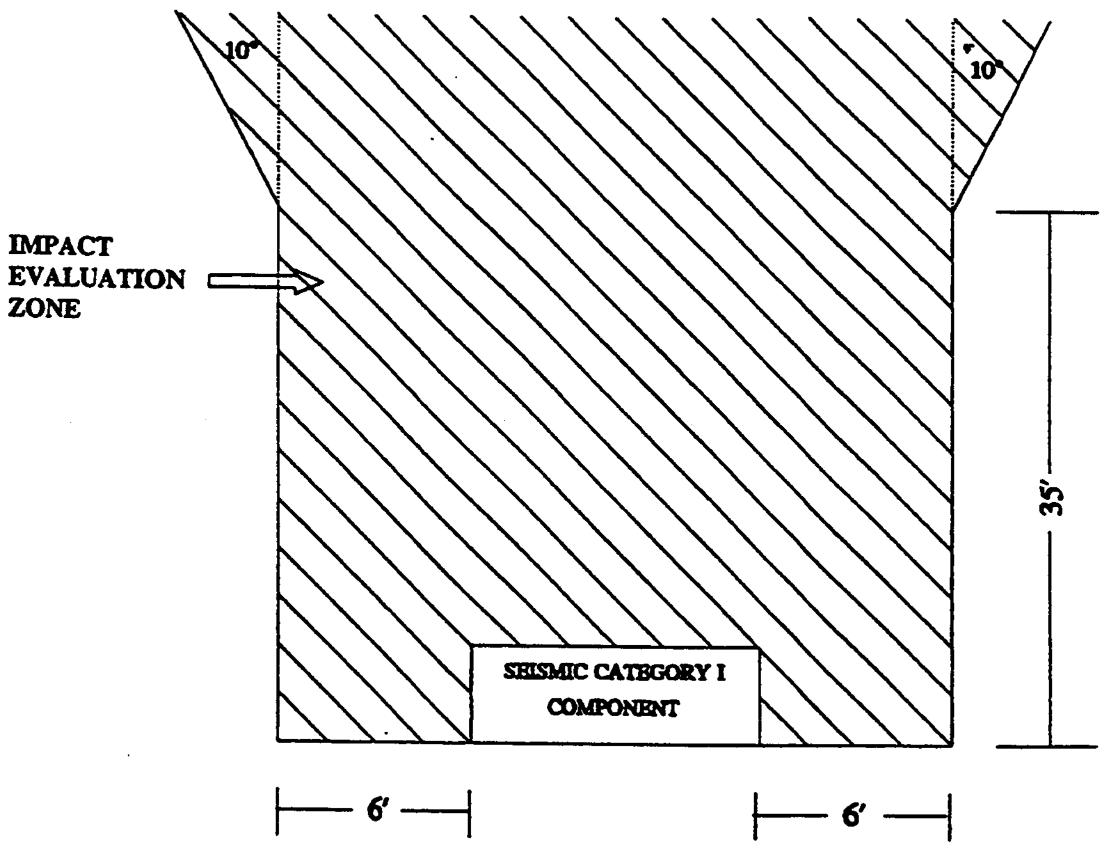


Figure 3.7.3-1

Impact Evaluation Zone

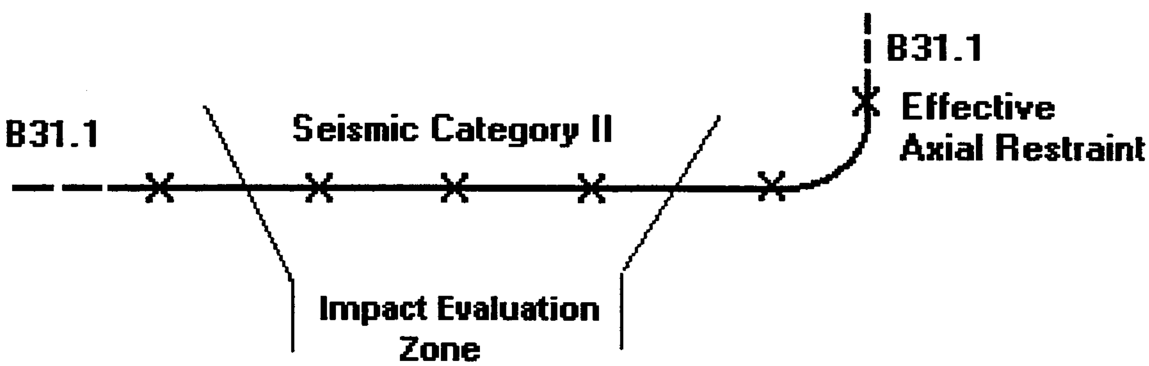


Figure 3.7.3-2

Impact Evaluation Zone and Seismic Supported Piping