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Your ref: Project Number 740
Our ref: DCP/NRC1845

March 16, 2007

Subject: AP1000 COL Response to Request for Additional Information (TR #3)

In support of Combined License application pre-application activities, Westinghouse is submitting a response to an NRC request for additional information (RAI) on AP1000 Standard Combined License Technical Report 3, APP-GW-S2R-010, Rev. 0, Extension of Nuclear Island Structures Seismic Analysis. This RAI response is submitted as part of the NuStart Bellefonte COL Project (NRC Project Number 740). The information included in the response is generic and is expected to apply to all COL applications referencing the AP1000 Design Certification.

The response is provided for request TR3-13, transmitted in NRC letter dated December 5, 2006 from Steven D. Bloom to Andrea Sterdis, Subject: Westinghouse AP1000 Combined License (COL) Pre-application Technical Report 3 – Request for Additional Information (TAC No. MD2358).

Pursuant to 10 CFR 50.30(b), the responses to requests for additional information on Technical Report 3 are submitted as Enclosure 1 under the attached Oath of Affirmation.

It is expected that when the RAIs on Technical Report 3 are complete, the technical report will be revised as indicated in the response and submitted to the NRC. The RAI response will be included in the document.

Questions or requests for additional information related to the content and preparation of this response should be directed to Westinghouse. Please send copies of such questions or requests to the prospective applicants for combined licenses referencing the AP1000 Design Certification. A representative for each applicant is included on the cc: list of this letter.

Very truly yours,

D. F. Hutchings for

A. Sterdis, Manager
Licensing and Customer Interface
Regulatory Affairs and Standardization

00090-psa.doc

D079

/Attachment

1. "Oath of Affirmation," dated March 16, 2007

/Enclosure

1. Response to Request for Additional Information on Technical Report No. 3, RAI-TR03-013

cc:	S. Bloom	-	U.S. NRC	1E	1A
	S. Coffin	-	U.S. NRC	1E	1A
	G. Curtis	-	TVA	1E	1A
	P. Grendys	-	Westinghouse	1E	1A
	P. Hastings	-	Duke Power	1E	1A
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	M. Moran	-	Florida Power & Light	1E	1A
	C. Pierce	-	Southern Company	1E	1A
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	G. Zinke	-	NuStart/Entergy	1E	1A

ATTACHMENT 1

"Oath of Affirmation"

ATTACHMENT 1

UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

In the Matter of:	)
NuStart Bellefonte COL Project	)
NRC Project Number 740	)

APPLICATION FOR REVIEW OF
"AP1000 GENERAL COMBINED LICENSE INFORMATION"
FOR COL APPLICATION PRE-APPLICATION REVIEW

W. E. Cummins, being duly sworn, states that he is Vice President, Regulatory Affairs & Standardization, for Westinghouse Electric Company; that he is authorized on the part of said company to sign and file with the Nuclear Regulatory Commission this document; that all statements made and matters set forth therein are true and correct to the best of his knowledge, information and belief.

W. E. Cummins
Vice President
Regulatory Affairs & Standardization

Subscribed and sworn to
before me this 16th day
of March 2007.

COMMONWEALTH OF PENNSYLVANIA
Notarial Seal
Debra McCarthy, Notary Public
Monroeville Boro, Allegheny County
My Commission Expires Aug. 31, 2009
Member, Pennsylvania Association of Notaries

Debra McCarthy
Notary Public

ENCLOSURE 1

Response to Request for Additional Information on Technical Report No. 3

RAI-TR03-013

RAI Response Number: RAI-TR03-013
Revision: 0

Question:

The first row of the first column of Table 4.2.4-1 describes the shield and auxiliary building model as "3D finite element coarse shell model of auxiliary and shield building [NI20] (including steel containment vessel, polar crane, RCL, and pressurizer). The staff's question is that should the CIS also be included in the model?

Westinghouse Response:

This RAI is similar to RAI-TR03-008 and a response is provided in the response to RAI-TR03-008.

This RAI number is used to track the proposed revisions to the DCD to address the material in the seismic report and the RAIs thereon.

Reference:
None

Design Control Document (DCD) Revision:
The following pages show the proposed revisions to the DCD to incorporate the material covered in detail in the technical report. The DCD revisions include changes to Section 3.7 and the addition of a new appendix providing a summary of the seismic analyses. This new Appendix 3G, "Nuclear Island Seismic Analyses" is added which summarizes the seismic analyses of the nuclear island building structures performed to support design certification. Results of analyses on stick models in Section 3.7 are deleted and replaced by information from the shell models in Appendix 3G.

The revised text of Section 3.7 (excluding subsections 3.7.3 and 3.7.4 which are not affected) is shown with bar marks indicating the locations of change. Existing Tables and Figures in section 3.7.1 are not changed. Figures 3.7.1-15 and 3.7.1-16 are added. Tables in 3.7.2 are deleted. Figures 3.7.2-12 **(as modified for the pressurizer design change in report APP-GW-GLR-016 [TR36])**, 3.7.2-14 and 3.7.2-19 are retained. All other figures in 3.7.2 are deleted.

The new Appendix 3G is shown following the text of Section 3.7.

PRA Revision:
None

Technical Report (TR) Revision:
The Technical Report will be revised to include the RAI responses in an appendix. Thus the proposed DCD revisions will become a part of the technical report.

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 AP1000: 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 AP1000 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 AP1000. 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 plant site are discussed in Section 2.5.

The peak ground acceleration of the safe shutdown earthquake has been established as 0.30g for the AP1000 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 AP1000 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 AP1000 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 AP1000 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 AP1000 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 AP1000 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 foundation level in the free field **at hard rock sites and at the finished grade in the free field at firm rock and soil sites. The definition (characteristics) of hard rock, firm rock, and soil sites are provided in subsection 3.7.1.4.**

3.7.1.2 Design Time History

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

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

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 foundation level 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 maximum frequency of interest in the horizontal and vertical seismic analysis of the nuclear island ~~for the hard rock site~~ is 33 hertz. Modes with higher frequencies are included in the analysis so that the mass in these higher modes is included in the member forces. **The cutoff frequencies used in the soil structure interaction analyses are 33 hertz.** 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 AP1000 (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 AP1000 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 AP1000 augmented power spectral density, anchored to 0.3 g, is as follows:

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

The AP1000 Minimum Power Spectral Density is presented in Figure 3.7.1-9. This AP1000 target power spectral density is compatible with the AP1000 horizontal design response spectra and envelops a target power spectral density compatible with the AP1000 vertical design response spectra. This AP1000 target power spectral density, therefore, is conservatively applied to the vertical response spectra.

The comparison plots of the power spectral density curve of the AP1000 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 4-98 (Reference 3), 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 stiffness-weighted method based on Reference 3. 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

3.7.1.4 Supporting Media for Seismic Category I Structures

The supporting media will be described by the Combined License applicant consistent with the information items in subsection 2.5.4. Seismic analyses for ~~a~~ **both** rock **and soil** sites are described in subsection 3.7.2 **and Appendix 3G.**

The AP1000 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 3.7.1-14.*]* The foundation embedment depth, foundation size, and total height of the seismic Category I structures are presented in Table 3.7.1-2.

For the design of seismic Category I structures, a set of four design soil profiles of various shear wave velocities is established from parametric studies as described in Appendix 3G. These four profiles are sufficient to envelope sites where the shear wave velocity of the supporting medium at the foundation level exceeds 1000 feet per second (see subsection 2.5.2). The design soil profiles include a hard rock site, a firm 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 as follows:

- **For the hard rock site, an upper bound case for rock sites using a shear wave velocity of 8000 feet per second.**
- **For the firm rock site, a shear wave velocity of 3500 feet per second to a depth of 120 feet, and base rock at the depth of 120 feet.**
- **For the upper bound soft-to-medium soil site, a shear wave velocity of 1414 feet per second at ground surface, increasing parabolically to 3394 feet per second at 240 feet, base rock at the depth of 120 feet, and ground water at grade level. The initial soil shear modulus profile is twice that of the soft-to-medium soil site.**
- **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.**

The strain-dependent shear modulus curves for the foundation materials, together with the corresponding damping curves are shown in Figures 3.7.1-15 and 3.7.1-16 for rock material and soil material respectively. The different curves for soil in Figure 3.7.1-16 apply to the range of depth within a soil column below grade.

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 AP1000 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.

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

~~Table 3.7.2-14 and Figure 3.7.2-13~~**Appendix 3G** summarizes the types of models and analysis methods that are used in the seismic analyses of the nuclear island, as well as the type of results that are obtained and where they are used in the design. ~~The dynamic analyses of the nuclear island building structures are performed using the following ANSYS models:~~

~~1. The finite element shell dynamic model of the coupled auxiliary and shield building is a finite element model using primarily shell elements. The portion of the model up to the elevation of the auxiliary building roof is developed using the solid model features of ANSYS, which allow definition of the geometry and structural properties. The nominal element size in the auxiliary building model is about 9 feet so that each wall has two elements for the wall height of about 18 feet between floors. This mesh size, which is the same as that of the solid model, has sufficient refinement for global seismic behavior. It is combined with a finite element model of the shield building roof and cylinder above the elevation of the auxiliary building roof. This model is used to develop modal properties (frequencies and mode shapes). Static analyses are also performed on portions of this model to define properties for the stick model. This model is shown in Figure 3.7.2-1.~~

2.The finite element shell model of the containment internal structures is a finite element model using primarily shell elements. It is developed using the solid model features of ANSYS, which allow definition of the geometry and structural properties. This model is used in both static and dynamic analyses. It models the concrete structures inside the shield building including the basemat. This model is used to develop modal properties (frequencies and mode shapes). Analyses are performed on portions of this model to define properties for the stick model. Static analyses are also performed on the model to obtain member forces in the walls. The walls and basemat inside containment for this model is shown in Figure 3.7.2-2. This model is also used as a superelement in both the finite element shell dynamic model of the nuclear island and in the 3D finite element basemat model (see subsection 3.8.5.4-1).

3.The finite element model of the containment vessel is an axisymmetric model fixed at elevation 100′. This model is used in both static and dynamic analyses. The model is used to develop modal properties (frequencies and mode shapes). Analyses are performed on portions of this model to define properties for the stick model. Static analyses are also performed on the model to obtain shell stresses. This model is shown in Figure 3.8.2-6.

4.The nuclear island lumped mass stick model consists of the stick models of the individual buildings interconnected by rigid links. Each individual stick model is developed to match the modal properties of the finite element models described in 1, 2, and 3 above. Modal analyses and seismic time history analyses are performed using this model. Plant design response spectra are developed from these analyses along with equivalent static seismic accelerations for analysis of the building structures. The individual stick models are shown in Figures 3.7.2-4, 3.7.2-5, and 3.7.2-6. The reactor coolant loop model is shown in Figure 3.7.2-7. The polar crane model is shown in Figure 3.7.2-8. The interconnection between the sticks is shown in Figure 3.7.2-18.

5.The finite element shell dynamic model of the nuclear island is also used in seismic time history analyses. This model uses the coupled auxiliary and shield building described in 1 above. It also includes the finite element model of the basemat inside the shield building and a superelement of the containment internal structures generated from the finite element model described in 2 above. Results from time history analyses from this model are compared to the results from the nuclear island lumped mass stick model. The results are used for development of vertical response spectra and for the equivalent static seismic acceleration of flexible floors and walls and the shield building roof.

The models of the containment internal structures and containment vessel described in 2 and 3 above are also used in equivalent static analyses to provide design member forces in each structure. A separate GTSTRUDL model as shown in Figure 3.8.4-3 is used for static analyses of the shield building roof. Member forces in the auxiliary and shield building are obtained from static analyses of the following model:

6.The equivalent static ANSYS finite element model of the auxiliary and shield building is more refined than the finite element model described in 1 above. This model is developed by meshing one area of the solid model with four finite elements. The nominal element size in this auxiliary building model is about 4.5 feet so that each wall has four elements for the wall height of about 18 feet between floors. This refinement is used to calculate the design member forces and moments for the equivalent static accelerations obtained from the time history analyses of the

~~nuclear island stick model. The stick model of the containment internal structures, which includes the basemat within the shield building, is also included.~~

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 and** fixed base 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 equivalent static acceleration method, ~~and~~ 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 Equivalent Static Acceleration Analysis

Equivalent static analyses, using computer program ANSYS (Reference 36), are performed to obtain the seismic forces and moments required for the structural design of the auxiliary building, the shield building, the steel containment vessel (see subsection 3.8.2.4.1.1), and the containment internal structures on the nuclear island. Equivalent static loads are applied to the finite element models using the maximum acceleration results from the time history analyses **for the four design soil profiles**~~of the stick models described in subsection 3.7.2.1.2~~. Accidental torsional moments are applied as described in subsection 3.7.2-11.

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 including the shield building roof. The effect of the containment internal structures are considered by inclusion of the ~~stick~~ **shell** models. **The equivalent static accelerations are** developed and discussed in ~~subsection 3.7.2.3~~**Appendix 3G,**~~or by use of substructures.~~. ~~Figure 3.7.2-1 shows the finite element model of the coupled shield and auxiliary buildings. In addition, a section of the coupled shield and auxiliary buildings is presented in Figure 3.7.2-3.~~

Equivalent static analyses are ~~performed for the hard rock site where the soil-structure interaction effect is negligible. The analyses are~~ performed using the fixed-base, three-dimensional, finite element models fixed at elevation 63′-6″. The support provided by the embedment below grade is not considered in these analyses.

Containment Internal Structures

Equivalent static 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~~**Appendix 3G**.~~Figure 3.7.2-2 shows the finite element model of the containment internal structures~~.

3.7.2.1.2 Time-History Analysis and Complex Frequency Response Analysis

Mode superposition time-history analyses using computer program ANSYS **and complex frequency response analysis using computer program SASSI** are performed to obtain the in-structure seismic response needed in the analysis and design of seismic subsystems. **Three-dimensional finite element shell models of the nuclear island structures are used in conjunction with the design soil profiles presented in subsection 3.7.1.4 to obtain the in-structure responses. Stick models are coupled to the shell models of the concrete structures for the containment vessel, polar crane, reactor coolant loop, pressurizer and core make up tanks. Two models are used. The fine (NI10) model, as described in subsection 3G.2.2.1, is used to define the seismic response for the hard rock site. The coarse (NI20) model, as described in subsection 3G.2.2.2, is used for the soil structure interaction (SSI) analyses and is set up in both ANSYS and SASSI. The models and analyses are described in Appendix 3G.**

~~The three-dimensional, lumped-mass stick models of the nuclear island structures developed as described in subsection 3.7.2.3 are used to obtain the in-structure responses. The lumped-mass stick models of the nuclear island structures are presented in Figure 3.7.2-4 for the coupled shield and auxiliary buildings, in Figure 3.7.2-5 for the steel containment vessel, in Figure 3.7.2-6 for the containment internal structures, and in Figure 3.7.2-7 for the reactor coolant loop model. The individual building lumped-mass stick models are interconnected with rigid links to form the overall dynamic model of the nuclear island.~~

~~The three-dimensional finite element model of the auxiliary and shield building, or a portion thereof, developed as described in subsections 3.7.2.3 and 3.7.2.3.1 is used to obtain the in-structure vertical response spectra of the auxiliary building including flexible floors. This model is used for the vertical analysis of the auxiliary building since the stick model is developed to match the fundamental vertical frequency of the shield building and does not represent the fundamental vertical frequencies of the auxiliary building, which is significantly lower than the shield building.~~

For the hard rock site, the soil-structure interaction effect is negligible. Therefore, for the hard rock site, the nuclear island is analyzed as a fixed-base structure, using computer program ANSYS without the foundation media. The three components of earthquake (two horizontal and one vertical time histories) are applied simultaneously in the analysis. The base of the stick model is fixed at the bottom of the basemat at elevation 60′-6″. The basemat is 6 feet thick. Since the **NI10** finite element model of the auxiliary and shield building uses shell elements to represent the 6-foot-thick basemat, the nodes of the basemat element are at the center of the basemat (elevation 63′-6″). The finite element model of the containment internal structures uses solid elements, which extend down to elevation 60'-6". When the finite element models are combined and used in the time history analyses, the auxiliary building finite element model is fixed at the shell element basemat nodes (elevation 63'-6") and the base of the containment internal structures is fixed at the bottom of the solid element base nodes (elevation 60'-6"). This difference in elevation of the base fixity is not significant since the concrete between elevations 60'-6" and 63'-6", below the auxiliary building, is nearly rigid. There is no lateral support due to soil or hard rock below grade. This case results in higher response than a case analyzed with full lateral support below grade.

3.7.2.1.3 Response Spectrum Analysis

Equivalent static acceleration and mode superposition time-history methods are primarily used for the evaluation of the nuclear island structures. Response spectrum analyses may be used to perform an analysis of a particular structure or portion of structure using the procedures described in subsections 3.7.2.6, 3.7.2.7, and 3.7.3.

3.7.2.2 Natural Frequencies and Response Loads

Modal analyses are performed for the **shell and** lumped-mass stick models of the seismic Category I structures on the nuclear island **as described in Appendix 3G. Typical results are shown in Appendix 3G.** ~~developed in subsection 3.7.2.3. Table 3.7.2-1 and Figure 3.7.2-9 summarize the modal properties of the stick model representing the coupled shield and auxiliary buildings. Table 3.7.2-2 and Figure 3.7.2-10 show the modal properties of the steel containment vessel. Table 3.7.2-3 (sheet 1) and Figure 3.7.2-11 show the modal properties for the containment internal structures without the reactor coolant loop stick model. Table 3.7.2-3 (sheet 2) shows the modal properties for the reactor coolant loop stick model. Table 3.7.2-4 shows the modal properties of the overall stick model of the nuclear island.~~

~~The time history seismic analysis of the nuclear island considers 200 vibration modes, extending up to a frequency of 83.8 hertz as shown in Table 3.7.2-4. The total cumulative mass participating in the seismic response constitute more than 80 percent of the total mass of the nuclear island.~~

~~Maximum absolute acceleration (ZPA) responses 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 the underside of basemat 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 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.~~

3.7.2.3 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**dynamic** model of the nuclear island structures.

The finite element models of the coupled shield and auxiliary buildings, and the containment internal structures are based on the gross concrete section with the modulus based on the specified compressive strength of concrete. When the finite element or stick models of these buildings are used in time history or response spectrum dynamic analyses, the stiffness properties are reduced by a factor of 0.8 to consider the effect of cracking as recommended in Table 6-5 of FEMA 356 (Reference 5).

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. **The modulus is reduced by a factor of 0.8 to consider the effect of cracking.** Furthermore, the weight density of concrete plus the uniformly distributed miscellaneous dead weights are considered by adding surface mass or 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. Major equipment weights are distributed over the floor area or are included as concentrated lumped masses at the equipment locations. Figures 3.7.2-1 and 3.7.2-2 show, respectively, tThe finite element models of the coupled shield and auxiliary buildings and the containment internal structures **are described in Appendix 3G**. The auxiliary and shield building is modeled with shell elements and the base of the finite element model is at the middle of the basemat at elevation 63′-6″. The bottom of the containment and internal structures are modeled with solid elements and the base of the finite element model is at the underside of the basemat at elevation 60′-6″. The interface between the models is at a radius of 69**71**′-0**6**″ at the inside face**mid surface** of the shield building.

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 shows a typical building section 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 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.

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**Figure 3G.2-4** 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**G.2-2**.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 265′ 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 228′-0″ as shown in Figure 3.8.2-1. It is modeled as a multi-degree of freedom system attached to the steel containment shell at elevation 224′ (midpoint of ring girder) as shown in Figure 3**G.2-4**~~.7.2-5~~. The polar crane is modeled as shown in Figure 3**G.2-5**~~.7.2-8~~ with five masses at the mid-height of the bridge at elevation 233′-6″ and one mass for the trolley. The polar crane model includes the flexibility of the crane bridge girders and truck assembly, and the containment shell's local flexibility. When fixed at the center of containment, the model shows fundamental frequencies of 3.7 hertz transverse to the bridge, 6.4 hertz vertically, and 8.5 hertz along the bridge.

[*During plant operating conditions, the polar crane is parked in the plant north-south direction with the trolley located at one end near the containment shell.*]* In the seismic model, the crane bridge spans 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**G.2-4**~~.7.2-5~~.

3.7.2.3.3 Nuclear Island Seismic Model

The nuclear island seismic models are described in Appendix 3G. The various building ~~lumped-mass stick~~ models are interconnected ~~with rigid links~~ 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 93 mass points and 403 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. **Since the water in the PCCS tank responds at a very low frequency (sloshing) and does not affect building response, the PCCS tank water horizontal mass is reduced to exclude the low frequency water sloshing mass.** ~~The convective (sloshing) effect of the water mass within the passive containment cooling system water tank on the shield building roof 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.

3.7.2.4 Soil-Structure Interaction

Soil-structure interaction is not significant for the nuclear island founded on rock with a shear wave velocity greater than 8000 feet per second. **The soil-structure interaction analyses for the firm rock and soil sites are described in Appendix 3G.**

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. The time-history responses for the hard rock condition are determined from a mode superposition time history analysis using computer program ANSYS. **The time-history responses for the firm rock and soil conditions are determined 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 broadening the **enveloped** nodal response spectra determined for the hard rock site **and soil sites**.

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 broadening procedure used to generate the design floor response spectra.

Floor response spectra for the auxiliary building are obtained from the three-dimensional model as described in ~~subsection 3.7.2.1.2~~**Appendix 3G**. These spectra are developed for the specific location in the auxiliary building. Where spectra at a number of nodes have similar characteristics, a single set of spectra may be developed by enveloping the broadened spectra at each of the nodes.

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 **Appendix 3G.**~~Figures 3.7.2-15 through 3.7.2-17~~.

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 ANSYS, the three components of earthquake are applied either simultaneously or separately. In the ANSYS 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 and equivalent static 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 SASSI analyses.**

- The peak responses due to the three earthquake components from the response spectrum and equivalent static 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 used in the nuclear island basemat analyses, the containment vessel analyses and the shield building roof 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~~**Appendix 3G**, the effect of three components of earthquake are considered as follows:

- Mode Superposition Time History Analysis (program ANSYS) **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~~**Appendix 3G**.

3.7.2.7 Combination of Modal Responses

The modal responses of the response spectrum system structural analysis are combined using the grouping 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~~**Appendix 3G**.

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 soil profiles described in subsection 3.7.1.4. For the hard rock site,** ~~assuming~~ a range of soil properties **are assumed** for the layer above rock at the level of the nuclear island foundation. 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 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~~ **SASSI** finite element soil-structure interaction analyses. The seismic response spectra input at the base of the annex building are the envelopes of the range of soil sites and also envelope the AP1000 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 1997 Uniform Building Code provisions for Zone 3 with an Importance Factor of 1.0. For an eccentrically braced structure the resistance modification factor is 7 (UBC-97, reference 1) using strength design. When using allowable stress design, the allowable stresses are not increased by one third for seismic loads and the resistance modification factor is increased to 10 (UBC-91).

- 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 7-98 (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. 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 **at each elevation**~~stick models shown in Figures 3.7.2-4, 3.7.2-5, and 3.7.2-6~~.
- The maximum absolute 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. ~~This was increased to ±10 percent to apply an additional torsional load to the model so that the member forces in the stick model would match those from the time history analyses.~~
- The torsional moments due to eccentricities of the masses at each elevation are assumed to act in the same direction on each structure.
- The torsional moments are applied in two load cases:
  - TOR-NS Case, T_{NS} – accidental torsional moment caused by a Y-eccentricity of the mass during a shock in the X direction
  - TOR-EW Case, T_{EW} – accidental torsional moment caused by a X-eccentricity of the mass during a shock in the Y direction
- The results of each of these torsional load cases are combined absolutely with the results of the corresponding translation acceleration case. The three directions are then combined as described in subsection 3.7.2.6, i.e.

$$R = \sqrt{\left(|A_{NS}| + |T_{NS}|\right)^2 + \left(|A_{EW}| + |T_{EW}|\right)^2 + A_{VT}^{\ 2}}$$

or

$$R = Fact(1)\left[SIGN(A_{NS})\left(|A_{NS}| + |T_{NS}|\right)\right] + Fact(2)\left[SIGN(A_{EW})\left(|A_{EW}| + |T_{EW}|\right)\right] + Fact(3)A_{VT}$$

where:

R	=	Seismic response (member force, stress or deflection)
A_{NS}	=	NS-Shock Case, response due to x-translation acceleration
A_{EW}	=	EW-Shock Case, response due to y-translation acceleration
A_{VT}	=	VT-Shock Case, response due to z-translation acceleration
Fact(i)	=	[±1.0, ±0.4, ±0.4]
SIGN()	=	Sign of variable in parentheses

3.7.2.12 Methods for Seismic Analysis of Dams

Seismic analysis of dams is site specific design.

3.7.2.13 Determination of Seismic Category I Structure Overturning Moments

Subsection 3.8.5.5.4 describes the effects of seismic overturning moments.

3.7.2.14 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.

3.7.5 Combined License Information

3.7.5.1 Seismic Analysis of Dams

Combined License applicants referencing the AP1000 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 AP1000 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 ~~at rock sites~~ 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.

3.7.5.5 Free Field Acceleration Sensor

The Combined License applicant will determine the location for the free-field acceleration sensor as described in subsection 3.7.4.2.1.

3.7.6 References

1. EPRI Report NP-5930, "A Criterion for Determining Exceedance of the Operating Basis Earthquake," July 1988.

2. Uniform Building Code, 1997.

3. ASCE Standard 4-98, "Seismic Analysis of Safety-Related Nuclear Structures and Commentary," American Society of Civil Engineers, September 1998~~6~~.

4. ASME B&PV Code, Code Case N-411.

5. FEMA 356, "Prestandard and Commentary for the Seismic Rehabilitation of Buildings," Federal Emergency Management Agency, November 2000.

6. Deleted

7. Deleted

8. Deleted

9. Deleted

10. Hyde, S. J., J. M. Pandya, and K. M. Vashi, "Seismic Analysis of Auxiliary Mechanical Equipment in Nuclear Plants," Dynamic and Seismic Analysis of Systems and Components, ASME-PVP-65, American Society of Mechanical Engineers, Orlando, Florida, 1982.

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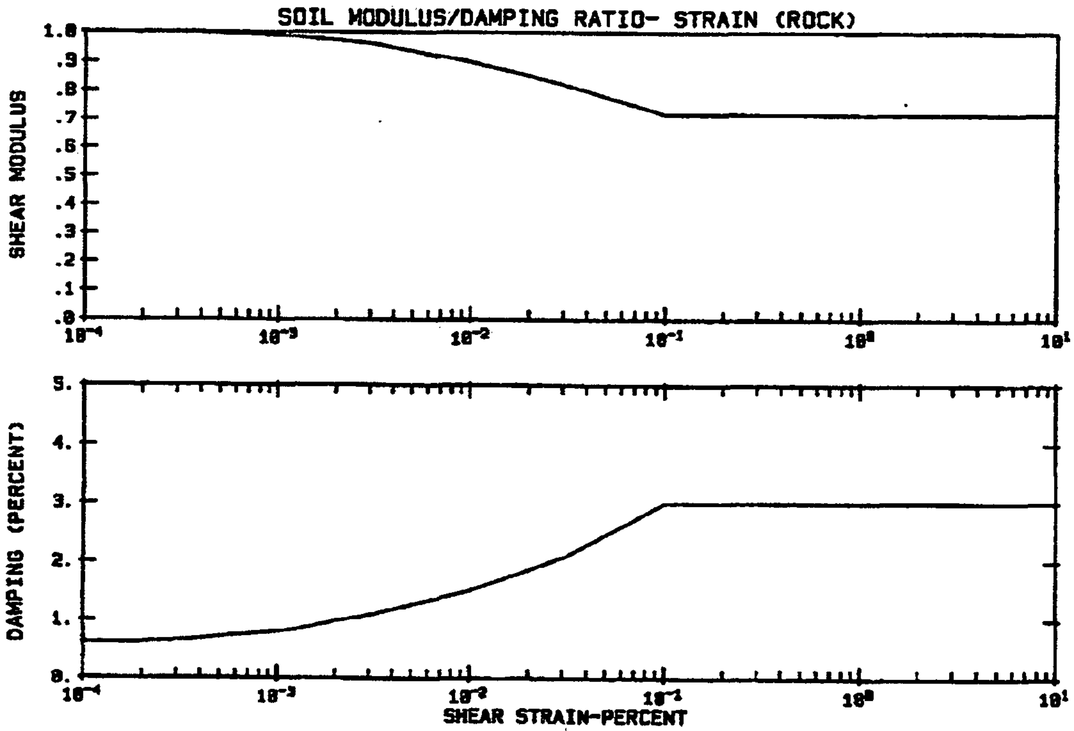

Modulus Reduction Curves for Generic Soil Sites

Figure 3.7.1-15

Strain Dependent Properties of Rock Material (Ref. 37)

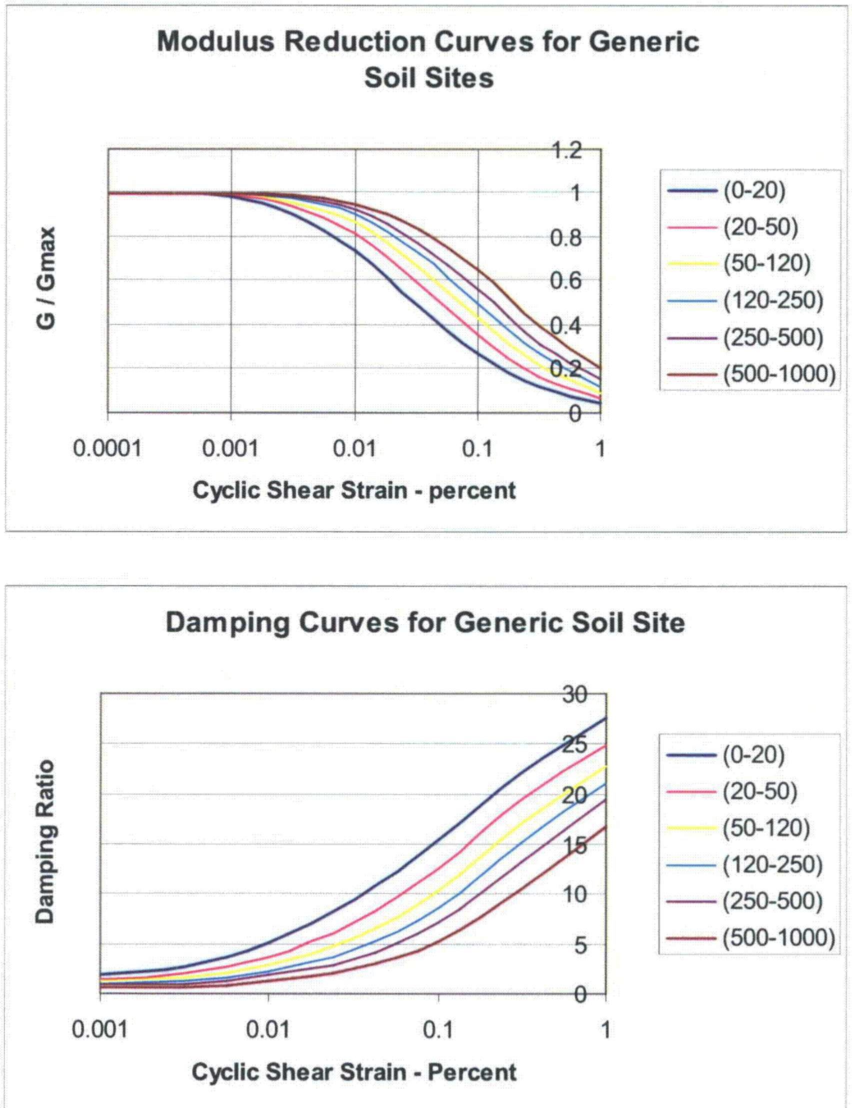

Figure 3.7.1-16

Strain Dependent Properties of Soil Material (Ref. 38)

ADD FILE FOR APPENDIX 3G

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APPENDIX 3G NUCLEAR ISLAND SEISMIC ANALYSES

3G.1 Introduction

This appendix summarizes the seismic analyses of the nuclear island building structures performed to support design certification. The seismic Category I building structures 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 Figures 3.7.1-14 and 3.7.2-12.

Analyses were performed in accordance with the criteria and methods described in Section 3.7. Section 3G.2 describes the development of the finite element models. Section 3G.3 describes the parametric analyses of a range of site parameters and the selection of the parameters used in the design analyses. Section 3G.4 describes the fixed base and soil structure interaction dynamic analyses and provides typical results from these dynamic analyses. Section 3G.5 describes the development of the equivalent static accelerations applied in the detailed seismic design analyses of the buildings. Section 3G.6 describes non-linear analyses considering the effect of lift-off of the basemat from the foundation soil or rock.

The seismic analyses of the nuclear island are summarized in a seismic analysis summary report. Deviations from the design due to as-procured or as-built conditions are acceptable based on an evaluation consistent with the methods and procedures of Sections 3.7 and 3.8 provided the following acceptance criteria are met.

- The structural design meets the acceptance criteria specified in Section 3.8
- The seismic floor response spectra meet the acceptance criteria specified in subsection 3.7.5.4

Depending on the extent of the deviations, the evaluation may range from documentation of an engineering judgment to performance of a revised analysis and design. The results of the evaluation will be documented in an as-built summary report by the Combined License applicant.

Table 3G.1-1 and Figure 3G.1-1 summarize the types of models and analysis methods that are used in the seismic analyses of the nuclear island, as well as the type of results that are obtained and where they are used in the design. Table 3G.1-2 summarizes the dynamic analyses performed and the methods used for combination of modal responses and directional input.

3G.2 Nuclear Island Finite Element Models

The AP1000 nuclear island (NI) consists of three distinct seismic Category I structures founded on a common basemat. The three building structures that make up the nuclear island are the coupled auxiliary and shield building (ASB), the steel containment vessel (SCV), and the containment internal structures (CIS). The shield building and the auxiliary building are monolithically constructed with reinforced concrete and therefore considered one structure. The nuclear island is embedded approximately forty feet with the bottom of basemat at Elevation 60'-6" and plant grade located at elevation 100'-0". The containment vessel is described in subsection 3.8.2, the containment internal structures in subsection 3.8.3, the auxiliary and shield building in subsection 3.8.4 and the nuclear island basemat in subsection 3.8.5.

Seismic systems are defined, according to SRP 3.7.2 (Reference 1), 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. Fixed base seismic analyses are performed for the Nuclear Island at a rock site. Soil-structure interaction analyses are performed for soil sites. The analyses generate a set of in-structure responses (design member forces, nodal accelerations, nodal displacements, and floor response spectra), which are used in the design and analysis of seismic Category I structures, components, and seismic subsystems. Concrete structures are modeled with linear elastic uncracked properties. However, the modulus of elasticity is reduced to 80% of the ACI code value to reduce stiffness to simulate cracking.

3G.2.1 Individual building and equipment models

3G.2.1.1 Coupled auxiliary and shield building

The finite element shell dynamic model of the coupled auxiliary and shield building is a finite element model using primarily shell elements. The portion of the model up to the elevation of the auxiliary building roof is developed using the solid model features of ANSYS, which allow definition of the geometry and structural properties. The nominal element size in the auxiliary building model is about 9 feet so that each wall has two elements for the wall height of about 18 feet between floors. This mesh size, which is the same as that of the solid model, has sufficient refinement for global seismic behavior. It is combined with a finite element model of the shield building roof and cylinder above the elevation of the auxiliary building roof. This model is shown in Figure 3G.2-1. This finite element shell dynamic model is part of the NI10 model.

Since the water in the PCCS tank responds at a very low frequency (sloshing) and does not affect building response, the PCCS tank water mass is reduced to exclude the low frequency water sloshing mass. The wall thickness of the bottom portion of the shield building (elevation 63.5' to 81.5') is modeled as one half (1.5') since the CIS model is connected to this portion and extends out to the mid radius of the shield building cylindrical wall. Local portions of the ASB floors and walls are modeled with sufficient detail to give the response of the flexible areas.

3G.2.1.2 Containment internal structures

The finite element shell model of the containment internal structures is a finite element model using primarily shell elements for the walls and floors and solid elements for the mass concrete. It is developed using the solid model features of ANSYS, which allow definition of the geometry and structural properties. This model is used in both static and dynamic analyses. It models the inner and outer mass concrete basemats embedding the lower portion of the containment vessel, and the concrete structures above the mass concrete inside the containment vessel. The walls and basemat inside containment for this model are shown in Figure 3G.2-2. The basemat (Dish) outside the containment vessel is shown in Figure 3G.2-3. This finite element shell dynamic model is part of the NI10 model. Static analyses are also performed on the model to obtain member forces in the walls. This model is also used in the 3D finite element basemat model (sees subsection 3.8.5.4.1).

3G.2.1.3 Containment vessel

The steel containment vessel is a freestanding, cylindrical, steel shell structure with ellipsoidal upper and lower steel domes. The finite element model of the containment vessel is an axisymmetric model fixed at elevation 100'. Static analyses are performed with this model to obtain shell stresses as described in subsection 3.8.2.4.1.1. The model is also used to develop modal properties (frequencies and mode shapes).

The three-dimensional, lumped-mass stick model of the steel containment vessel is developed based on the axisymmetric shell model. Figure 3G.2-4 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.

The stick model is combined with the polar crane stick model as shown in Figure 3G.2-4. Modal properties of the containment vessel with and without the polar crane are shown in Table 3G.2-1. It is connected to nodes on the dish model. NI10 node numbers are shown in red and NI20 node numbers are shown in black.

The 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 3G.2-2. The shell of revolution vertical model (n = 0 harmonic) has a series of local shell modes of the top head above elevation 265′ 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.

3G.2.1.4 Polar crane

The polar crane is supported on a ring girder which is an integral part of the steel containment vessel at elevation 228′-0″ as shown in Figure 3.8.2-1. It is modeled as a multi-degree of freedom system attached to the steel containment shell at elevation 224′ (midpoint of ring girder) as shown in Figure 3G.2-4. The polar crane is modeled as shown in Figure 3G.2-5 with five masses at the mid-height of the bridge at elevation 233′-6″ and one mass for the trolley. The polar crane model includes the flexibility of the crane bridge girders and truck assembly, and the containment shell's local flexibility. When fixed at the center of containment, the model shows fundamental frequencies of 3.3 hertz transverse to the bridge, 7.0 hertz vertically, and 6.4 hertz along the bridge.

3G.2.1.5 Major Equipment and Structures using Stick Models

The major equipment supported by the CIS is represented by stick models connected to the CIS. These stick models are the reactor coolant loop (RCL) model shown in Figure 3G.2-6, the pressurizer (PZR) model shown in Figure 3G.2-7, and the core make-up tank (CMT) model shown in Figure 3G.2-8. Modal properties of the reactor coolant loop are shown in Table 3G.2-3. The core make-up tank model is only used in the nuclear island fine (NI10) model; the core make-up tank is represented by mass in the nuclear island coarse model (NI20).

3G.2.2 Nuclear Island Dynamic Models

Finite element shell models (3-D) of the nuclear island concrete structures are used for the time history seismic analyses. Stick models are coupled to the shell models of the concrete structures for the containment vessel, polar crane, the reactor coolant loop and pressurizer. Two models are used. The fine (NI10) model is used to define the seismic response for the hard rock site. The coarse (NI20) model is used for the soil structure interaction (SSI) analyses. It is similar to the NI10 model with the exception that the mesh size for the ASB and CIS is approximately 20' instead of 10'. This model is set up in both ANSYS and SASSI. The NI10 and NI20 models are described in the subsections below.

3G.2.2.1 NI10 Model

The large solid-shell finite element model of the AP1000 nuclear island shown in Figure 3G.2-9 combines the auxiliary and shield building (ASB) solid-shell model described in subsection 3G.2.1.1, and the containment internal structure (CIS) solid-shell model described in subsection 3G.2.1.2. The containment vessel and major equipment that are supported by the CIS are represented by stick models and are connected to the CIS. These stick models are the Steel Containment Vessel (SCV) and the polar crane models, the reactor coolant loop (RCL) model, core make-up tank (CMT) models, and the pressurizer (PZR) model. The stick models are described in subsections 3G.2.1.3 and 3G.2.1.4. The CIS and attached sticks are shown in Figure 3G.2-10. This AP1000 nuclear island model is referred to as the NI10 or fine model. The ASB portion of this model has a mesh size of approximately 10 feet.

The SCV is connected to the CIS model using constraint equations. The SCV at the bottom of the stick at elevation 100' (node 130401) is connected to CIS nodes at the same elevation. Figure 3G.2-4 shows the SCV stick model with the constraint equation nodes. The nodes are defined using a cylindrical coordinate system whose origin coincides with the center of containment (node 130401). The CIS vertical displacement is tied rigidly (constrained) to the vertical displacement and RX and RY rotations of node 130401. The CIS tangential displacement is tied rigidly (constrained) to the horizontal displacement and RZ rotation of node 130401.

3G.2.2.2 NI20 Model

The NI20 coarse model has fewer nodes and elements than the NI10 model. It captures the essential features of the nuclear island configuration. The nominal shell and solid element dimension is about 20 feet. It is used in the soil-structure interaction analyses of the nuclear island are performed using the program SASSI. The stick models are the same as used for the NI10 model except that the CMT is not included. This model is shown in Figures 3G.2-11 and 3G.2-12. Modal properties of the auxiliary and shield building are summarized in Table 3G.2-4. Modal properties of the containment internal structures are summarized in Table 3G.2-5. Results of fixed base analyses of the NI20 model were compared to those of the NI10 model to confirm the adequacy of the NI20 model for use in the soil-structure-interaction analyses.

3G.2.2.3 Nuclear Island Stick Model

The nuclear island lumped mass stick model consists of the stick models of the individual buildings interconnected by rigid links. Each individual stick model is developed to match the modal properties of the finite element models described in subsections 3G.2.1.1 and 3G.2.1.2 above. Modal analyses and seismic time history analyses were performed using this model for the hard rock design certification.

The nuclear island lumped mass stick model has been replaced in the design analyses described in this appendix by the NI10 and NI20 finite element shell dynamic models of the nuclear island described in subsections 3G.2.2.1 and 3G.2.2.2 above. A 2D stick model is used in the soil sensitivity parametric analyses described in subsection 3G.3, and in non-linear lift-off analyses as described in subsection 3G.6.

3G.2.3 Static models

The models of the containment internal structures and containment vessel described in subsections 3G.2.1.2 and 3G.2.1.3 above are also used in equivalent static analyses to provide design member forces in each structure. A separate GTSTRUDL model as shown in Figure 3.8.4-3 is used for static analyses of the shield building roof.

Member forces in the auxiliary and shield building are obtained from static analyses of a model that is more refined than the finite element model described in subsection 3G.2.1.1. This model is developed by meshing one area of the solid model with four finite elements. The nominal element size in this auxiliary building model is about 4.5 feet so that each wall has four elements for the wall height of about 18 feet between floors. This finite element shell model is referred to as the NI05 model. This refinement is used to calculate the design member forces and moments using the equivalent static accelerations obtained from the time history analyses of the nuclear island models. The finite element shell model of the containment internal structures described in subsections 3G.2.1.2, which includes the basemat within the shield building and the containment vessel stick model, is also included.

3G.3 2D SASSI Analyses and Parameter Studies

This section describes the parametric analyses performed using 2D models in SASSI to select the design soil cases for the AP1000. The AP1000 footprint, or interface to the soil medium, is identical to the AP600. The AP1000 containment and shield building are 25’ 6” taller than AP600. Results and conclusions from the AP600 soil studies (Reference 2) are considered in establishing the design soil profiles for the AP1000.

Analyses were performed using 2D stick models of the AP1000 for horizontal seismic input with and without adjacent structures for four soil profiles previously evaluated for the AP600. The soil profiles included a hard rock site (HR), a firm rock site, a soft rock site (SR), a soft-to-medium soil site (SMS) and a soft soil site (SS). Analyses were also performed without adjacent structures for firm rock and the upper bound soft to medium sites previously analyzed for the AP600. The soil damping and degradation curves are described in subsection 3.7.1.4. The soil profiles selected for the AP1000 utilize the same parameters on depth to bedrock, depth to water table and variation of shear wave velocity with depth as those used in the AP600 design analyses. The Poisson’s ratio is 0.25 for rock sites (hard and firm rock) and 0.35 for soil sites (soft-to-medium soil , and upper bound soft-to-medium soil). For all the soil profiles defined, the base rock has been taken to be at 120 feet below grade level. The soil profiles are shown in Figure 3G.3-1. The shear wave velocity profiles and related governing parameters are as follows:

- For the hard rock site, an upper bound case for rock sites using a shear wave velocity of 8000 feet per second.
- For the firm rock site, a shear wave velocity of 3500 feet per second to a depth of 120 feet, and base rock at the depth of 120 feet.

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

- 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 soft soil site, a shear wave velocity of 1000 feet per second at ground surface, increasing linearly to 1200 feet per second at 240 feet, base rock at the depth of 120 feet, and ground water is assumed at grade level.

The analyses with and without adjacent structures demonstrated that the effect of adjacent buildings on the nuclear island response is small. Based on this the 3D SASSI analyses of the AP1000 nuclear island can be performed without adjacent buildings similar to those performed for the AP600.

The maximum acceleration values obtained from the AP1000 analyses without adjacent structures are given in Table 3G.3-1. The soil cases giving the maximum response are shown in bold. Floor response spectra associated with nodes 41, 120, 310, 411 and 535 for the six AP1000 soil cases are shown in Figures 3G.3-2 to 3G.3-11.

Based on review of the above results, three soil conditions were selected for 3D SASSI analyses in addition to the hard rock condition evaluated in the existing AP1000 Design Certification. Thus, the following four soil and rock cases identified in subsection 3.7.1.4 are considered: hard rock; firm rock; upper bound soft to medium soil and soft to medium soil.

3G.4 Nuclear island dynamic analyses

3G.4.1 ANSYS fixed base analysis

The NI10 model described in subsection 3G.3.2.2.1 was analyzed by time history modal superposition. To perform the time history analysis of this large model, the ANSYS superelement (substructuring) techniques were applied. Substructuring is a procedure that condenses a group of finite elements into one element represented as a matrix. The reasons for substructuring are to reduce computer time of subsequent evaluations. Two sets of analyses were performed. To obtain the time history response of the ASB, the ASB finite element model was merged with the superelement of the CIS and its major components. To obtain the time history response of the CIS, the CIS finite element model was merged with the superelement of the ASB.

Deflection time history responses were obtained at selected representative locations. These locations included major wall and floor intersections and nodes at the cardinal orientations at key elevations of the shield building. Nodes were also selected at mid span on flexible walls and floors. Typical locations are shown for the auxiliary and shield building at elevation 135' on Figures 3G.4-1 and 3G.4-2. Figure 3G.4-1 shows the "rigid" locations and Figure 3G.4-2 shows the "flexible" locations.

3G.4.2 3D SASSI Analyses

The computer program SASSI2000 is used to perform Soil-Structure Interaction analysis with the NI20 Coarse Finite Element Model. The SASSI Soil-Structure Interaction analyses are performed for the three soil conditions established from the AP1000 2D SASSI analyses. These soil conditions are firm rock, upper bound soft-to-medium soil, and soft-to-medium soil. The model includes a surrounding layer of excavated soil and the existing soil media as shown in Figures 3G.4-3 and 3G.4-4. Acceleration time histories and floor response spectra are obtained. Adjacent structures have a negligible effect on the nuclear island structures and thus are not considered in the 3D SASSI analyses.

In these analyses, the three components of ground motions (N-S, E-W and vertical direction) are input separately. Each design acceleration time history (N-S, E-W, & Vertical) is applied separately and the time history responses are calculated at the required nodes. The resulting co-linear time history responses at a node due to the three earthquake components are then combined algebraically.

3G.4.3 Seismic Analysis Dynamic Results

3G.4.3.1 Absolute accelerations

Maximum absolute accelerations are described in section 3G.5.

3G.4.3.2 Relative deflections

The maximum seismic relative deflections that were obtained from the hard rock time history analyses and SASSI analyses are given in Tables 3G.4-1 to 3G.4-3 for the auxiliary and shield building, containment internal structure, and steel containment vessel.

3G.4.3.3 Floor response spectra

Seismic response spectra are developed at the locations of the selected nodes. Typical results are provided in Figures 3G.4-5 to 3G.4-10. The figures show results for each of the four design soil cases and are broadened as defined in the AP1000 DCD subsection 3.7.2.5. Spectra are shown for the following six key locations for evaluation of any site-specific analyses.

Containment internal structures at elevation of reactor vessel support	Figure 3G.4-5
Containment operating floor	Figure 3G.4-6
Auxiliary building NE corner at control room ceiling	Figure 3G.4-7
Shield building at fuel building roof	Figure 3G.4-8
Shield building roof	Figure 3G.4-9
Steel containment vessel at polar crane support	Figure 3G.4-10

The response spectra are grouped and enveloped to define the seismic design response spectra. The grouping is based on the building (i.e., ASB and CIS) and elevation. If equipment or a structure is supported at more than one elevation, then the analysts of such equipment will define the seismic input as an envelope of multiple groups based on the support locations. Therefore, if the equipment or structure is supported on rigid and flexible floor areas the response spectra (horizontal and vertical directions) used by the analysts will be the envelope of the rigid and flexible areas that include inside and outside nodes.

3G.5 Equivalent Static Accelerations

Equivalent static accelerations are a set of accelerations applied to the masses in a finite element model such that static analyses give member forces similar to the maximum member forces in a dynamic analysis. In many cases, the equivalent static accelerations are taken equal to the maximum values resulting from the dynamic analysis.

Equivalent static accelerations are applied to detailed three dimensional finite element models to generate (1) the in-plane and out-of-plane forces for the design of floors and walls of the ASB and CIS, (2) the design bearing reaction and member forces in the basemat, (3) the design member forces for the shield building roof structures, and (4) stresses for the containment vessel design. The analysis for each earthquake component is performed by applying equivalent static loads to the structural model at each finite element nodal point. The static load at each nodal mass point is the corresponding mass times the maximum absolute acceleration response at the corresponding elevation. The accelerations are the maximum accelerations from the time history results of the shell model at representative locations of the following portions of the nuclear island:

Shield building cylinder and roof
Auxiliary building – south side
Auxiliary building – north side
Containment internal structures – east side
Containment internal structures – west side
Steel containment vessel

Results of the time history analyses are obtained at representative locations, such as major wall and floor intersections and nodes at the cardinal orientations at key elevations of the shield building, as described in subsection 3G.4.1. Results at locations without local flexibility are considered in establishing the equivalent static accelerations.

Equivalent static accelerations are developed for application to detailed 3D finite element models that are conservative for the full range of soil sites at which the AP1000 may be located. Two sets of loads are specified. The first set is intended for use in design of the buildings. The second set is intended for seismic stability of the Nuclear Island and non-linear global analyses that consider uplift of the nuclear island from the soil. The results of these nonlinear analyses are used for the design of the base mat.

The following procedure is used for design of the buildings:

- Equivalent static accelerations based on the response at "rigid" locations of the structure are applied to all of the building structures in linear analyses. These are applied in separate load vectors for each direction. The design and overturning accelerations are applied uniformly for the region that they apply. Linear interpolation is used to define seismic accelerations between elevations.

- For those local flexible structures that are amplified, an additional acceleration is applied to these structures equal to the difference between the average uniform amplified component accelerations and rigid body component equivalent static accelerations. These accelerations are considered in local design of the flexible portion of the structure but do not need to be considered in areas of the structure away from the local flexibility. They can be applied in a series of individual load vectors.

- An accidental torsional moment is included in the design of the nuclear island structures as described in subsection 3.7.2.11.

Non-linear analyses are performed to address lift off of the basemat from the soil as described in subsection 3.8.5.4. These analyses are used for design of the nuclear island basemat. They are also used to check the walls that act as buttresses to transfer loads from the shield building into the portion of basemat in contact with the soil. These analyses use the equivalent static accelerations described in subsection 3G.5.5 for overturning based on the response at "rigid" locations of the structure.

3G.5.1 Equivalent Static Accelerations for Shield Building

The maximum seismic acceleration values obtained from the seismic time history analyses of the different soil cases and hard rock case are used to define the equivalent static seismic accelerations for the Shield building. Table 3G.5-1 shows the values for the South, East, North, and West sides of the shield building. The table also shows average values which are used as the equivalent static accelerations applied for design of the shield building. These design values are also shown in Table 3G.5-3. The seismic accelerations are averaged to obtain the representative acceleration associated with a specific elevation on the shield building. It is recognized that the nodes in the radial direction of excitation are influenced by local mode effects. Consequently, the nodes that are tangent to the direction of excitation are used to define the equivalent static seismic accelerations for this seismic component. The average value of the North and South Sides of the shield building is used for an East-West Earthquake and the average value of the East and West sides of the shield building for a North-South Earthquake. The vertical acceleration is the average of the four nodes defined by the nodes on the North, South, East, and West sides of the shield building. The vertical equivalent static seismic accelerations at elevations 294.93' and 333.13' are obtained directly from the maximum time history results by taking the average of the four nodes on the North, South, East, and West sides of the shield building. The vertical accelerations from the 3D finite element model at the shield building edges at these elevations are significantly influenced by the horizontal loading. If they are used for the vertical equivalent accelerations, the horizontal response would be double counted in the vertical direction.

3G.5.2 Equivalent Static Accelerations for Auxiliary Building

The maximum accelerations throughout the ASB (auxiliary shield building) are obtained from the seismic time history analyses for the hard rock and soil cases. They are evaluated separately for the South Side and North Side of the building. For each side accelerations at the corners are enveloped and the maximum value is specified for design. Since the south and north sides are found to have comparable accelerations the values for the two sides of the building are then enveloped to specify a single design value for all of the auxiliary building. Table 3G.5-2 shows the values for each side of the building and the enveloped values used in the finite element analysis to determine member forces for building design. The response of the auxiliary building in the vertical direction is influenced by horizontal input and by the height of the Auxiliary Building. Therefore, the vertical seismic acceleration values used for design are taken as the average of the accelerations of the shield building cylinder shown in Table 3G.5-1. The design values of the shield building and auxiliary building are summarized in Table 3G.5-3.

The equivalent static seismic accelerations in Tables 3G.5-3 are applied to all of the ASB structures. An additional uniform acceleration is applied for flexible walls and floors over local portions of the building structure. This acceleration is determined from the maximum response of a node representing this flexible location in the time history analyses. The peak magnitude is adjusted based on the deflection of the flexible location (e.g. cantilever beam, pin end supported beam), and applied uniformly to the flexible member so that the resulting member forces are consistent with the flexible response. The combined results from the "rigid" acceleration (Table 3G.5-3) in a given direction, and the additional seismic acceleration in the same direction due to flexibility are combined absolutely to define the member forces for the building design of the flexible structures.

3G.5.3 Equivalent Static Accelerations for Steel Containment Vessel

The steel containment vessel is represented by a stick model in the time history analyses. The equivalent static seismic acceleration values are the maximum accelerations of the masses on the containment vessel stick and are given in Table 3G.5-4. They are based on the maximum values obtained from the time history analyses of the hard rock and different soil cases.

3G.5.4 Equivalent Static Accelerations for Containment Internal Structure

Maximum seismic accelerations from the time history analyses are used to define the equivalent static seismic accelerations. Nodes are grouped according to different general areas within the containment internal structure (CIS): base & center; steam generator compartments (East & West); edges & sides; pressurizer compartment. The accelerations associated with the nodes within these groups are then averaged to obtain the equivalent static seismic acceleration values, and are given in Table 3G.5-5 for the CIS.

3G.5.5 Equivalent Static Accelerations for Evaluation of Building Overturning

Table 3G.5-6 shows the equivalent static seismic accelerations used in the evaluation of the basemat and overturning stability of the Nuclear Island. These equivalent static seismic accelerations consider that the dynamic response of the structure affecting overturning and basemat lift off is primarily the first mode response at about 3 hertz on hard rock. This reduces to about 2.4 hertz on soil sites as shown in the 2D ANSYS and SASSI analyses. The accelerations of the shield building are also applied to the auxiliary building which is integral with the shield building. The higher auxiliary building accelerations of Table 3G.5-2 are not considered in overturning since they are from higher frequency modes greater than 2.4 hertz. Amplified response of individual walls in the Auxiliary Building and the IRWST are not considered since they are local responses that do not effect overturning. Torsional building response is not considered since it will not contribute to overturning and uplift since loads on the building will be increased on one side and reduced on the other. Support loads from the Reactor Coolant Loop and Pressurizer are not considered in the overturning analysis since they are not significant to overturning; their mass is small compared to the rest of the nuclear island.

3G.6 Nuclear Island Liftoff Analyses and Bearing

The effects of basemat uplift for the hard rock and soft to medium sites were evaluated using non-linear seismic time history analyses. The East-West lumped-mass stick model of the NI structures was supported on a rigid plate with nonlinear springs that transmit reactions in horizontal and vertical directions to simulate the foundation contact area. Peak accelerations, floor response spectra, and member forces from seismic time history analyses that included basemat uplift were compared to seismic time history analyses that did not include these effects. The comparisons show that the basemat uplift effect is insignificant for the SSE input of 0.30g and has small effect at the Review Level Earthquake (RLE) of 0.50g that is part of the seismic margin analysis (Chapter 19).

Lift off was evaluated using an East-West lumped-mass stick model of the nuclear island structures supported on a rigid basemat with nonlinear springs. This model is shown in Figure 3G.6-1. The liftoff analysis model consists of the following two elements:

- The nuclear island (NI) combined stick model (ASB, CIS and SCV). The three sticks are concentric and the reactor coolant loop is included as mass only.
- The rigid basemat model of the basemat footprint with horizontal and vertical rock springs.

The hard rock or soil is modeled as horizontal and vertical spring elements with viscous damping at each node of the rigid beam. The stiffness of the springs in the vertical direction for the hard rock profile are calculated for a semi-infinite medium. The stiffness of the springs in the vertical direction for the soft to medium soil profile is calculated for elastic layers of finite depth by means of the Steinbrenner approximation (Reference 3). The stiffness of springs in the horizontal direction is calculated from that in the vertical direction assuming that the ratio of horizontal and vertical stiffness for the layered site has the same relationship as for a semi-infinite medium. The NI combined stick is attached to the rigid basemat at the NI gravity center, which is about 9 feet from the center of the rigid basemat. In the north-south direction, the stick is fixed at the bottom (EL. 60.5'). The stiffness properties of the ASB and CIS in the NI combined stick model are reduced by a factor of 0.8 to consider the effect of cracking.

Time history analyses are run by direct integration for dead load plus safe shutdown earthquake for two cases:

- linear springs able to take both tension and compression

- non-linear springs where the vertical springs act in compression only and the horizontal springs are active when the vertical spring is closed and inactive when the vertical spring lifts off.

Damping is included as mass and stiffness proportional damping matching the modal damping specified for each structure at frequencies of 3 and 25 Hertz. The value of modal damping for the springs is selected to match member forces from the linear analyses to the member forces in the corresponding 2D SASSI analyses described in subsection 3G.3.

Linear analyses of the ANSYS models showed that the soft-to-medium soil case gave the maximum base shear force and overturning moment. Hence, a non-linear lift off analysis was performed for the soft-to-medium soil case. Linear and non-linear (liftoff) analyses were performed for the SSE input of 0.3g and the RLE (review level earthquake) input of 0.5g.

Typical floor response spectra results for hard rock are shown in Figures 3G.6-2 and 3G.6-3 for the SSE and RLE spectra at elevation 116.5' in the ASB. The SSE figure also shows results with the soil springs reduced to 50% of the hard rock spring. Typical floor response spectra results for the soft to medium soil are shown in Figures 3G.6-4 and 3G.6-5. The results show that the liftoff has insignificant effect on the SSE response and a small increase at high frequencies for the RLE.

3G.7 References

(1) NUREG-800, Review of Safety Analysis Reports for Nuclear Power Plants, Section 3.7.2, Seismic System Analysis, Rev. 2.
(2) GW-GL-700, AP600 Design Control Document, Appendices 2A and 2B, Revision 4.
(3) Steinbrenner, W., 1934, "Tafelin zur Setzungs berechnung," Die Strasse, Vol. 1, October, pp. 121-124.

Table 3G.1-1

SUMMARY OF MODELS AND ANALYSIS METHODS

Model	Analysis Method	Program	Type of Dynamic Response/Purpose
3D (ASB) solid-shell model	-	ANSYS	Creates the finite element mesh for the ASB finite element model
3D (CIS) solid-shell model	-	ANSYS	Creates the finite element mesh for the CIS finite element model
3D finite element model including shield building roof (ASB10)	-	ANSYS	ASB portion of NI10
3D finite element model including dish below containment vessel	Equivalent static analysis using accelerations from time history analyses	ANSYS	CIS portion of NI10 To obtain SSE member forces for the containment internal structures.
3D finite element shell model of nuclear island [NI10](coupled auxiliary/shield building shell model, containment internal structures, steel containment vessel , polar crane, RCL, pressurizer and CMTs)	Mode superposition time history analysis	ANSYS	Performed for hard rock profile for ASB with CIS as superelement and for CIS with ASB as superelement. To develop time histories for generating plant design floor response spectra for nuclear island structures. To obtain maximum absolute nodal accelerations (ZPA) to be used in equivalent static analyses. To obtain maximum displacements relative to basemat. To obtain maximum member forces and moments in selected elements for comparison to equivalent static results.
3D finite element coarse shell model of auxiliary and shield building and containment internal structures [NI20] (including steel containment vessel , polar crane, RCL, and pressurizer)	Mode superposition time history analysis	ANSYS	Performed for hard rock profile for comparisons against more detailed NI10 model
2D finite element lumped mass stick model of auxiliary and shield building.	Time history analysis	SASSI	Performed parametric soil studies to help establish the bounding generic soil conditions.

3D finite element coarse shell model of auxiliary and shield building and containment internal structures [NI20] (including steel containment vessel , polar crane, RCL, and pressurizer)	Time history analysis	SASSI	Performed for the three soil profiles of firm rock, upper bound soft to medium soil, and soft to medium soil. To develop time histories for generating plant design floor response spectra for nuclear island structures. To obtain maximum absolute nodal accelerations (ZPA) to be used in equivalent static analyses To obtain maximum displacements relative to basemat. To obtain maximum member forces and moments in selected elements for comparison to equivalent static results.
3D shell of revolution model of steel containment vessel	Modal analysis Equivalent static analysis using accelerations from time history analyses	ANSYS	To obtain dynamic properties. To obtain SSE stresses for the containment vessel.
3D lumped mass stick model of the SCV	-	ANSYS	Used in the NI10 and NI20 models
3D lumped mass stick model of the RCL	-	ANSYS	Used in the NI10 and NI20 models
3D lumped mass stick model of the Pressurizer	-	ANSYS	Used in the NI10 and NI20 models
3D lumped mass stick model of the CMT	-	ANSYS	Used in the NI10 model
Static analyses			
3D finite element refined shell model of auxiliary and shield building (ASB05)	Equivalent static analysis using accelerations from time history analyses	ANSYS	To obtain SSE member forces for the auxiliary and shield building.
3D finite element model of the shield building roof	Equivalent static analysis using accelerations from time history analyses	GT STRUDL	To obtain SSE member forces for the shield building roof.
3D finite element refined shell model of nuclear island (NI05)	Equivalent static non-linear analysis using accelerations from time history analyses	ANSYS	To obtain SSE member forces for the nuclear island basemat

Table 3G.1-2

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	ANSYS	Algebraic Sum	n/a
3D finite element, fixed base models, coupled auxiliary/shield building shell model, with superelement of containment internal structures	Mode superposition time history analysis	ANSYS	Algebraic Sum	n/a
3D finite element, fixed base models, coupled auxiliary/shield buildings and containment internal structures	Equivalent static analysis using nodal accelerations from 3D stick model	ANSYS	SRSS or 100%, 40%, 40%	n/a
3D finite element model of the nuclear island basemat	Equivalent static analysis using nodal accelerations 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	ANSYS	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 GT STRUDL	SRSS	n/a
PCS valve room and miscellaneous steel frame structures, miscellaneous flexible walls, and floors	Response spectrum analysis	ANSYS	SRSS	Grouping

Table 3G.2-1 (Sheet 1 of 2)

STEEL CONTAINMENT VESSEL LUMPED-MASS STICK MODEL (WITHOUT POLAR CRANE) MODAL PROPERTIES

Mode	Frequency	Effective Mass		
		X Direction	Y Direction	Z Direction
1	6.309	2.380	159.153	0.005
2	6.311	159.290	2.382	0.000
3	12.942	0.018	0.000	0.000
4	16.970	0.000	0.006	171.030
5	18.960	0.102	40.263	0.002
6	18.970	40.161	0.102	0.000
7	28.201	0.000	0.000	28.073
8	31.898	0.054	2.636	0.000
9	31.999	2.789	0.057	0.000
10	37.990	0.909	0.007	0.000
11	38.634	0.022	4.846	0.009
12	38.877	3.758	0.014	0.000
13	47.387	0.000	0.000	5.066
14	54.039	4.649	0.633	0.000
15	54.065	0.624	4.693	0.002
16	60.628	0.002	0.042	3.389
17	62.734	0.147	0.001	0.018
18	63.180	0.000	0.050	7.069
19	63.613	0.002	0.001	0.003
20	65.994	0.022	0.659	0.041
Sum of Effective Masses		214.929	215.545	214.706

Notes:

1. Fixed at Elevation 100′.
2. The total mass of the containment vessel is 225.697 kip-sec^2/ft.

Table 3G.2-1 (Sheet 2 of 2)

STEEL CONTAINMENT VESSEL LUMPED-MASS STICK MODEL (WITH POLAR CRANE) MODAL PROPERTIES

Mode	Frequency	Effective Mass		
		X Direction	Y Direction	Z Direction
1	3.619	0.000	41.959	0.000
2	5.387	175.274	0.000	0.175
3	6.192	0.000	148.385	0.005
4	6.415	3.321	0.000	24.074
5	9.422	0.002	1.017	0.000
6	9.674	10.510	0.000	0.532
7	12.811	0.015	0.001	0.000
8	15.757	0.004	0.320	0.010
9	16.367	3.103	0.003	159.153
10	17.495	28.537	0.001	19.546
11	18.944	0.000	40.053	0.001
12	21.043	10.724	0.000	0.426
13	22.102	0.000	0.005	0.000
14	27.340	0.054	0.000	18.661
15	30.387	2.978	0.001	1.559
16	31.577	0.002	3.526	0.004
17	35.033	0.194	0.006	3.895
18	35.535	0.211	0.027	0.399
19	35.646	0.000	1.451	0.019
20	37.599	0.325	0.426	0.007
Sum of Effective Masses		235.254	237.181	228.465

Notes:

1. Fixed at Elevation 100′.
2. The total mass of the containment vessel with the polar crane is 255.85 kip-sec^2/ft.

Table 3G.2-2

**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	16.51 hertz	16.97 hertz	6.20 hertz	6.31 hertz
2	23.26 hertz	28.20 hertz	18.58 hertz	18.96 hertz

Note:
1. Fixed at elevation 100'.

Table 3G.2-3

RCL LUMPED-MASS STICK MODEL
MODAL PROPERTIES

Mode	Frequency	Effective Mass		
		X Direction	Y Direction	Z Direction
1	4.211	0.000	0.000	0.001
2	4.216	45.174	0.112	0.000
3	8.110	15.825	73.633	0.000
4	8.477	0.000	0.000	1.181
5	8.627	18.084	3.670	0.000
6	8.671	0.000	0.000	10.486
7	8.701	15.028	83.412	0.000
8	9.260	0.001	13.517	0.000
9	9.279	0.000	0.000	111.275
10	9.750	0.000	0.000	5.115
11	9.830	0.007	0.627	0.000
12	10.365	0.000	0.000	0.968
13	10.799	0.000	0.000	0.001
14	10.903	0.491	0.004	0.000
15	11.898	19.209	1.293	0.000
16	11.913	13.286	1.888	0.000
17	13.414	22.697	0.010	0.000
18	13.459	0.000	0.000	3.165
19	13.465	1.011	0.784	0.000
20	15.411	0.606	5.228	0.000
21	16.197	0.000	0.000	0.009
22	16.250	30.402	0.101	0.000
23	21.731	2.133	0.000	0.000
24	22.101	0.006	1.518	0.000
25	28.236	0.000	0.000	39.954
26	28.258	0.002	0.384	0.000
27	29.292	0.000	0.000	0.501
28	29.850	0.925	0.206	0.000
29	30.416	0.000	0.000	0.156
30	31.012	2.248	0.000	0.000
Sum of Effective Masses		187.132	186.387	172.811

Notes:
1. Fixed at building end of RCL supports.
2. The total mass of the RCL is 187.84 kip-sec^2/ft.

Table 3G.2-4

COUPLED SHIELD AND AUXILIARY BUILDINGS NI20 MODEL

MODAL PROPERTIES

Mode	Frequency	Effective Mass		
		X Direction	Y Direction	Z Direction
6	2.372	0	122.217	0
7	2.922	0.96	1331.88	0.117
8	3.065	1135.35	0.089	0.064
16	4.825	185.648	0.027	0.056
18	5.053	152.635	0.074	0.92
19	5.109	0.991	0.001	855.255
21	5.928	0.043	276.168	0.03
25	6.243	18.722	4.095	44.702
27	6.35	2.368	470.713	0.82
28	6.496	251.398	0.621	0.728
29	6.649	125.132	8.924	0.531
31	6.682	0.505	118.954	0.432
34	7.046	36.842	14.654	0.117
35	7.157	0.932	70.731	1.464
36	7.198	51.672	3.435	0.061
44	7.94	83.179	40.848	2.124
46	8.12	127.04	2.932	0.006
50	8.219	19.562	258.199	0.03
52	8.543	0.012	33.728	0.37
54	8.624	0.005	0.01	122.774
56	8.726	3.873	44.7	0.01
62	8.869	1.326	499.45	2.744
64	9.335	28.014	306.635	4.462
66	9.528	66.799	1.628	0.001
68	9.671	668.782	3.024	0.11
69	9.687	215.109	0.178	0.127
73	9.979	6.392	66.222	0.306
80	10.329	36.36	8.995	1.296
83	10.716	33.056	2.753	0.002
84	10.785	60.284	5.884	0.001
95	11.39	82.52	31.927	4.76
98	11.575	44.252	2.16	1.329

Table 3G.2-4

COUPLED SHIELD AND AUXILIARY BUILDINGS NI20 MODEL

MODAL PROPERTIES

Mode	Frequency	Effective Mass		
		X Direction	Y Direction	Z Direction
99	11.604	44.066	59.547	0.06
107	12.306	0.073	0.293	66.691
116	12.8	10.261	73.775	0.533
118	12.928	0	36.342	3.025
122	13.278	5.268	11.961	241.083
123	13.343	3.467	0.22	444.104
124	13.515	46.333	19.027	10.239
128	13.682	173.291	17.517	45.261
129	13.778	11.188	0.737	156.529
131	13.925	2.832	0.042	144.645
133	14.12	68.195	33.09	22.654
139	14.372	0.204	48.323	0.135

Note:
1. Fixed at base.

Table 3G.2-5

CONTAINMENT INTERNAL STRUCTURES NI20 MODEL

MODAL PROPERTIES

Mode	Frequency	Effective Mass		
		X Direction	Y Direction	Z Direction
15	9.377	78.64	113.18	0.18
16	9.668	0.30	107.44	0.24
18	10.403	88.08	38.02	0.38
20	10.854	12.73	130.23	0.43
22	11.310	63.94	0.79	1.26
23	11.346	169.55	20.13	0.90
36	14.703	31.59	87.71	2.99
42	16.468	52.74	43.17	0.85
46	18.320	168.10	7.18	0.76
51	20.522	47.74	6.87	0.52
78	26.229	3.51	22.38	80.85
81	27.393	20.16	45.98	27.58
82	27.807	3.64	173.88	0.99
88	28.331	4.23	11.66	40.82
93	28.870	47.87	0.84	24.11
96	29.886	35.20	60.27	3.02
98	30.445	3.83	112.56	4.50
99	30.694	11.39	80.23	26.11
103	31.942	32.71	72.53	7.10
108	33.327	189.18	4.98	18.16
109	33.339	344.61	4.24	16.87
110	33.468	16.31	255.00	9.24

Note:
1. Fixed at base.

Table 3G.3-1 – AP1000 ZPA for 2D SASSI Cases

	North-South		Hard Rock	Firm Rock	Soft Rock	UBSM	SM	Soft soil
	node	El. feet	ZPA [g]	ZPA [g]	ZPA [g]	ZPA [g]	ZPA [g]	ZPA [g]
ASB	21	81.5	0.326	0.326	0.345	**0.358**	0.306	0.249
	41	99	0.348	0.327	0.347	**0.361**	0.308	0.227
	120	179.6	**0.571**	0.501	0.469	0.498	0.529	0.247
	150	242.5	0.803	0.795	0.816	**0.819**	0.787	0.29
	310	333.1	1.449	1.561	**1.567**	1.524	1.226	0.453
SCV	407	138.6	0.405	**0.424**	0.408	0.387	0.407	0.232
	411	200	0.82	**0.916**	0.672	0.541	0.484	0.263
	417	281.9	1.396	**1.465**	1.031	0.723	0.598	0.372
CIS	535	134.3	**0.548**	0.45	0.347	0.368	0.355	0.229
	538	169	**1.517**	0.874	0.45	0.441	0.397	0.317

	East-West		Hard Rock	Firm Rock	Soft Rock	UBSM	SM	Soft soil
	node	El. feet	ZPA [g]	ZPA [g]	ZPA [g]	ZPA [g]	ZPA [g]	ZPA [g]
ASB	21	81.5	0.309	0.318	0.359	**0.376**	0.311	0.235
	41	99	0.318	0.336	0.367	**0.385**	0.317	0.237
	120	179.6	**0.607**	0.561	0.546	0.549	0.605	0.295
	150	242.5	0.84	0.823	0.854	0.912	**0.962**	0.557
	310	333.1	1.449	1.536	1.624	**1.74**	1.506	0.891
SCV	407	138.6	0.528	0.529	**0.535**	0.513	0.38	0.247
	411	200	0.817	**0.95**	0.816	0.741	0.515	0.429
	417	281.9	1.251	**1.503**	1.136	0.985	0.716	0.675
CIS	535	134.3	**0.52**	0.404	0.391	0.404	0.365	0.259
	538	169	**1.679**	1.052	0.755	0.553	0.526	0.441

Table 3G.4-1 – Maximum Seismic Deflections for Auxiliary and Shield Building

Units - inches

Elevation feet	Shield Building	Auxiliary Building	Shield Building	Auxiliary Building	Shield Building	Auxiliary Building
	North-South		East-West		Vertical	
333.13	1.4398		1.6984		0.6482	
294.93	1.1086		1.3138		0.6350	
265	0.9400		1.2045		0.3996	
222.75	0.7073		0.9323		0.3619	
179.19	0.4782	0.1513	0.6656	0.2734	0.3013	0.1351
160	0.3724	0.1728	0.5327	0.3236	0.2570	0.1950
134.88	0.2340	0.0991	0.3588	0.2313	0.1990	0.1405
99	0.0370	0.0353	0.0672	0.0672	0.0920	0.1036

Table 3G.4-2 – Maximum Seismic Deflections for Containment Internal Structure

Units - inches

Elevation feet	North-South		East-West		Vertical	
	East	West	East	West	East	West
160		0.0733		0.1544		0.0519
153	0.1440	0.0703	0.1550	0.1216	0.0592	0.0517
134	0.1042	0.0644	0.1221	0.1180	0.0684	0.0511
100	0.0270	0.0270	0.0396	0.0396	0.0084	0.0084

Table 3G.4-3 – Maximum Seismic Deflections from SCV Stick Model

Units - inches

Elevation feet	North-South	East-West	Vertical
282	0.4590	0.4335	0.0601
224	0.3404	0.3212	0.0335
170	0.1983	0.1907	0.0253
132	0.1001	0.0988	0.0174

Table 3G.5-1 – Shield Building Seismic Acceleration Distribution

Units: g

Maximum Value from Each Individual Soil Case

Elevation feet	Shield Building South Side			Shield Building East Side			Shield Building North Side			Shield Building West Side			Average Values		
	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z	X	Y	Z
99	0.362	0.367	0.376	0.352	0.360	0.388	0.376	0.365	0.349	0.345	0.370	0.343	0.35	0.37	0.36
134.88	0.515	0.416	0.397	0.489	0.584	0.447	0.551	0.452	0.424	0.452	0.725	0.470	0.47	0.43	0.41
179.19	0.650	0.533	0.468	0.587	0.647	0.559	0.802	0.591	0.463	0.536	1.045	0.582	0.55	0.56	0.51
222.75	0.802	0.731	0.565	0.659	0.912	0.676	0.745	0.718	0.616	0.724	0.990	0.704	0.69	0.72	0.64
265	0.802	0.847	0.649	0.777	1.032	0.693	0.911	0.868	0.704	0.855	1.062	0.747	0.79	0.85	0.69
294.93	1.069	1.028	1.309	0.934	1.194	1.223	0.918	1.119	1.081	1.029	1.007	1.045	0.98	1.07	1.09 (1)
333.13	1.258	1.334	1.329	1.210	1.364	1.253	1.268	1.393	1.102	1.294	1.363	1.061	1.25	1.36	1.10 (1)

Notes to Table 3G.5-1:

(1) These values have been obtained by averaging the time history response of the nodes on the North, South, East, and West sides of the shield building diameter to provide the response on the center line at the axis of the shield building. This avoids double counting the horizontal seismic component, because the Z component (vertical) values given for the South, East, North, and West side have the effect of the horizontal component.

Table 3G.5-2 – Auxiliary Building Equivalent Static Seismic Acceleration Summary

Maximum Value from Each Individual Soil Case at Corners of Area

Elevation feet	South Side			North Side			Equivalent Static Seismic Accelerations		
	X	Y	Z	X	Y	Z	X	Y	Z(1)
66.5 (2)							0.32	0.37	0.36
81.5 (2)							0.36	0.37	0.36
99	0.42	0.40	0.41	0.38	0.40	0.39	0.42	0.40	0.36
116.5				0.43	0.43	0.40	0.43	0.43	0.37
134.88	0.58	0.52	0.53	0.55	0.58	0.45	0.58	0.58	0.41
152.19									
152.96									
154.69				0.71	0.58	0.46	0.71	0.58	0.44
159.69									
160.56									
162.19	0.66	0.58	0.47	0.66	0.69	0.48	0.71(3)	0.69	0.46
179.19	0.86	0.73	0.64				0.86	0.73	0.51

Notes to Table 3G.5-2:

(1) The values in the vertical direction are the average values at the edge of the shield building see Table 3G.5-1. Linear interpolation is used for intermediate elevations.

(2) Value is linear interpolated for hard and firm rock using 0.3g at 66.5' elevation, or represents the value at 99' for upper bound soft to medium or soft to medium soil sites.

(3) Value increased to equal value at elevation 154.69'.

Table 3G.5-3 - ASB Design Accelerations

Units: g

Elevation Feet	North South		East West		Vertical
	Shield Building	Auxiliary Building	Shield Building	Auxiliary Building	Shield and Auxiliary Building
333.13	1.25		1.36		1.10
294.93	0.98		1.07		1.09
265	0.79		0.85		0.69
242.5	0.74		0.78		0.66
222.75	0.69		0.72		0.64
200	0.62		0.64		0.57
180	0.55	0.86	0.56	0.73	0.51
162	0.52	0.71	0.51	0.69	0.46
153.98	0.51	0.71	0.49	0.58	0.44
134.88	0.47	0.58	0.43	0.58	0.41
116.5	0.41	0.43	0.38	0.43	0.37
99	0.35	0.42	0.37	0.40	0.36
81.5	0.32	0.36	0.37	0.37	0.36
66.5	0.32	0.32	0.37	0.37	0.36

Notes to Table 3G.5-3:

(1) Linear interpolation can be used between elevations.

Table 3G.5-4 –SCV Design Accelerations

Units: g

Elevation feet	Equivalent Static Seismic Accelerations (1)		
	X	Y	Z
281.9	1.18	1.37	1.21
273.83	1.14	1.33	1.03
265.83	1.1	1.28	0.86
255.02	1.04	1.22	0.75
244.21	0.98	1.15	0.7
224	0.87	1.03	0.66
169.93	0.56	0.65	0.55
131.68	0.41	0.48	0.44
99	0.33	0.36	0.36

Notes to Table 3G.5-4:

(1) X = North-South; Y = East-West; Z = Vertical
(2) Linear interpolation can be used between elevations.

Table 3G.5-5 – CIS Design Accelerations

Units: g

Elevation (2)	East Side			West Side		
	X	Y	Z	X	Y	Z
164.95				0.85	0.83	0.41
153	0.71	0.59	0.39	0.74	0.66	0.40
134.25	0.58	0.56	0.39	0.59	0.56	0.39
107.17	0.37	0.36	0.36	0.37	0.36	0.36
103	0.36	0.36	0.36	0.36	0.36	0.36
99	0.35	0.36	0.36	0.35	0.36	0.36
82.5	0.33	0.36	0.36	0.33	0.36	0.36
66.5	0.33	0.36	0.36	0.33	0.36	0.36

Notes to Table 3G.5-5:

(1) X = North-South; Y = East-West; Z = Vertical
(2) Linear interpolation can be used between elevations.

Table 3G.5-6 – Equivalent Seismic Static Accelerations for Overturning Evaluation

	Elevation feet	Equivalent Static Seismic Accelerations (1) X	Y	Z	Notes
ASB	66.5	0.32	0.37	0.36	Table 3G.5-3 Shield Bldg
	81.5	0.32	0.37	0.36	
	99	0.35	0.37	0.36	
	116.5	0.41	0.40	0.38	
	134.88	0.47	0.43	0.41	
	179.19	0.55	0.56	0.51	
	222.75	0.69	0.72	0.64	
	265	0.79	0.85	0.69	
	294.93	0.98	1.07	1.09	
	333.13	1.25	1.36	1.10	
SCV	99.00	0.33	0.36	0.36	Table 3G.5-4
	131.68	0.41	0.48	0.44	
	169.93	0.56	0.65	0.55	
	224.00	0.87	1.03	0.66	
	244.21	0.98	1.15	0.70	
	255.02	1.04	1.22	0.75	
	265.83	1.10	1.28	0.86	
	273.83	1.14	1.33	1.03	
	281.90	1.18	1.37	1.21	
CIS	66.5	0.33	0.36	0.36	Table 3G.5-5
	82.5	0.33	0.36	0.36	
	99	0.35	0.36	0.36	
	103	0.36	0.37	0.36	
	107.17	0.37	0.38	0.37	
	134.25	0.58	0.56	0.39	
	153	0.73	0.62	0.39	
	164.95	0.85	0.83	0.41	

Notes to Table 3G.5-6:

(1) X = North-South; Y = East-West; Z = Vertical
(2) Linear interpolation can be used between elevations.

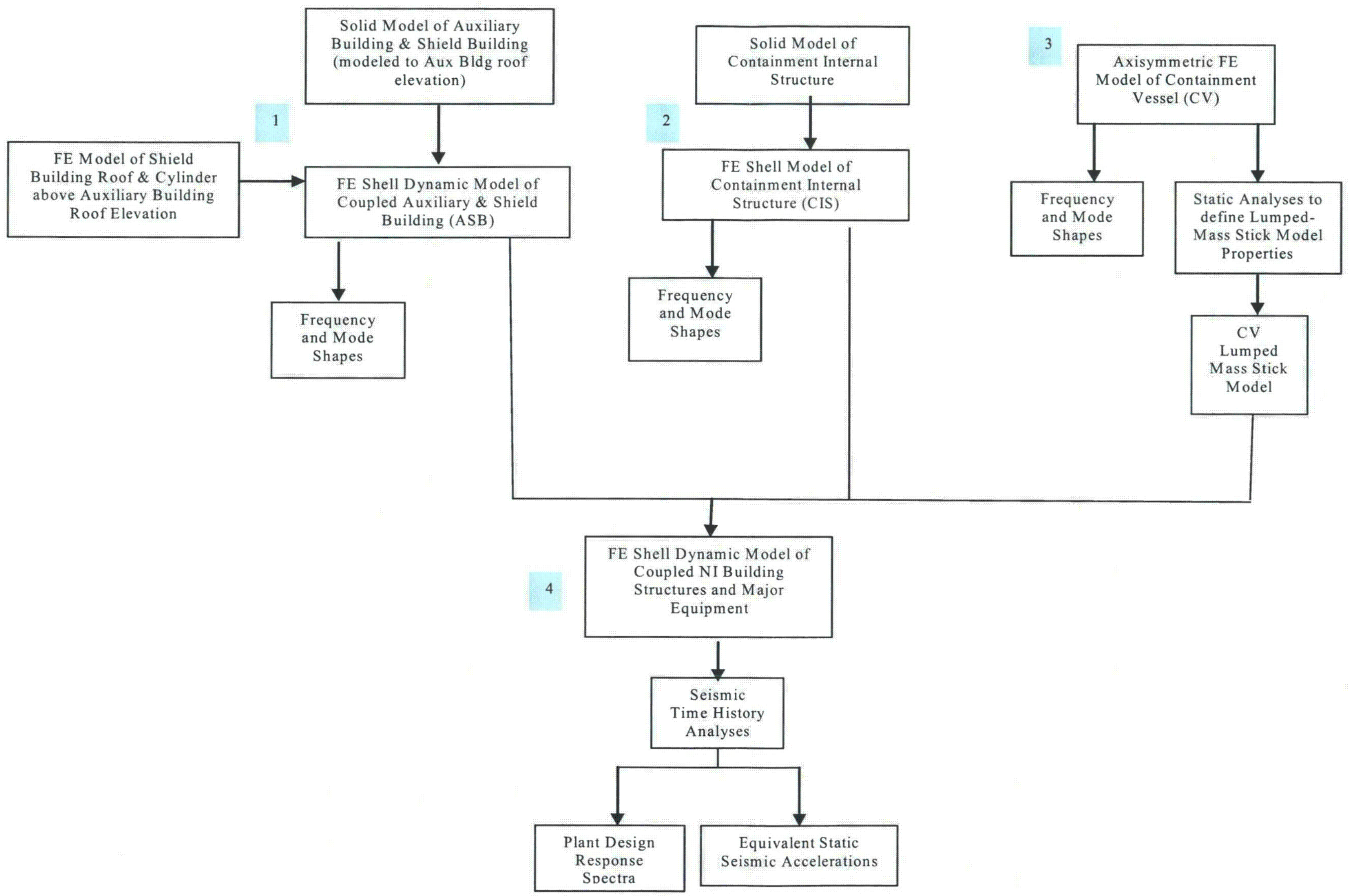


Figure 3G.1-1

Nuclear Island Seismic Analysis Models

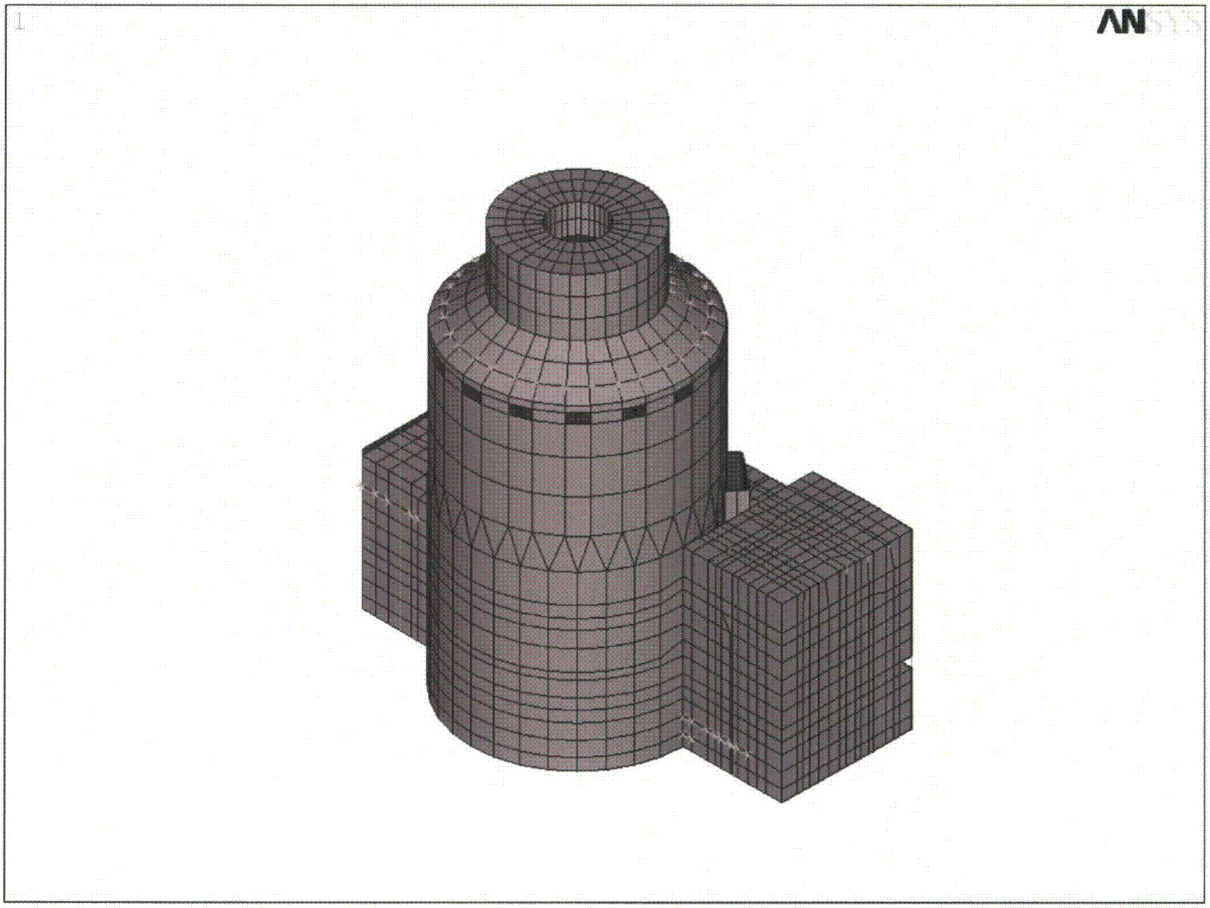
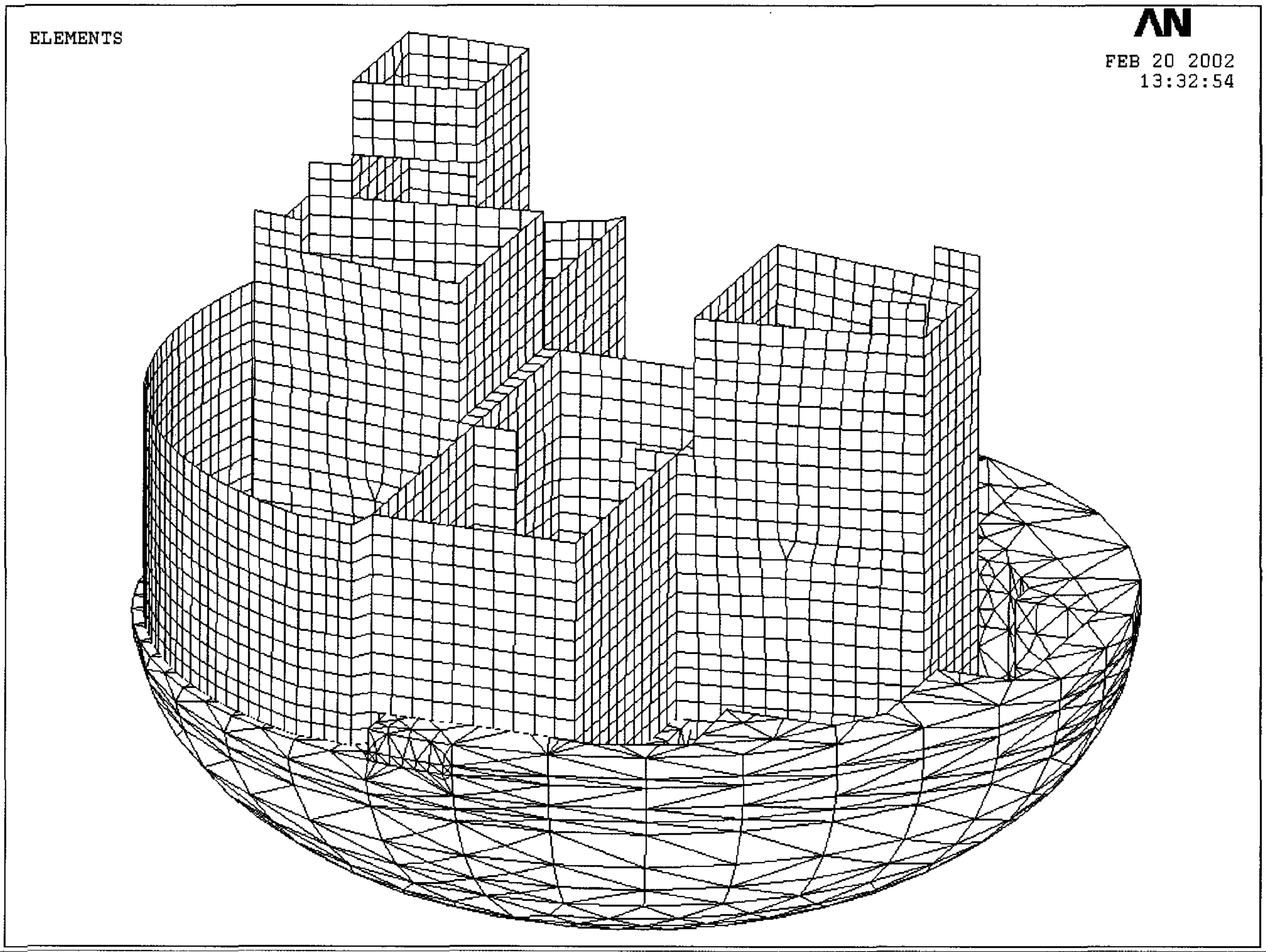


Figure 3G.2-1

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



Note: This figure shows the finite element model of walls and basemat inside containment. Floors are not shown.

Figure 3G.2-2

3-D Finite Element Model of Containment Internal Structures

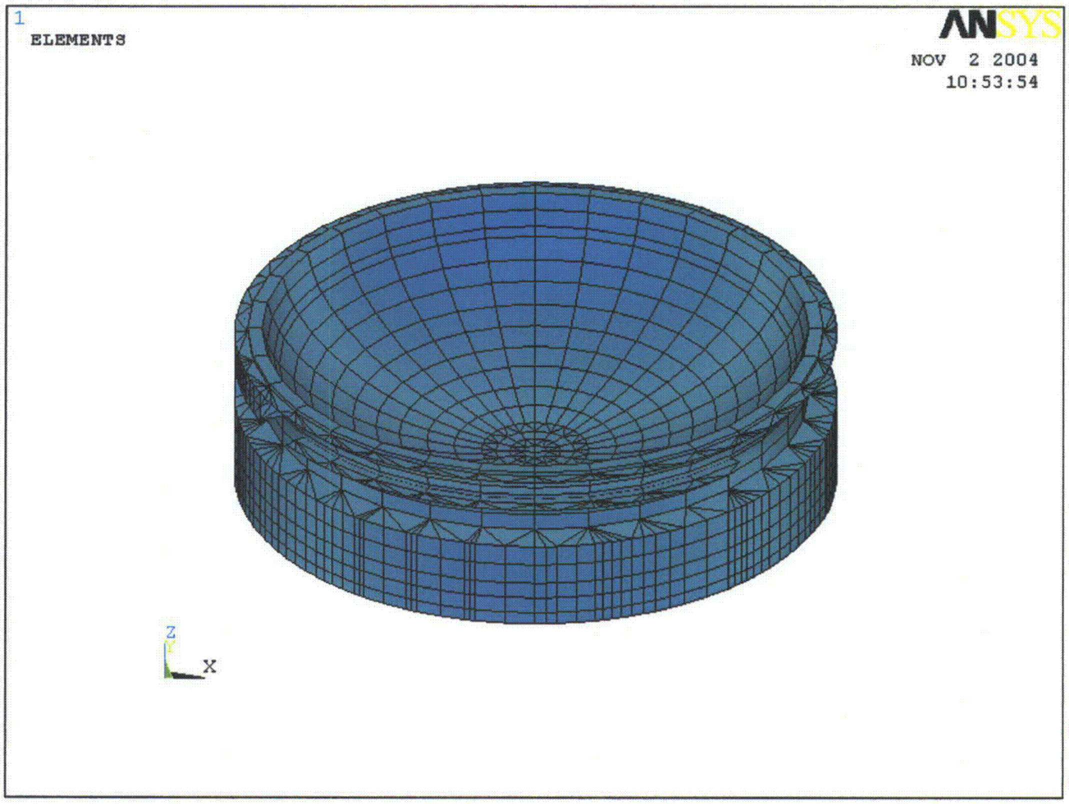


Figure 3G.2-3

3-D Finite Element Model of Containment Outer Basemat (Dish)

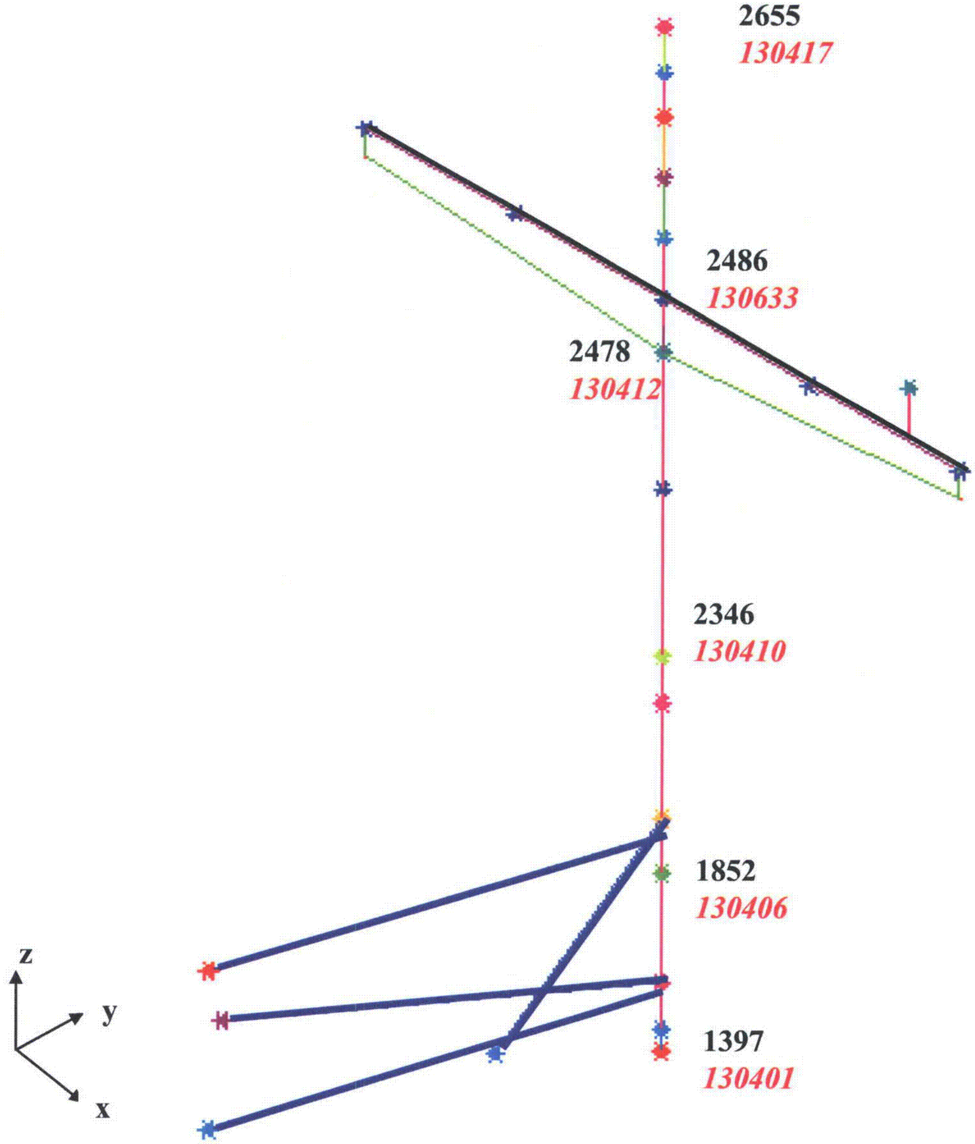
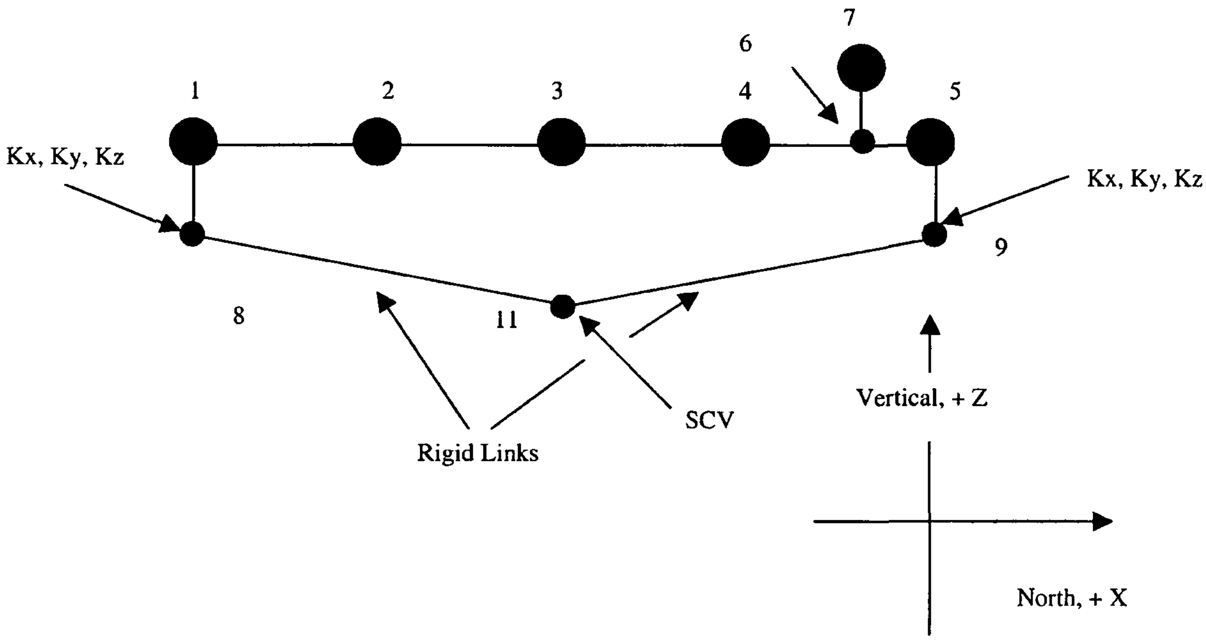


Figure 3G.2-4 - Steel Containment Vessel and Polar Crane Models



Local SCV Stiffness are Kx, Ky, Kz

Dynamic Degrees of Freedom

- Masses at nodes 1, 2, 3, 4, 5, and 7
- All Mass nodes have DOFs in X, Y, and Z directions

Comments:

1. Cross Beams between girders are represented by rotation spring constants Kxx and Kzz
2. Cross Beam rotational spring constant Kyy is negligible compared to girder stiffness

Figure 3G.2-5

Polar Crane Model

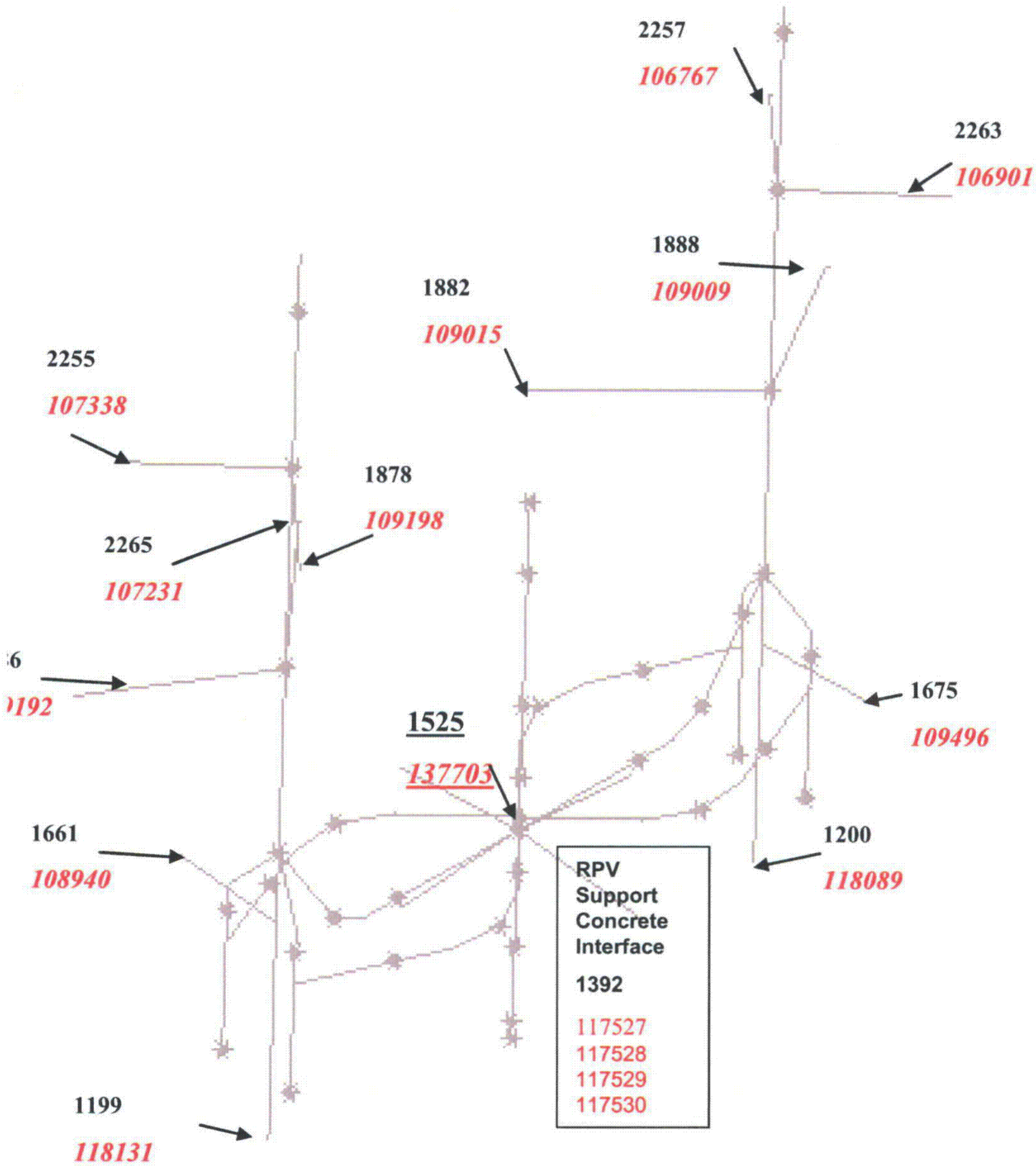


Figure 3G.2-6

Reactor Coolant Loop Lumped Mass Stick Model

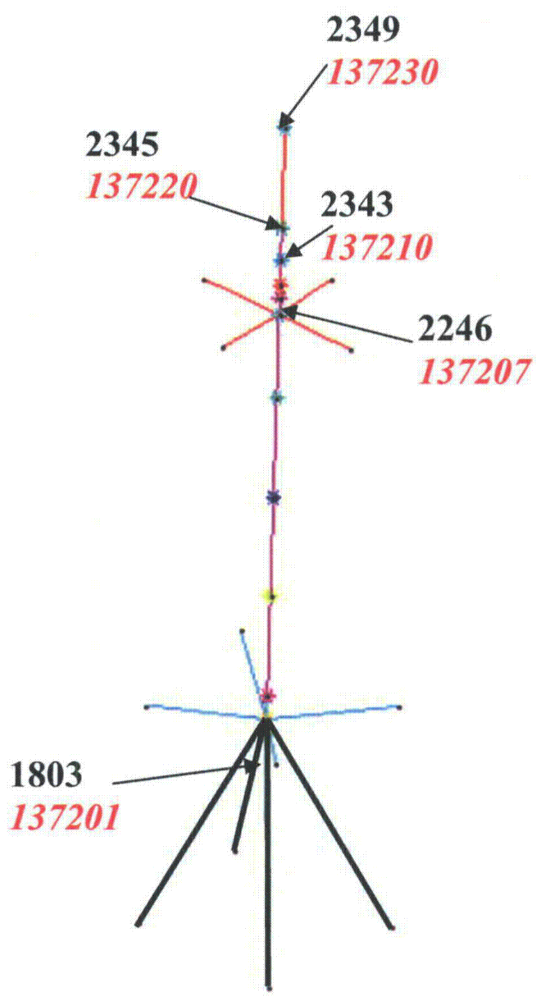


Figure 3G.2-7 – Pressurizer Model

Figure 3G.2-8 – Core Make-Up Tank Models

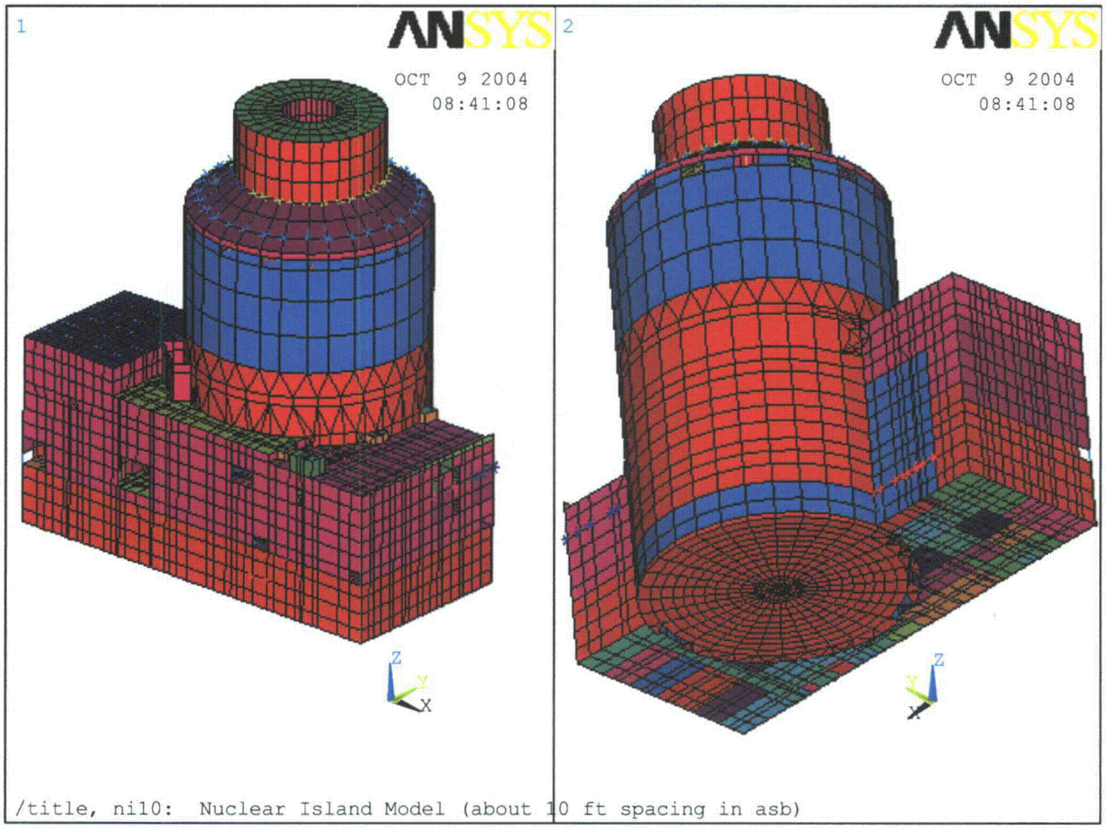


Figure 3G.2-9 - AP1000 Nuclear Island solid-shell model (NI10)

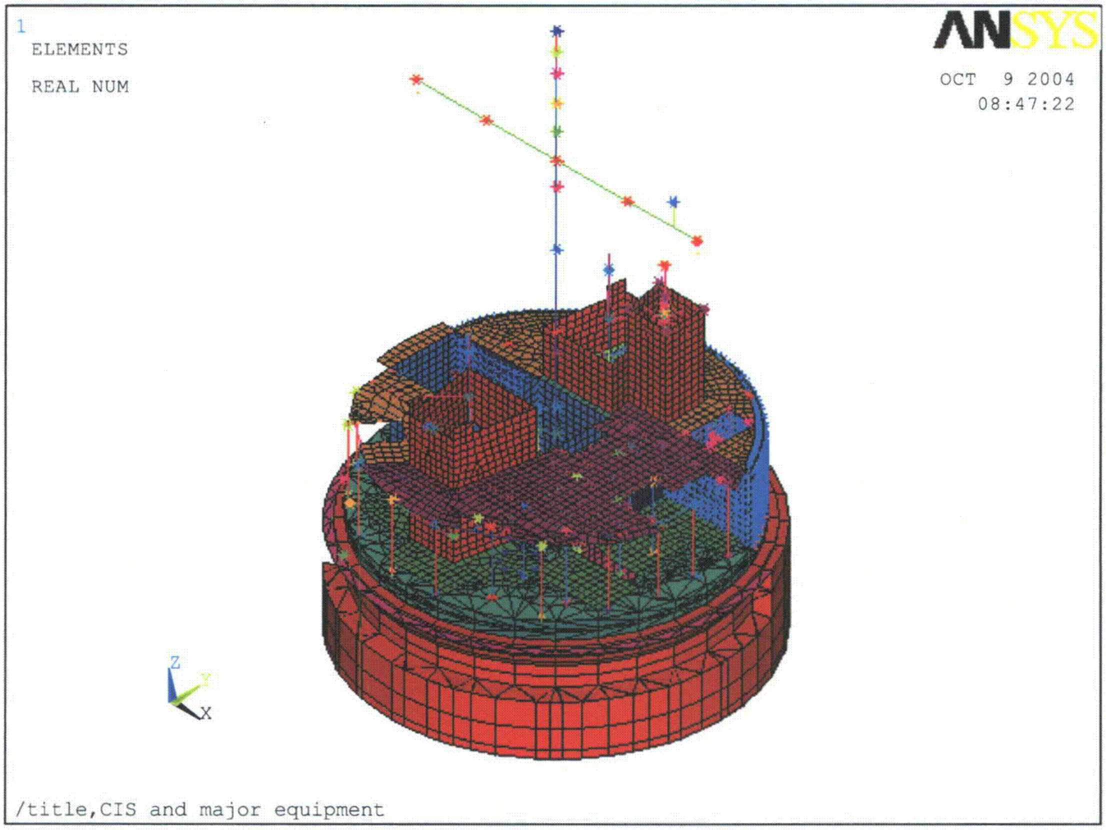
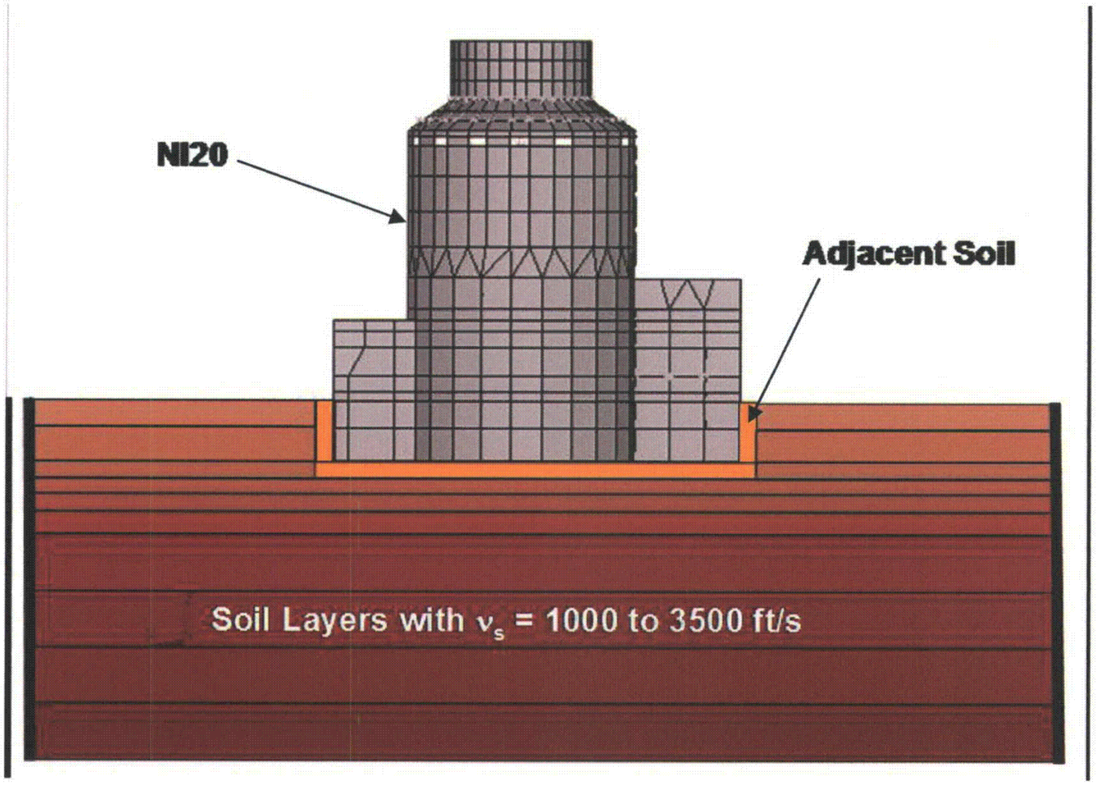
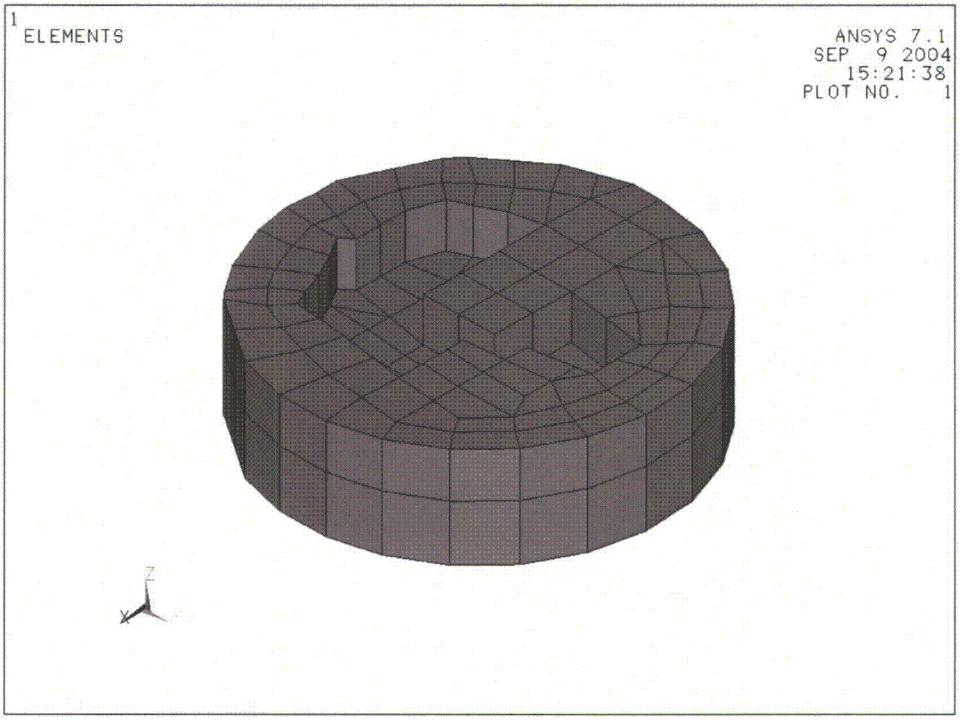


Figure 3G.2-10 - CIS with the SCV, PC, RCL and PZR



Note: The adjacent soil elements are part of the structural portion of SASSI and have the same material properties as the soil. These elements are used to obtain soil lateral and bearing soil pressures.

Figure 3G.2-11 - Soil Structure Interaction Model – NI20 Looking East



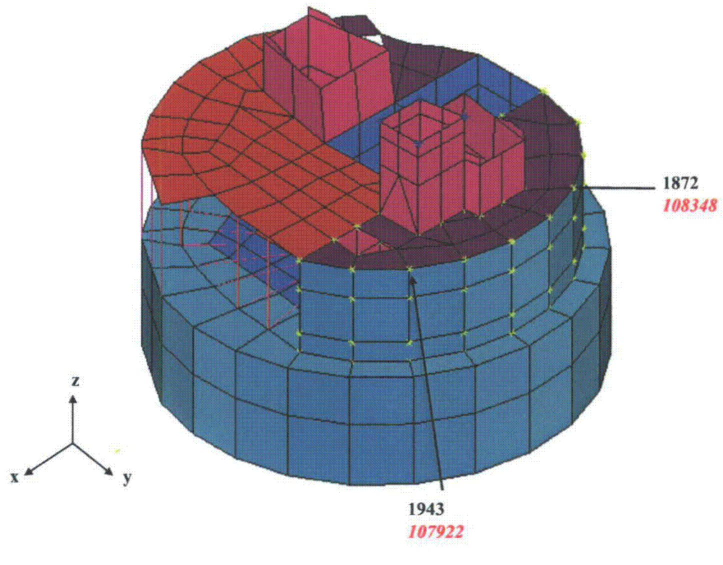
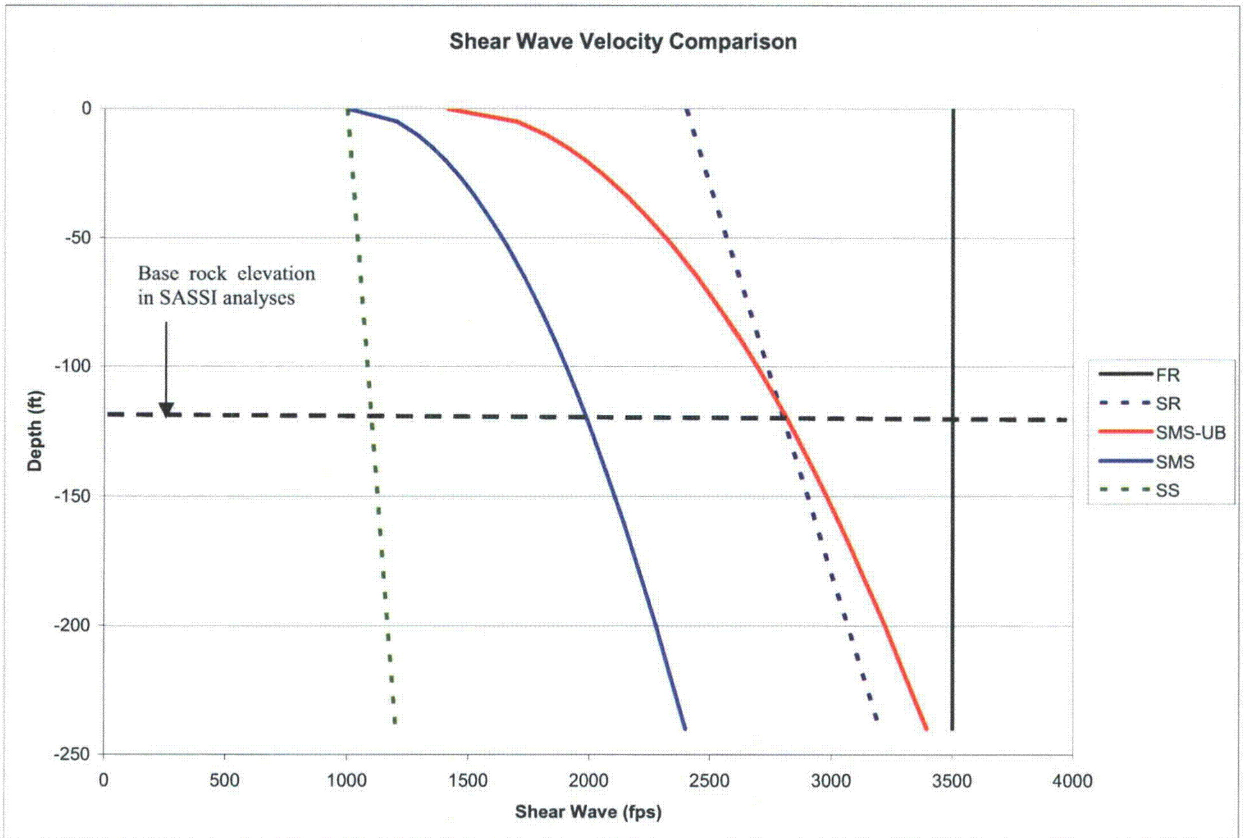


Figure 3G.2-12 – Coarse model of containment internal structures.



Note: Fixed base analyses were performed for hard rock sites. These analyses are applicable for shear wave velocity greater than 8000 feet per second.

Figure 3G.3-1 Generic Soil Profiles

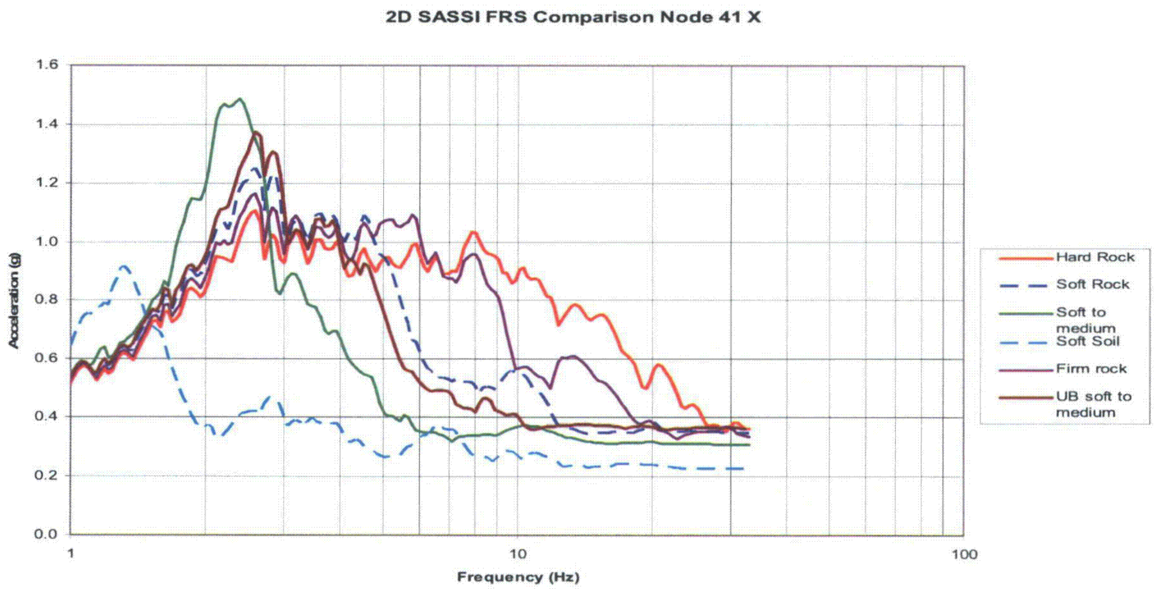


Figure 3G.3-2 - 2D SASSI FRS - Node 41 X (ASB El. 99')

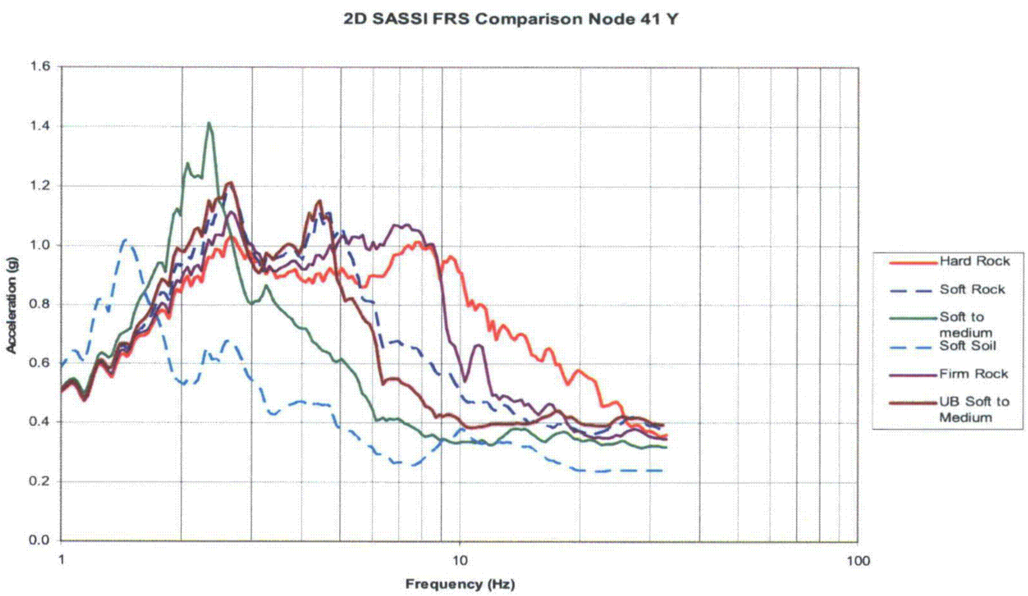


Figure 3G.3-3 - 2D SASSI FRS - Node 41Y (ASB El. 99')

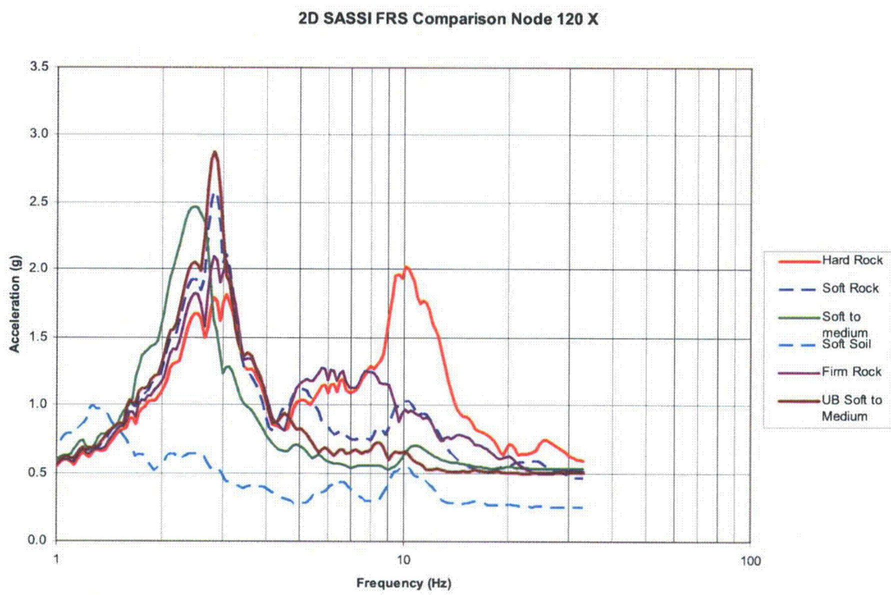


Figure 3G.3-4 - 2D SASSI FRS - Node 120 X (ASB El. 179.6')

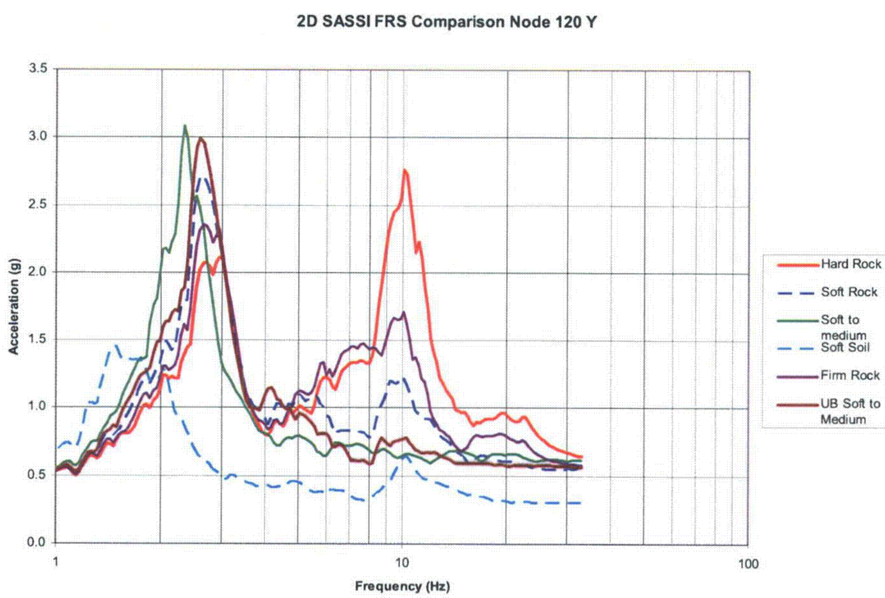


Figure 3G.3-5 - 2D SASSI FRS - Node 120 Y (ASB El. 179.6')

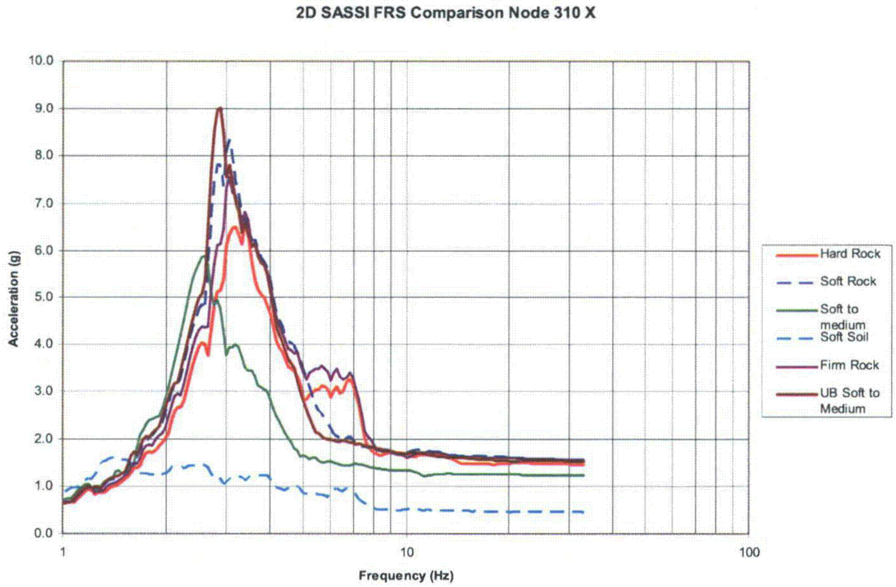


Figure 3G.3-6 - 2D SASSI FRS - Node 310 X (ASB El. 333.2')

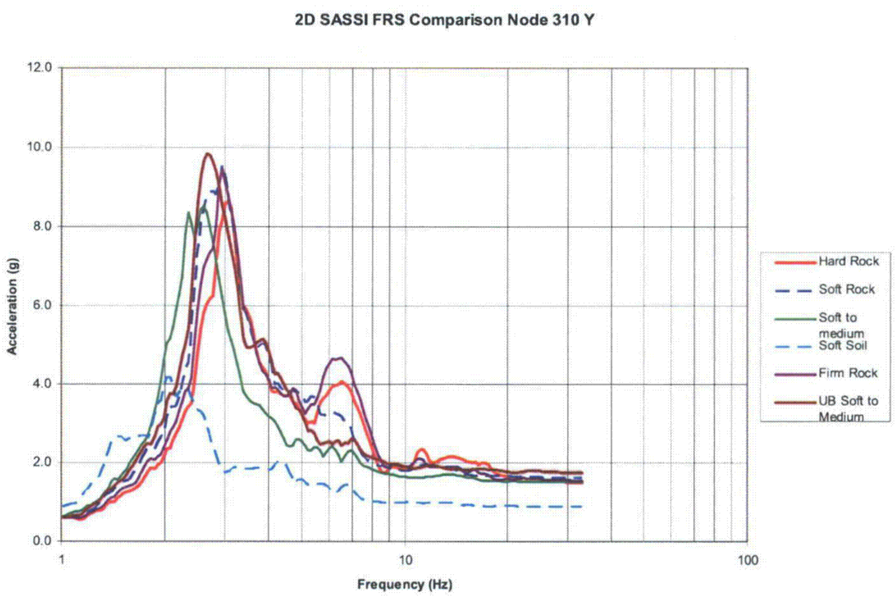


Figure 3G.3-7 - 2D SASSI FRS - Node 310 Y (ASB El. 333.2')

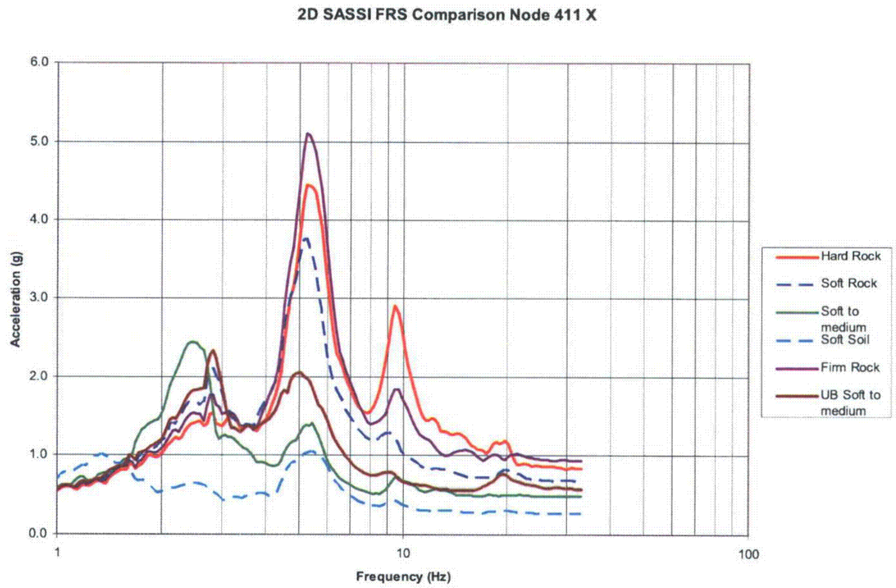


Figure 3G.3-8- 2D SASSI FRS - Node 411 X (SCV El. 200.0')

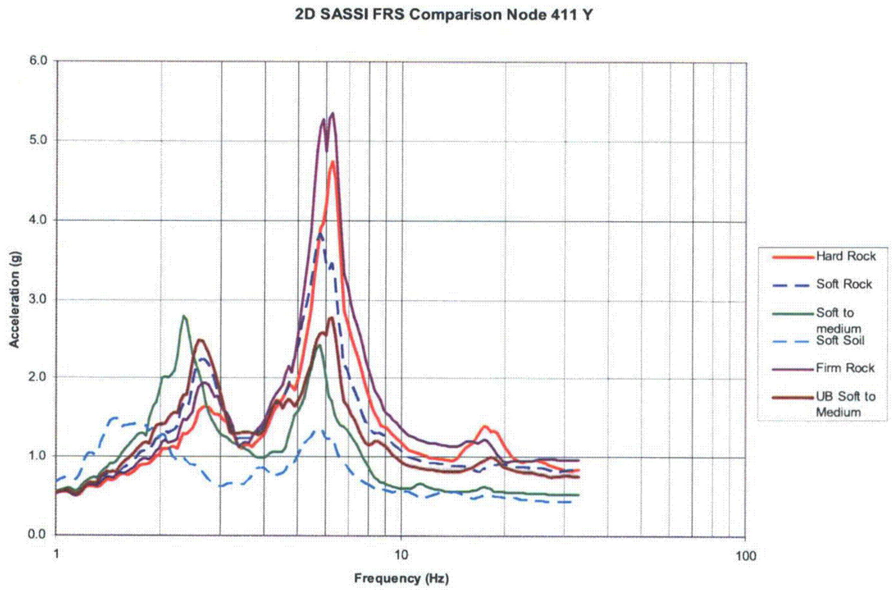


Figure 3G.3-9 - 2D SASSI FRS - Node 411 Y (SCV El. 200.0')

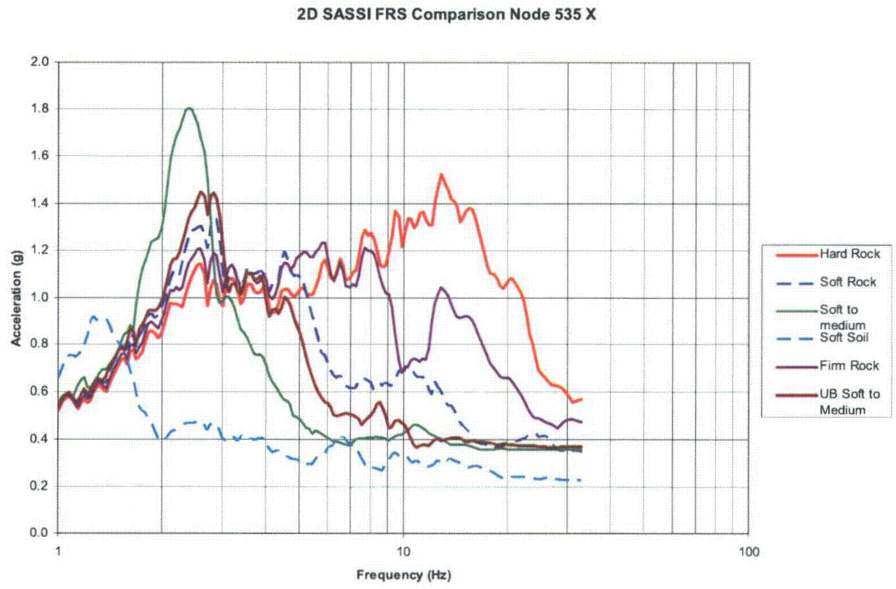


Figure 3G.3-10 - 2D SASSI FRS - Node 535 X (CIS El. 134.3')

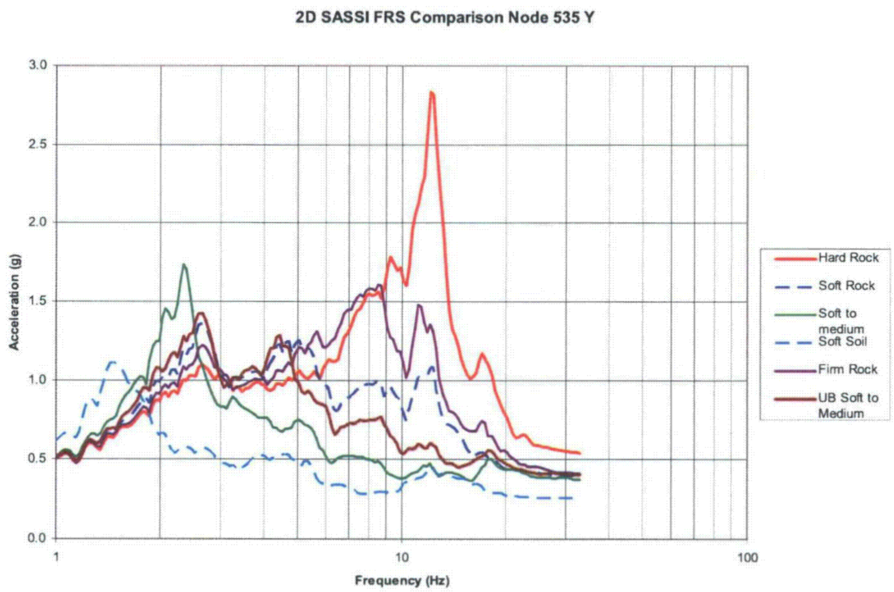


Figure 3G.3-11 - 2D SASSI FRS - Node 535 Y (CIS El. 134.3')

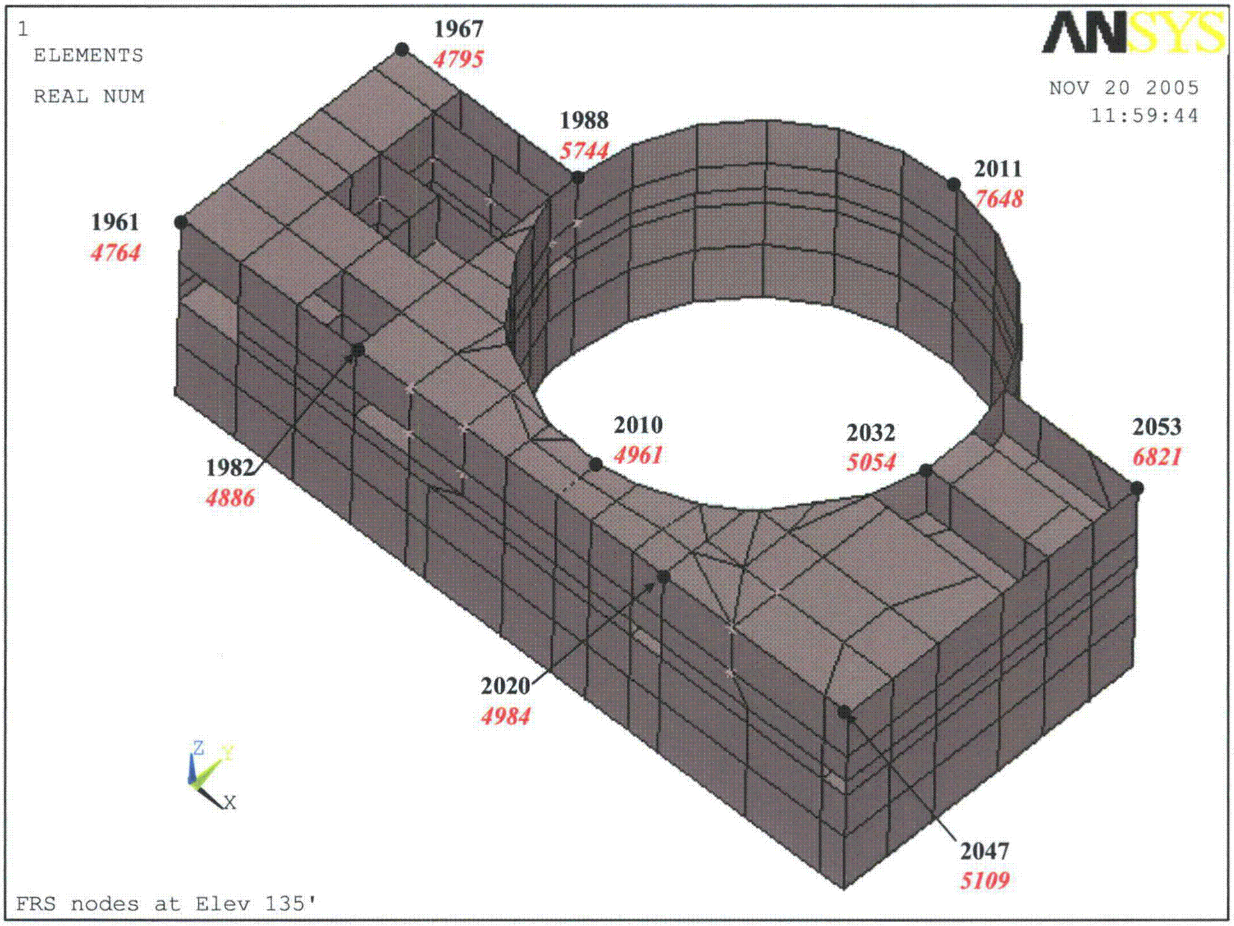


Figure 3G.4-1 – ASB "Rigid" Nodes at El. 135'

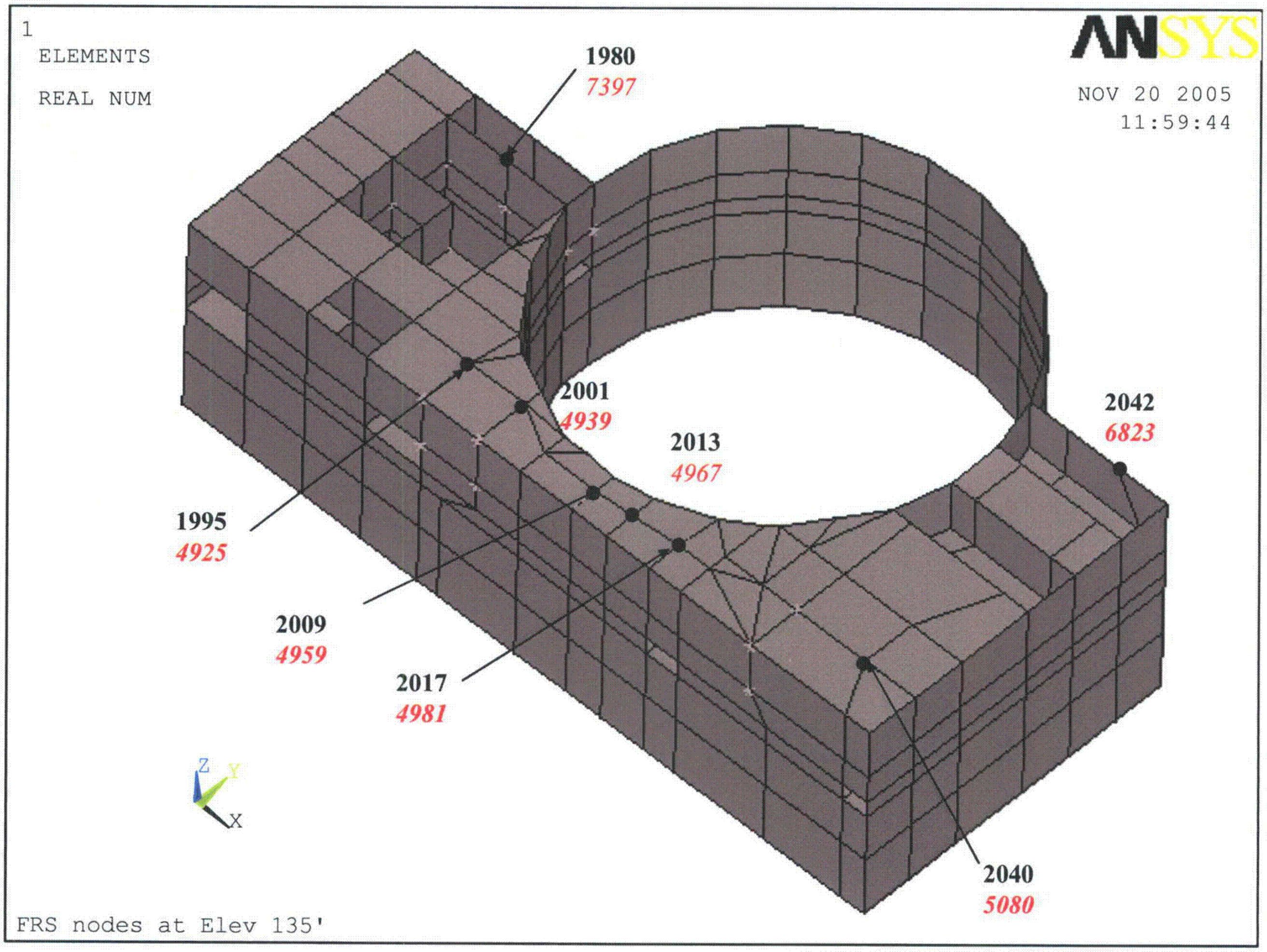


Figure 3G.4-2 – ASB "Flexible" Nodes at El. 135'

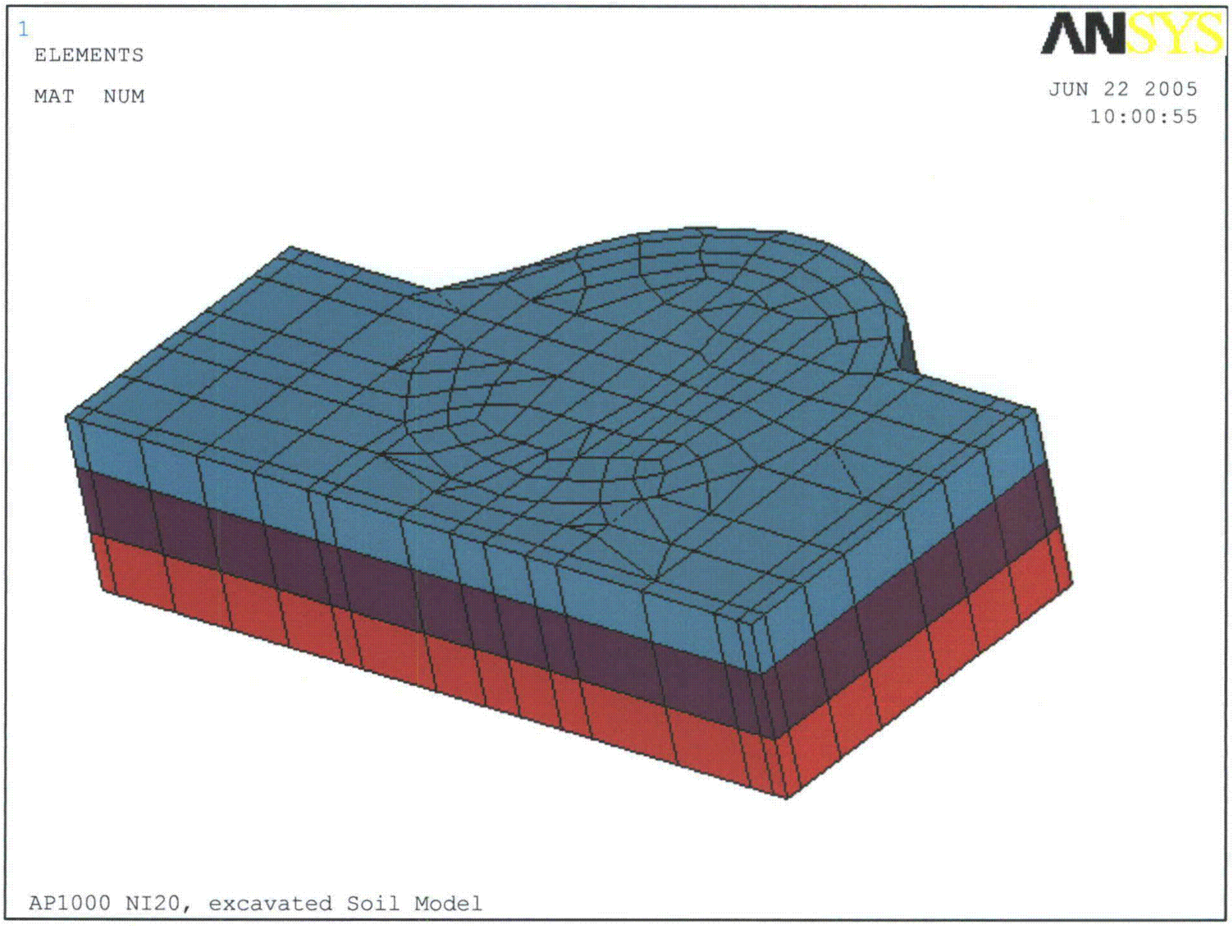


Figure 3G.4-3 - Excavated Soil

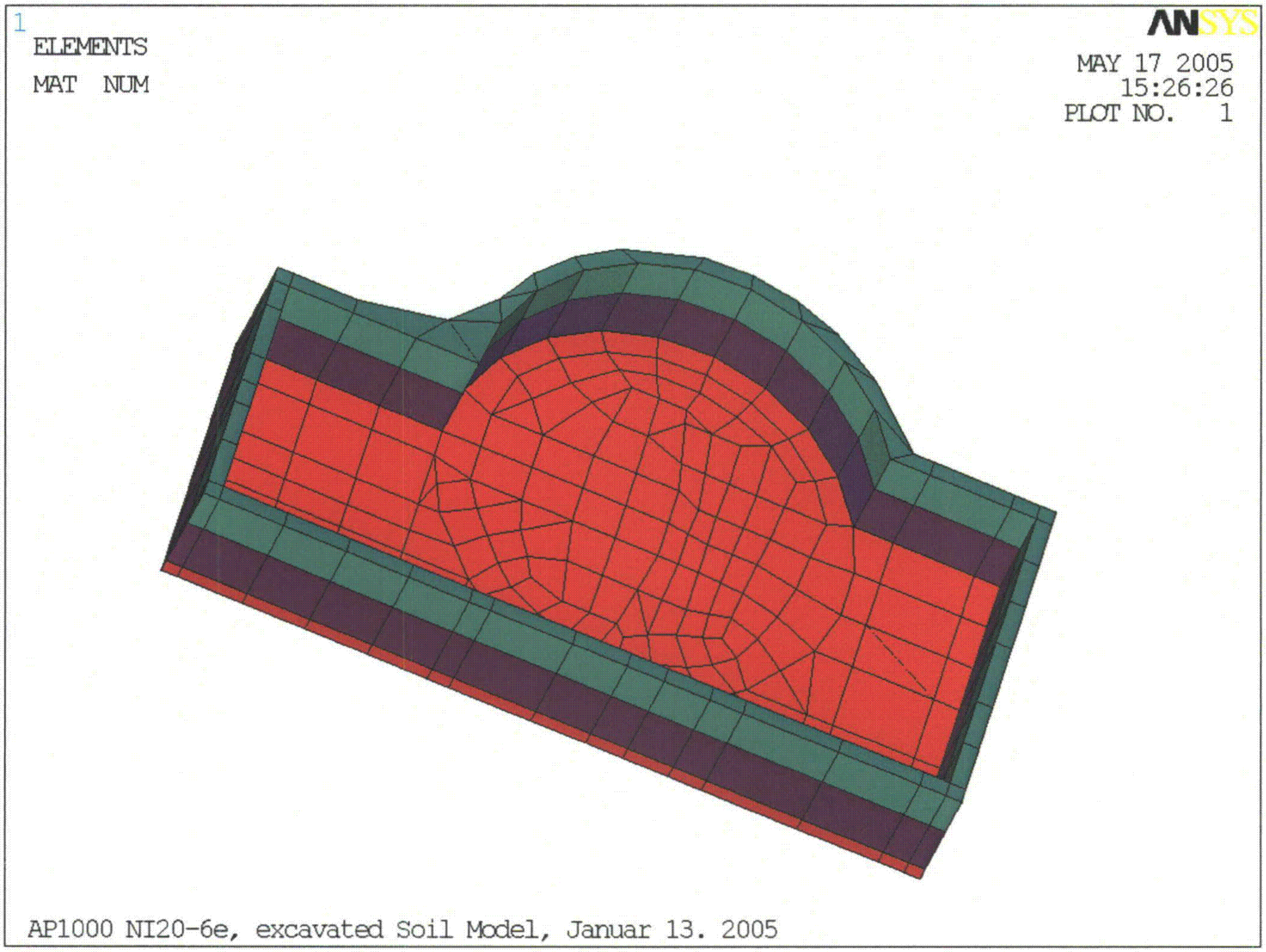
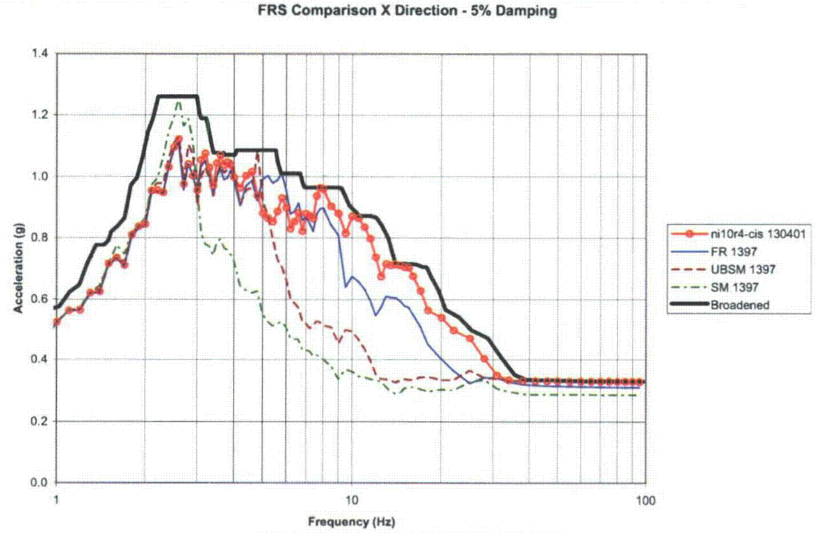
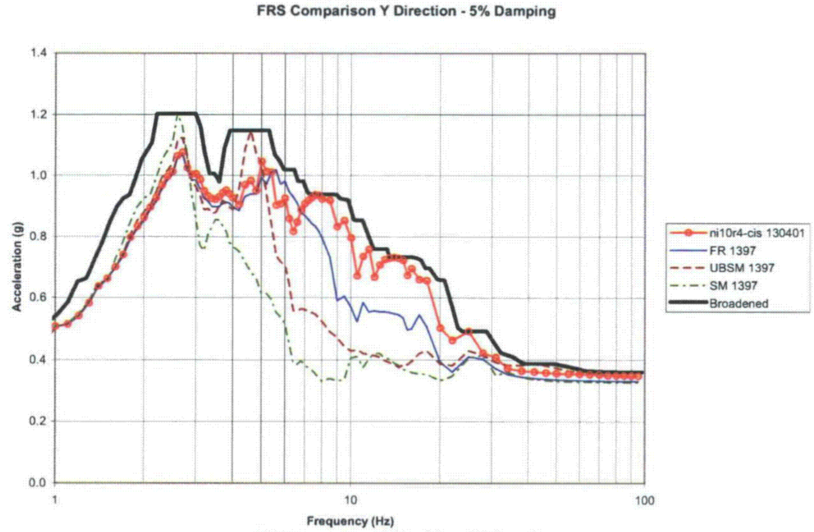


Figure 3G.4-4 - Additional Elements for Soil Pressure Calculations





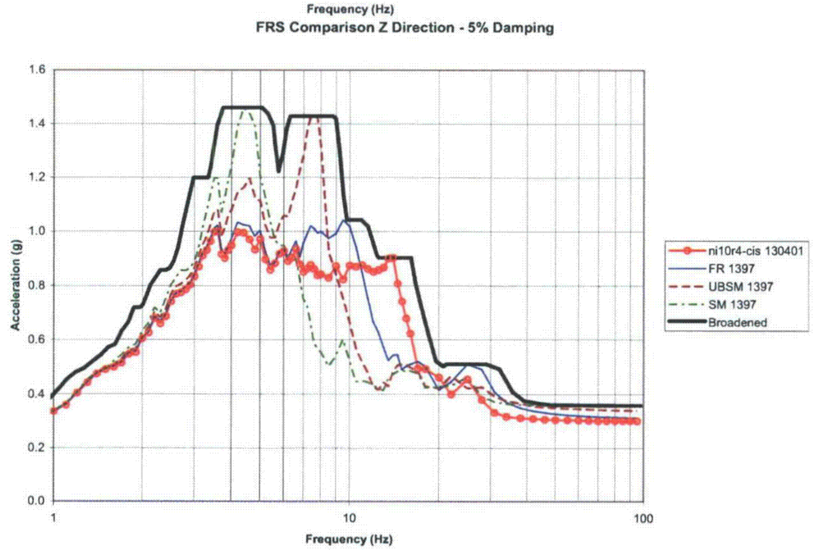
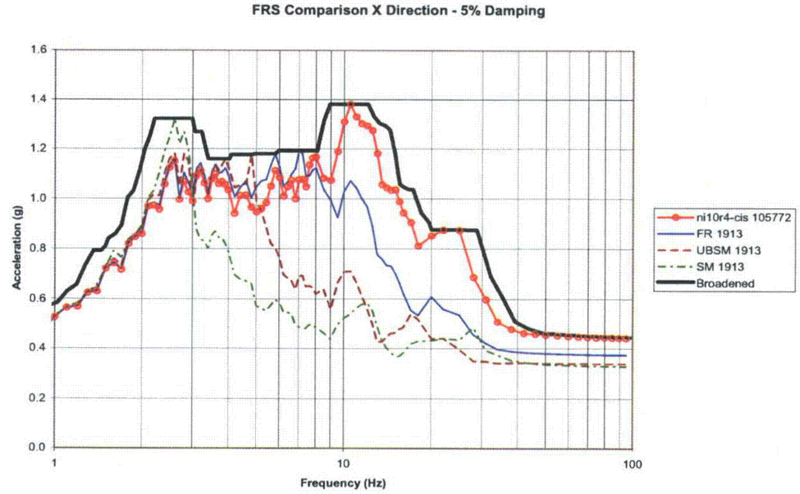
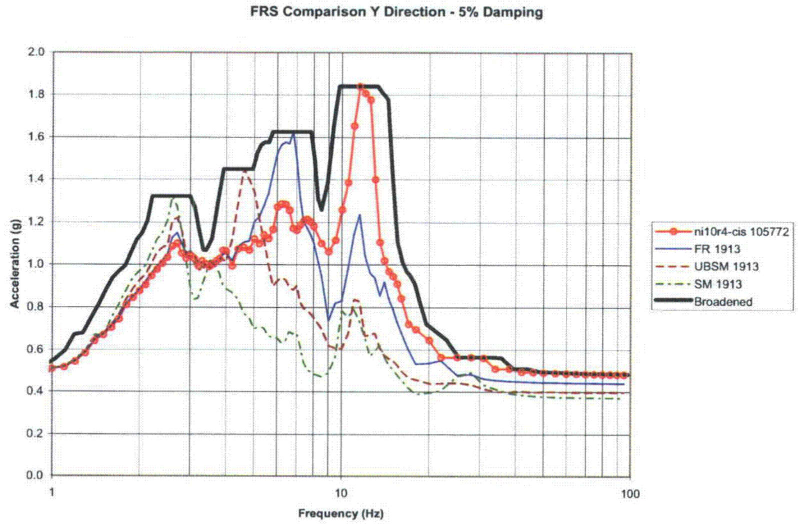


Figure 3G.4-5 –FRS for node 130401 (NI10) or 1397 (NI20)
CIS at Reactor Vessel Support Elevation of 100'





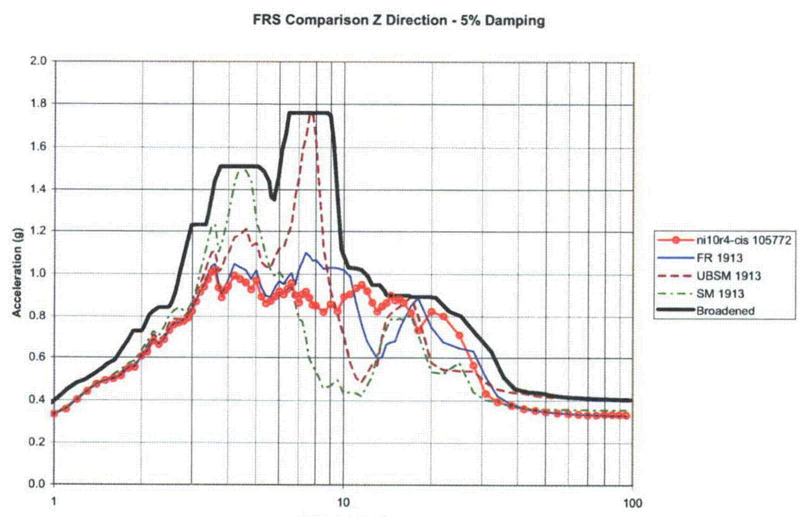


Figure 3G.4-6 – FRS for node 105772 (NI10) or 1913 (NI20) CIS at Operating Deck Elevation 134.25'

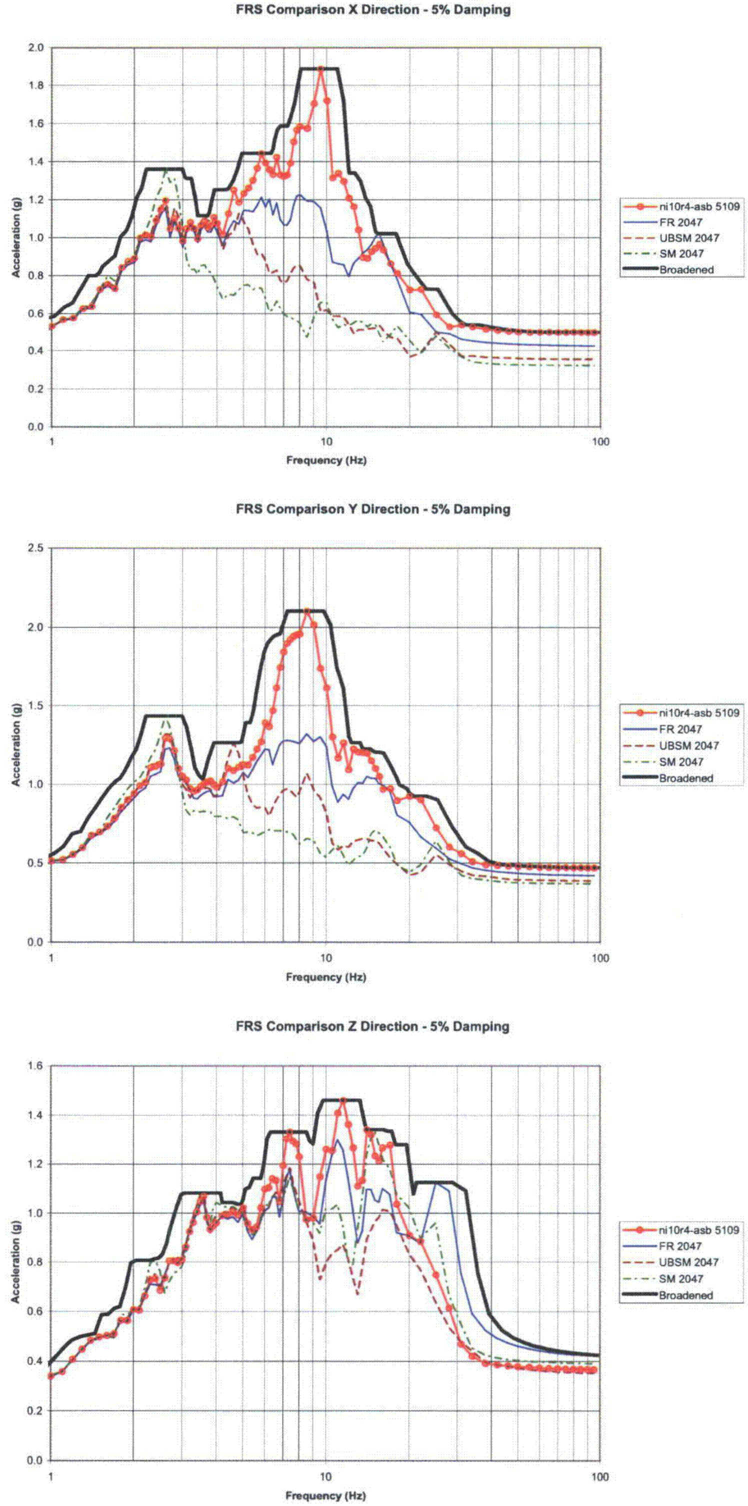


Figure 3G.4-7 – FRS for node 5109 (NI10) or 2047 (NI20)
ASB Control Room Side Elevation 134.88'

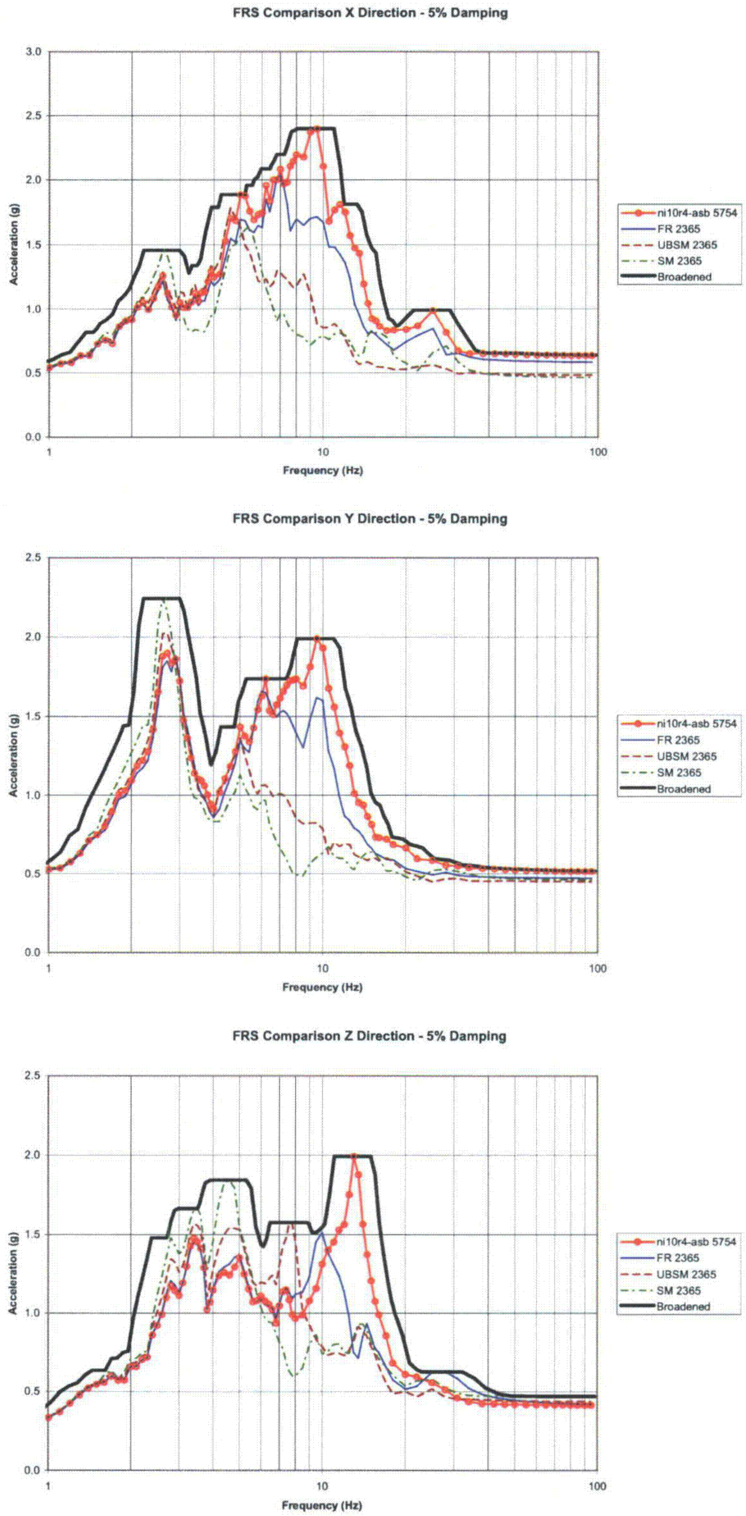


Figure 3G.4-8 –FRS for node 5754 (NI10) or 2365 (NI20)
ASB Fuel Building Roof Elevation 179.19'

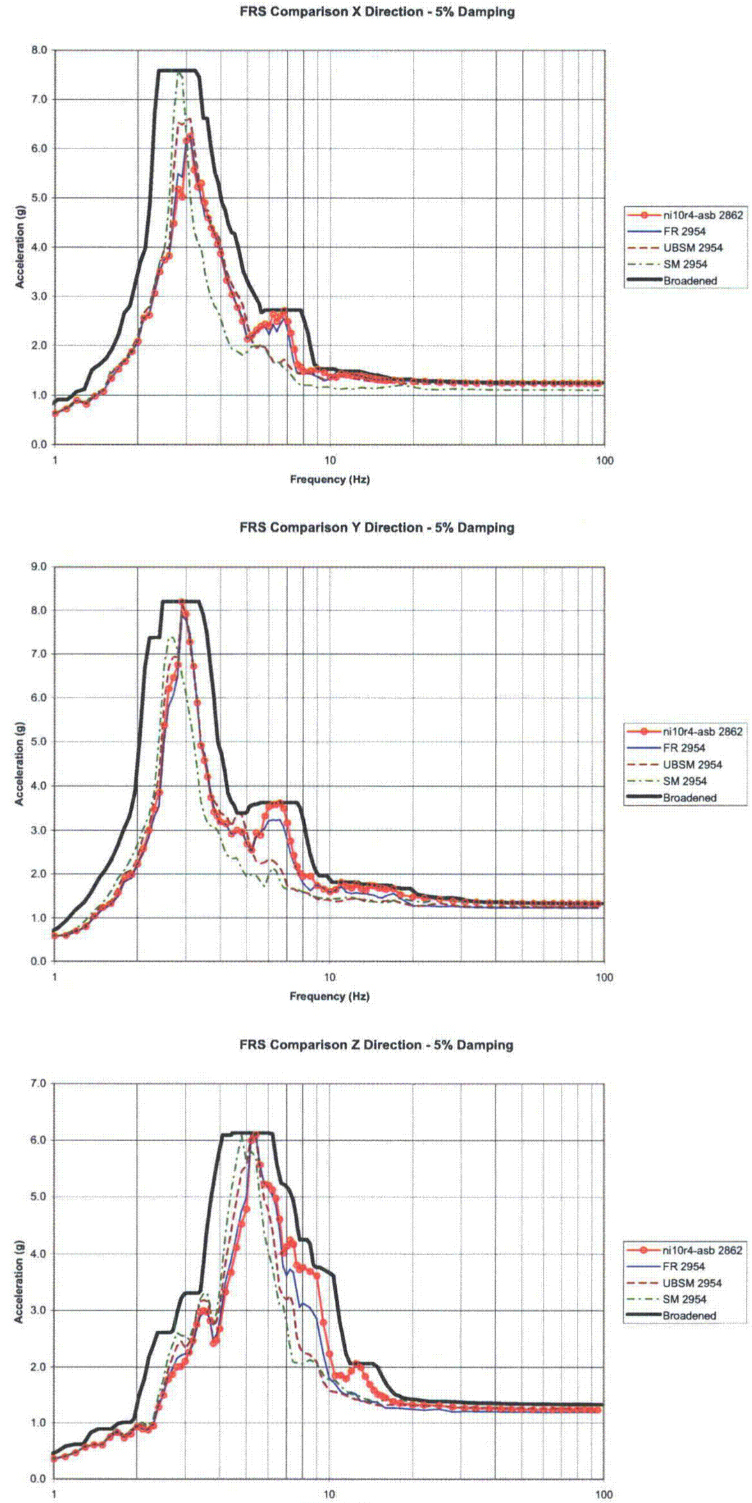


Figure 3G.4-9 –FRS for node 2862 (NI10) or 2954 (NI20)
ASB Shield Building Roof Elevation 333.12'

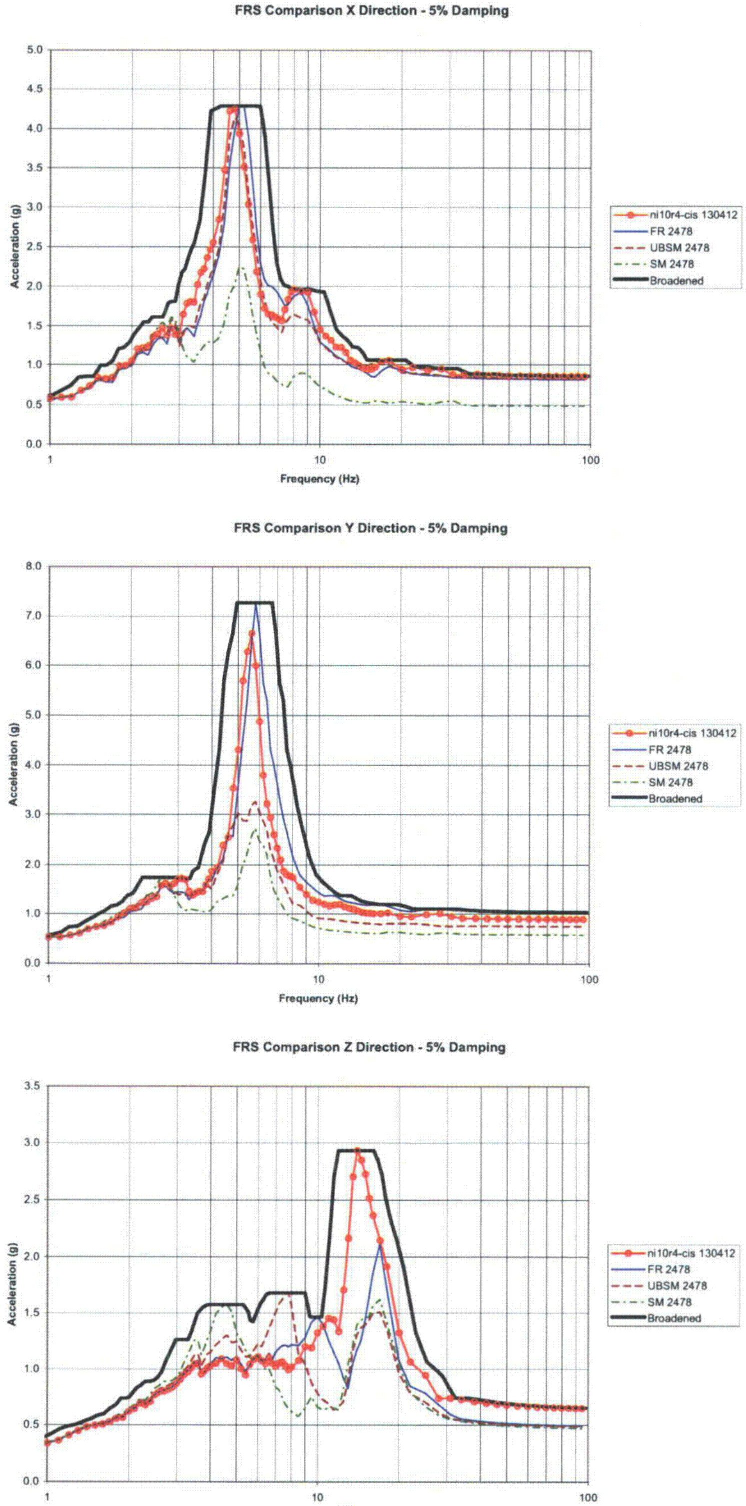


Figure 3G.4-10 – FRS for node 130412 (NI10) or 2478 (NI20)
SCV near Polar Crane elevation 224.00'

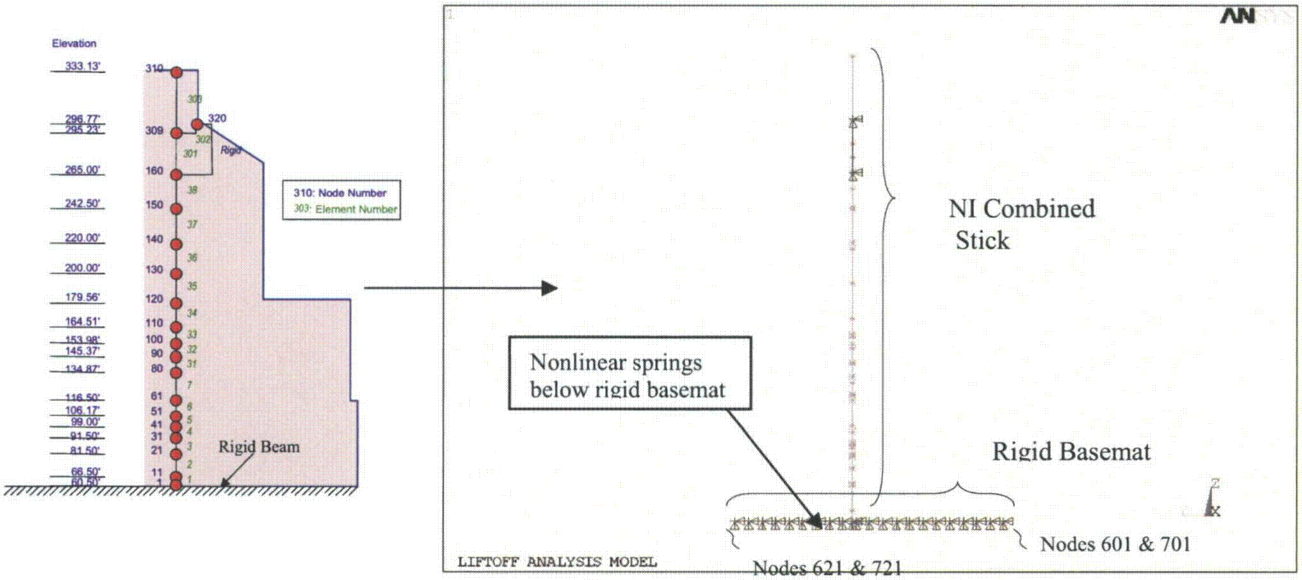
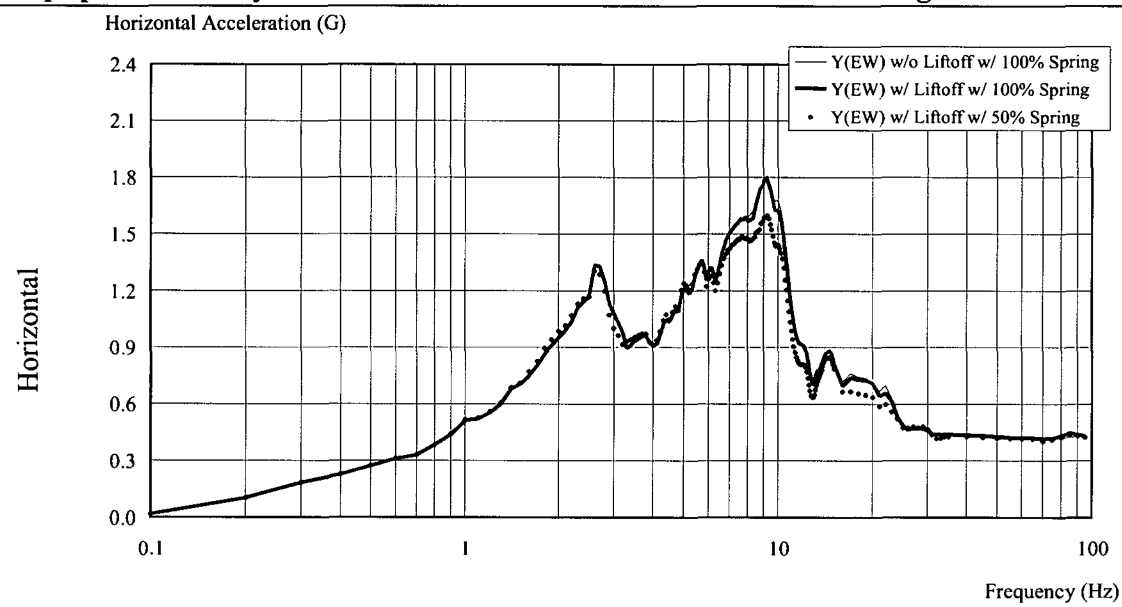


Figure 3G.6-1 - ASB Stick portion of NI combined model



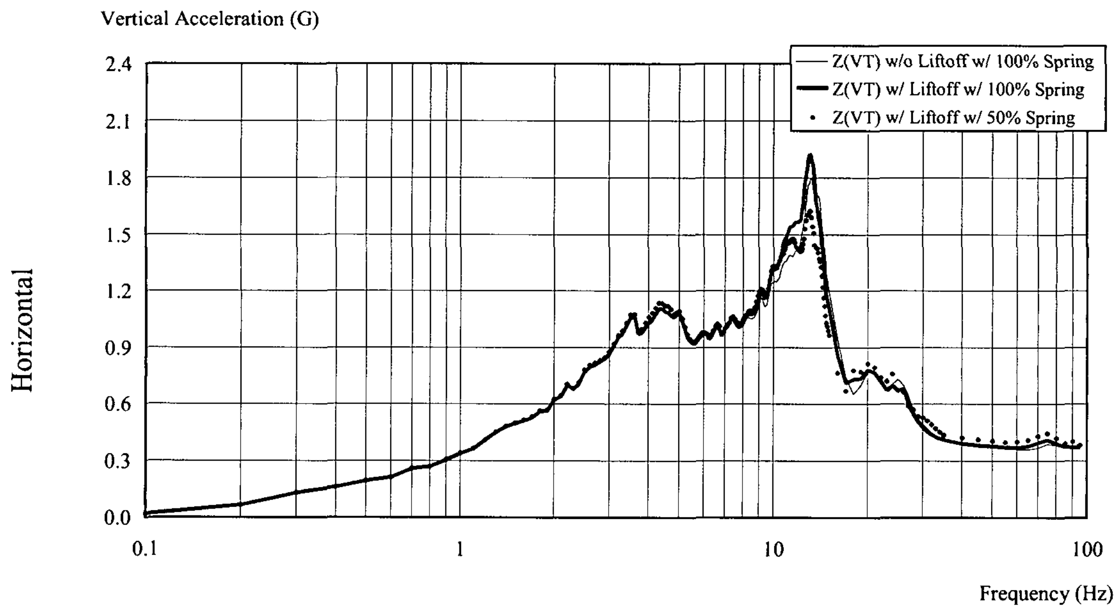
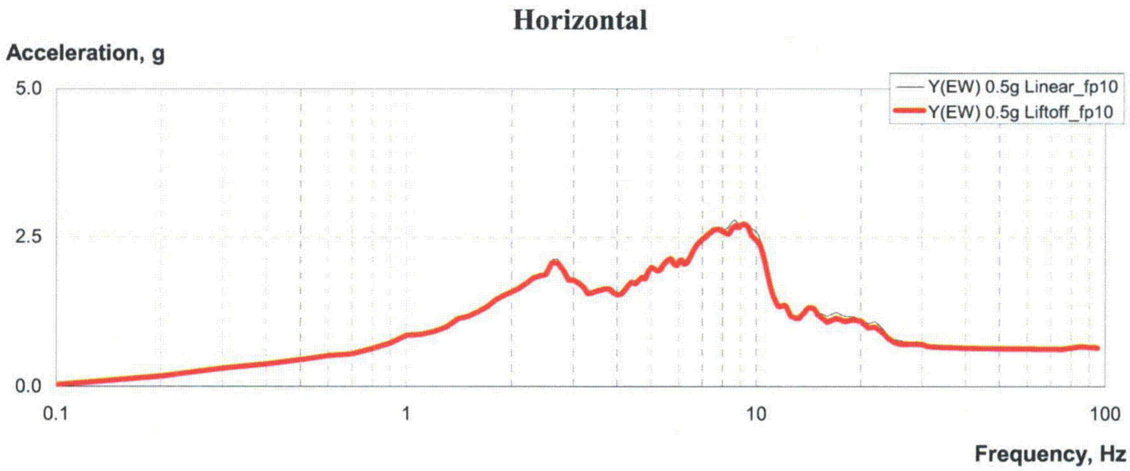


Figure 3G.6-2
SSE Floor Response Spectra at 5 % Damping – Hard Rock – ASB Node 61 (EL. 116.50')



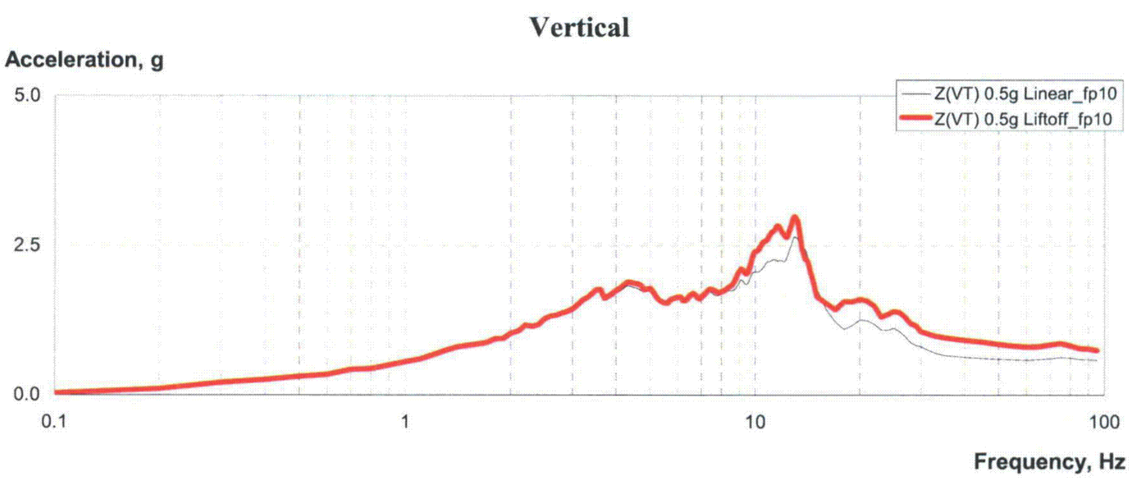


Figure 3G.6-3
RLE Floor Response Spectra at 5 % Damping – Hard Rock - ASB Node at EL. 116.50'

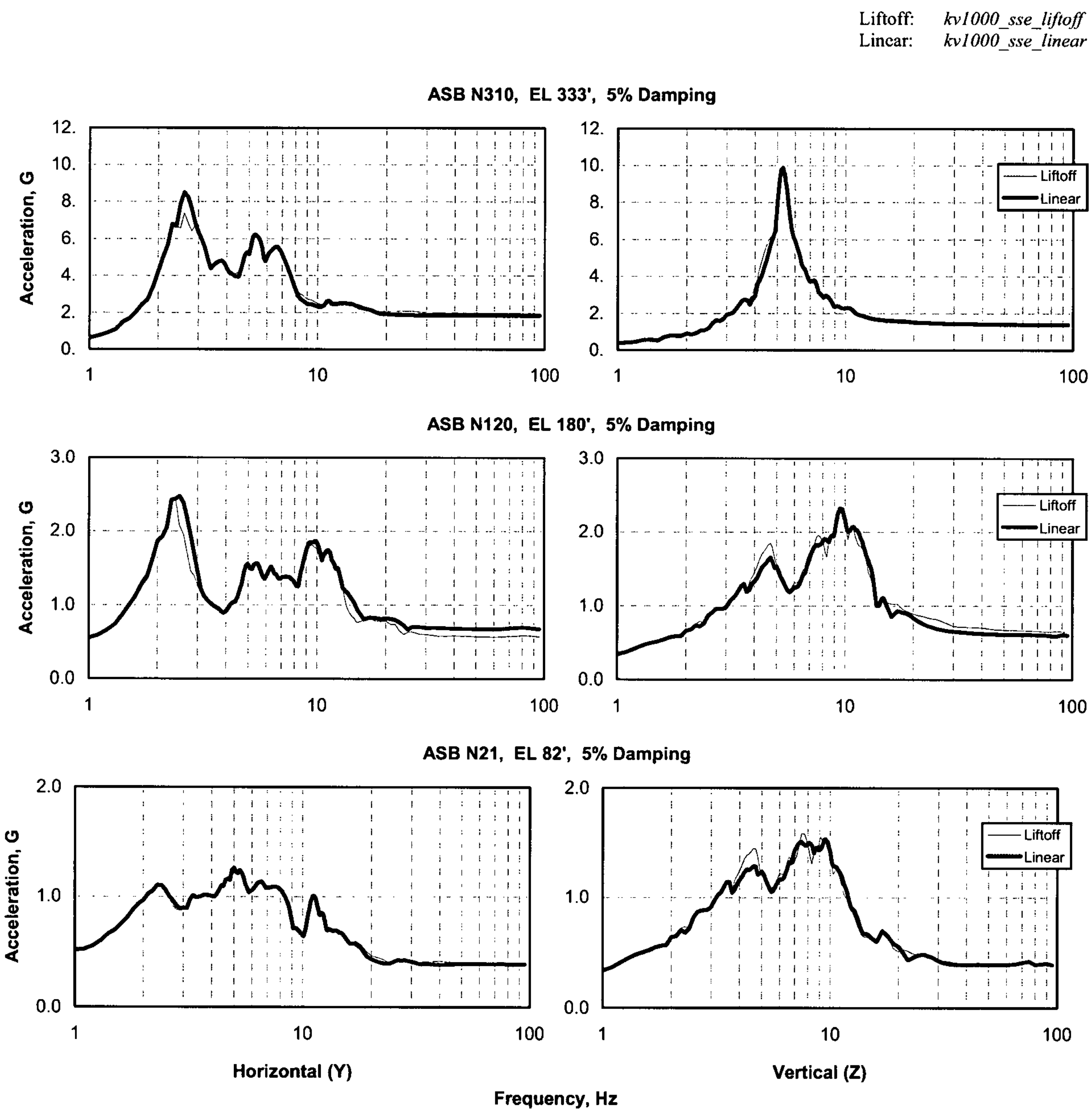


Figure 3G.6-4 - ANSYS Lift Off Effects on FRS (SSE) Soft to medium Soil

Linear: *kv1000_rle_linear*

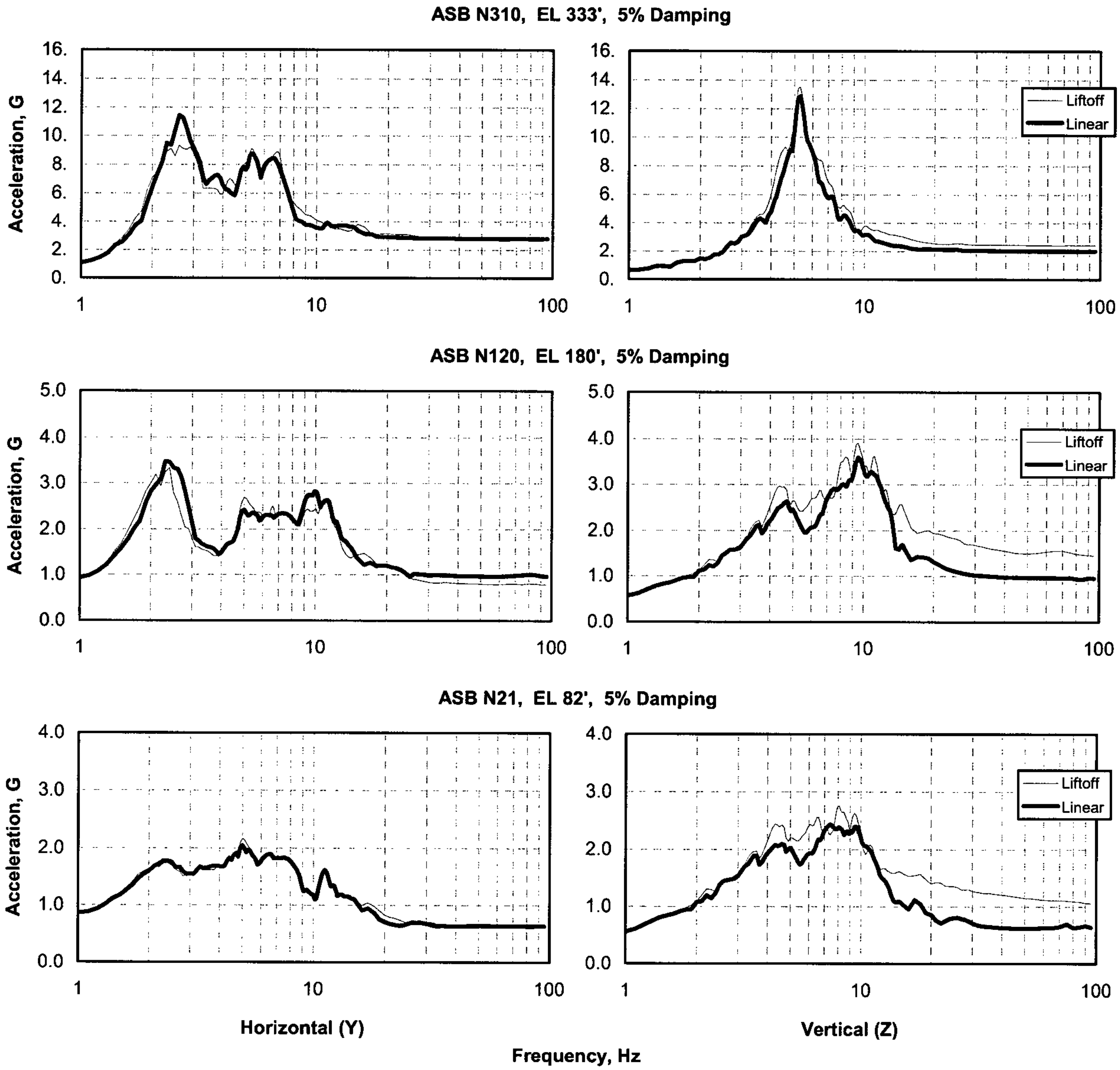


Figure 3G.6-5 - ANSYS Lift off Effects on FRS (RLE) Soft to Medium soil