DCP/NRC1526

October 4, 2002

Attachment 1

ATTACHMENT 1

Table 1 "List of Westinghouse's Responses to RAIs Transmitted in DCP/NRC1526"
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DCP/NRC1526

October 4, 2002

Attachment 2

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 220.003

Question:

In DCD Section 3.8.2, Westinghouse stated that the containment shell material is SA738, Grade B. Westinghouse further stated, in the same DCD subsection, that this material is included in the ASME Code but is not applicable for containment vessel in the 2000 Addenda. The material has been approved for containment vessels by Code Case N655. This code case was approved by the ASME Code committee on February 25, 2002, but it is not yet published.

The code case approves the use of SA-738, Grade B for Class MC components. Based on paragraph (b) of the reply to the inquiry, the allowable stress intensity (S_{mc} or S) for SA-738, Grade B used in Class MC components is 1.1 x 24.0 = 26.4 ksi at 300°F. This is based on the 1998 ASME Code, Section II, Part D, Table 1A value for S at 300°F, which is 24.0 ksi. This stress intensity limit is applied to the general primary membrane stress intensity at the design pressure and temperature. The hoop stress in the cylinder is +26,297 pounds-per-square inch (psi), based on 59 psi design pressure, t = 1.75", and r = 65' x 12 = 780". The radial stress is -59 psi at the inside shell surface and zero at the outside shell surface, resulting in an average radial stress of -59/2 = -29.5 psi. Therefore, the general primary membrane stress intensity is 26,297 + 29.5 = 26,326.5 psi, which is just below S_{mc} = 26,400 psi, at the design temperature.

Please provide justification for adopting allowable stress values for SA738 Grade B material, which are not of yet included in the current version of the ASME Code, for Class MC components.

Westinghouse Response:

SA738, grade B material is included as an acceptable material for containment vessels in the 2002 Addenda to the ASME Code. The allowable stress values are now included in the Code. The DCD will be revised to show design of the vessel to this latest addendum

Design Control Document (DCD) Revision:

3.8.2.2 Applicable Codes, Standards, and Specifications

[*The containment vessel is designed and constructed according to the ~~1998~~2001 edition of the ASME Code, Section III, Subsection NE, Metal Containment, including the 2002 Addenda*]* ~~including the 1999 and 2000 Addenda. The Combined License applicant may update the Code edition and addenda as defined in subsection 5.2.1.1. The shell material is SA738, Grade B. This material is included in the ASME code but is not applicable for containment vessels in the 2000 Addenda. The material has been approved for containment vessels by~~

~~Code Case N655. A change is being processed by ASME to include the material for containment vessels in the 2002 edition of the ASME code. The code case will be annulled once the material is included for containment vessels in the code.~~ Stability of the containment vessel and appurtenances is evaluated using ASME Code, Case N-284-1, Metal Containment Shell Buckling Design Methods, Class MC, Section III, Division 1, as published in the 2001 Code Cases, 2001 Edition, July 1, 2001.

Revise third paragraph of subsection 6.2.1.1.2

The containment vessel is designed and constructed in accordance with the ASME Code, Section III, Subsection NE, Metal Containment, ~~including Addenda through 2001,~~ as described in subsection 3.8.2.

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 220.004

Question:

Code Case N-284-1 (revision 1 to N-284) on metal containment buckling is currently unacceptable to the staff because of errors contained in the initial version. Before formal staff acceptance, the staff must confirm that the previously identified errors in N-284-1 have been corrected in the latest version. The existing staff qualification relating to the application of axi-symmetric-based design criteria to non-symmetric details such as hatches will be retained. In the AP1000 DCD, Westinghouse has referenced Code Case N-284-1, as documented in the 2001 edition of the Nuclear Code Case volume, as one technical basis for demonstrating the buckling resistance of the AP1000 containment shell. In the AP600 DCD, Westinghouse referenced Code Case N-284, Revision 0, with supplemental requirements as documented in Appendix 3G of the AP600 DCD. The staff found this acceptable for AP600. Therefore, Westinghouse is requested to provide its technical justification for the acceptability of the latest version of Code Case N-284-1 by identifying the differences and demonstrating an equivalent level of safety when compared to Code Case N-284, Revision 0, plus the supplemental requirements of AP600 DCD Appendix 3G.

Westinghouse Response:

There are no differences in the buckling criteria for the AP1000 containment vessel between the criteria given in the latest version of Code Case N-284-1 and those given by Code Case N-284, Revision 0, with supplemental requirements as documented in Appendix 3G of the AP600 DCD.

Design Control Document (DCD) Revision:

None

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 220.005

Question:

In the AP1000 DCD Section 3.8.2 "Steel Containment," Subsection 3.8.2.6, "Materials, Quality Control, and Special Construction Techniques" (Page 3.8-15), Westinghouse states that "The basic material is SA738, Grade B, plate. This material has been selected to satisfy the lowest service metal temperature requirement of -15°F. This temperature is established by analysis for the portion of the vessel exposed to the environment when the minimum ambient air temperature is -40°F. Impact requirements are as specified in NE-2000." The staff notes that for AP600 the lowest service metal temperature requirement is -40°F. Westinghouse is requested to provide the details of the analysis conducted for AP1000 that justifies the change in the minimum service temperature, from -40°F for AP600 to -15°F for AP1000, and also to indicate whether SA738, Grade B meets the impact requirements of NE-2000 if the minimum service temperature requirement is -40°F.

Westinghouse Response:

The minimum environment temperature is specified in Table 2-1 to be –40 °F. For operating conditions, the inside containment atmosphere temperature can be as low as 50 °F. Natural convection and radiation heat transfer is assumed along the inside and outside containment shell surface. Air enters the passive containment cooling system air inlets at –40 °F and is heated as it contacts the outer containment shell. The lower density associated with the warmer air causes buoyancy-induced flow and enhances heat transfer on the outside shell, so the natural circulation heat transfer coefficient is conservatively assumed to be doubled. For this case, about 63% of the overall thermal resistance occurs between the inside atmosphere and the inside shell surface, 36% occurs between the outside boundary temperature and the outside shell surface, and 1% occurs due to heat conduction through the shell itself. The temperature changes through the three regions are proportional to the resistances. Thus, for a total temperature change of 90 °F, there is a 56 °F inner film drop, a 1 °F drop through the shell, and a 33 °F drop through the outer film, and the minimum service temperature is approximately –7 °F. A minimum service temperature of –15 °F is specified for the AP1000 for added conservatism.

A738, Grade B has been used with plate thickness up to 1.15 inches in applications with a design metal temperature of –55° F. It is expected that it could be procured to meet the impact requirements of NE-2000 if the minimum service temperature requirement were -40°F.

Design Control Document (DCD) Revision:

None

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 220.010

Question:

AP1000 DCD Subsection 3.8.3.5, "Design Procedures and Acceptance Criteria," indicates that the SSE loads are derived from the response spectrum analysis of a 3D finite element model representing the containment internal structures and refers to Section 3.7.2 for the analysis method. Subsection 3.7.2.1.1 discusses the use of equivalent static acceleration analysis for containment internal structures and the coupled shield and auxiliary buildings. However, no details of the analysis method are provided. There is no discussion of response spectrum analysis in Section 3.7.2. Table 3.8.3-2 identifies that an equivalent static analysis of the 3D finite element model is utilized to obtain in-plane seismic forces for the design of floors and walls for the containment internal structures fixed at Elevation 82'-6". It is unclear what method is used to obtain out-of-plane seismic forces for design of floors and walls for the containment internal structures. The staff notes that this is a departure from the AP600 approach, which utilized the response spectrum analysis method.

In order to clarify the analysis method that is actually employed, please provide information regarding the following issues:

A. (1) a description of the use of response spectrum analysis and equivalent static analysis in defining the seismic design loads for the containment internal structures, specifically identifying where each of the methods was employed, either singly or in combination, and (2) an indication of how the three simultaneous components of seismic input motion are applied in the analyses and design.

B. a detailed description of how the equivalent static analysis method was implemented for the containment internal structures, the auxiliary building, and the shield building, including: (1) how possible seismic amplification due to out-of-plane flexibility of walls and floors was considered; (2) how the equivalent static acceleration was calculated; (3) numerical values for the significant modal frequencies; and (4) numerical values for the equivalent static accelerations used in the analyses.

C. the technical basis for concluding that a comparable level of safety is achieved for AP1000, compared to AP600.

These concerns are applicable to Section 3.8.4, "Other Category I Structures."

Westinghouse Response:

The Westinghouse responses to RAI 230.006 and 230.007 provide the information requested. Further, Tables 3.7.2-1 to 3.7.2-7 of the DCD provide numerical values for frequency and accelerations. Both the AP1000 and AP600 will maintain a comparable level of safety which is based on the code criteria stress limits that are being used.

Design Control Document (DCD) Revision:

Revise fifth paragraph of subsection 3.8.3.5

The methods described in subsection 3.7.2 are employed to obtain the safe shutdown earthquake loads at various locations in the containment internal structures. The safe shutdown earthquake loads are derived from the ~~response spectrum~~**equivalent static** analysis of a three-dimensional, finite element model representing the entire containment internal structures.

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 220.016

Question:

The third paragraph of Subsection 3.8.5.1 states that resistance to sliding of the concrete basement foundation is provided by passive soil pressure and soil friction. For the case of the AP1000 nuclear island founded on a hard rock site, Westinghouse is requested to:

A. Provide a description of construction techniques and sequence to ensure that the surrounding soil or rock (embedment) will provide enough passive pressure to prevent the nuclear island from sliding.

B. Clarify the applicability of the words "soil friction" to the AP1000 design.

C. Indicate how passive lateral pressures and base soil friction components can be properly estimated, considering consistent lateral displacements for both forces.

Westinghouse Response:

A. The excavation technique for the AP1000 is described in subsection 2.5.4.1. It is the same as that of the AP600. It may vary depending on the depth of soil over the rock. As shown in DCD Figure 3.4-1 a mud mat is placed which includes a cementitious crystalline waterproofing additive. The nuclear island basemat is then placed on top of this mud mat.

The passive pressure is calculated using an internal friction angle of 35° for the surrounding soil. The lateral resistance due to the passive pressure is then included in calculating the factor of safety against sliding following the methodology given in subsection 3.8.5.5.

B. The term "soil friction" is used in the global soil mechanics meaning. For the AP1000 plant, the basic soil mechanics formulas are used. The hard rock and concrete interface is considered to have a coefficient of friction of 0.55 per subsection 3.8.5.5.3 of the DCD. Soil friction is calculated by the following formula:

$F_f = \mu \times$ (Deadweight – Buoyant Force – 0.4 x SSE Vertical)

The 0.4 coefficient in front of the SSE Vertical force reflects the [1.0, 0.4, 0.4] combination of directional input with the horizontal load having a 1.0 coefficient.

μ =0.55 per subsection 3.8.5.5.3 of the DCD.

Buoyant Force = 74,990 kips

C. Active soil pressure acts such that it reduces the factor of safety related to sliding. Passive soil pressure provides resistance by the surrounding soil material to sliding.

The formula for active and passive pressure are given in References 1 and 2:

P_A = Active Pressure = $\frac{1}{2}\gamma H^2 \tan^2(45° - \phi/2)$

$\gamma = 0.0876$ kips/ft^3,
$\phi = 35°$
H = 99' 6" – 60' 6" = 39'

P_A = 18.05 kips/ft of wall width

P_P = Passive Pressure = $\frac{1}{2}\gamma H^2 \tan^2(45° + \phi/2)$
P_P = 245.8 kips/ft of wall width

The frictional sliding resistance (F_f) between the basemat and rock interface is calculated using formulas given in subsection 3.8.5.5.

References:

1. Terzaghi, Karl, and Ralph B. Peck, Soil Mechanics in Engineering Practice, John Wiley & Sons, Inc., New York, 1948.
2. Taylor, Donald W., Fundamentals of Soil Mechanics, John Wiley & Sons, Inc., New York, 1948.

Design Control Document (DCD) Revision:

None

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 220.017

Question:

Section 3.8.5 contains a number of apparent inconsistencies related to the designation of Tier 2* material and the status of the AP1000 basemat design.

Subsection 3.8.5.4.3, "Design Summary of Critical Sections" references both Table 3.8.5-3 and Figure 3.8.5-3 as showing the basemat reinforcement details for the basemat critical sections. The design of the critical sections is designated Tier 2* in the text of Subsection 3.8.5.4.3, and Table 3.8.5-3 is also designated Tier 2*. However, in Figure 3.8.5-3, only sheets 1,2, and 5 are designated Tier 2*, while sheets 3 and 4 are unmarked.

Table 3.8.5-3 includes Note (5), indicating that "The results are representative for the AP1000 and may be updated when structural calculations are completed." However, Figure 3.8.5-3 does not have a comparable note, implying that the information provided reflects the AP1000 final basemat design.

Please provide an explanation for these apparent inconsistencies by (1) identifying what is Tier 2*, what is not Tier 2*, and the technical basis for the proposed designation, and (2) describing the status of the AP1000 final basemat design and the relationship between the information in Table 3.8.5-3 and Figure 3.8.5-3 to the AP1000 final basemat design.

Westinghouse Response:

The final design of the two critical sections of the nuclear island basemat defined in the DCD is in progress and will be available for the structural audit. The required reinforcement in Table 3.8.5-3 will be updated once the design calculation is completed. The figures may not need to be revised since it is anticipated that the structural demand at a hard rock site is lower than that at the AP600 envelope of sites which is the basis for the reinforcement provided in Figure 3.8.5-3.

Tier 2* material was selected for the AP600 by NRC staff. The markings of Tier 2* in the AP600 DCD on Figure 3.8.5-3 also show that only sheets 1,2, and 5 are designated Tier 2*, while sheets 3 and 4 are unmarked.

The AP1000 Tier 2* designations were based on those in AP600 and will be revised. Westinghouse proposes that Tier 2* be applied to the critical sections where the design calculations will be subject to NRC staff review in the proposed structural audit. Since Figure 3.8.5-3 contains significantly more information than is covered by the critical sections, the Tier 2* designation will be removed from all sheets of the figure but will be retained on Table 3.8.5-3.

Design Control Document (DCD) Revision:

In Figure 3.8.5-3, sheets 1,2, and 5, delete Tier 2* designation

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 220.018

Question:

Table 3.8.5-2 lists factors of safety for floatation, overturning, and sliding applicable to the "hard rock condition," calculated in accordance with Subsection 3.8.5.5, "Structural Criteria." Since there is no indication that these factors of safety are subject to change, the staff concludes that the factors of safety are based on the actual AP1000 basemat loads due to deadweight, flood, groundwater, wind, tornado, and earthquake. To facilitate the staff's review, Westinghouse is requested to provide the numerical values of the basemat loads used in the above calculations, for both the AP1000 and AP600, and to describe any basemat design changes, from AP600 to AP1000, necessary to meet the minimum factor of safety requirements listed in Table 3.8.5-1.

Westinghouse Response:

The plan dimensions of the Nuclear Island for the AP1000 plant are the same as for the AP600 plant. The Basemat design is the same for the AP1000 and AP600. The AP1000 shield building and steel containment are 25' 6" higher. Given below is a summary of pertinent loads for the AP600 and AP1000 plants.

The weight and center of gravity (in the plant coordinate system) are provided in the following table for both the AP1000 and AP600 plants.

Weight

	AP1000	AP600
Weight	280,715 kips	255,380 kips
Xcg	992.34'	992.315'
Ycg	986.45'	985.632'
Zcg	125.14'	117.426'

The buoyant force, lateral forces due to active and passive soil pressures, and overburden pressure are given in the table below. They are the same for the both the AP600 and AP1000 plants.

Hydrodynamic and Soil Pressures

Buoyant Force	Vertical	74,990 kips
Active Soil Pressure Force	North-South	2,906 kips
Active Soil Pressure Force	East-West	4,621 kips
Passive Soil Pressure Force	North-South	39,574 kips
Passive Soil Pressure Force	East-West	62,925 kips
Surcharge Pressure Force	North-South	1,927 kips
Surcharge Pressure Force	East-West	3,805 kips

The seismic reactions at elevation 60.5' for the AP1000 and AP600 plants are given in the tables below.

AP1000 Seismic Loads
Units: kips & ft-kip

Moments Relative to Center of Containment		Moments Relative to End Boundaries of NI Basemat	
Seismic Reactions	**Absolute Value**	**Seismic Reactions**	**Absolute Value**
Vertical	101,495	Moment Line 1	14,903,017
Shear NS	99,814	Moment Line 11	14,791,352
Moment about NS	8,547,756	Moment Line I	12,639,190
Shear EW	93,516	Moment West Side of Shield Building	13,644,293
Moment about EW	10,662,959		

AP600 Seismic Loads
Units: kips & ft-kip
Moments Relative To Center Of Containment

Seismic Reactions	Hard Rock
Axial	89,234
Shear NS	91,368
Moment about NS	7,715,877
Shear EW	91,868
Moment about EW	7.478,927

The factors of safety associated with the AP1000 plant for the stability evaluation are given in Table 3.8.5-2, which will be revised in the DCD as shown below. The minimum required factors of safety for overturning, sliding, and flotation of structures are given in Table 3.8.5-1.

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

The factors of safety associated with the AP1000 and AP600 plants from the stability evaluation for the hard rock case are given in the table below.

FACTORS OF SAFETY FOR FLOTATION, OVERTURNING AND SLIDING OFNUCLEAR ISLAND STRUCTURES

Environmental Effect	AP1000	AP600
Flotation		
High Ground Water Table	3.7	3.4
Design Basis Flood	3.5	3.2
Sliding		
Design Wind, North-South	18.4	> 12.5
Design Wind, East-West	14.0	> 9.5
Design Basis Tornado, North-South	10.3	> 6.9
Design Basis Tornado, East-West	8.6	> 6,1
Safe Shutdown Earthquake, North-South	1.25	1.2
Safe Shutdown Earthquake, East-West	1.5	1.3
Overturning		
Design Wind, North-South	~~45.5~~ **51.7**	62.4
Design Wind, East-West	28.0	24.8
Design Basis Tornado, North-South	~~15.6~~ **17.7**	18.1
Design Basis Tornado, East-West	9.6	8.7
Safe Shutdown Earthquake, North-South	1.75	2.0
Safe Shutdown Earthquake, East-West	1.2	1.2

Design Control Document (DCD) Revision:

Revise Table 3.8.5-2 as follows:

FACTORS OF SAFETY FOR FLOTATION, OVERTURNING AND SLIDING OF NUCLEAR ISLAND STRUCTURES

HARD ROCK CONDITION

Environmental Effect	Factor of Safety [(1)]
Flotation	
High Ground Water Table	3.7
Design Basis Flood	3.5
Sliding	
Design Wind, North-South	18.4
Design Wind, East-West	14.0
Design Basis Tornado, North-South	10.3
Design Basis Tornado, East-West	8.6
Safe Shutdown Earthquake, North-South	1.25
Safe Shutdown Earthquake, East-West	1.5
Overturning	
Design Wind, North-South	~~45.5~~51.7
Design Wind, East-West	28.0
Design Basis Tornado, North-South	~~15.6~~17.7
Design Basis Tornado, East-West	9.6
Safe Shutdown Earthquake, North-South	1.75
Safe Shutdown Earthquake, East-West	1.2

Notes:

1. Factor of safety is calculated for a site with rock below the underside of the base mat (elevation 60'-6") and soil adjacent to the exterior walls above this elevation

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 230.001

Question:

In the first paragraph of Subsection 3.7.1.2 (Page 3.7-1), Westinghouse states that site-specific time histories may be used as defined in Subsection 2.5.4.5.5. However, Subsection 2.5.4.5.5 does not exist. Please clarify this definition in the DCD.

Westinghouse Response:

The reference to site specific time histories will be removed.

Design Control Document (DCD) Revision:

3.7.1.2 Design Time History

A "single" set of three mutually orthogonal, statistically independent, synthetic acceleration time histories is used as the input in the dynamic analysis of seismic Category I structures. ~~Site-specific time histories may be used as defined in subsection 2.5.4.5.5.~~ The synthetic time histories were generated by modifying a set of actual recorded "TAFT" earthquake time histories.

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 230.002

Question:

For the case of the AP1000 nuclear island founded on a hard rock site with a 40 feet embedment, described in Subsection 3.7.1.1, "Design Response Spectra," and in the last sentence in Page 3.7-2, you stated that the design ground response spectra are applied at the foundation level in the free field. However, in Subsection 3.7.1.2, "Design Time History," you stated that the design time histories are applied at the finished grade in the free field. Also, as stated in Subsection 3.7.1.2, the three components of the ground motion time history were derived from the design response spectra. Please address the following issues:

A. The location where the ground motion (design response spectra and design time histories) is applied should be consistent throughout the entire AP1000 DCD.

B. Subsection 3.7.2.1 indicates that soil-structure interaction (SSI) effects are negligible for the AP1000 nuclear island founded on hard rock and that the effect of embedment below grade is not considered in the equivalent static and time history analyses of the structure. The staff's concern is that if the plant is founded on a hard rock surface and is surrounded by soil, the application of the design ground motion at the ground surface may result in underestimation of the seismic responses of the plant without considering the SSI effects. Please elaborate regarding the staff's concern.

C. Please provide a description in the DCD that explains: (1) how lateral soil pressures (dynamic, active and/or passive) due to the embedment (plant embedded in the rock and plant founded on rock and surrounded by soil) are to be calculated, (2) how the out of phase motion between the soil burden and the side walls of the nuclear island structures will be accounted for in the assessment of the lateral soil pressure, and (3) how the soil lateral pressure will be incorporated into the design of the nuclear island side walls, basemat and below grade interior members.

Westinghouse Response:

A. The ground motion is applied at the foundation level in the free field. DCD subsection 3.7.1.2 will be revised as shown below.

B. The design ground motion is not applied at the ground surface. Deconvolution through a soil column is not performed. The effect of lateral support due to the side soils is addressed in the response to RAI 230.014.

C. The exterior walls below grade are designed for embedment in soil above the foundation level at elevation 60' 6". Lateral earth pressures for this case are conservatively taken equal to the passive earth pressure.

Design Control Document (DCD) Revision:

Revise paragraph in subsection 3.7.1.2:

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

Revise paragraph in subsection 3.8.4.4.1:

The seismic Category I structures are reinforced concrete and structural module shear wall structures consisting of vertical shear/bearing walls and horizontal slabs supported by structural steel framing. In-plane seismic forces are obtained from the equivalent static analysis of the three dimensional finite element models described in Table 3.7.2-14. These results are modified to account for accidental torsion as described in subsection 3.7.2. Where the refinement of these finite element models is insufficient for design of the reinforcement, for example in walls with a large number of openings, detailed finite element models are used. Also evaluated and considered in the shear wall and floor slab design are out-of-plane bending and shear loads, such as live load, dead load, seismic, lateral earth pressure, hydrostatic, hydrodynamic, and wind pressure. These out-of-plane bending and shear loads are obtained from the equivalent static analyses supplemented by hand calculations. The exterior walls of the seismic Category I structures below the grade are designed to resist the worst case lateral earth pressure loads (static and dynamic), soil surcharge loads, and loads due to external flooding as described in Section 3.4. ~~Two dimensional SASSI models are used to calculate the lateral earth pressures on the exterior walls below grade.~~**The worst case lateral earth pressure loads in the seismic event are taken equal to full passive earth pressure.**

PRA Revision:

None.

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 230.003

Question:

In DCD Subsection 3.7.2.1, "Seismic Analysis Methods," Westinghouse stated that the computer program ANSYS is to be used to perform equivalent static analyses and mode superposition time history analyses. The following items were identified by the staff for clarification:

A. Subsection 3.7.1.3 provides a description of how the composite modal damping is calculated for the seismic analysis. Please demonstrate, in the AP1000 DCD, that the method for calculating the modal damping adopted in the ANSYS computer code is consistent with the method described in Subsection 3.7.1.3.

B. In Table 3.7.2-16, Westinghouse stated that the 100%, 40%, 40% combination technique is applied to combine the three components of the seismic responses when the ANSYS computer program is used. Please clarify whether the ANSYS computer program has the option of using the 100%, 40%, 40% combination technique.

Westinghouse Response:

A. The method for calculating the modal damping adopted in the ANSYS computer code is described in equation 17.7-1 of the ANSYS Theory Manual (see attached page 17-34). The damping ratio is input in ANSYS as a material property. The damping ratio in each mode is proportional to the strain energy in the mode. This is the same as the stiffness weighted method described in DCD subsection 3.7.1.3.

B. The 100%, 40%, 40% combination technique is incorporated in a post processor written by the user. This approach was also used for the AP600 ANSYS analyses of the nuclear island basemat.

Design Control Document (DCD) Revision:

None

PRA Revision:

None

Table 17.7.1 Types of Spectrum Loading

	Excitation Option			
	Excitation at Support			Excitation Away From Support
Spectrum Input	Response spectrum table (**FREQ** and **SV** commands)			Amplitude multiplier table (**FREQ** and **SV** commands)
Orientation of Load	Direction vector (input on **SED** command)			X, Y, or Z direction at each node (selected by FX, FY, or FZ on **F** command)
Distribution of Loads	Constant on all support points			Amplitude in X, Y, or Z directions (selected by *VALUE* on **F** command)
Type of Input	Velocity	Acceleration	Displacement	Force
Response spectrum type (*KSV* on **SVTYP** command)	0	2	3,4	1

Damping

Damping is evaluated for each mode and is defined as:

$$\xi_i' = \frac{\beta\omega_i}{2} + \xi_c + \frac{\sum_{m=1}^{NMAT} \xi_m E_m^s}{\sum_{m=1}^{NMAT} E_m^s} + \xi_i \qquad (17.7\text{-}1)$$

where:

ξ_i' = effective damping ratio for mode i

β = beta damping (input as *VALUE*, **BETAD** command)

ω_i = undamped natural circular frequency of the i^th mode

ξ_c = damping ratio (input as *RATIO*, **DMPRAT** command)

ξ_m = damping ratio for material m (input as DAMP on **MP** command)

E_m^s = $\frac{1}{2}\{\phi_i\}^T[K_m]\{\phi_i\}$ = strain energy

$\{\phi_i\}$ = displacement vector for mode i

$[K_m]$ = stiffness matrix of part of structure of material m

ξ_i = modal damping ratio of mode i (**MDAMP** command)

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 230.004

Question:

For the damping values shown in Table 3.7.1-1, identify (a) those damping values that are based on ASCE Standard 4-98, and (b) bases or source references for the damping values for fuel assembly, control rod drive mechanisms, cabinets and panels for electrical equipment, and equipment such as welded instrument racks and tanks. Since the AP1000 nuclear island structures are to be analyzed for the safe shutdown earthquake (SSE), the damping values, recommended in Regulatory Guide (RG) 1.61, "Damping Values for Seismic Design of Nuclear Power Plants," for an SSE, are acceptable to the staff. Westinghouse should clarify when, where, and how the ASCE Standard 4-98 damping values are to be used.

Westinghouse Response:

The damping values that are shown in Table 3.7.1-1 are the same as those approved by the NRC for the AP600 plant. The bases or source references for the damping values were provided to the NRC when requested as part of the AP600 licensing process.

The damping values are the same as given in RG 1.61 (and in ASCE Standards 4-98 and 4-86) for: bearing bolted structures and equipment; prestressed concrete structures; reinforced concrete structures. As stated in the response to AP600 RAI 230.29, the HVAC damping values are also chosen in conformance with RG 1.61.

The NRC was provided proprietary and non-proprietary information (NTD-NRC-95-4460 dated May 10, 1995) related to the issue of the damping value assigned for the fuel assemblies (20 percent). Also, Westinghouse provided a response to AP600 RAI 230.29 that requested the bases of the 20% damping value for the fuel assemblies. The NRC concluded in Chapter 3 of the AP600 FSER (NUREG-1512), page 3-247 that the use of this damping value was acceptable.

In the response to AP600 RAI 230.29 Westinghouse stated that the damping used for electrical raceway systems, including cable trays and related supports is based on tests (Bechtel/ANCO test results – Reference 19 of the AP1000 DCD). This reference is also recognized in ASCE 4-98 for cable tray damping. Also, as stated in the responses to AP600 RAI 230.7 and AP600 RAI 210.13, the damping value for conduits is based on the same tests reported in Reference 19 of the AP1000 DCD.

The damping values for control rod drive mechanisms, cabinets and panels for electrical equipment, and equipment such as welded instrument racks and tanks are based on recognized industry practice and were approved by the NRC for the AP600 plant. These values are equally applicable for the AP1000 plant.

Design Control Document (DCD) Revision:

None

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 230.005

Question:

The horizontal design ground motion response spectrum (and associated enveloping time histories) is indicated to be appropriate for application to the Eastern United States (EUS) sites at a mean annual probability of exceedance of 10^{-4} per year. Recent developments in ground motion assessment show that spectral shapes applicable to the EUS rock sites are rich in the high frequency range. These shapes of ground response spectra indicate that the peaks at high frequencies of 10 hertz (hz) and above are higher than those used for the AP1000 design. Please demonstrate that the design of the AP1000 structures, systems and components, based on its proposed design ground response spectra shown in Figure 3.7.1.1, will be an adequate design. The attached figures (Figures 1 through 3) show the result of recent development on the ground motions.

Westinghouse Response:

Westinghouse is requesting design certification of the AP1000 for the same seismic input motions as were specified for the AP600. It is recognized that recent developments show spectral peaks that are rich in the high frequency range. This was considered during the AP600 development and Westinghouse designed for input motions that were enhanced in the high frequency range above the values that would have been suggested by Regulatory Guide 1.60.

The subject of high frequency input motions is being discussed separately by industry and NRC staff. Generally, high frequency input is not expected to be damaging to nuclear power plant systems, structures and equipment. Westinghouse will continue to monitor the progress of the industry and NRC discussions.

The Combined License applicant will demonstrate that his site falls within the interface parameters for which the AP1000 is designed. These parameters include the ground motion design spectra and require demonstration that the site spectra are equal or less than those specified for the design of the AP1000.

Design Control Document (DCD) Revision:

None

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 230.006

Question:

In Subsection 3.7.2.1, "Seismic Analysis Methods," Westinghouse replaced the description of the response spectrum analysis method (Subsection 3.7.2.1.1) with the equivalent static acceleration analysis method when converting the AP600 DCD to AP1000 DCD. Also, the application of the response spectrum analysis method was eliminated from Table 3.7.2-14, "Summary of Models and Analysis Methods." The following areas were identified by the staff for clarification:

A. If the response spectrum analysis method will not be used for the AP1000 design, Westinghouse should clearly state that this method will be excluded from the AP1000 DCD. Otherwise, a new subsection should be developed specifically address the use of the response analysis method.

B. If the response spectrum analysis method will not be used for the AP1000 design, any description related to this method should be deleted from Section 3.7.2. Examples are found in: (1) the third paragraph of Subsection 3.7.2.6 (Page 3.7-13), (2) Subsection 3.7.2.7 (Page 3.7-14), (3) Table 3.7.2-16 (Page 3.7-74), and Figures (sic, Tables?) 3.7.2-17 through 3.7.2-19.

Westinghouse Response:

Westinghouse has not eliminated response spectrum analysis as an acceptable method for the design and analysis of the nuclear island structures, substructures and equipment; however, it is not the method of analysis used for the global analyses. The global analyses of the nuclear island structures use equivalent static acceleration and mode superposition time-history methods. Subsection 3.7.2.1.3 will be added to the DCD as requested by the NRC.

Design Control Document (DCD) Revision:

Add new subsection 3.7.2.1.3

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.

Revise fifth paragraph of subsection 3.7.2-6 as follows:

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

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

- Mode Superposition Time History Analysis (program ANSYS) – the time history responses from the three components of earthquake motion are combined algebraically at each time step.

Delete subsection 3.7.2.12

3.7.2.12 Deleted~~Comparison of Responses~~

~~The three dimensional lumped mass fixed base stick model of the nuclear island was analyzed by mode superposition time history analysis and by the response spectrum analysis method for the hard rock site condition. Tables 3.7.2-17, 3.7.2-18, and 3.7.2-19 compare the maximum absolute nodal accelerations, member forces, and moments, respectively. The time history analyses considered vibration modes up to 118.6 hertz. In the response spectrum analyses, the combination of modal responses used the grouping method for vibration modes up to 33 hertz. The two methods of analysis give similar results with the response spectrum analysis being generally more conservative.~~

Table 3.7.2-16

3D lumped mass stick, fixed base models	Mode superposition time history analysis	ANSYS	Algebraic Sum	n/a
	~~Response spectrum analysis~~	~~ANSYS~~	~~SRSS~~	~~Grouping~~

Delete Tables 3.7.2-17, 3.7.2-18 and 3.7.2-19

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 230.007

Question:

A discussion of seismic analyses using equivalent static acceleration analysis and time history analysis methods is provided in Section 3.7.2. Describe how the results (member forces and floor response spectra) from either method will be used in the AP1000 plant design. Also, when using the equivalent static acceleration analysis method in conjunction with a three dimensional (3D) finite element model, describe how the seismic effects (member forces or accelerations) obtained based on the nuclear island stick model will be used to calculate the seismic design forces in all elements, and how you account for the out-of-plane effects due to seismic excitation in the design of walls, floors and attached safety-related subsystems.

Westinghouse Response:

Seismic in-structure (floor) response spectra are developed using the time history analysis methods described in subsection 3.7.2.1.2. These floor response spectra are used to perform seismic response spectrum analyses of attached equipment and structures as described in subsection 3.7.3. Floor response spectra are also developed for flexible floors in safety related areas using appropriate portions of the finite element models described in subsection 3.7.2.3. Revisions will be included in DCD subsection 3.7.2.1.2 to describe these analyses.

Subsection 3.7.2.6 describes how seismic member forces are calculated when the equivalent static acceleration analysis method is used in conjunction with a three-dimensional (3D) finite element model. The three seismic components of earthquake are applied separately. An analysis for each earthquake component is made by applying equivalent static loads to the structural model at each finite element node with mass equal to the mass times the maximum absolute acceleration value (obtained from the time history analysis of the stick models) for the earthquake component being evaluated. The results obtained for each of the three components of earthquake motion are combined by one of two methods:

- Each of the member forces due to the three earthquake components calculated from the equivalent static analyses are combined using the square root of the sum of squares (SRSS) method.
- Each of the member forces 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.

The seismic analysis methodology for out-of-plane effects on flexible floors and walls is described in subsection 3.7.3. Member forces in the floors, walls and slabs used for the design of nuclear island structures are developed using either the equivalent static acceleration method or the mode superposition time-history method. DCD subsection 3.8.4.4.1 will be modified to cross reference subsection 3.7.3.

Torsion effects are accounted for using the method described in subsection 3.7.2.11. DCD subsection 3.7.2.11 will be revised as shown below to state that these torsion effects are added absolutely to the corresponding translation case prior to combination of the effects due to the three directions of input.

Design Control Document (DCD) Revision:

Add new paragraph after second paragraph in subsection 3.7.2.1.2:

The three-dimensional finite element model of the auxiliary and shield building, or a portion thereof, developed as described in subsection 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.

Revise paragraph in subsection 3.7.2.11:

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

- Horizontal mass properties of the building stick models shown in Figures 3.7.2-4, 3.7.2-5, and 3.7.2-6,
- The ~~enveloping~~ **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.
- The torsional moments due to eccentricities of the masses at each elevation are assumed to act in the same direction on each structure. ~~Both positive and negative values are considered.~~

- 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 $R = Fact1\left(|A_{NS}| + |T_{NS}|\right) + Fact2\left(|A_{EW}| + |T_{EW}|\right) + Fact3\left(|A_{VT}|\right)$

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]

Revise paragraph in subsection 3.8.4.4.1:

The seismic Category I structures are reinforced concrete and structural module shear wall structures consisting of vertical shear/bearing walls and horizontal slabs supported by structural steel framing. ~~In-plane s~~Seismic forces are obtained from the equivalent static analysis of the three dimensional finite element models described in Table 3.7.2-14. **The out-of-plane bending and shear loads for flexible floors and walls are analyzed using the methodology described in subsections 3.7.2.6 and 3.7.3.** These results are modified to account for accidental torsion as described in subsection 3.7.2.11. Where the refinement of these finite element models is insufficient for design of the reinforcement, for example in walls with a large number of openings, detailed finite element models are used. Also evaluated and considered in the shear wall and floor slab design are out-of-plane bending and shear loads, such as live load, dead load, seismic, lateral earth pressure, hydrostatic, hydrodynamic, and wind pressure. These out-of-plane bending and shear loads are obtained from the equivalent static analyses supplemented by hand calculations.

The exterior walls of the seismic Category I structures below the grade are designed to resist the worst case lateral earth pressure loads (static and dynamic), soil surcharge loads, and loads due to external flooding as described in Section 3.4. Two dimensional SASSI models are used to calculate the lateral earth pressures on the exterior walls below grade.

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 230.008

Question:

During the AP1000 pre-application review, Westinghouse indicated that only the hard rock site will be considered in the design certification application of the AP1000 (see Westinghouse letter dated February 13, 2002, ADAMS Accession No. ML020640065). Westinghouse also stated, in DCD Subsection 3.7.2.1.2, that for the hard rock site, the SSI effect is negligible. Therefore, for the hard rock site, the nuclear island is analyzed as a fixed-base structure, using ...foundation media. Based on the definition provided in AP600 DCD, Appendix 2B (Table 2B-1), the shear wave velocity for hard rock sites should be 8,000 ft/sec or higher. However, Westinghouse defined, in DCD Tier I material, Table 5.0-1, the rock site as a site with the shear wave velocity equal to or higher than 3,500 ft/sec, and stated, in DCD Section 3.7.2, that fixed-base seismic analyses are performed for nuclear island at a rock site. In the DCD, Westinghouse should address the following:

A. State whether the site condition used in the design certification application for the AP1000 is a hard rock site or rock site.

B. If the hard rock site condition is to be used, specify the shear wave velocity for a hard rock site (i.e., 8,000 ft/sec or higher based on the AP600 definition) which can reasonably simulate the fixed base condition for the AP1000 design. In this scenario, you are also requested to delete any design information not related to the hard rock site.

C. If the rock site with a shear wave velocity equal to 3,500 ft/sec or higher is to be used, provide your basis to justify that the use of a fixed-base structural model can lead to adequate calculation of the seismic responses (member forces and floor response spectra) of the nuclear island structures founded on a rock site.

Westinghouse Response:

Westinghouse is requesting design certification based on the fixed base seismic analyses. Table 5.0-1 in Tier 1 and Table 2-1 in Tier 2 will be revised to show that the shear wave velocity should exceed 8000 feet per second. Westinghouse expect that the nuclear island design using the results of the fixed-base seismic analyses will be adequate for sites with lower shear wave velocities. However, such justification is not part of the current application and may be provided as part of a Combined License application.

Design Control Document (DCD) Revision:

See response to 240.002

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 230.009

Question:

The fifth paragraph of Section 3.7.2 (page 3.7-6) states that Table 3.7.2-14 summarizes the types of model and analysis methods that are used in the seismic analyses of the nuclear island. It also summarizes the type of results that are obtained and where they are used in the design. With regard to the modeling of the nuclear island, the staff identified the following items for clarification:

A. The location (elevation) of "fixed base" should be clearly specified in the DCD.

B. Westinghouse should provide information regarding the model, analysis methods, and computer codes to be used for calculating the overturning moment, sliding force, floating force, etc. Also, describe how the calculated overturning moment, sliding force, and floating force are to be used for evaluating the dynamic stability of the nuclear island and the foundation mat design.

Westinghouse Response:

A. The location of the fixed base has been revised to the bottom of the foundation mat (Elevation 60'-6") instead of the top of the mat (Elevation 66'-6"). See also the Westinghouse response to RAI 230.011.

B. An ANSYS deadweight run (stick model) was made to determine the dead weight and center of gravity of the Nuclear Island. The seismic reactions on the Nuclear Island Basemat (Elevation 60.5') are calculated from the ANSYS safe-shutdown earthquake (SSE) time history analysis. A post processor is used to combine the loads so that the overturning moment along column lines 1, 11, I, and the West side of the Shield Building are obtained. The seismic overturning moments are adjusted for nuclear island missing mass above 88.466 hertz. Hand calculations were made to determine factors of safety for stability following subsection 3.8.5.5 methodology.

Design Control Document (DCD) Revision:

None

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 230.010

Question:

Figure 3.7.2-18 shows the combined fixed-base stick model for the time history analyses. In this figure, three lateral supports are provided to represent the support for hard rock to grade. The staff's review identified the following items for clarification:

A. The last sentence of the third paragraph of Subsection 3.7.2.1.1 states that the support provided by the embedment below grade is not considered in the seismic analyses. This statement is not consistent with the lateral support provided for the coupled shield and auxiliary building stick model. This inconsistency should be clarified.

B. Since there is no contact between the containment internal structures and the hard rock foundation, Westinghouse should explain why these two lateral supports were included in the containment internal structure lumped mass stick model to consider the effects of the hard rock support.

Westinghouse Response:

A. The lateral supports for hard rock to grade will be removed from this figure.

B. A horizontal connection will be shown between the containment internal structures and the auxiliary and shield building at grade. This restrains the two structures horizontally at the center of the containment and represents bearing between the two structures at the containment vessel interface.

The figure is also being revised to reflect revised models where both the auxiliary and shield building stick and the containment internal structures stick were extended down to the underside of the basemat at elevation 60' 6".

Design Control Document (DCD) Revision:

See the revised figure 3.7.2-18 in the proposed revision to DCD section 3.7 transmitted by letter DCP/NRC1526.

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 230.011

Question:

In the last paragraph of Subsection 3.7.2.1 (Page 3.7-8) and Table 3.7.2-4 (Page 3.7-61), Westinghouse states that the combined lumped-mass stick model of the nuclear island structures is fixed at the top of the basemat at Elevation 66'-6" to simulate the hard rock foundation media. The staff's review identified the following items for clarification:

A. Regarding the appropriateness of using a fixed-base model to simulate the nuclear island structures founded on a hard rock foundation media, Westinghouse should provide a detailed description of construction technique and construction sequence (either in Section 2.5 or in Section 3.8.5) to demonstrate that the nuclear island structures founded on hard rock foundation media can be reasonably represented by a fixed-base structural model.

B. In order to include the mass effects of the foundation mat, the fixed base should be located at the bottom of the foundation mat (Elevation 60'-0") instead of the top of the mat (Elevation 66'-6"). The staff is unclear about your rationale for excluding the mass effects from the foundation mat in the seismic analyses that are based on the current fixed base model (at Elevation 66'-6"). The staff suspects that the calculated seismic responses (member forces, nodal accelerations, overturning moments, base shear, etc.) will be underestimated. Westinghouse should provide a justification to show that the seismic responses calculated based on the seismic model fixed at the top of the foundation mat (Elevation 66'-6") will result in a more conservative design than that in which the model is fixed at the bottom of the basemat (Elevation 60'-0").

Westinghouse Response:

A. The excavation technique for the AP1000 is described in subsection 2.5.4.1. It is the same as that of the AP600. It may vary depending on the depth of soil over the rock. As shown in DCD Figure 3.4-1 a mud mat is placed which includes a cementitious crystalline waterproofing additive. The nuclear island basemat is then placed on top of this mud mat.

B. The location of the fixed base has been revised to the bottom of the foundation mat (Elevation 60'-6") instead of the top of the mat (Elevation 66'-6").

Design Control Document (DCD) Revision:

See proposed revisions to DCD Section 3.7 transmitted by letter DCP/NRC1526.

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 230.012

Question:

Subsection 3.7.2.2 states that the time history seismic analyses of the nuclear island consider vibration modes having a frequency up to 114 hz. Subsection 3.7.1.2 states that in the fixed-base modal superposition time history analyses of the nuclear island, the time step of the ground motion time histories is 0.005 second. Given the time step of 0.005 second, a time history analysis is typically accurate only for modes having a frequency up to about 50 hz. Therefore, Westinghouse should provide verification of the accuracy of the time history analysis results for modes having a frequency between 50 hz and 114 hz.

Westinghouse Response:

The fixed base nuclear island analyses use 200 modes in the time history analyses. High frequency modes are included in the analysis in order to capture the effect of the "rigid body modes". In the most recent AP1000 analyses in the proposed revision to DCD Section 3.7 transmitted by letter DCP/NRC1526, the 200th mode has a frequency of about 85 hertz. The first 200 modes include 85% of the horizontal mass in the X direction, 84% of the horizontal mass in the Y direction and 80% of the vertical mass.

There will be no loss of accuracy since the modes in question reflect the "rigid body effect." This can be seen from the formulations associated with a single degree of freedom system.

Let $y = x - x_o$

Where

y = displacement of mass relative to base
x_o = displacement of base
x = total displacement of mass

Dynamic equation of motion:

$$\ddot{y} + 2\zeta\omega y' + \omega^2 y = -x''_o$$

Where

$\ddot{y}$ = relative acceleration to base = $x'' - x''_o$
y' = relative velocity to base = $x' - x'_o$
ζ = Coefficient of damping
ω = Frequency in radians / second

x''_o = acceleration of base

For the higher modes in question, the system will respond as a "rigid" structure. Therefore,

$y \rightarrow 0$ since $x \simeq x_o$

$y' \rightarrow 0$ since $x' \simeq x_o'$

Noting the above, the equation of motion becomes:

$\ddot{y} = -x''_o$

It can be concluded that the mass associated with the higher modes will be excited with the base acceleration (floor or ground as appropriate). There will be no missing mass effect in the solution, and there will be no loss of accuracy since these modes respond in the rigid range of response. This statement is confirmed by performing single degree of freedom analyses with different time steps. The adequacy of the time history analyses for high frequency modes was verified by analyzing a single degree of freedom oscillator with a frequency of 100 hertz for the horizontal time history used in the north-south direction. Analyses were conducted with time steps of 0.01 seconds, 0.005 seconds, and 0.00025 seconds. Figures 230.012-1 shows the relative displacement between time 7.5 seconds and 8.5 seconds. Figure 230.012-2 shows the relative displacement between time interval 7.9 seconds to 8.1seconds. As seen from these figures, the displacement of the mass point relative to the input is essentially the same. Therefore, it can be concluded that the member forces will not be affected by the larger time step for "rigid modes." Figure 230.012-3 shows the absolute acceleration of the mass point for the three time steps. Figure 230.012-4 shows a portion of response within this time interval (7.94 seconds to 8.14 seconds). As seen from these curves the absolute acceleration is equivalent to the input acceleration with a peak of 0.3g as predicted above from the dynamic equation of motion.

The solution method that was used in ANSYS was the Newmark method that uses finite difference expansions in the time interval Δt, in which it is assumed that:

$$\{U'_{n+1}\} = \{u'_n\} + [(1-\delta)\{u''_n\} + \delta\{u''_{n+1}\}]\,\Delta t \qquad (230.012\text{-}1)$$
$$\{u_{n+1}\} = \{u_n\} + (u'_n)\,\Delta t + [(0.5-\alpha)\{u''_n\} + \alpha\{u''_{n+1}\}]\,\Delta t^2 \qquad (230.012\text{-}2)$$

Where: α, δ = Newmark integration parameters
$\Delta t = t_{n+1} - t_n$
$\{u_n\}$ = nodal displacement vector at time t_n
$\{u'_n\}$ = nodal velocity vector at time t_n
$\{u''_n\}$ = nodal acceleration vector at time t_n

In ANSYS the Newmark parameters are related to a γ factor as follows:

$$\alpha = 0.25(1+\gamma)^2 \qquad (230.012\text{-}3)$$

$$\delta = 0.5 + \gamma \qquad (230.012\text{-}4)$$

The ANSYS default value for γ, used by Westinghouse, is 0.005. Based on equations 230.012-3 and 230.012-4, the values of α and δ are calculated to be $\alpha = 0.505$, and $\delta = 0.2525$.

As stated in the ANSYS Theory Manual (Chapter 17, page 17-7), the solution of the above equations (230.012-1 & 230.012-2) is unconditionally stable when:

$$\delta = 0.5 + \gamma \qquad (230.012\text{-}5)$$
$$\alpha \geq 0.25(1+\gamma)^2 \qquad (230.012\text{-}6)$$
$$\gamma \geq 0 \qquad (230.012\text{-}7)$$

Therefore, all the Westinghouse time history solutions are stable using the ANSYS program with the default values since equations 230.012-5 to 230.012-7 are met. That is:

$$\delta = 0.505 = 0.5 + \gamma$$
$$\alpha = 0.2525 = 0.25(1+\gamma)^2$$
$$\gamma = 0.005 \geq 0$$

Thus, it can be concluded that the time history solution obtained using the time step of 0.005 seconds is adequate to avoid missing mass with no loss of accuracy when considering the higher (rigid) modes.

Note that the results presented in the DCD are from analyses that actually used a time step of 0.00025 seconds. However, as described above, a time step of 0.005 seconds would have been adequate.

Design Control Document (DCD) Revision:

None

PRA Revision:

None

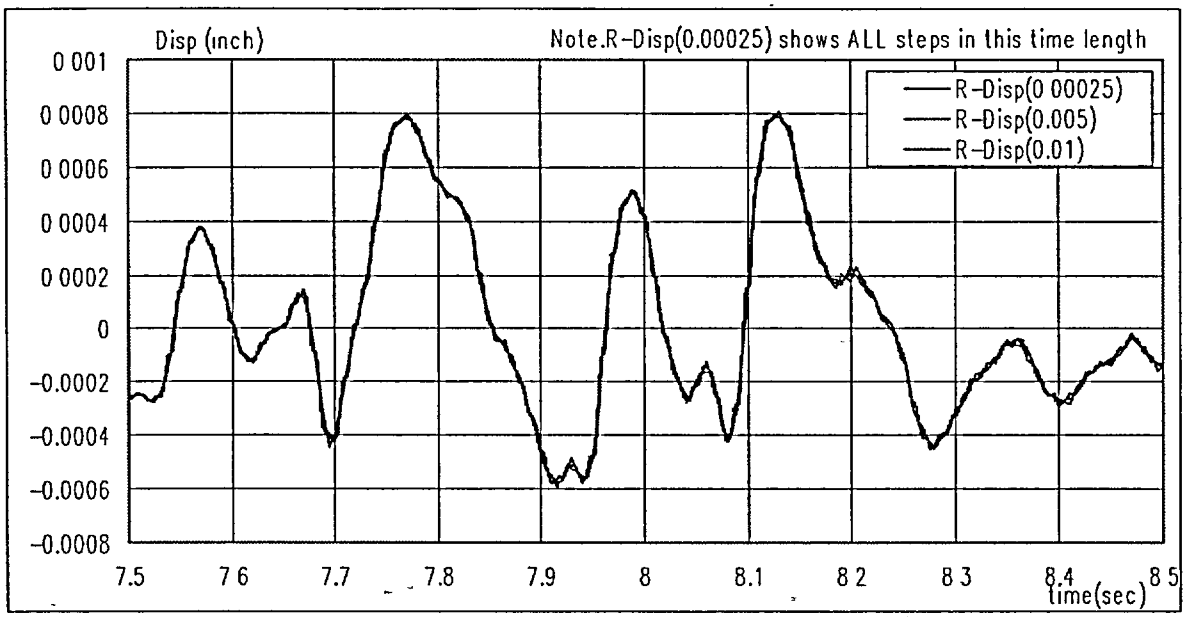


Fig 230.012-1 - Relative Displacement from 7.5 sec to 8.5 sec

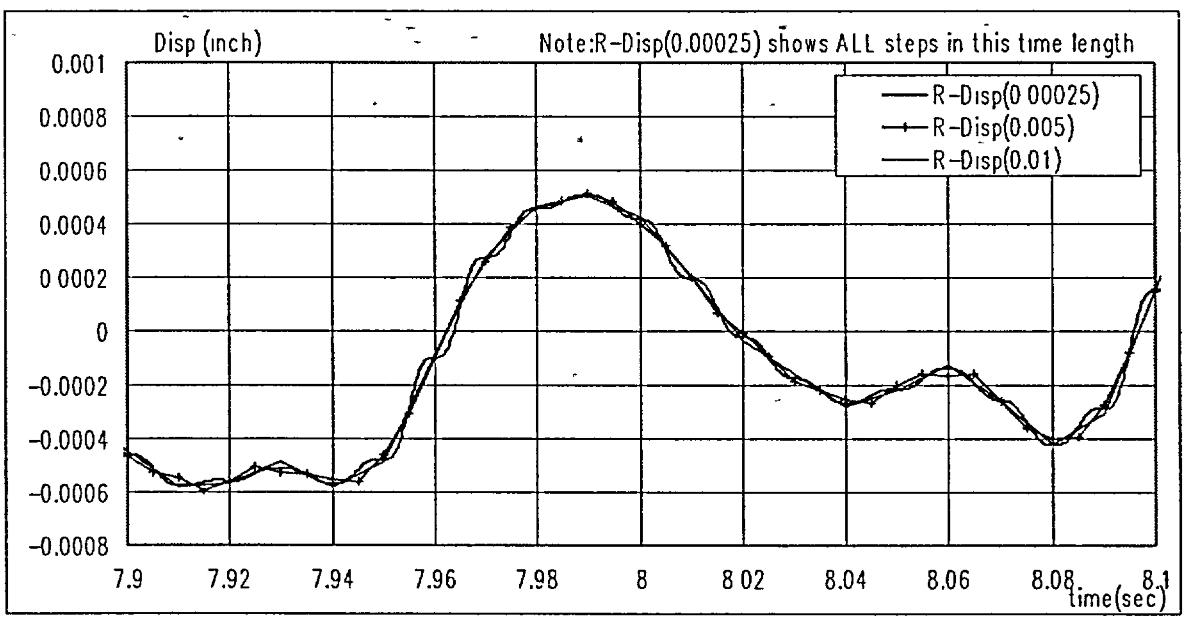
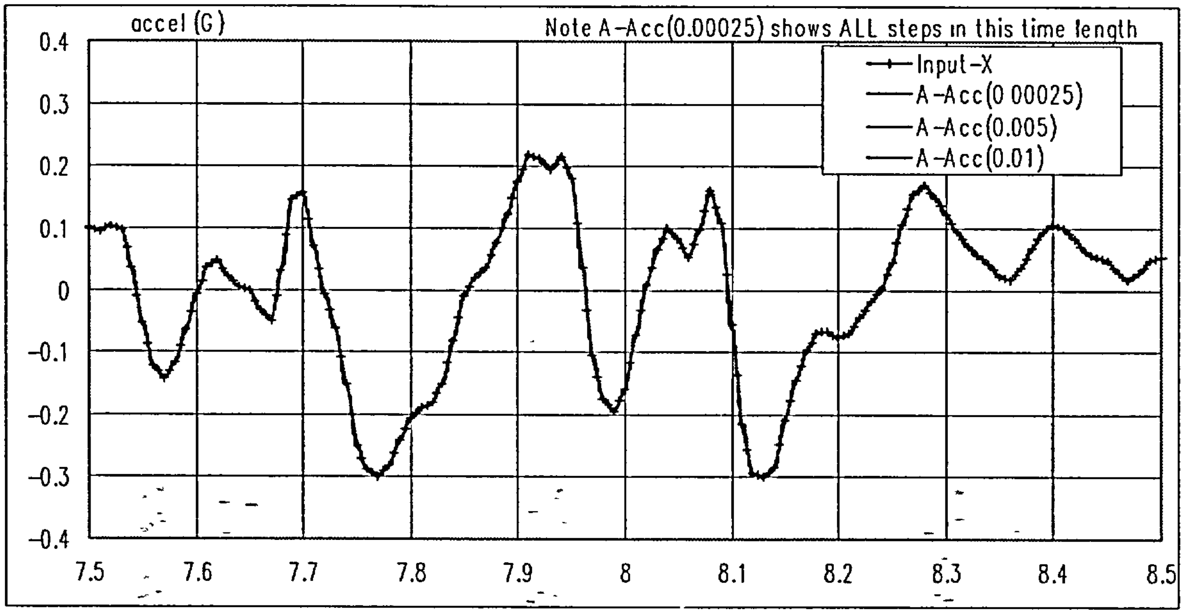


Fig 230.012-2 - Relative Displacement from 7.9 sec to 8.1 sec



- Fig 230.012-3 - Absolute Acceleration from 7.5 sec to 8.5 sec

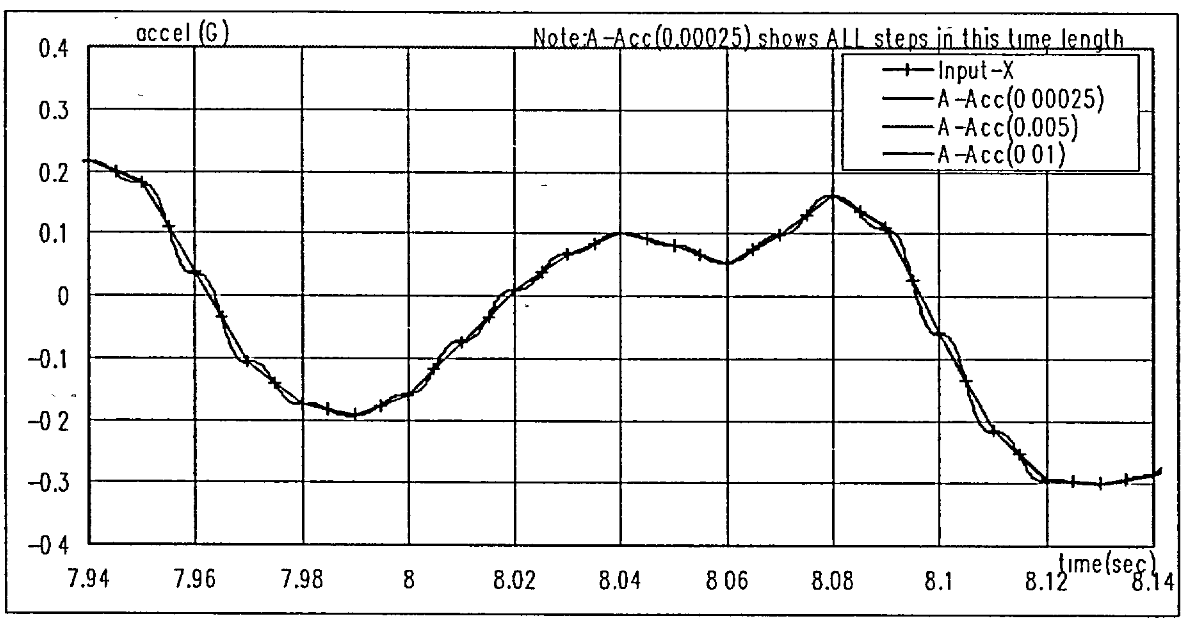


Fig 230.012-4 - Absolute Acceleration from 7.94 sec to 8.14 sec

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 230.013

Question:

Both Subsection 3.7.2.1 and Table 3.7.2-14 specify only two methods for the seismic analyses of the nuclear island. These two methods are the equivalent static acceleration analysis method and the modal superposition time history analysis method. Table 3.7.2-16, however, states that both the modal superposition time history analysis method and the response spectrum analysis method are used in the dynamic seismic analysis of the 3D stick model of the nuclear island. Westinghouse is requested to:

A. Reconcile the contradiction between Subsection 3.7.2.1/Table 3.7.2-14 and Table 3.7.2-16,

B. Clarify the purpose of applying both the time history method and response spectrum method of analysis to the 3D stick model of the nuclear island, and

C. Clarify the purpose of the comparison of responses between the time history method and response spectrum method of analysis as stated in Subsection 3.7.2.12.

Westinghouse Response:

Subsection 3.7.2-12 and Table 3.7.2-16 will be revised to delete reference to response spectrum analyses. Tables 3.7.2-17 to 19 will be deleted.

Design Control Document (DCD) Revision:

See response to RAI 230.006

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 230.014

Question:

Subsection 3.7.2.1.2 states that, in the time history analysis of the 3D stick model of the nuclear island, the base of the stick model is fixed at the top of the basemat at Elevation 66'-6" and lateral supports due to soil or rock below grade is omitted thus resulting in higher responses than analyses considering full lateral support below grade. This may be true for floor response spectra provided that the issue associated with location of the point of application of the design ground motion is clarified. However, considering full lateral support due to structural embedment may result in a shift of the floor spectrum peaks toward higher frequencies. Please verify that the 15% peak broadening of the floor response spectra resulting from the fixed-base stick model neglecting the lateral support below grade would account for the spectrum peak shifting due to the lateral support below grade. In addition, please verify the accuracy of Fig. 3.7.2-18 that shows lateral supports below grade and hence appears to contradict Subsection 3.7.2.1.2.

Westinghouse Response:

Table 230.014-1 and Figures 230.014-1 to 230.014-4 show the effect of the lateral support below grade. The analyses with lateral support are those previously submitted in WCAP-15614, "AP1000 Seismic and Structural Design Activities". The analyses without lateral support are those described in Revision 0 of the Design Control Document with the stick models fixed at the top of the basemat. The table compares responses at key locations. The figures show horizontal floor response spectra at the fuel building roof (elevation 180') and at the top of the shield building (elevation334'). Vertical response spectra are identical for the two cases.

The differences between the two cases are small. Results for the case with no lateral support are slightly larger than those for the case with lateral support below grade. The 15% peak broadening of the floor response spectra resulting from the fixed-base stick model neglecting the lateral support below grade would account for the spectrum peak shifting due to the lateral support below grade

Figure 3.7.2-18 will be revised as described in the response to RAI 230.010.

Design Control Document (DCD) Revision:

None

PRA Revision:

None

Table 230.014-1
Summary of Seismic Responses

Frequencies (Hertz)						
	Fixed base with lateral support			Fixed base - no lateral support		
	N-S	E-W	VERT	N-S	E-W	VERT
	3.87	3.57	5.94	3.82	3.52	5.94
	10.96	11.03	16.50	10.42	10.56	16.45
Maximum Absolute Nodal Acceleration, ZPA (g)						
	Fixed base with lateral support			Fixed base - no lateral support		
Elevation	N-S	E-W	VERT	N-S	E-W	VERT
Top of shield building	1.44	1.54	0.89	1.52	1.64	0.89
Shield building air inlet (el.265')	0.86	0.86	0.53	0.90	0.93	0.53
Maximum Relative Displacement (in.)						
	Fixed base with lateral support			Fixed base - no lateral support		
Elevation	N-S	E-W	VERT	N-S	E-W	VERT
Top of shield building	0.95	1.10	0.25	1.02	1.17	0.25
Shield building air inlet (el.265')	0.56	0.63	0.06	0.61	0.68	0.06
Maximum Forces ($x10^3$ Kips)						
	Fixed base with lateral support			Fixed base - no lateral support		
Elevation	Axial	N-S Shear	E-W Shear	Axial	N-S Shear	E-W Shear
Shield building air inlet (el. 265')	14.83	14.71	15.75	14.81	15.77	16.75
Aux. building - El. 100'	41.61	46.8	38.69	41.80	50.11	41.30
Maximum Moment ($x10^3$ K-ft)						
	Fixed base with lateral support			Fixed base - no lateral support		
Elevation	Torque	about N-S Axis	about E-W Axis	Torque	about N-S Axis	about E-W Axis
Shield building air inlet (el.265')	36	891	804	56	950	845
Aux. building - El. 100'	1640	5564	6048	1743	5785	6517

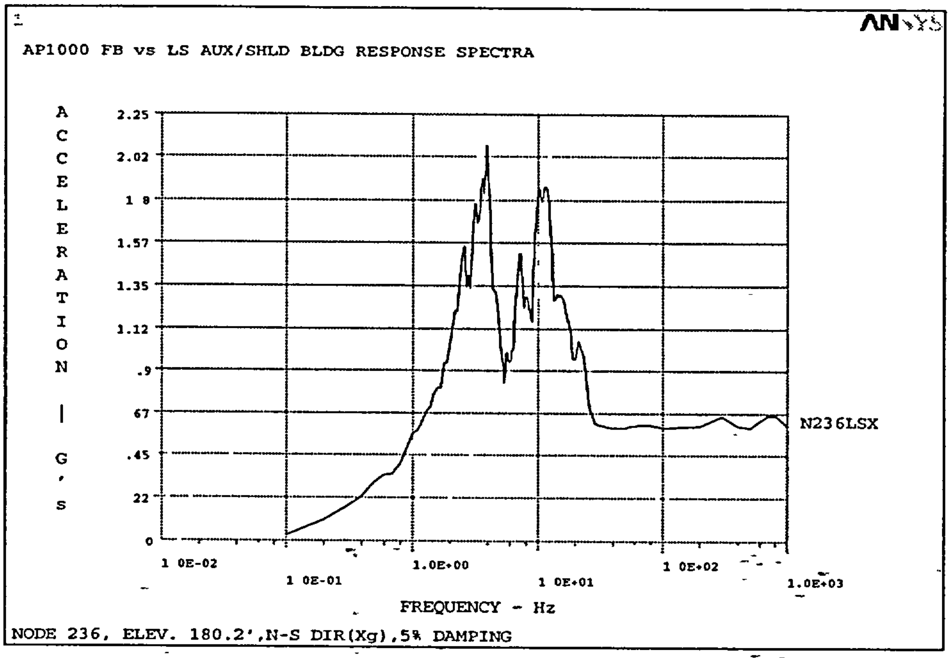


Figure 230.014-1a Response spectra with lateral support at Elevation 180 in X direction

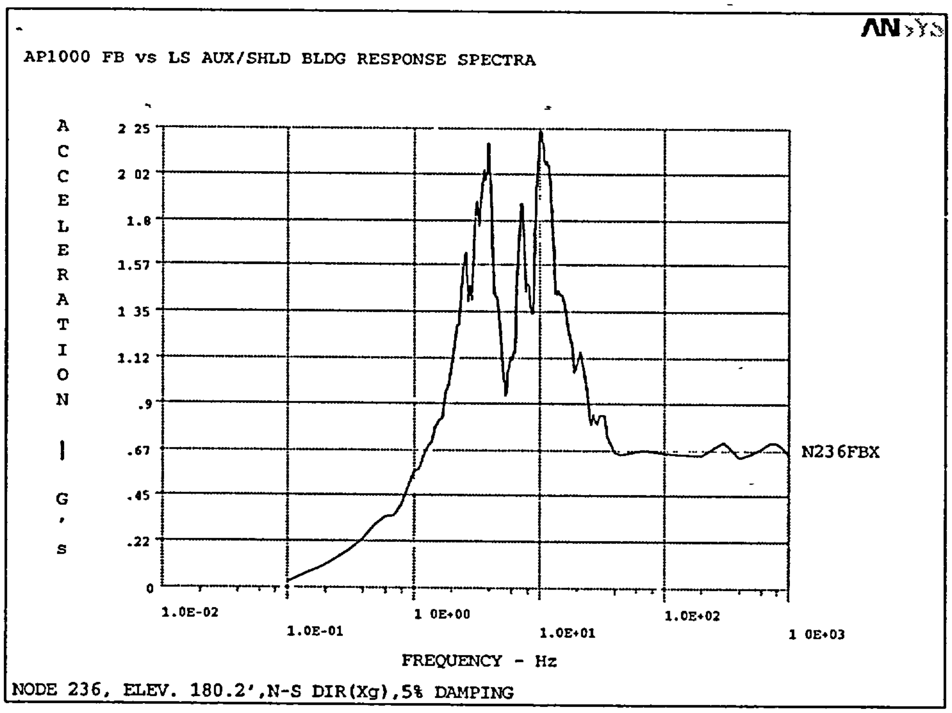


Figure 230.014-1b Response spectra without lateral support at Elevation 180 in X direction

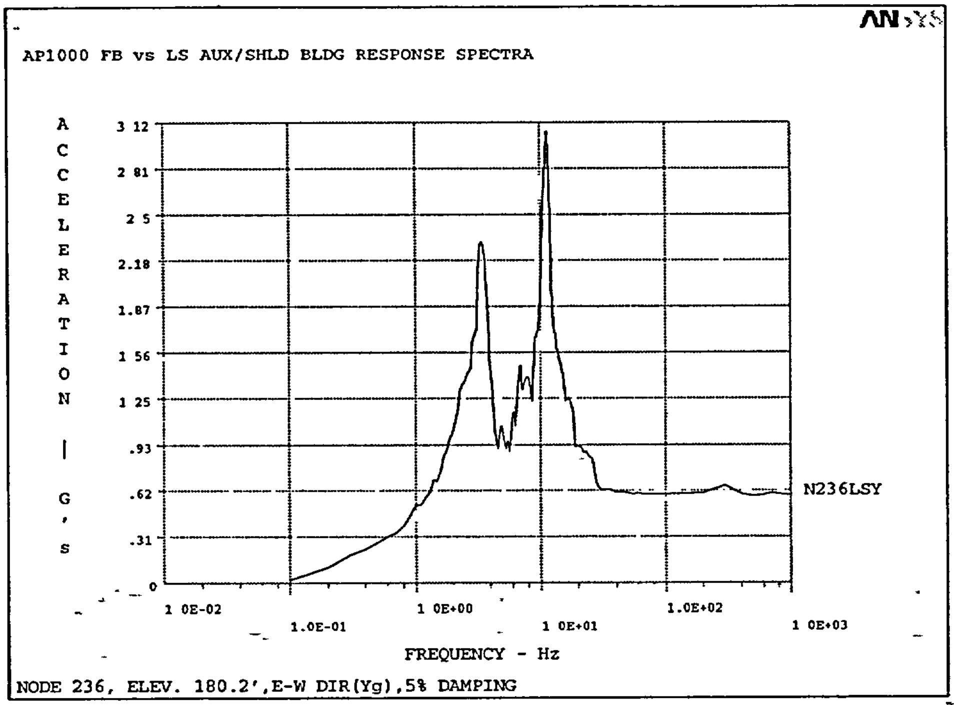


Figure 230.014-2a Response spectra with lateral support at Elevation 180 in Y direction

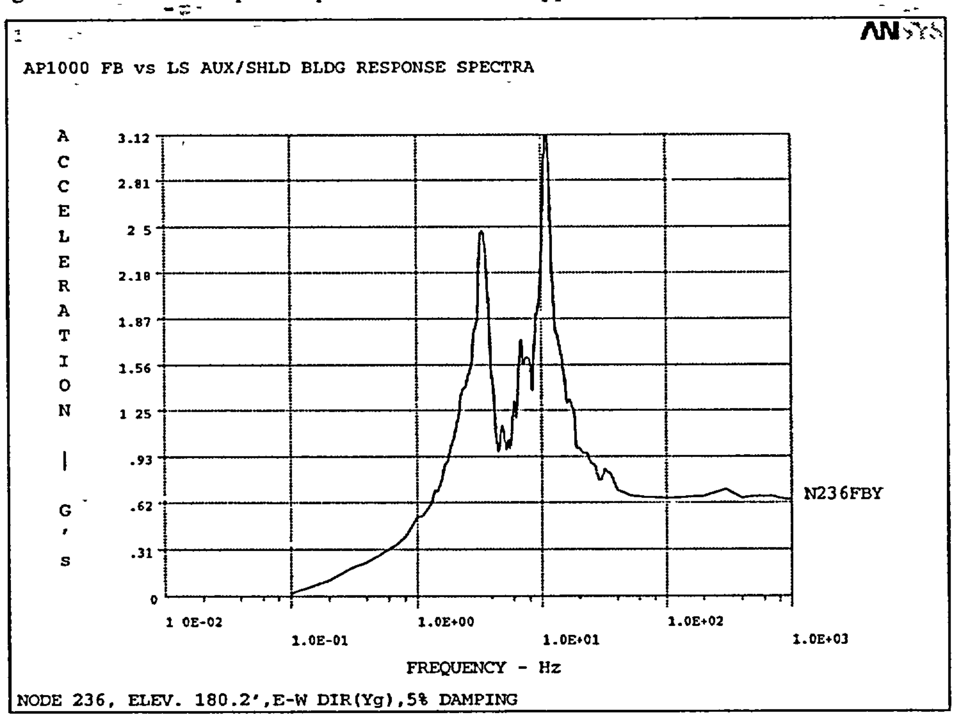


Figure 230.014-2b Response spectra without lateral support at Elevation 180 in Y direction

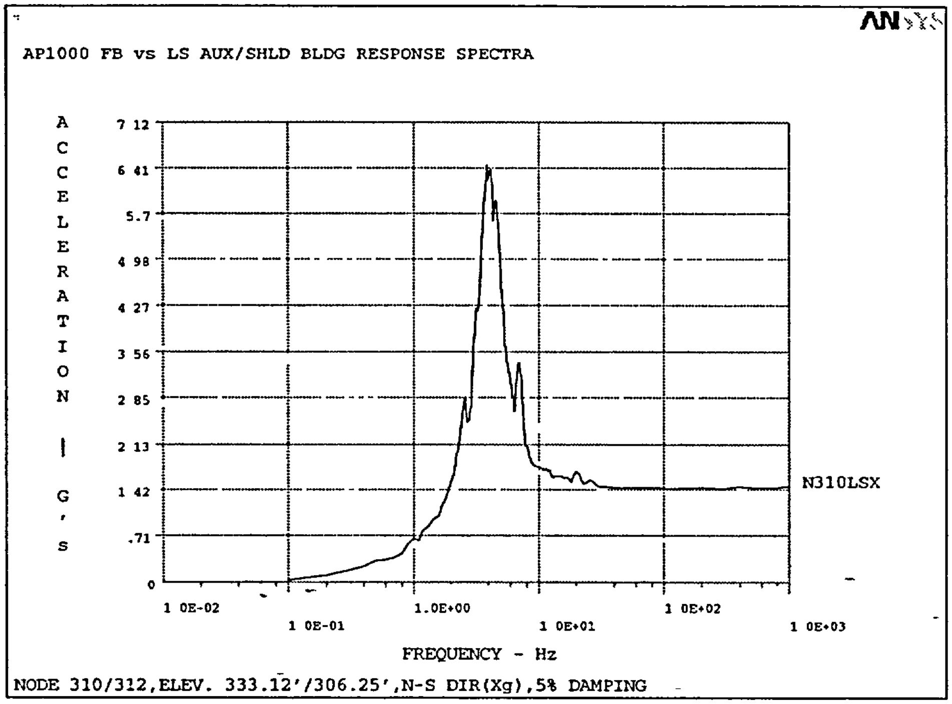


Figure 230.014-3a Response spectra with lateral support at Elevation 334 in X direction

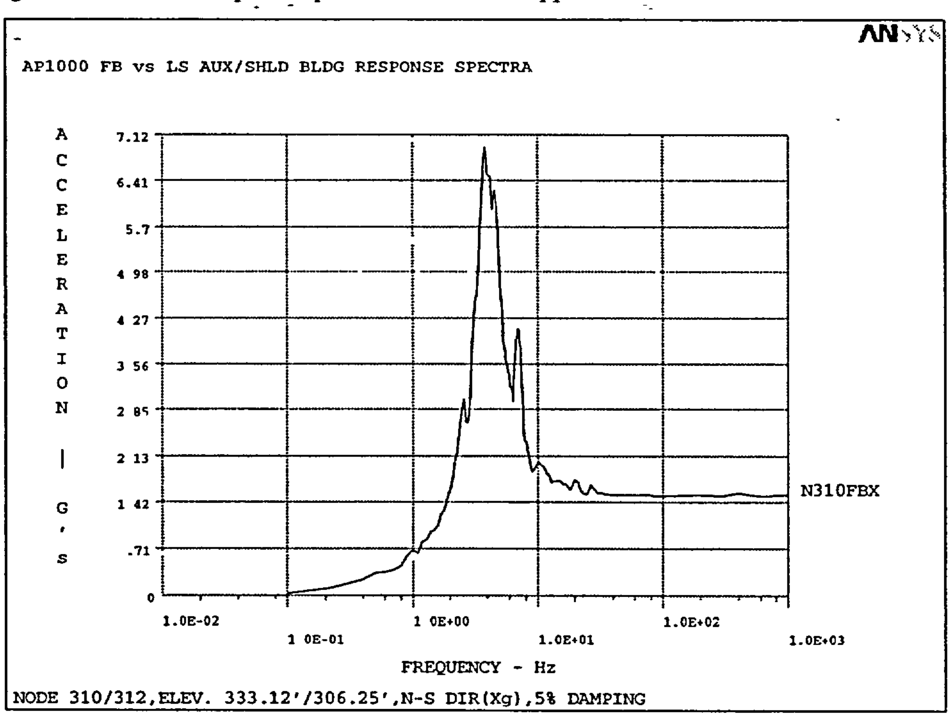


Figure 230.014-3b Response spectra without lateral support at Elevation 334 in X direction

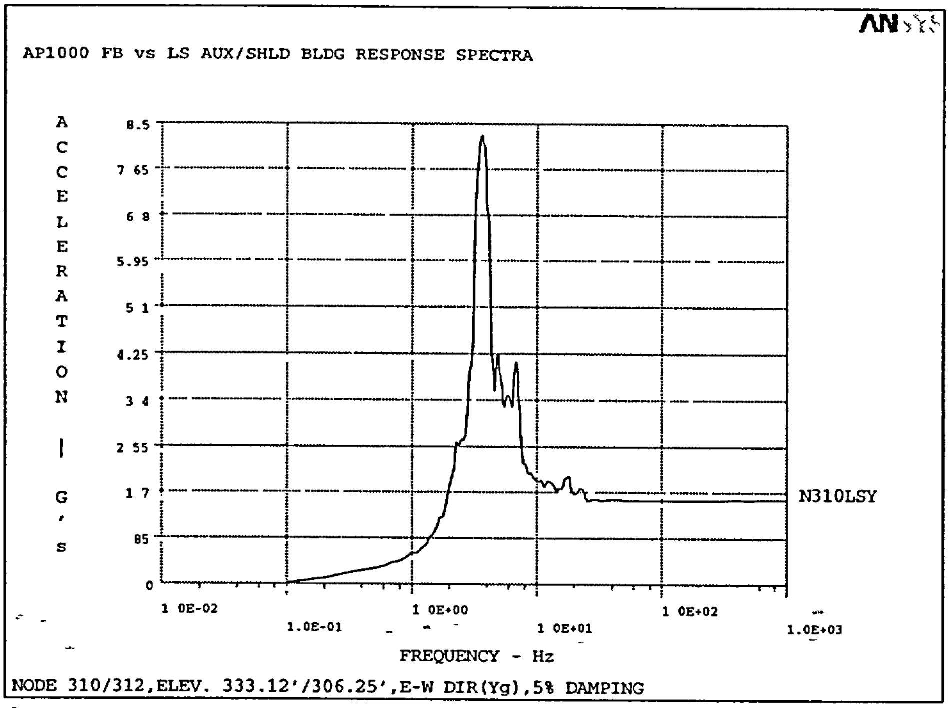


Figure 230.014-4a Response spectra with lateral support at Elevation 334 in Y direction

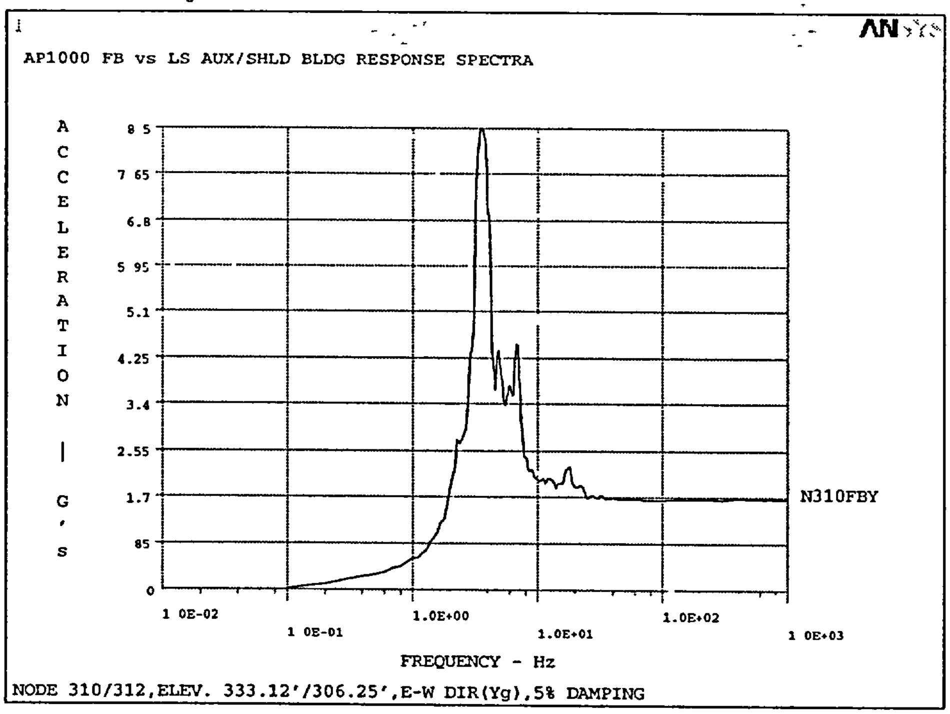


Figure 230.014-4b Response spectra without lateral support at Elevation 334 in Y direction

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 230.015

Question:

For the development of seismic model of the containment vessel, Westinghouse stated in Subsection 3.7.2.3.2 (the last paragraph of Page 3.7-11) that the polar crane is parked in the plant north-south direction with the trolley located at one end near the containment shell. This requirement should be specified as an interface item for the COL applicant.

Westinghouse Response:

The orientation of the polar crane in the parked condition is part of the plant design and is under configuration control. It establishes the location of the access platforms and ladders to the polar crane on the plant general arrangement. It is also specified in the Containment Vessel Design Specification. It is not necessary to specify the polar crane orientation in the parked condition as an interface item for the COL applicant

Design Control Document (DCD) Revision:

None

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 230.016

Question:

The second paragraph of Subsection 3.7.2.5 (Page 3.7-12) states that the floor response spectra for the design of subsystems and components are generated by enveloping the nodal response spectra determined for the hard rock site. Please explain how and where the enveloping technique is applied.

Westinghouse Response:

No enveloping is required since there is only one analysis on hard rock. The third and fourth paragraphs of the DCD will be revised as shown below.

Design Control Document (DCD) Revision:

The floor response spectra for the design of subsystems and components are generated by **broadening**~~enveloping~~ the nodal response spectra determined for the hard rock site.

The ~~enveloped floor response spectra are smoothed, and the~~ spectral peaks associated with the structural frequencies are broadened by ±15 percent to account for the variation in the structural frequencies, due to the uncertainties in parameters such as material and mass properties of the structure and soil, damping values, seismic analysis technique, and the seismic modeling technique. Figure 3.7.2-14 shows the ~~smoothing and~~ broadening procedure used to generate the design floor response spectra.

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 230.017

Question:

When the design of the AP600 coupled shield and auxiliary building complex (the building complex) was converted to the AP1000 building complex, Westinghouse increased the height of the shield building cylindrical wall and the volume of the passive containment cooling tank (i.e., the mass of water contained in the tank and the tank structural mass) by a significant percentage. The other portions of the AP1000 building complex remain the same as those of the AP600. With these design changes, the staff expects that the fundamental frequency of the AP1000 building complex should be lower than that of the AP600 building complex. However, when Table 3.7.2-1 of these two designs was compared, the staff finds that the fundamental frequency of AP1000 is 13% higher than that of AP600. Please provide additional information addressing the staff's observation.

Westinghouse Response:

The fundamental frequencies in Table 3.7.2-1 are the sloshing frequency of the water in the PCS tank. This frequency is dependent on tank internal and external radii and on depth. The Increase in tank external radius and depth results in a higher frequency for the AP1000.

The staff's expectation that the fundamental frequency of the AP1000 building complex should be lower than that of the AP600 building complex is correct for the fundamental structural horizontal modes (4.31 and 4.77 hertz for AP600 versus 3.48 and 3.76 for the AP1000). In the most recent AP1000 analyses in the proposed revision to DCD Section 3.7 transmitted by letter DCP/NRC1526, the fundamental frequencies are 3.26 and 3.50 hertz.

Design Control Document (DCD) Revision:

None

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 230.018

Question:

It is the staff's understanding that the layouts of the coupled shield and auxiliary buildings for AP1000 and AP600 are the same and only the height of the shield building and the size of the passive containment cooling water storage tank were increased. As a result of these design changes, the dominating frequency (6.065 hz) of the AP1000 in the vertical direction is lower than that of the AP600 (6.77 hz). From Figure 3.7.1-2, "Vertical Design Response Spectra - Safe Shutdown Earthquake," one can find that the vertical responses (accelerations) of the coupled shield and auxiliary buildings for the AP1000 should be higher than those of the AP600. However, the comparison of the two designs summarized in Table 3.7.2-5 and Figure 3.7.2-4 shows an opposite conclusion. The staff's review identified the following areas for clarification:

A. Westinghouse used a detailed model between Elevation 306'-3" (the top of the tank roof) and Elevation 241'-0" (the bottom of the air vent columns) for AP600, while it used a less detailed model for AP1000. Please provide an explanation for the change in models and reason for using the less detailed model for the AP1000.

B. As summarized in Table 3.7.2-5 of DCD, Revision 0, the comparison of the vertical seismic responses (maximum absolute nodal accelerations) of the two designs indicates that the dynamic amplification in the vertical direction is higher for the AP600 than for the AP1000. Based on our engineering judgement, it is the staff's expectation that the results should be reversed, because there is no change to the building wall thickness for both designs and the shield building complex of the AP1000 is more massive than that of the AP600. Westinghouse is requested to provide an explanation to address the staff's observation.

The staff's observation regarding the dynamic amplification discussed in (a) and (b) above are also applicable for the steel containment vessel.

Westinghouse Response:

A. The AP1000 shield building roof is represented in the stick model by masses at the top of the roof and at the elevation of the intersection of the exterior wall of the PCS tank with the conical roof. The AP600 model also had a mass at the mid height of the tank. The roof response is primarily influenced by the conical roof and the additional mass at mid height of the tank was not necessary. Both the AP600 and AP1000 models were developed to match the dynamic properties of a detailed axi-symmetric model of the roof.

B. The maximum vertical absolute acceleration of the roof is 0.90g for the AP600 and 0.89g for the AP1000. In the most recent AP1000 analyses in the proposed revision to the DCD Section 3.7 transmitted by letter number DCP/NRC1526, the frequency is 5.81 hertz and the maximum acceleration is 0.96g. These differences in response are partly due to changes in modal properties but are also affected by the time history which envelopes the ground input spectrum of Figure 3.7.1-2 as shown in Figure 3.7.1-8.

The maximum vertical absolute acceleration of the steel containment vessel is 1.49g for the AP600 and 1.40g for the AP1000. In the most recent AP1000 analyses in the proposed revision to DCD Section 3.7 transmitted by letter DCP/NRC1526, the maximum acceleration is 1.13g. The reduction in vertical response is associated with use of a multi-mass model of the polar crane instead of the single mass used in the AP600 analyses and the initial AP1000 analyses. The description of the polar crane model is included in the proposed revision to the DCD.

Design Control Document (DCD) Revision:

Revise fourth paragraph of subsection 3.7.2.3.2

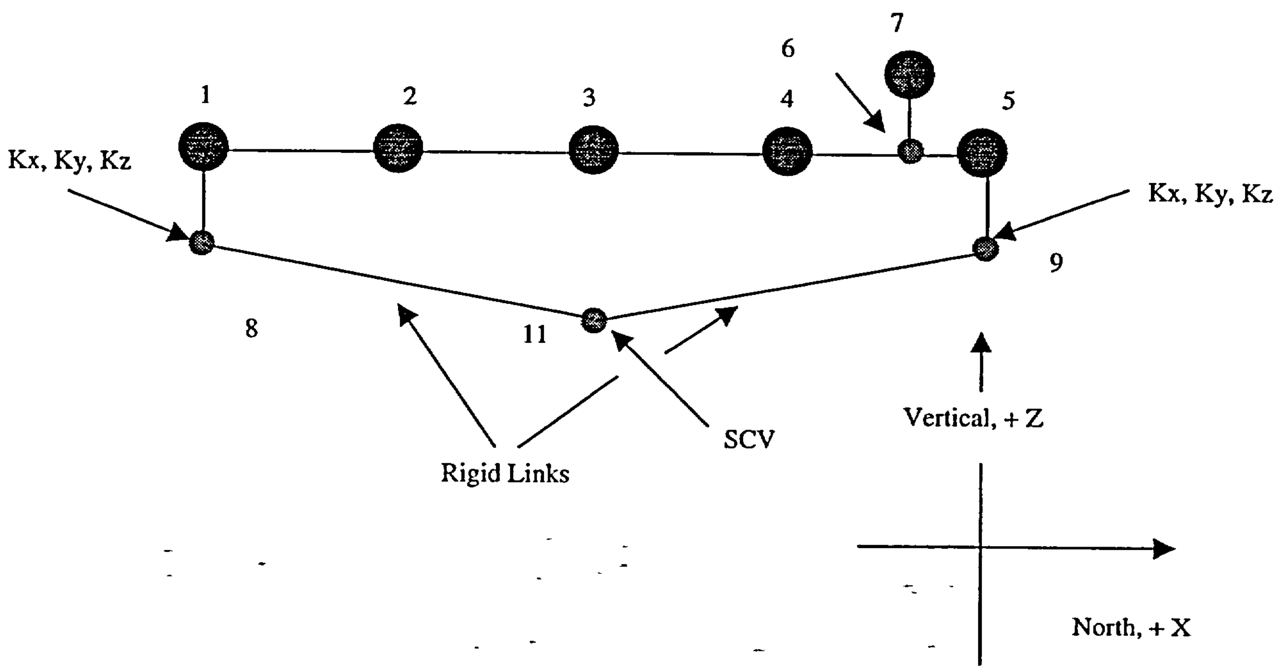
The polar crane is supported on a ring girder which is an integral part of the steel containment vessel at elevation 228'-0" **as shown in Figure 3.8.2-1**. It is modelled as a ~~single~~ **multi**-degree of freedom system attached to the steel containment shell **at elevation 224' (mid point of ring girder)** as shown in Figure 3.7.2-5. **The polar crane is modeled as shown in Figure 3.7.2-8 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.**

Revise Figure 3.7.2-5 as shown in the proposed revision to DCD Section 3.7 transmitted by letter number DCP/NRC1526.

Add Figure 3.7.2-8 as shown on page 230.018-3

PRA Revision:

None



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 3.7.2-8

Polar Crane Model

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 230.019

Question:

As shown in Table 3.7.2-1, "Modal Properties for the Coupled Shield and Auxiliary Buildings Lumped-Mass Stick Model," the dominating frequency in the vertical direction is 6.055 hz for the AP1000. This frequency is lower than that of the AP600 (6.77 hz). From Figure 3.7.1-2, "Vertical Design Response Spectra - Safe Shutdown Earthquake," one can find that the vertical response (acceleration) of the coupled shield and auxiliary buildings for the AP1000 should be higher than that of the AP600. However, as indicated in AP1000 Table 3.7.2-5 and AP600 Table 3.7.2-5 (Sheet 1 of 4), the vertical responses of the AP600 coupled shield and auxiliary buildings are consistently higher than those of the AP1000 coupled shield and auxiliary buildings. Please provide a discussion addressing these expectations.

Westinghouse Response:

See response to RAI Number 230.018.

Design Control Document (DCD) Revision:

None

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 240.001

Question:

The discussion in Section 2.4 (and Section 3.4.1) of the Tier 2 information on the effect of probably maximum precipitation is not clear. Without adequate site drainage, AP1000 design flood level, which is set at the finished grade level, will be exceeded. Clarify whether adequate site drainage is specified in the DCD.

Westinghouse Response:

Subsection 2.4.1.2 requires the Combined License applicant to address flooding including maximum precipitation. This evaluation will include consideration of site drainage. Maximum flood level must be compared against plant elevation 100' given in Table 2-1.

Design Control Document (DCD) Revision:

None

PRA Revision:

None

RAI Number: 240.002

Question:

Section 2.5.4 states that seismic analysis and foundation design for rock sites is described in Sections 3.7 and 3.8 and that the AP1000 certified design is based on the nuclear island being founded on rock. In Table 2.1 (Sheet 1 of 2), the foundation is characterized by a low strain shear wave velocity equal to 3,500 feet-per-second (ft/sec). Section 3.7.2 states that fixed base seismic analyses are performed for the nuclear island at a rock site, and Subsection 3.7.2.4 states that soil-structure interaction effect is not significant for the nuclear island founded on rock with a shear wave velocity greater than 3,500 ft/sec.

For the AP600 nuclear island, a shear wave velocity of 3,500 ft/sec is associated with soft rock sites and the effect of soil-structure interaction (SSI) was found to be not negligible. The SSI effect is negligible only for hard rock sites that are characterized by a shear wave velocity of 8,000 ft/sec. The AP1000 nuclear island is taller and more massive than the AP600 nuclear island. Therefore, the validity for performing a fixed-base analysis that neglects the effect of SSI for the AP1000 nuclear island founded on a rock site with a shear wave velocity of 3,500 ft/sec needs be substantiated. Please provide clarifying information.

Westinghouse Response:

Westinghouse is requesting design certification based on the fixed base seismic analyses. Table 5.0-1 in Tier 1 and Table 2-1 in Tier 2 will be revised to show that the shear wave velocity should exceed 8000 feet per second. Westinghouse expect that the nuclear island design using the results of the fixed-base seismic analyses will be adequate for sites with lower shear wave velocities. However, such justification is not part of the current application and may be provided as part of a Combined License application.

Design Control Document (DCD) Revision:

Revise Tier 1 Table 5.0-1 and Tier 2 Table 2-1 as follows:

Shear Wave Velocity	Greater than or equal to ~~3500~~ **8000** ft/sec based on low strain best estimate soil properties over the footprint of the nuclear island at its excavation depth

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 240.003

Question:

Subsection 2.5.4.3, "Settlement," does not address the hard rock site conditions. Please provide a discussion addressing the hard rock site conditions or your basis for this omission.

Westinghouse Response:

Settlement at a hard rock site is negligible. Westinghouse will delete the requirement for the Combined License applicant at a soil site to address settlement.

Design Control Document (DCD) Revision:

2.5.4.3 Settlement

~~The Combined License applicant will address short-term (elastic) and long-term (heave and consolidation) settlement for soil sites for the history of loads imposed on the foundation consistent with the construction sequence. The resulting time-history of settlements includes construction activities such as dewatering, excavation, bearing surface preparation, placement of the basemat and construction of the superstructure. The settlement under the nuclear island footprint is represented in the distribution of subgrade stiffness.~~

Settlement at a hard rock site is small and is not significant to the design of the AP1000. The AP1000 does not rely on structures, systems, or components located outside the nuclear island to provide safety-related functions. Differential settlement between the nuclear island foundation and the foundations of adjacent buildings does not have an adverse effect on the safety-related functions of structures, systems, and components. Differential settlement under the nuclear island foundation could cause the basemat and buildings to tilt. Much of this settlement occurs during civil construction prior to final installation of the equipment. Differential settlement of a few inches across the width of the nuclear island would not have an adverse effect on the safety-related functions of structures, systems, and components.

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 240.004

Question:

Section 3.3.3 requires the COL applicant to address the site interface criteria for wind and tornado. However, no description was provided. Westinghouse is requested to describe these interface criteria in the DCD.

Westinghouse Response:

The interface criteria for wind and tornado are provided in Table 2-1. Information to be provided by the Combined License applicant is described in subsection 2.3.

Design Control Document (DCD) Revision:

None

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 241.001

Question:

The staff's review of Table 2-1 identified the following issues:

A. Shear wave velocity of 3,500 ft/sec is defined for soil. All other references to shear wave velocity refer to rock or hard rock. The DCD does not specifically clarify the definition of assumed foundation properties for the design. Please clarify your position regarding the shear wave velocity versus restriction of the AP1000 design to rock or hard rock site.

B. The "average allowable static soil bearing capacity" of 8,400 pounds-per-square foot (psf) was specified in this table. If the DCD is applicable to hard rock sites only, Westinghouse needs to demonstrate the appropriateness of this definition. In addition, it is not clear if the definition is based on an assessment of the average strength of the hard rock or if it refers to the load associated with a given relative displacement of the foundation. Please clarify.

Westinghouse Response:

A. Westinghouse is requesting design certification based on the fixed base seismic analyses. Table 5.0-1 in Tier 1 and Table 2-1 in Tier 2 will be revised to show that the shear wave velocity should exceed 8000 feet per second. Westinghouse expect that the nuclear island design using the results of the fixed-base seismic analyses will be adequate for sites with lower shear wave velocities. However, such justification is not part of the current application and may be provided as part of a Combined License application.

B. Table 2-2 of the AP600 DCD provided typical net allowable static bearing capacities for various soils. It shows an allowable bearing capacity of 220 kips per square foot for soft rock and 450 kips per square foot for hard rock. The nuclear island analyses described in Section 3.7 show that the maximum membrane vertical compression in the walls of the nuclear island is less than 200 kips per square foot (for dead, live and seismic loads). The maximum bearing reaction on the hard rock will be smaller than the compressive stress in the walls since the reactions will be distributed through the basemat which is 22 feet thick below the shield building where the maximum wall load occurs. Thus bearing strength at a hard rock site exceeds the demand. The site interface parameter for bearing capacity will be removed.

Design Control Document (DCD) Revision:

2.5.4.2 Bearing Capacity

The maximum vertical stress in the nuclear island walls is less than 200,000 pounds per square foot under all combined loads including the safe shutdown earthquake. The maximum bearing reaction on the hard rock will be smaller than the compressive stress in the walls since the reactions will be distributed through the thickness of the basemat. Bearing capacity at a hard rock site will exceed this demand.
~~The average bearing reaction of the AP1000 is about 8,400 pounds per square foot. The minimum average allowable static soil bearing capacity is 8,400 pounds per square foot over the footprint of the nuclear island at its excavation depth (see Table 2-1).~~

~~The Combined License applicant will perform field and laboratory investigations to establish the material type and the associated strength parameters in order to determine the site-specific bearing capacity value.~~

2.5.4.6.7 ~~Bearing Capacity — The Combined License applicant will verify that the site-specific soil static bearing capacity is equal to or greater than the value documented in Table 2-1 of the DCD. The Combined License applicant will verify that the dynamic site specific bearing capacity is equal or greater than the seismic bearing demand.~~**Deleted.**

Table 2-1

~~Average allowable static soil bearing capacity~~	~~Greater than or equal to 8,400 pounds per square foot over the footprint of the nuclear island at its excavation depth~~

For other related revisions see response to RAI 240.002

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 241.002

Question:

Subsection 2.5.4.6.5 indicates that dynamic characteristics for rock, namely low strain shear velocity and material damping, need to be compared to the assumptions included in the analysis. It is not clear where either parameter has been used in the analysis and design. Please provide clarifying information. Please provide additional discussion on this issue.

Westinghouse Response:

The dynamic characteristics of the rock are not used in the fixed base seismic analyses. The DCD will be revised.

Design Control Document (DCD) Revision:

2.5.4.6.5 ~~Response of Soil and Rock to Dynamic Loading — The Combined License applicant will establish the dynamic characteristics of the soil and rock to be used in the soil structure interaction analyses and the foundation design for soil sites. For rock sites the dynamic characteristics will be compared to the assumptions made in the standard design regarding the variation of shear wave velocity and material damping.~~ **Deleted.**

PRA Revision:

None

AP1000 DESIGN CERTIFICATION REVIEW

Response to Request For Additional Information

RAI Number: 241.003

Question:

Subsection 2.5.4.6.7 refers to static soil bearing capacity and that the Combined License (COL) applicant will evaluate the site-specific dynamic bearing capacity. It is not clear where either parameter will be used in the design. Please clarify.

Westinghouse Response:

See response to RAI 241.001

Design Control Document (DCD) Revision:

None

PRA Revision:

None

DCP/NRC1526

October 4, 2002

Attachment 3

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.

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 maximum "cut-off" frequency for 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), and 5 percent damping for piping, except for the damping value of the primary coolant loop piping, which is based on Reference 22, and conduits, cable trays and their related supports.

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

For structures or components composed of different material types, the composite modal damping is calculated using the 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 rock site are described in subsection 3.7.2.

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 3.7.1-16.*]* The foundation embedment depth, foundation size, and total height of the seismic Category I structures are presented in Table 3.7.1-2.

* NRC Staff approval is required prior to implementing a change in this information, see DCD Introduction Section 3 5.

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 seismic analyses are performed for the nuclear island at a rock site. 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.

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

The seismic analyses of the nuclear island are summarized in a seismic analysis summary report. This report describes the development of the finite element models, the 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.

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

Seismic analyses, using the equivalent static acceleration method, and the mode superposition time-history 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, 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 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 models developed and discussed in subsection 3.7.2.3, 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. 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. Figure 3.7.2-2 shows the finite element model of the containment internal structures.

3.7.2.1.2 Time-History Analysis

Mode superposition time-history analyses using computer program ANSYS are performed to obtain the in-structure seismic response needed in the analysis and design of seismic subsystems.

The three-dimensional, lumped-mass stick models of the nuclear island structures developed as described in subsection 3.7.2.3 are used 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 subsection 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 base of the finite element model is fixed at the middle of the basemat at elevation 63′ 6″. 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 lumped-mass stick models of the seismic Category I structures on the nuclear island developed in subsection 3.7.2.3. Table 3.7.2-1 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 88.5 hertz. Modes below 33 hertz are 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 model of the nuclear island structures.

Three-dimensional, lumped-mass stick models are developed to represent the steel containment vessel, the containment internal structures, and the coupled shield and auxiliary buildings. Discrete mass points are provided at major floor elevations and at locations of structural discontinuities. The structural eccentricities between centers of rigidity and the centers of mass of the structures are considered. These seismic models consist of lumped masses connected to vertical elastic structural elements by horizontal stiff beam elements to simulate eccentricity. The individual building lumped-mass stick models are interconnected with other stiff beam elements to form the overall dynamic model of the nuclear island.

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

3.7.2.3.1 Coupled Shield and Auxiliary Buildings and Containment Internal Structures

The finite element models of the coupled shield and auxiliary buildings and the reinforced concrete portions of the containment internal structures are based on the gross concrete section with the modulus based on the specified compressive strength of concrete of contributing structural walls and slabs. The properties of the concrete-filled structural modules are computed using the combined gross concrete section and the transformed steel face plates of the structural modules. Furthermore, the weight density of concrete plus the uniformly distributed miscellaneous dead weights are considered by 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, the finite element models of the coupled shield and auxiliary buildings and the containment internal structures. Each of these models start at the underside of the basemat at elevation 60′ 6″. The interface between the models is at a radius of 69′ 6″ at the inside face 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 presents the steel containment vessel stick model. In the stick model, the properties are calculated as follows:

- Members representing the cylindrical portion are based on the properties of the actual circular cross section of the containment vessel.

- Members representing the bottom head are based on equivalent stiffnesses calculated from the shell of revolution analyses for static 1.0g in vertical and horizontal directions.

- Shear, bending and torsional properties for members representing the top head are based on the average of the properties at the successive nodes, using the actual circular cross section. These are the properties that affect the horizontal modes. Axial properties, which affect the vertical modes, are based on equivalent stiffnesses calculated from the shell of revolution analyses for static 1.0g in the vertical direction.

This method used to construct a stick model from the axisymmetric shell model of the containment vessel is verified by comparison of the natural frequencies determined from the stick model and the shell of revolution model as shown in Table 3.7.2-15. The shell of revolution vertical model (n = 0 harmonic) has a series of local shell modes of the top head above elevation 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′ (mid point of ring girder) as shown in Figure 3.7.2-5. The polar crane is modeled as shown in Figure 3.7.2-8 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 3.7.2-5.

3.7.2.3.3 Nuclear Island Seismic Model

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. 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.

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. 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 nodal response spectra determined for the hard rock site.

The 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.

The safe shutdown earthquake floor response spectra for 5 percent damping, at representative locations of the coupled auxiliary and shield buildings, the steel containment vessel, and the containment internal structures are presented in Figures 3.7.2-15 through 3.7.2-17.

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.
- 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, the effect of three components of earthquake are considered as follows:

- Mode Superposition Time History Analysis (program ANSYS) – the time history responses from the three components of earthquake motion are combined algebraically at each time step.

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

3.7.2.7 Combination of Modal Responses

The modal responses of the response spectrum system structural analysis are combined using the 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.

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

3.7.2.10 Use of Constant Vertical Static Factors

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

3.7.2.11 Method Used to Account for Torsional Effects

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

- Horizontal mass properties of the building stick models shown in Figures 3.7.2-4, 3.7.2-5, and 3.7.2-6,

- The 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.
- 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.

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

or

$$R = \text{Fact1}(|A_{NS}| + |T_{NS}|) + \text{Fact2}(|A_{EW}| + |T_{EW}|) + \text{Fact3}(|A_{VT}|)$$

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]

3.7.2.12 Deleted

3.7.2.13 Methods for Seismic Analysis of Dams

Seismic analysis of dams is site specific design.

3.7.2.14 Determination of Seismic Category I Structure Overturning Moments

Subsection 3.8.5.5.4 describes the effects of seismic overturning moments.

3.7.2.15 Analysis Procedure for Damping

Subsection 3.7.1.3 presents the damping values used in the seismic analyses. For structures comprised of different material types, the composite modal damping approach utilizing the strain energy method is used to determine the composite modal damping values. Subsection 3.7.2.4 presents the damping values used in the soil-structure interaction analysis.

3.7.3 Seismic Subsystem Analysis

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

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

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

3.7.3.1 Seismic Analysis Methods

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

3.7.3.2 Determination of Number of Earthquake Cycles

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

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

3.7.3.3 Procedure Used for Modeling

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

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

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

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

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

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

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

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

3.7.3.4 Basis for Selection of Frequencies

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

3.7.3.5 Equivalent Static Load Method of Analysis

The equivalent static load method involves equivalent horizontal and vertical static forces applied at the center of gravity of various masses. The equivalent force at a mass location is computed as the product of the mass and the seismic acceleration value applicable to that mass location. Loads, stresses, or deflections, obtained using the equivalent static load method, are adjusted to account for the relative motion between points of support when significant.

3.7.3.5.1 Single Mode Dominant or Rigid Structures or Components

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

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

- A rigid component (fundamental frequency ≥ 33 hertz), whose support can be represented by a flexible spring, can be modelled as a single degree of freedom model in the direction of excitation (horizontal or vertical directions). The equivalent static seismic load for the direction of excitation is defined as the product of the component mass and the seismic acceleration value at the natural frequency from the applicable floor response spectra. If the frequency is not determined, the peak acceleration from the applicable floor response spectrum is used.

- If the component has a distributed mass whose dynamic response will be single mode dominant, the equivalent static seismic load for the direction of excitation is defined as the product of the component mass and the seismic acceleration value at the component natural frequency from the applicable floor response spectra times a factor of 1.5. A factor of less than 1.5 may be used if justified. Static factors smaller than 1.5 are not used for piping systems. A factor of 1.0 is used for structures or equipment that can be represented as uniformly loaded cantilever, simply supported, fixed-simply supported, or fixed-fixed beams (References 10 and 11). If the frequency is not determined, the peak acceleration from the applicable floor response spectrum is used.

3.7.3.5.2 Multiple Mode Dominant Response

This procedure applies to piping, instrumentation tubing, cable trays, and HVAC that are multiple span models. The equivalent static load method of analysis can be used for design of piping systems, instrumentation and supports that have significant responses at several vibrational frequencies. In this case, a static load factor of 1.5 is applied to the peak accelerations of the applicable floor response spectra. For runs with axial supports which are rigid in the axial direction (fundamental frequency greater than or equal to 33 hertz), the acceleration value of the mass of piping in its axial direction may be limited to 1.0 times its calculated spectral acceleration value. The spectral acceleration value is based on the frequency of the piping system along the axial direction. The relative motion between support points is also considered.

3.7.3.6 Three Components of Earthquake Motion

Two horizontal components and one vertical component of seismic response spectra are employed as input to a modal response spectrum analysis. The spectra are associated with the safe shutdown earthquake. In the response spectrum and equivalent static analyses, the effects of the three components of earthquake motion are combined using one of the following methods:

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

- The peak responses due to the three earthquake components are combined directly, using the assumption that when the peak response from one component occurs, the responses from the other two components are 40 percent of the peak (100 percent-40 percent-40 percent method). Combinations of seismic responses from the three earthquake components, together with variations in sign (plus or minus), are considered. This method is not used for piping systems.

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

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

The time-history safe shutdown earthquake analysis of a subsystem can be performed by simultaneously applying the displacements and rotations at the interface point(s) between the subsystem and the system. These displacements and rotations are the results obtained from a model of a larger subsystem or a system that includes a simplified representation of the subsystem. The time-history safe shutdown earthquake analysis of the system is performed by applying three mutually orthogonal and statistically independent, artificial time histories. Possible examples of the use of this method of seismic analysis include the following:

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

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

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

3.7.3.7 Combination of Modal Responses

For the seismic response spectra analyses, the zero period acceleration cut-off frequency is 33 hertz. High frequency or rigid modes are considered using the left-out-force method or the missing mass method described in subsection 3.7.3.7.1. The method to combine the low frequency modes is described in subsection 3.7.3.7.2. The rigid mode results in the three perpendicular directions of the seismic input are combined by the SRSS method. The resultant response of the rigid modes is combined by SRSS with the flexible mode results. The combination of modal responses in time history analyses of piping systems is described in subsection 3.7.3.17. Modal responses in time history analyses of other subsystems are combined as described in subsection 3.7.2.6.

3.7.3.7.1 Combination of High-Frequency Modes

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

3.7.3.7.1.1 Left-Out-Force Method or Missing Mass Correction for High Frequency Modes

The left-out-force method is based on the Left-Out-Force Theorem. This theorem states that for every time history load there is a frequency, f_r, called the "rigid mode cutoff frequency" above which the response in modes with natural frequencies above f_r will very closely resemble the applied load at each instant of time. These modes are called "rigid modes." The left-out-force method is used in program PIPESTRESS.

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

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

where:

f (t) = the applied load vector
M = the mass matrix
e_j = the eigenvector

Note that Σ is only for all the flexible modes, not including the rigid modes.

In the response spectra analysis, the total inertia force contribution of higher modes can be interpreted as:

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

where:

Am = the maximum spectral acceleration beyond the flexible modes
[M] = the mass matrix
{r} = the influence vector or displacement vector due to unit displacement
P_j = participation factor

Since,

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

In PIPESTRESS, the low frequency modes are combined by one of the Regulatory Guide 1.92 methods in the response spectrum analysis. For each support level, there is a pseudo-load vector or left-out-force vector in the X, Y and Z directions. These left-out-force vectors are used to generate left-out-force solutions which are multiplied by a scalar amplitude equal to a magnification factor specified by the user. This factor is usually the ZPA (zero period acceleration) of the response spectrum for the corresponding direction. The resultant low frequency responses are combined by square root of the sum of the squares with the high frequency responses (rigid modes results).

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

3.7.3.7.1.2 SRP 3.7.2 Method

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

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

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

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

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

where:

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

C_n is the participation factor given by:

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

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

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

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

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

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

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

where:

P_I = force or moment to be applied by degree of freedom i
M_i = mass or mass moment of inertia associated with degree of freedom i.

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

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

3.7.3.7.2 Combination of Low-Frequency Modes

This subsection describes the method for combining modal responses in the seismic response spectra analysis. The total unidirectional seismic response for subsystems is obtained by combining the individual modal responses using the square root sum of the squares method. For subsystems having modes with closely spaced frequencies, this method is modified to include the possible effect of these modes. For piping systems, the methods in Regulatory Guide 1.92 are used for modal combinations. For other subsystems, the methods in Regulatory Guide 1.92 or the following alternative methods may be used. The groups of closely spaced modes are chosen so that the differences between the frequencies of the first mode and the last mode in the group do not exceed 10 percent of the lower frequency.

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

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

where:

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

$$\varepsilon_{k\ell} = (1 + \frac{(w_k' - w_\ell')^2}{(\beta_k' w_k + \beta_\ell' w_\ell)^2})^{-1}$$

and,

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

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

where:

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

Alternatively, a more conservative grouping method can be used in the seismic response spectra analyses. The groups of closely spaced modes are chosen so that the difference between two frequencies is no greater than 10 percent. Therefore,

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

where:

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

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

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

3.7.3.8 Analytical Procedure for Piping

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

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

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

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

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

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

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

3.7.3.8.1 Supporting Systems

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

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

Stiffness Effect

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

Mass Effect

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

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

Required Coupled Branch Piping

If the stiffness and/or mass effects are considered significant, the branch piping is included in the piping analysis for the run pipe analysis. The portion of branch piping considered in the analysis adequately represents the behavior of the run pipe and branch pipe. The branch line model ends in one of the following ways:

- First six-way anchor
- Four rigid/seismic supports in each of the three perpendicular directions
- Rigidly supported zone as described in subsection 3.7.3.13.4.2

3.7.3.8.2 Supported Systems

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

3.7.3.8.2.1 Large Diameter Auxiliary Piping

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

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

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

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

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

3.7.3.8.2.2 Small-Diameter Auxiliary Piping

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

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

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

3.7.3.8.3 Piping Systems on Modules

Many portions of the systems for the AP1000 are assembled as modules offsite and shipped to the plant as completed units. This method of construction does not result in any unique requirements for the analysis of these structures, systems, or components. Existing industry standards and regulatory requirements and guidelines are appropriate for the evaluation of structures, systems, and components included in modules.

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

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

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

- Rm = mass of supported component or piping/mass of supporting structural frame
- Rf = frequency of the component or piping/frequency of the structural frame

Decoupling may be done when:

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

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

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

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

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

3.7.3.8.4 Piping Systems with Gapped Supports

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

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

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

The basis of the concept is to find an equivalent linear spring with a response-dependent stiffness for each nonlinear rigid gapped support, or limit stop, in the mathematical model of the piping system. The equivalent linearized stiffness minimizes the mean difference in force in the support between the equivalent spring and the corresponding original gapped support. The mean difference is estimated by an averaging process in the time domain, that is, across the response duration, using the concept of random vibration. Details of the design and analysis methods and modeling assumptions are described in Reference 12.

3.7.3.9 Combination of Support Responses

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

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

The effect of relative seismic anchor displacements is obtained either by using the worst combination of the peak displacements or by proper representation of the relative phasing characteristics associated with different support inputs. For components supported by a single concrete building (coupled shield and auxiliary buildings, or containment internal structures), the seismic motions at all elevations above the basemat are taken to be in phase. When the component supports are in the same structure, the relative seismic anchor motions are small and the effects are neglected. This is applicable to building structures and to those supplemental steel frames that are rigid in comparison to the components. Supplemental steel frames that are flexible can have significant seismic anchor motions which are considered. When the components supports are in different structures, the relative seismic anchor motion between the structures is taken to be out-of-phase and the effects are considered. The results of the modal spectra analysis (multiple input or envelope) are combined with the results from seismic anchor motion by the absolute sum method.

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

Seismic Response Spectra Peak Shifting

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

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

If no equipment or piping system natural frequencies exist in the ±15 percent interval associated with the maximum spectrum acceleration peak, then the interval associated with the next highest spectrum acceleration peak is selected and used in the following procedure.

Consider all N natural frequencies in the interval

$$f_j - 0.15f_j \leq (f_e)_n \leq f_j + 0.15f_j$$

where:

f_j = the frequency of maximum acceleration in the envelope spectra
n = 1 to N

The system is then evaluated by performing N + 3 separate analyses using the envelope unbroadened floor design response spectrum and the envelope unbroadened spectrum modified by shifting the frequencies associated with each of the spectral values by a factor of +0.15; -0.15; and

$$\frac{(f_e)_n - f_j}{f_j}$$

where:

n = 1 to N

The results of these separate seismic analyses are then enveloped to obtain the final result desired (e.g., stress, support loads, acceleration, etc.) at any given point in the system. If three different floor response spectrum curves are used to define the response in the two horizontal and the vertical directions, then the shifting of the spectral values as defined above may be applied independently to these three response spectrum curves.

Independent Support Response Spectrum Methods

The use of multiple-input response spectra accounts for the phasing and interdependence characteristics of the various support points. The following alternative methods are used for the AP1000 plant. These are based on the guidelines provided by the "Pressure Vessel Research Committee Technical Committee on Piping Systems" (Reference 14).

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

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

where:

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

Enveloped response spectra are developed as the seismic input in three perpendicular directions of the piping coordinate system to include the spectra at the floor elevations of the attachment points and the piping module or equipment if applicable. The mode shapes and frequencies below the cut-off frequency are calculated in the response spectrum analysis. The modal participation factors in each direction of the earthquake motion and the spectral accelerations for each significant mode are calculated. Based on the calculated mode shapes, participation factors, and spectral accelerations of individual modes, the modal inertia response forces, moments, displacements, and accelerations are calculated. For a given direction, these modal inertia responses are combined based on consideration of closely spaced modes and high frequency modes to obtain the resultant forces, moments, displacements, accelerations, and support loads. The total seismic responses are combined by square-root-sum-of-the-squares method for all three earthquake directions.

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

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

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

A support group is defined by supports that have the same time-history input. This usually means all supports located on the same floor (or portions of a floor) of a structure.

3.7.3.10 Vertical Static Factors

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

3.7.3.11 Torsional Effects of Eccentric Masses

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

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

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

3.7.3.12 Seismic Category I Buried Piping Systems and Tunnels

There are no seismic Category I buried piping systems and tunnels in the AP1000 design.

3.7.3.13 Interaction of Other Systems with Seismic Category I Systems

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

- Separation – separation with the use of physical barriers

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

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

- Support as seismic Category II

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

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

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

1. Define targets susceptible to damage (sensitive targets);
   Sensitive targets are those seismic Category I components for which adverse spatial interaction can result in loss of safety function.

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

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

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

5. Correct adverse seismic interaction conditions.

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

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

3.7.3.13.1 Separation and Segregation

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

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

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

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

3.7.3.13.2 Impact Analysis

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

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

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

- Robustness of target

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

- Energy of impact

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

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

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

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

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

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

- Tanks, "heavy" equipment (for example, heat exchangers)
- Mechanical or electrical penetrations
- Heating, ventilation, and air conditioning (HVAC)
- Adjacent piping
- Conduits

- Cable trays
- Structures

3.7.3.13.3 Seismic Category II Supports

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

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

Nonseismic equipment identified as a source within the impact evaluation zone is supported as seismic Category II. Support seismic loads include seismic inertia loads of the equipment determined as described in subsection 3.7.3.5 and nozzle loads from attached piping determined as described in subsection 3.7.3.13.4.2. Adequate free space is assessed considering a 6-inch deflection envelope for equipment identified as a source, or calculated deflections obtained using the safe shutdown earthquake load combination and elastic analysis.

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

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

3.7.3.13.4.1 Seismic Category I Piping

The safe shutdown earthquake piping displacements obtained for the seismic Category I piping are used for the evaluation of seismic interaction with sensitive equipment. Adequate free space between a source and a target is checked adding absolutely the piping safe shutdown earthquake deflection and the safe shutdown earthquake target deflection along with the other loads (e.g., dead weight, thermal) that are in the appropriate design criteria load combinations. Sensitive equipment for piping as the source is seismic Category I equipment shown in Table 3.7.3-2 along with the portion that must be protected ("zone of protection"). Supports may be added to limit seismic movement to eliminate potential adverse interaction.

3.7.3.13.4.2 Seismic Category II Piping

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

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

Single Interface Anchor

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

Anchor Followed by a Series of Seismic Supports

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

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

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

RF = M_L/M_a

M_a = Resultant moment at elbow/bend. Use maximum value if several elbows/bends are within seismically supported region.

$M_L = 0.8h^{0.6} D^2 t\ Sy$ for $h < 1.45$

$M_L = D^2 t\ Sy$ for $h > 1.45$

h = Flexibility characteristic of elbow/bend

D = Outside diameter of elbow/bend

t = Thickness of elbow/bend

R = Bend radius of elbow/bend

Rigid Region

The seismic Category II piping may be terminated at the last seismic support of a rigidly supported region of the piping system. The rigid region is typically defined as either four bi-lateral supports around an elbow or six bilateral supports around a tee. The structural behavior of the rigid region is similar to that of a six-way anchor. The frequency of the piping system in the rigid region is greater than or equal to 33 hertz. This last seismic support in the rigid region and the supports on the seismic Category II piping are evaluated for safe shutdown earthquake loadings using the rules of ASME III Subsection NF.

3.7.3.13.4.3 Nonseismic Piping

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

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

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

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

- The piping segment identified as the source has at least one effective axial support.
- Adequate free space between a source and a target is checked adding absolutely the piping safe shutdown earthquake deflections (defined following seismic Category II piping analysis methodology) and the safe shutdown earthquake target deflection. Also included are the displacements associated with the appropriate load cases.
- When the anchor is an equipment nozzle, the equipment is supported as seismic Category II as described in subsection 3.7.3.13.3.

3.7.3.14 Seismic Analyses for Reactor Internals

See subsection 3.9.2 for the dynamic analyses of reactor internals.

3.7.3.15 Analysis Procedure for Damping

Damping values used in the seismic analyses of subsystems are presented in subsection 3.7.1.3. Safe shutdown earthquake damping values used for different types of analysis are provided in Table 3.7.1-1. For subsystems that are composed of different material types, the composite modal damping approach with the weighted stiffness method is used to determine the composite modal damping value. Alternately, the minimum damping value may be used for these systems. Composite modal damping for coupled building and piping systems is used for piping systems that are coupled to the primary coolant loop system and the interior concrete building. Composite modal damping is used for piping systems that are coupled to flexible equipment or flexible valves. Piping systems analyzed by the uniform envelope response spectra method with rigid valves can be evaluated with 5 percent damping. Five percent damping is not used in piping systems that are susceptible to stress corrosion cracking.

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

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

3.7.3.16 Analysis of Seismic Category I Tanks

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

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

3.7.3.17 Time History Analysis of Piping Systems

The time history dynamic analysis is an alternate seismic analysis method for response spectrum analysis when time history seismic input is used. This method is also used for dynamic analyses of piping systems subjected to time history hydraulic transient loadings or forcing functions induced by postulated pipe breaks. Direct integration or the modal superposition method is used to solve the equations of motion. The computer programs used are GAPPIPE, PIPESTRESS, ANSYS, and WECAN. WECAN is not used for linear time history analyses or response spectra analyses of piping systems.

The modal superposition method is based on the equations of motion which can be decoupled as long as the piping system is within its elastic limit. The modal responses are obtained from integrating the decoupled equations. The total responses are obtained by the algebraic sum of the individual responses of the individual modes at each time step. The cutoff frequency is selected based on the frequency content of the input forcing function and the highest significant frequency of the piping system. The integration time step is no larger than 10 percent of the period of the cutoff frequency.

For dynamic analysis, including seismic analysis at a hard rock site, three separate analyses are performed for each loading case to account for uncertainties. The three analyses correspond to three different time scales: normal time, time expanded by 15 percent, and time compressed by 15 percent. Alternatively, when the results are shown to be acceptable based on comparison with test data, one time history analysis is performed using normal time. For time history analysis of piping system models that include a dynamic model of the supporting concrete building either the building stiffness is varied by + or - 30 percent, or the time scale is shifted by + or - 15 percent. Alternately, when uniform enveloping time history analysis is performed, modeling uncertainties are accounted for by the spreading that is included in the broadened response spectra.

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

Alternately, for time history analysis using the PIPESTRESS, GAPPIPE, ANSYS, or WECAN programs, the number of modes used in the modal analysis is chosen so that the results of the dynamic analysis based on the chosen number of modes are within 10 percent of the results of the dynamic analysis based on the next higher number of modes used. The number of modes analyzed is selected to account for the principal vibration modes of the piping system. The modes are combined by algebraic sum. Composite modal damping is used with the ANSYS or WECAN programs. The damping of the individual components is as listed in Table 3.7.1-1.

3.7.4 Seismic Instrumentation

3.7.4.1 Comparison with Regulatory Guide 1.12

Compliance with Regulatory Guide 1.12 and draft Regulatory Guide DG-1016 is discussed in this section and in subsection 1.9.1.

3.7.4.1.1 Safety Design Basis

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

3.7.4.1.2 Power Generation Design Basis

The seismic instrumentation is designed to provide the following:

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

3.7.4.2 Location and Description of Instrumentation

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

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

This solid-state recording and analysis system has internal batteries and a charger to prevent the loss of data during a power outage, and to allow data collection and analysis in a seismic event during which the power fails. Normally 120 volt alternating current power is supplied from the non-Class 1E dc and uninterruptible power supply system. The system uses triaxial acceleration sensor input signals to initiate the time-history analyzer recording and main control room alarms. The system initiation value is adjustable from 0.002g to 0.02g.

The time-history analyzer starts recording triaxial acceleration data from each of the triaxial acceleration sensors after the initiation value has been exceeded. Pre-event recording time is adjustable from 1.2 to 15.0 seconds, and will be set to record at least 3 seconds of pre-event signal. Post-event run time is adjustable from 10 to 90 seconds. A minimum of 25 minutes of continuous recording is provided. Each recording channel has an associated timing mark record with 2 marks per second, with an accuracy of about 0.02 percent.

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

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

3.7.4.2.1 Triaxial Acceleration Sensors

Each sensor unit contains three accelerometers mounted in a mutually orthogonal array mounted with one horizontal axis parallel to the major axis assumed in the seismic analysis. The triaxial acceleration sensors have a dynamic range of 1000 to 1 (0.001 to 1.0g) and a frequency range of 0.2 to 50 hertz.

One sensor unit will be located in the free field. Because this location is site-specific, the planned location will be determined by the Combined License applicant. The AP1000 seismic monitoring system will provide for signal input from the free field sensor.

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

A third sensor unit is located on the shield building structure at elevation 266′ near column lines 4-1 and K.

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

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

3.7.4.2.2 Time-History Analyzer

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

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

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

The time-history analyzer can compute cumulative absolute velocity (CAV) and the 5 percent of critical damping response spectrum for frequencies between 1 and 10 Hz. The operator may select the analysis of either CAV or the response spectrum. Analysis results are printed out on a dedicated graphics printer that is part of the system and is located in the same panel as the time-history analyzer.

3.7.4.3 Control Room Operator Notification

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

3.7.4.4 Comparison of Measured and Predicted Responses

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

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

3.7.4.5 Tests and Inspections

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

3.7.5 Combined License Information

3.7.5.1 Seismic Analysis of Dams

Combined License applicants referencing the 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.

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3.7.6 References

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Table 3.7.1-1

SAFE SHUTDOWN EARTHQUAKE DAMPING VALUES

	Percent
Welded aluminum structures	4
Welded and friction-bolted steel structures and equipment	4
Bearing bolted structures and equipment	7
Prestressed concrete structures	5
Reinforced concrete structures	7
Concrete filled steel plate structures	5
Piping (for uniform envelope response spectra analysis)	5
Piping (alternative for time history analysis and independent support motion response spectra analysis)	
Less than or equal to 12-inch diameter	2
Greater than 12-inch diameter	3
Primary coolant loop	4
Fuel assemblies	20
Control rod drive mechanisms	5
Full cable trays and related supports	10[1]
Empty cable trays and related supports	7
Conduits and related supports	7
HVAC ductwork	7
HVAC welded ductwork	4
Cabinets and panels for electrical equipment	5
Equipment such as welded instrument racks and tanks	3

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

Table 3.7.1-2

EMBEDMENT DEPTH AND RELATED DIMENSIONS OF CATEGORY I STRUCTURES

Structure	Foundation Embedment Depth (ft)	Least Foundation Width (ft)	Structure Height (ft)
Shield Building	See Note	See Note	273.25
Steel Containment Vessel	See Note	See Note	215.33
Auxiliary Building	See Note	See Note	119.50

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

* NRC Staff approval is required prior to implementing a change in this information; see DCD Introduction Section 3 5

Table 3.7.1-3

**AP1000 DESIGN RESPONSE SPECTRA
AMPLIFICATION FACTORS FOR CONTROL POINTS**

HORIZONTAL

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

VERTICAL

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

Note:

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

Table 3.7.2-1

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

		Effective Mass		
Mode	Frequency	X Direction	Y Direction	Z Direction
1	0.136	1.761	1.182	0.000
2	0.136	1.391	1.531	0.000
3	0.136	116.363	6.398	0 000
4	0.136	6.360	116.911	0.000
5	3.026	0.016	1774.770	11.212
6	3.317	1586.760	0.011	6.317
7	5.783	3.587	5.983	912.825
8	6.602	261.230	48.164	0.009
9	6.792	3.757	944.846	0.127
10	7.066	538.350	0.614	0.220
11	8.870	4.536	834.204	0.766
12	9.490	1066.390	1.333	1.435
13	12.343	133.858	7.669	325.571
14	13.976	9.536	45.488	73.456
15	14.620	26.746	92.708	385.481
16	15.530	33.225	68.190	594.891
17	16.440	85.342	0.355	63.498
18	18.802	1.309	8.509	242.579
19	20.274	79.210	58.069	20.546
20	20.805	99.243	69.774	53.702
Sum of Effective Masses		4058.970	4086.710	2692.640

Note:
1. Fixed at elevation 60.5′.
2. The first four modes are principally water sloshing in the passive containment system tank.

Table 3.7.2-2

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

Mode	Frequency	Effective Mass X Direction	Effective Mass Y Direction	Effective Mass 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

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

Table 3.7.2-3 (Sheet 1 of 2)

CONTAINMENT INTERNAL STRUCTURES LUMPED-MASS STICK MODEL MODAL PROPERTIES

		Effective Mass		
Mode	Frequency	X Direction	Y Direction	Z Direction
1	10.803	3.030	654.204	1.240
2	11.903	494.857	0.179	0.113
3	12.812	72.800	83.341	0.442
4	16.581	79.974	192.555	0.016
5	17.129	57.072	71.529	0.075
6	18.742	192.755	0.779	0 055
7	23.101	0.827	766.212	19.606
8	25.155	0.033	124.256	38.462
9	26.089	0.192	76.777	0.599
10	27.438	16.393	53 887	12.908
11	28.538	1070.300	0 030	0.589
12	30.900	1.903	0.294	8.625
13	32.733	1.260	2.006	23.440
14	33.215	0.009	7.807	2.483
15	35.625	52.624	0.144	25.008
16	39.109	14.825	58.622	763.856
17	39.948	5.796	87.639	391.125
18	40.879	0.008	14.856	23.059
19	44.784	0.011	0.816	11.729
20	45.220	0 046	1.794	29.567
Sum of Effective Masses		2064.710	2197.730	1353.000

Note:
1. Fixed at Elevation 60.50′.
2. The total mass of the containment internal structure is 3242.1 kip-sec^2/ft.

Table 3.7.2-3 (Sheet 2 of 2)

RCL LUMPED-MASS STICK MODEL MODAL PROPERTIES

Mode	Frequency	Effective Mass X Direction	Effective Mass Y Direction	Effective Mass 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

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

Table 3.7.2-4 (Sheet 1 of 3)

NUCLEAR ISLAND COMBINED LUMPED-MASS STICK MODEL MODAL PROPERTIES

Mode	Frequency	Effective Mass X Direction	Effective Mass Y Direction	Effective Mass Z Direction
1	0.136	0.22	2.88	0.00
2	0.136	3.22	0.18	0.00
3	0.136	121.05	1.33	0.00
4	0.136	1.32	121.52	0.00
5	3.263	1.94	1659.47	5.91
6	3.501	1480.63	1.56	6.49
7	3.619	0.02	3.06	0.03
8	4.111	52.49	0.03	0.01
9	4.159	7.08	0.14	0.00
10	5.044	0.02	90.52	0.12
11	5.321	178.23	0.00	0.14
12	5.808	2.52	0.11	884.02
13	6.068	0.03	251.53	8.23
14	6.409	4.81	0.16	21.92
15	7.074	269.31	8.15	0.04
16	7.218	4.11	644.60	0.21
17	7.348	281.60	10.62	0.08
18	7.714	0.57	115.90	0.50
19	8.077	32.87	79.26	0.02
20	8 461	0.08	0.02	0.42
21	8.612	81.35	57.07	0.03
22	9.210	0.03	10.29	4.57
23	9.241	0.00	4.99	130.94
24	9.412	0.91	6 43	0.00
25	9.637	39.76	0.01	0.73
26	9.734	0.00	4.46	4.25
27	9.827	0.00	17.24	0.00
28	10.110	0.46	1544.32	0.77
29	10.346	0.07	9.63	0.51
30	10.753	1590.44	0.06	0.04
31	10.800	0.24	0.01	0.00
32	10.908	19.99	0.01	0.00
33	11.620	0.00	8.39	0.38
34	11.896	2.98	9.22	0.94
35	11.916	13.05	1.53	0.34
36	12.218	69.19	1.12	4.30
37	12.582	1.49	2.03	1.04
38	12.953	57.18	57.78	26.96
39	13.260	196.22	2.40	497.27
40	13.429	0.08	1.31	27.52

Table 3.7.2-4 (Sheet 2 of 3)

NUCLEAR ISLAND COMBINED LUMPED-MASS STICK MODEL MODAL PROPERTIES

		Effective Mass		
Mode	Frequency	X Direction	Y Direction	Z Direction
41	13 457	0.13	0 58	2.24
42	13.481	0 48	0.29	0.22
43	14.697	12.30	44.44	69.54
44	15.314	31.56	174.29	45.86
45	15.540	47.13	155.91	273.87
46	15 687	21.51	12.79	45.09
47	15.756	0.72	2.95	1.71
48	15.979	23.25	196.17	459.98
49	16.088	30.24	1.58	0.25
50	16.397	50.37	14.61	78.28
51	16.859	22.08	7.13	61.31
52	17.228	227.03	0 00	31.25
53	17.410	2.72	1.56	1.52
54	17.562	32.84	0.09	7.10
55	18.758	0.75	63.74	0.06
56	19.059	86.38	0.21	0.76
57	19.213	0.24	13.60	219.05
58	20.975	36.95	0 01	0.15
59	21.672	33.56	331.30	7.98
60	21.728	3.06	75.77	3.08
61	22.091	3.29	1.35	0.02
62	22.148	11.22	22.27	0.36
63	22.462	255.81	25.02	5.00
64	22.785	101.53	49.10	18.50
65	23.192	8.37	27.46	7.44
66	24.108	5.59	156.68	9.83
67	24.995	3.78	7.69	106.76
68	26.161	3.91	0.66	10.05
69	26.602	150.62	9.94	216.32
70	27.115	27.67	29.05	128.87
71	27.311	63.23	199.65	9.40
72	28.059	4.29	0.10	135.57
73	28.254	2.59	1.85	4.09
74	28.690	199.65	0.03	34.44
75	29.131	1.95	19.66	7.79
76	29.433	11.71	15.45	7.68
77	29.589	0.08	1.42	26.06
78	29.620	3.32	3.99	171.16
79	29.868	0.08	6.92	10.40
80	30.099	32.87	74.60	193.96

Table 3.7.2-4 (Sheet 3 of 3)

NUCLEAR ISLAND COMBINED LUMPED-MASS STICK MODEL MODAL PROPERTIES

		Effective Mass		
Mode	**Frequency**	**X Direction**	**Y Direction**	**Z Direction**
81	30.350	11.63	0.71	0.10
82	30.443	45.36	15.78	49.99
83	30.887	64.65	12.30	167.55
84	31.000	0.17	0.61	14.95
85	31.313	25.49	32.58	3.89
86	31.404	131.56	5.83	31.75
87	31.954	71.16	10.48	2.11
SUMMATIONS		6420.4	6557.5	4312.1
TOTAL MASS		8724.9	8724.9	8724.9

Note:
1. Fixed at Elevation 60.5′.

Table 3.7.2-5

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

HARD ROCK SITE CONDITION

Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g)					
	N-S Direction		E-W Direction		Vertical Direction	
	Mass Center	Edge	Mass Center	Edge	Mass Center	Edge
333.13	1.36	1.42	1.77	1.80	0.96	1.52
295.23	1.07	1.07	1.24	1.31	0.95	1.49
265.00	0.90	0.92	0.90	1.00	0.58	0.78
242.50	0.81	0.84	0.82	0.88	0.56	0.73
220.00	0.74	0.77	0.75	0.81	0.52	0.66
200.00	0 68	0.70	0.70	0.75	0.48	0.60
179.56	0.60	0.62	0.71	0.76	0.43	0.57
164.51	0.55	0.59	0.69	0.74	0.39	0.54
153.98	0 53	0.60	0.67	0.71	0.39	0.52
134.87	0.50	0.55	0.59	0.66	0.39	0.65
116.50	0.45	0.46	0.47	0.54	0.34	0.57
99.00	0.37	0.38	0.41	0.45	0.33	0.45
81.50	0.32	0.33	0.33	0.35	0.31	0.32
66.50	0.30	0.30	0.30	0.30	0.30	0.30

Note:

1. The results at the edges are on the auxiliary building at and below the elevation 134.87′ and on the shield building above this elevation. This is the maximum value of the response at any of these edge nodes.

Table 3.7.2-6

MAXIMUM ABSOLUTE NODAL ACCELERATION (ZPA)
STEEL CONTAINMENT VESSEL

HARD ROCK SITE CONDITION

Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g)					
	N-S Direction		E-W Direction		Vertical Direction	
	Mass Center	Edge	Mass Center	Edge	Mass Center	Edge
281.90	1.27		1.42		1.13	
273.83	1.22		1.38		0.85	
265.83	1.17		1.34		0.71	
255.02	1.10		1.28		0.62	
244.21	1.03	1.07	1.22	1.22	0.58	0.62
233.50 P.C.	1.52		2.07		1.29	
224.00	0.90	0.95	1.09	1.10	0.56	0.60
200.00	0.76	0.81	0.93	0.93	0.52	0.58
169.93	0.63	0.67	0.70	0.71	0.46	0.53
162.00	0.59	0.62	0.64	0.65	0.45	0.52
141.50	0.47	0.50	0.53	0.53	0.40	0.47
131.68	0.41	0.44	0.49	0.50	0.38	0.44
112.50	0.37	0.38	0.41	0.42	0.34	0.41
104.12	0.37	0.38	0.39	0.41	0.34	0.40
100.00	0.36	0.38	0.40	0.42	0.33	0.37

Note:
1. Enveloped response results at the north, south, east, and west edge nodes of the structure are shown. This is the maximum value of the response at any of these edge nodes.
2. Results at elevation 233.50′ are mid span of polar crane bridge.

Table 3.7.2-7

MAXIMUM ABSOLUTE NODAL ACCELERATION (ZPA) CONTAINMENT INTERNAL STRUCTURES

HARD ROCK SITE CONDITION

Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g)					
	N-S Direction		E-W Direction		Vertical Direction	
	Mass Center	Edge	Mass Center	Edge	Mass Center	Edge
169.00 (PRZ Compartment)	1.33		1.44		0.43	
155.00 (SG-West Compartment)	0.73		0.65		0.39	
155.00 (SG-East Compartment)	1.15		0.59		0.40	
135.25	0.56	0.63	0.51	0.60	0.32	0.49
107.17	0.38	0.39	0.41	0.42	0.31	0.34
103.00	0.37	0.39	0.40	0.41	0.31	0.34
98.00	0.36	0.38	0.39	0.40	0.30	0.34
82.50	0.32	0.33	0.33	0.34	0.30	0.31
66.50	0.30	0.30	0.30	0.30	0.30	0.30

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

Table 3.7.2-8

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

HARD ROCK SITE CONDITION

Elevation (ft)	Maximum Relative Displacement (in.)					
	N-S Direction		E-W Direction		Vertical Direction	
	Mass Center	Edge	Mass Center	Edge	Mass Center	Edge
333.13	1.18	1.19	1.37	1.39	0.28	0.53
295.23	0.90	0.91	0.98	1.00	0.28	0.52
265.00	0.70	0.72	0.76	0.79	0.08	0.34
242.50	0.59	0.61	0.62	0 66	0.08	0.33
220.00	0.48	0.49	0.49	0.52	0.07	0.30
200.00	0.37	0.39	0.39	0.41	0.06	0.28
179.56	0.27	0.29	0.29	0.30	0.06	0.25
164.51	0.21	0.22	0.22	0.23	0.06	0.26
153.98	0.18	0.18	0.18	0.19	0.06	0.21
134.87	0.12	0.12	0.13	0.14	0.06	0.22
116.50	0.07	0.08	0.08	0.09	0.01	0.16
99.00	0.03	0.03	0.03	0.04	0.01	0.09
81.50	0 01	0.01	0.01	0.01	0.00	0.03
66.50	0 00	0.00	0.00	0.00	0.00	0.00

Note:

1. The results at the edges are on the auxiliary building at and below the elevation 134.87′ and on the shield building above this elevation. This is the maximum value of the response at any of these edge nodes.

Table 3.7.2-9

MAXIMUM DISPLACEMENT RELATIVE TO BOTTOM OF BASEMAT STEEL CONTAINMENT VESSEL

HARD ROCK SITE CONDITION

Elevation (ft)	Maximum Relative Displacement (in.)					
	N-S Direction		E-W Direction		Vertical Direction	
	Mass Center	Edge	Mass Center	Edge	Mass Center	Edge
281.90	0.46		0.53		0.05	
273.83	0.45		0.51		0.05	
265.83	0.43		0.49		0.04	
255.02	0.41		0.47		0.04	
244.21	0.38	0.39	0.44	0 44	0.04	0.17
233.50 P.C.	0.55		1.45		0.33	
224.00	0.34	0.34	0.39	0.39	0.04	0.17
-200.00	0.27	0.27	0.31	0.32	0.03	0.16
169.93	0.18	0.19	0.22	0.22	0.03	0.15
162.00	0.16	0.17	0.20	0.20	0.03	0.14
141.50	0.11	0.12	0.14	0.14	0.03	0.13
131.68	0.09	0.09	0.11	0.11	0.03	0.12
112.50	0 05	0.05	0.06	0.06	0.03	0.10
104.12	0.03	0.04	0.04	0.04	0.03	0.09
100.00	0 03	0.03	0.03	0 04	0.02	0.07

Note:
1. Enveloped relative displacements at the north, south, east and west edge nodes of the structure are shown. This is the maximum value of the relative displacement at any of these edge nodes.
2. Results at elevation 233.50′ are mid span of polar crane bridge.

Table 3.7.2-10

MAXIMUM DISPLACEMENT RELATIVE TO BOTTOM OF BASEMAT CONTAINMENT INTERNAL STRUCTURES

HARD ROCK SITE CONDITION

Elevation (ft)	Maximum Relative Displacement (in.)					
	N-S Direction		E-W Direction		Vertical Direction	
	Mass Center	Edge	Mass Center	Edge	Mass Center	Edge
169.00 (PRZ Compartment)	0.12		0.16		0.02	
155.00 (SG-West Compartment	0.07		0.10		0.02	
155.00 (SG-East Compartment)	0.10		0.09		0.01	
134.25	0.06	0.06	0.06	0.08	0.01	0.03
107.17	0.03	0.03	0.04	0.04	0.00	0.02
103.00	0.03	0.03	0.03	0.04	0.00	0.02
98.00	0.03	0.03	0.03	0.03	0.00	0.02
82.50	0.01	0.01	0.01	0.01	0.00	0.01
66.50	0.00	0.00	0.00	0.00	0.00	0.00

Note:

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

Table 3.7.2-11

MAXIMUM MEMBER FORCES AND MOMENTS COUPLED AUXILIARY & SHIELD BUILDINGS

HARD ROCK SITE CONDITION

Elevation (ft)	Maximum Forces ($x10^3$ Kips)				Maximum Moment ($x10^3$ K-ft)	
	Axial	N-S Shear	E-W Shear	Torque	About N-S Axis	About E-W Axis
333.13						
	2.78	6.01	7.74	7.47		
295.23					330.16	239.31
	15.86	15.47	16.64	38.57		
265.00					1063.69	826.93
	19.07	21.62	22.47	144.94		
242.50					1565.58	1312.17
	21.18	24.85	25.23	210.93		
220.00					2185.60	1911.83
	23.07	27.47	27.09	265.65		
200.00					2771.73	2497.83
	24.72	29.60	28.27	309.85		
179.56					3388.05	3126.97
	26.04	22.16	22.33	779.66		
164.51					3753.83	3473.23
	28.14	22.18	18.79	848.70		
153.98					3991.97	3717.34
	31.75	17.06	14.33	413.23		
134.88					4271.73	4212.15
	37.55	45.18	36.44	1087.18		
116.50					5423.90	6289.15
	45.75	52.71	45.27	1201.33		
99.00					5969.36	7281.73
	56.99	21.69	21.64	1486.92		
81.50					6858.13	7911.27
	61.70	13.70	12.99	645.18		
66.50					2203.61	3705.18
	84.13	47.30	52.58	1467.04		
60.50					3188.77	9520.76

Note:

1. The forces in the shear beam between elevation 60.5′ and 99′ are those in the auxiliary and shield building stick. There is a parallel shear beam for the base mat of the containment internal structures.

Table 3.7.2-12

MAXIMUM MEMBER FORCES AND MOMENTS
STEEL CONTAINMENT VESSEL

HARD ROCK SITE CONDITION

Elevation	Maximum Forces ($x10^3$ Kips)			Maximum Moment ($x10^3$ K-ft)		
(ft)	Axial	N-S Shear	E-W Shear	Torque	About N-S Axis	About E-W Axis
281.90						
	0.21	0.23	0 26	0.00		
273.83					2.07	1.87
	0.49	0.63	0.70	0 85		
265.83					9.04	8.24
	0.73	1.03	1.14	2.36		
255.02					23.78	21.66
	0.96	1.43	1.60	4.58		
244.21					44.63	40.60
	1.25	1.96	2.21	7.90		
224.00					94.41	85.45
	2.30	4.45	3.97	24.57		
200.00					206.01	225.75
	2.76	5.10	4.69	28.35		
169.93					355.66	387.42
	3.18	5.55	5.31	31.69		
162.00					405.23	438.31
	3.43	5.80	5.66	34.44		
141.50					524.21	561.19
	3.65	5.96	5.89	37.27		
138.58					545.77	581.18
	3.65	5.96	5.89	37.27		
131.68					585.67	622.32
	3.88	6.09	6 09	39.61		
112.50					705.34	741.66
	4.04	6.14	6.18	42.35		
110.50					720.45	755.19
	4.04	6.14	6.18	42.35		
104.12					759.84	794.35
	4.10	6.16	6.20	43.06		
100.00					786.28	820.04

Table 3.7.2-13

MAXIMUM MEMBER FORCES AND MOMENTS CONTAINMENT INTERNAL STRUCTURES

HARD ROCK SITE CONDITION

Elevation (ft)	Maximum Forces ($x10^3$ Kips) Axial	N-S Shear	E-W Shear	Maximum Moment ($x10^3$ K-ft) Torque	About N-S Axis	About E-W Axis
Above Elevation 134.25', West SG Compartment						
169 00						
	0.01	0.46	0.52	3.92		
163.79					3.57	3.05
	0.14	0.64	0.69	4.90		
153.00					11.03	9.96
	0.51	1.43	2.10	14.73		
134.25					53.72	35.01
Above Elevation 134.25', East SG Compartment						
155.00						
	0.19	0.61	2.15	0.66		
134.25					46.95	12.58
Below Elevation 134.25'						
	0.00	7.33	7.11	106.43		
121.50					172.45	142.95
	3.32	7.33	7.11	106.43		
107.17					258 27	242.74
	6.73	11.94	10.84	288.48		
103.00					378.30	320.18
	10.49	49.22	41.09	406.69		
98.00					366.01	389.45
	17.09	53.24	45.91	380.03		
82.50					1039.66	1189.70
	83.62	71.62	65.38	1107.35		
66.50					3952.44	6330.69

<u>Note:</u>

1. The forces in the shear beam between elevation 60.5' and 98' are those in the containment internal structures stick. There is a parallel shear beam for the base mat of the auxiliary and shield building.

Table 3.7.2-14

SUMMARY OF MODELS AND ANALYSIS METHODS

Model	Analysis Method	Program	Type of Dynamic Response/Purpose
3D lumped mass stick, fixed base models	Mode superposition time history analysis	ANSYS	Performed for hard rock profile. To develop time histories for generating floor response spectra. To obtain the following: Maximum absolute nodal accelerations (ZPA). Maximum displacements relative to basemat. Maximum member forces and moments for all structures.
3D finite element, fixed base models, coupled aux/shield buildings, with stick models of containment internal structures	Mode superposition time history analysis	ANSYS	Performed for hard rock profile. To develop time histories for generating vertical floor response spectra in auxiliary building. To obtain the following: Maximum absolute nodal accelerations (ZPA). Maximum displacements relative to basemat.
3D finite element, fixed base models, coupled aux/shield buildings, with stick models of containment internal structures	Equivalent static analysis using nodal accelerations from 3D stick model	ANSYS	Performed for the hard rock profile with equivalent static acceleration input. To obtain the in-plane forces for the design of floors and walls of the auxiliary and shield building. To obtain SSE bearing reactions and member forces in the basemat.
3D finite element, fixed base model of containment internal structures	Equivalent static analysis using nodal accelerations from 3D stick model	ANSYS	Performed for the hard rock profile with equivalent static acceleration input. To obtain the in-plane forces for the design of floors and walls of the containment internal structures.
3D shell of revolution model of steel containment vessel	Equivalent static analysis using nodal accelerations from 3D stick model	ANSYS	To obtain SSE stresses for the containment vessel
3D finite element model of the shield building roof	Equivalent static analysis using nodal accelerations from 3D stick model	ANSYS STRUDL	To obtain SSE member forces for the shield building roof

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Table 3.7.2-15

COMPARISON OF FREQUENCIES FOR CONTAINMENT VESSEL SEISMIC MODEL

	Vertical Model		Horizontal Model	
Mode No.	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

Table 3.7.2-16

SUMMARY OF DYNAMIC ANALYSES & COMBINATION TECHNIQUES

Model	Analysis Method	Program	Three Components Combination	Modal Combination
3D lumped mass stick, fixed base models	Mode superposition time history analysis	ANSYS	Algebraic Sum	n/a
3D finite element, fixed base models, coupled aux/shield buildings, with stick models of containment internal structures	Mode superposition time history analysis	ANSYS	Algebraic Sum	n/a
3D finite element, fixed base models, coupled Aux/Shield buildings and Cont. 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 STRUDL	SRSS	n/a

Table 3.7.2-17 not used.

Pages 3.7-76 through 3.7-77 intentionally blank.

Table 3.7.2-18 not used.

Pages 3.7-79 through 3.7-80 intentionally blank.

Table 3.7.2-19 not used.

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Table 3.7.3-1 (Sheet 1 of 3)

SEISMIC CATEGORY I EQUIPMENT OUTSIDE CONTAINMENT BY ROOM NUMBER

Room No.	Room Name	Equipment Description
12101	Division A battery room	Batteries
12102	Division C battery room 1	Batteries
12103	Spare battery room	Spare batteries
12104	Division B battery room 1	Batteries
12105	Division D battery room	Batteries
12113	Spare battery charger room	
12162	RNS pump room A	RNS pressure boundary
12163	RNS pump room B	RNS pressure boundary
12201	Division A dc equipment room	dc equipment
12202	Division C battery room 2	Batteries
12203	Division C dc equipment room	dc equipment
12204	Division B battery room 2	Batteries
12205	Division D dc equipment room	dc equipment
12207	Division B dc equipment room	dc equipment
12211	Corridor	Divisional cables
12212	Division B RCP trip switchgear room	RCP trip switchgear
12244	Lower annulus valve area	CVS/WLS containment isolation valves
12251	Demineralizer/filter access area	CVS/DWS isolation valves
12254	SFS penetration room	SFS containment isolation valve
12256	Containment isolation valve room	RNS containment isolation valves
12259	Pipe chase	RNS piping
12262	Piping/Valve room	RNS pressure boundary, SFS piping
12265	Waste monitor tank room C	SFS piping
12269	Pipe chase	RNS pressure boundary
12300	Corridor	Divisional cable
12301	Division A I&C room	Divisional I&C
12302	Division C I&C room	Divisional I&C

Table 3.7.3-1 (Sheet 2 of 3)

SEISMIC CATEGORY I EQUIPMENT OUTSIDE CONTAINMENT BY ROOM NUMBER

Room No.	Room Name	Equipment Description
12303	Remote shutdown workstation	Remote shutdown workstation
12304	Division B I&C/penetration room	Divisional I&C/electrical penetrations
12305	Division D I&C/penetration room	Divisional I&C/electrical penetrations
12306	Valve/piping penetration room	CCS/CVS/DWS/FPS/SGS containment isolation valves
12311	Corridor	Divisional cabling
12312	Division C RCP trip switchgear room	RCP trip switchgear
12313	Division C I&C/penetration room	Divisional I&C/electrical penetrations
12321	Non-1E equipment/penetration room	Divisional cabling
12341	Middle annulus	Class 1E electrical penetrations Various mechanical piping penetrations
12351	Maintenance floor staging area	Divisional cabling (ceiling)
12352	Personnel hatch	Personnel airlock (interlocks)
12354	Middle annulus access room	PSS/SFS containment isolation valves
12362	RNS HX room	RNS pressure boundary
12365	Waste monitor tank room B	SFS piping
12400	Control room vestibule	Control room access
12401	Main control room	Main control panels VBS HVAC dampers VES isolation valves Lights
12404	Lower MSIV compartment B	SGS containment isolation valves, instrumentation and controls
12405	Lower VBS B and D equipment room	VWS/PXS/CAS containment isolation valves
12406	Lower MSIV compartment A	SGS containment isolation valves, instrumentation and controls
12412	Electrical penetration room Division A	Divisional electrical penetration

Table 3.7.3-1 (Sheet 3 of 3)

SEISMIC CATEGORY I EQUIPMENT OUTSIDE CONTAINMENT BY ROOM NUMBER

Room No.	Room Name	Equipment Description
12421	Non 1E equipment/penetration room	Divisional cabling
12422	Reactor trip switchgear II	Reactor trip switchgear
12423	Reactor trip switchgear I	Reactor trip switchgear
12452	VFS penetration room	VFS containment isolation valves, divisional cabling
12454	VFS/SFS/PSS penetration room	SFS/PSS/VFS containment isolation valves, RNS pressure boundary
12462	Cask washdown pit	SFS piping
12504	Upper MSIV compartment B	SGS CIVs, instrumentation and controls
12506	Upper MSIV compartment A	SGS CIVs, instrumentation and controls
12541	Upper annulus	PCS piping and cabling PCS air baffle
12553	Personnel access area	Personnel airlock (interlocks)
12555	Operating deck staging area/VES air storage	VES high pressure air bottles
12562	Fuel handling area	Spent fuel storage racks
12701	PCS valve room	PCS isolation valves/instrumentation
12703	PCS water storage tank	PCS piping, level and temperature instrumentation

Table 3.7.3-2

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

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

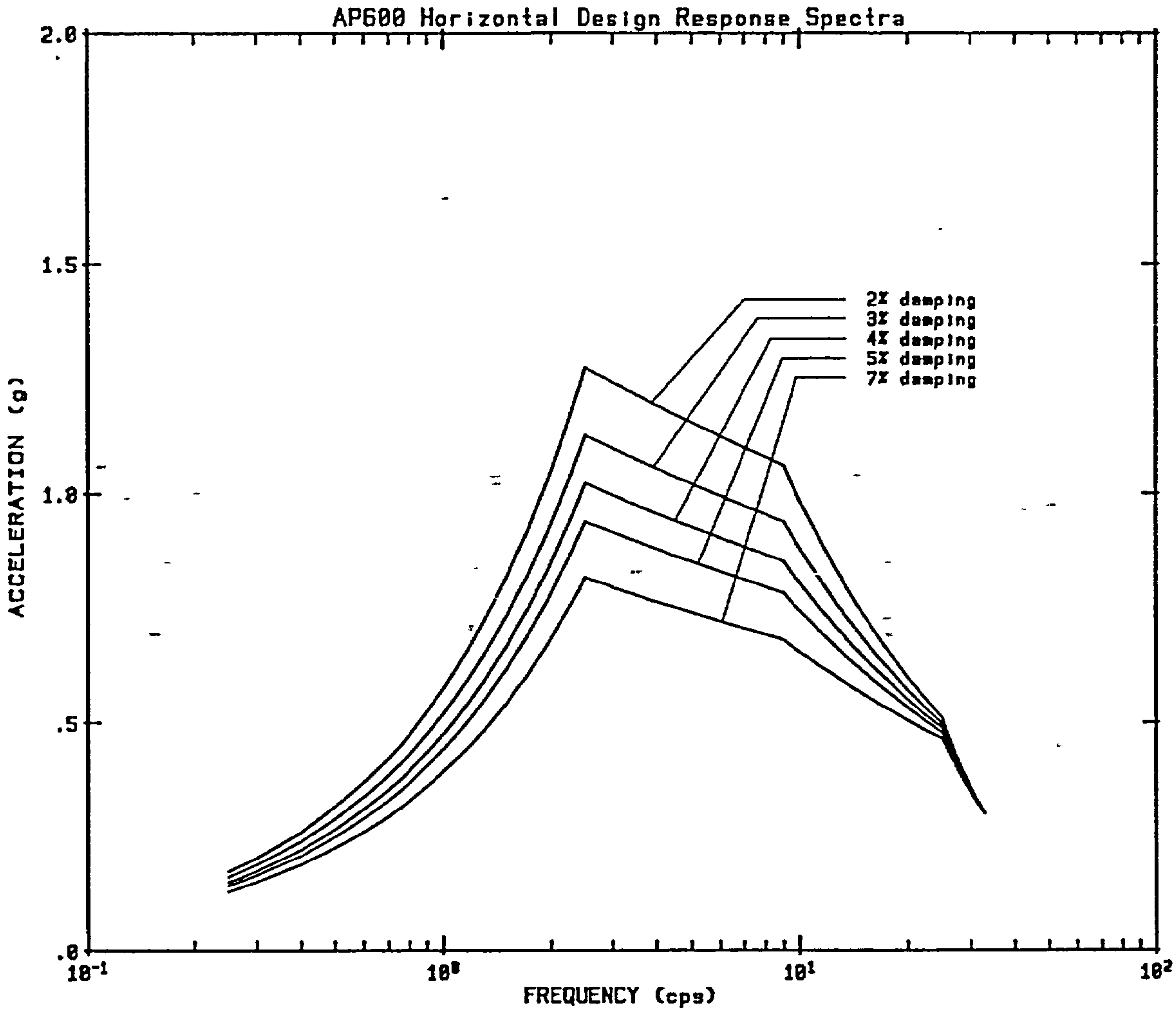


Figure 3.7.1-1

Horizontal Design Response Spectra
Safe Shutdown Earthquake

Note: AP600 and AP1000 seismic inputs are identical

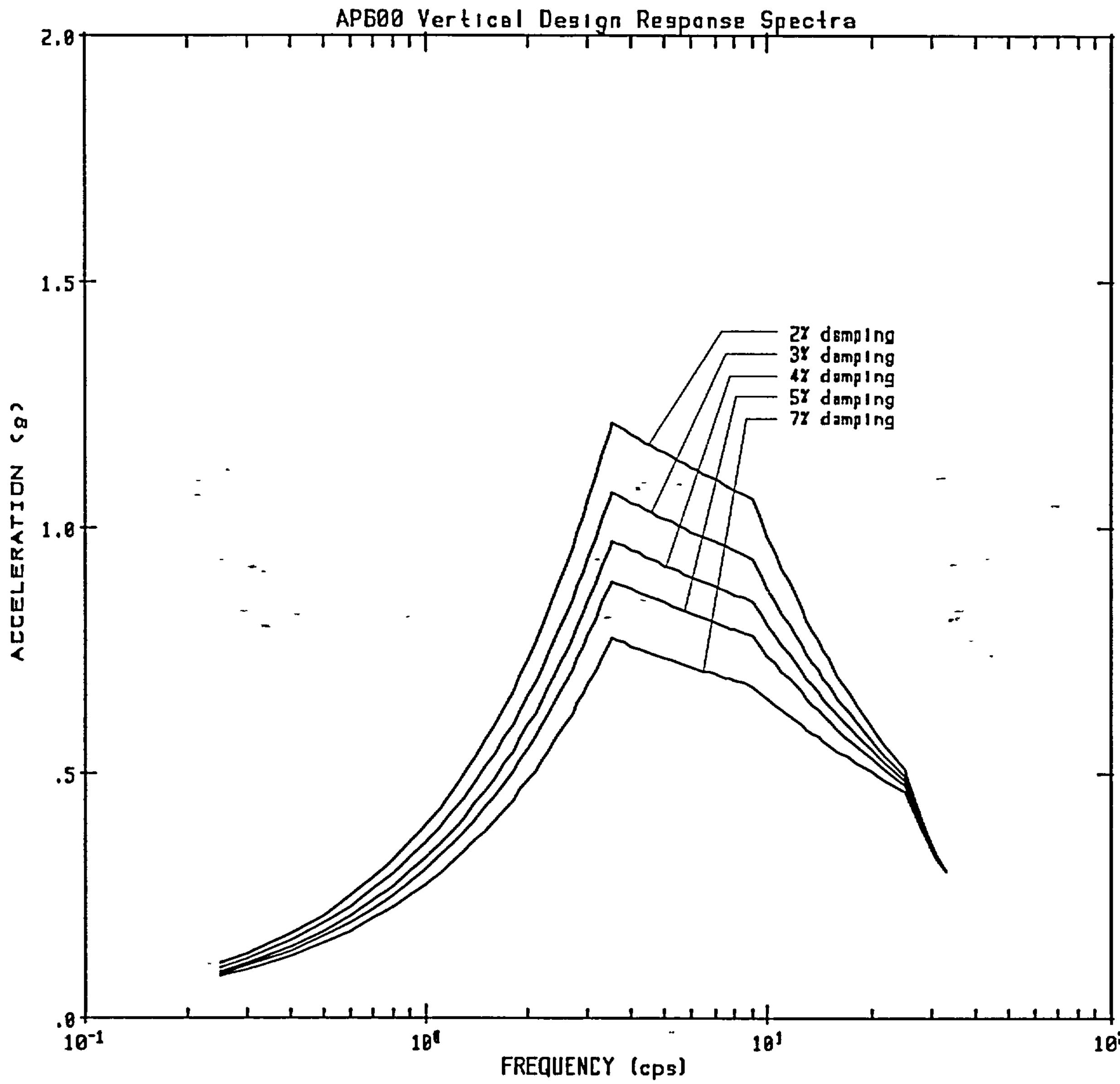


Figure 3.7.1-2

Vertical Design Response Spectra
Safe Shutdown Earthquake

Note: AP600 and AP1000 seismic inputs are identical

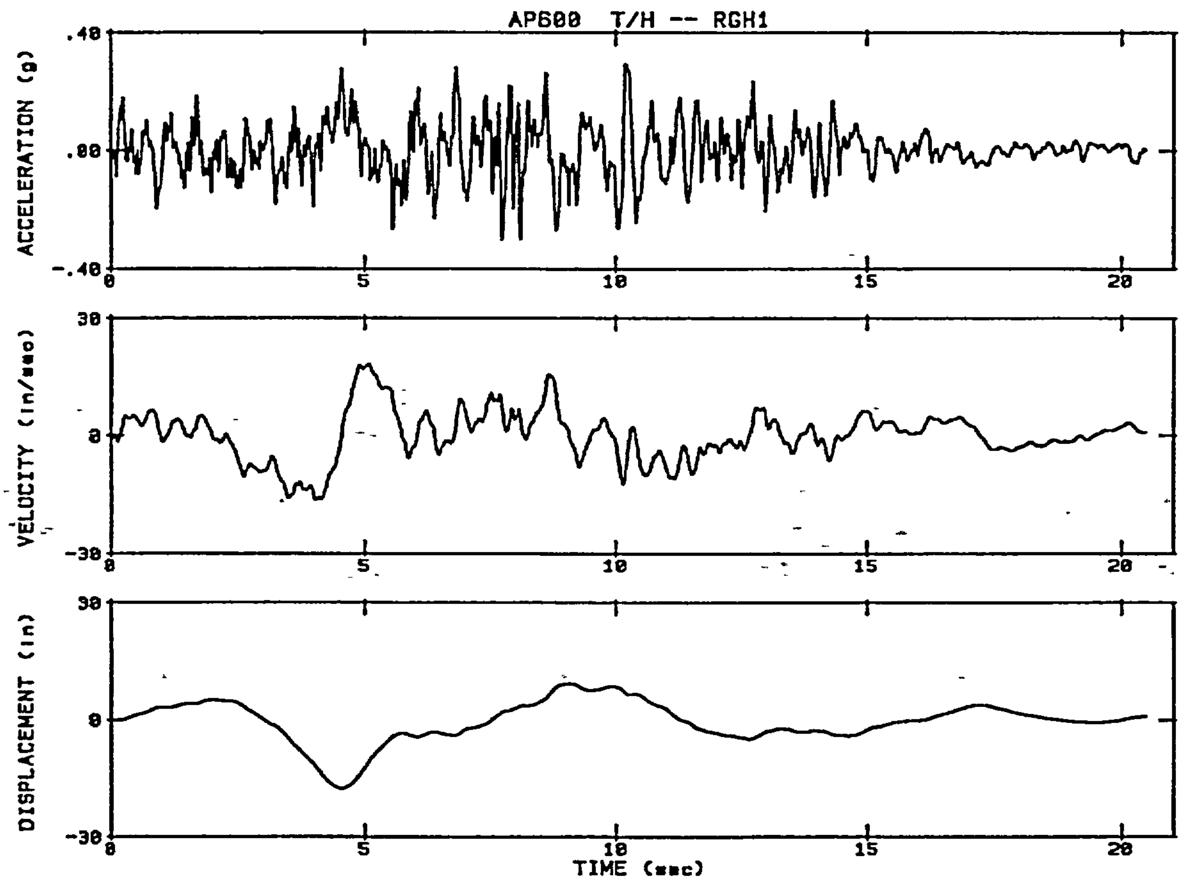


Figure 3.7.1-3

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

Note: AP600 and AP1000 seismic inputs are identical

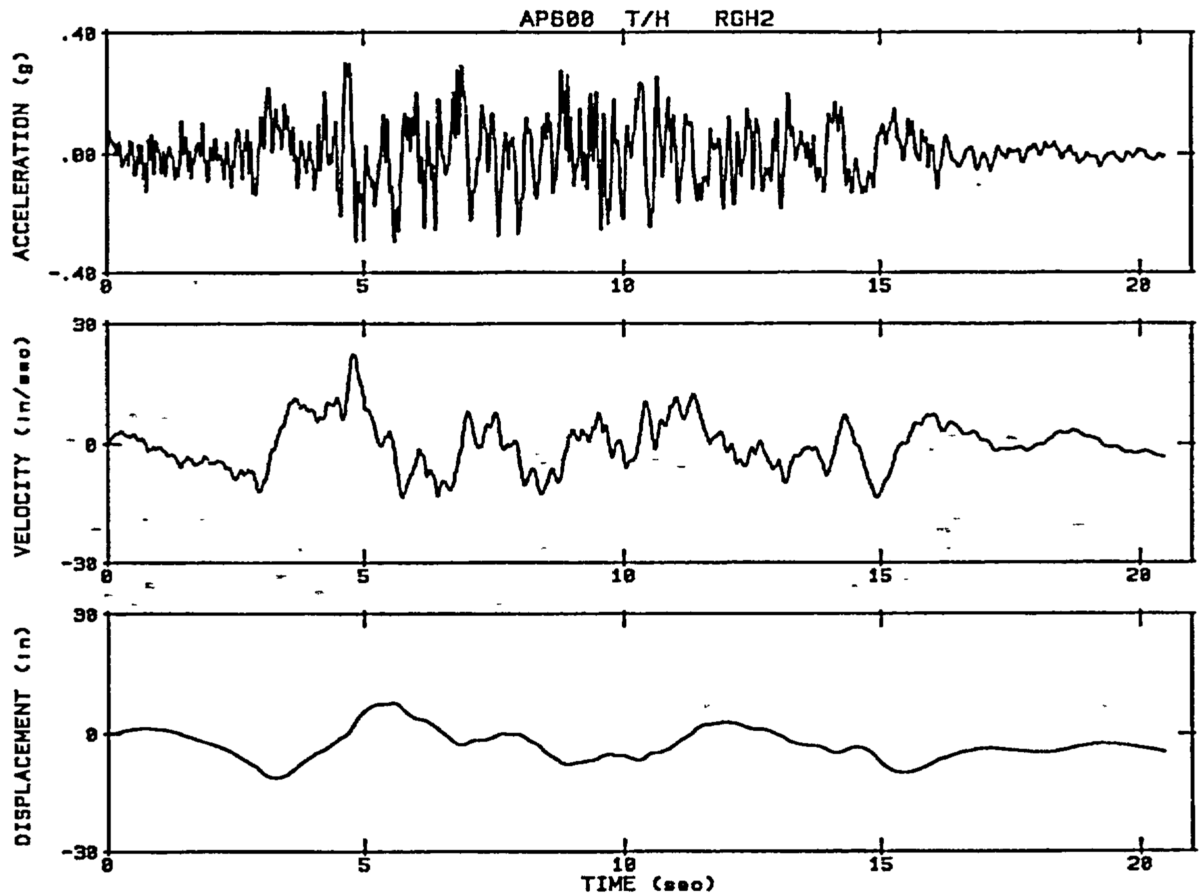


Figure 3.7.1-4

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

Note: AP600 and AP1000 seismic inputs are identical

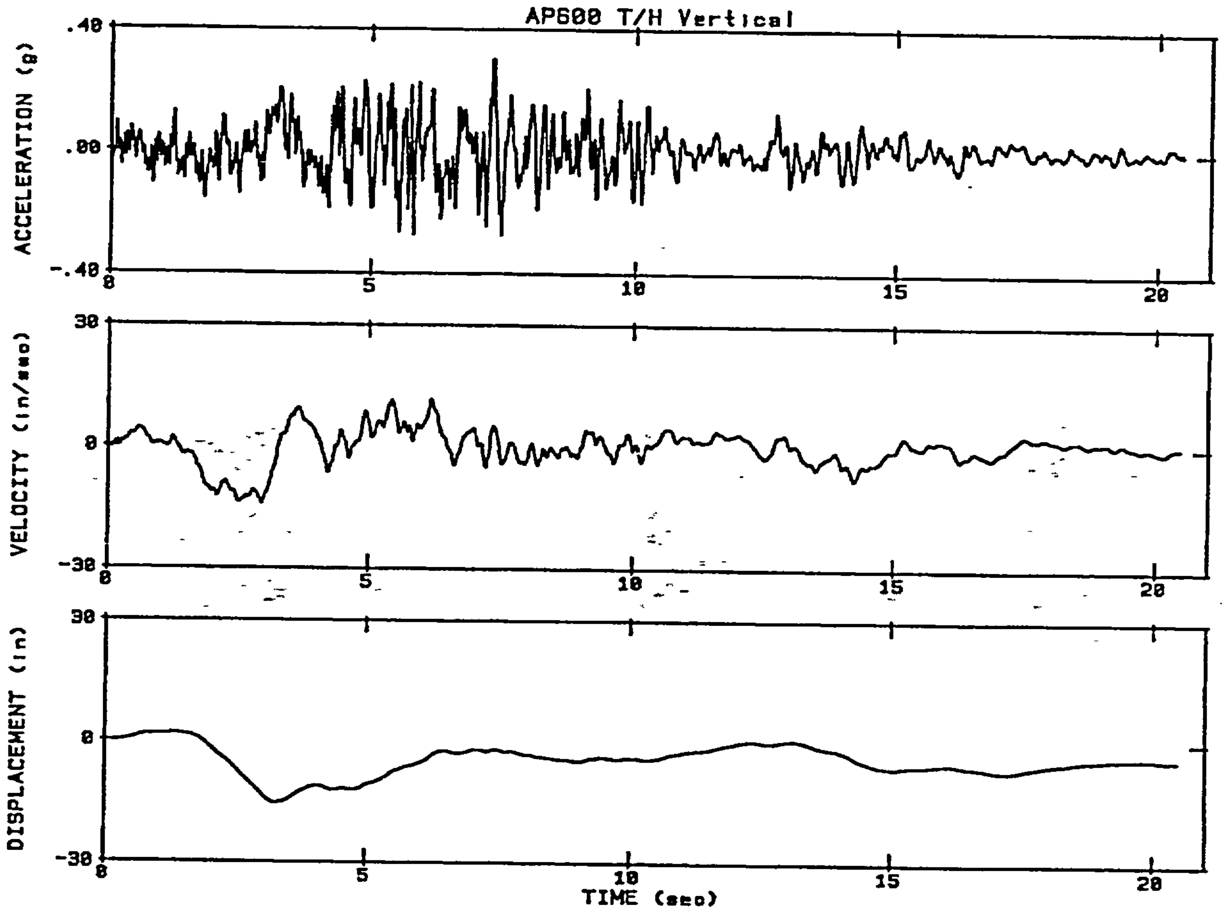


Figure 3.7.1-5

Design Vertical Time History Acceleration, Velocity & Displacement Plots

Note: AP600 and AP1000 seismic inputs are identical

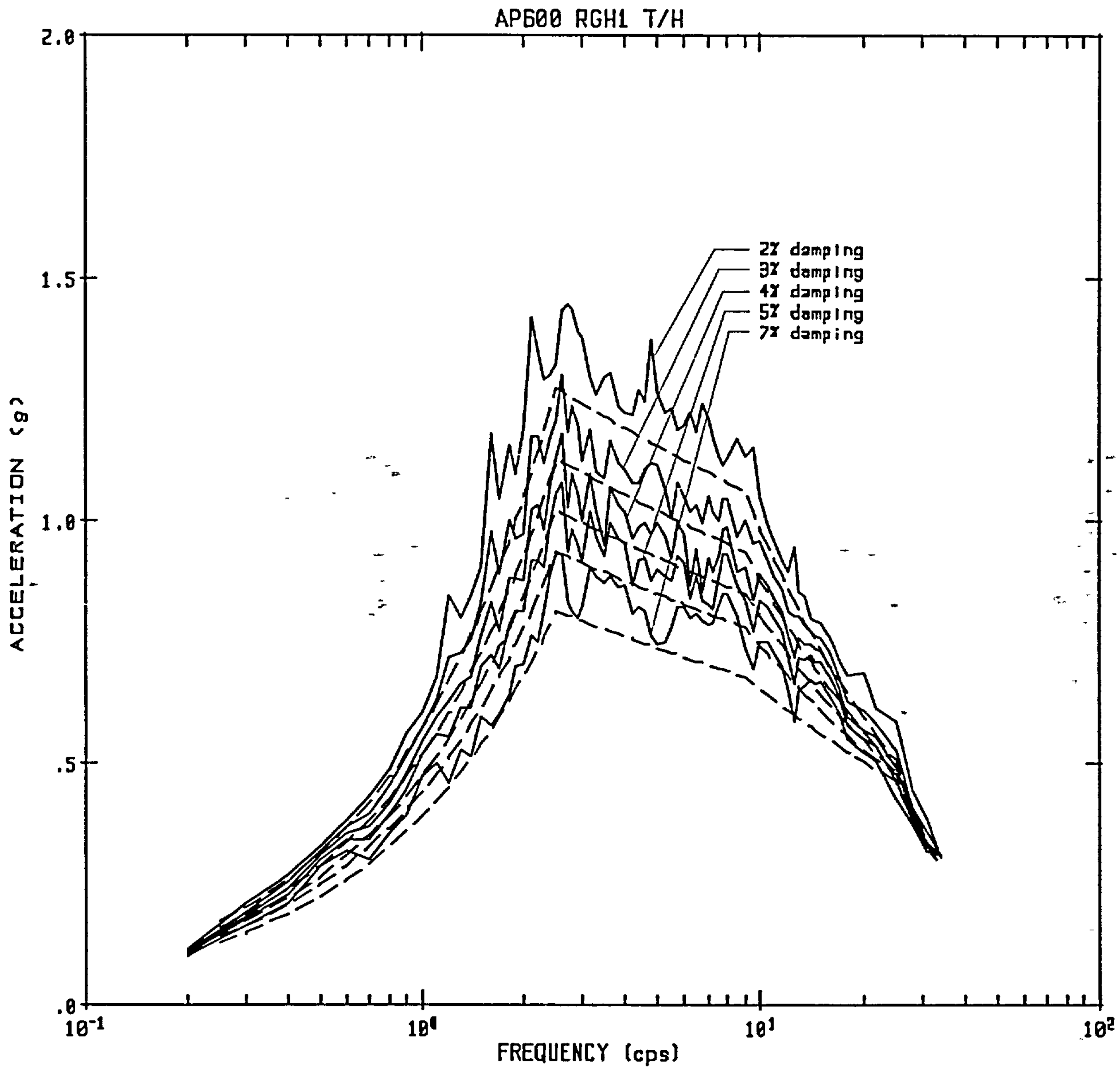


Figure 3.7.1-6

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

Note: AP600 and AP1000 seismic inputs are identical

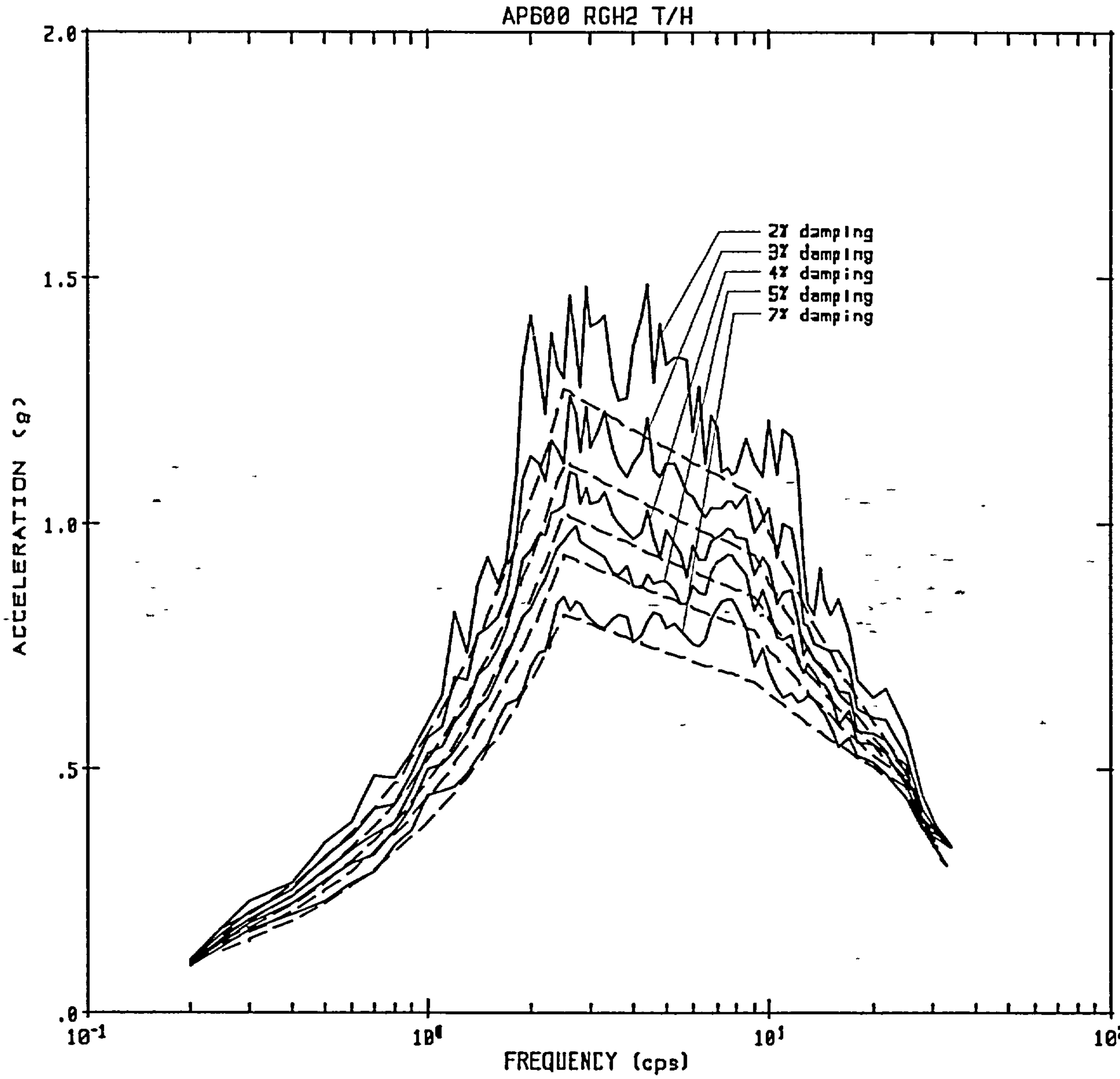


Figure 3.7.1-7

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

Note: AP600 and AP1000 seismic inputs are identical

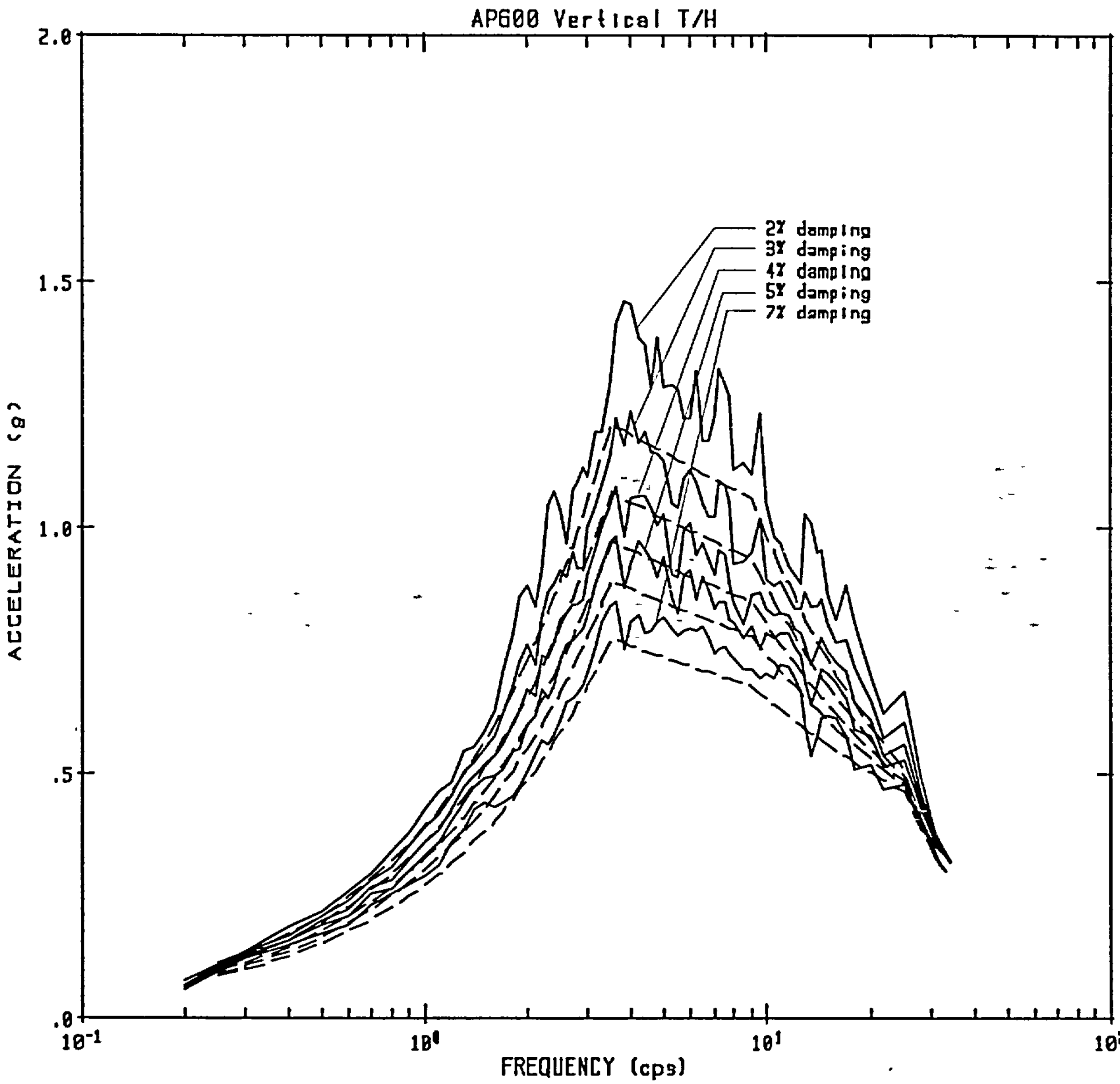


Figure 3.7.1-8

Acceleration Response Spectra of Design Vertical Time History

Note: AP600 and AP1000 seismic inputs are identical

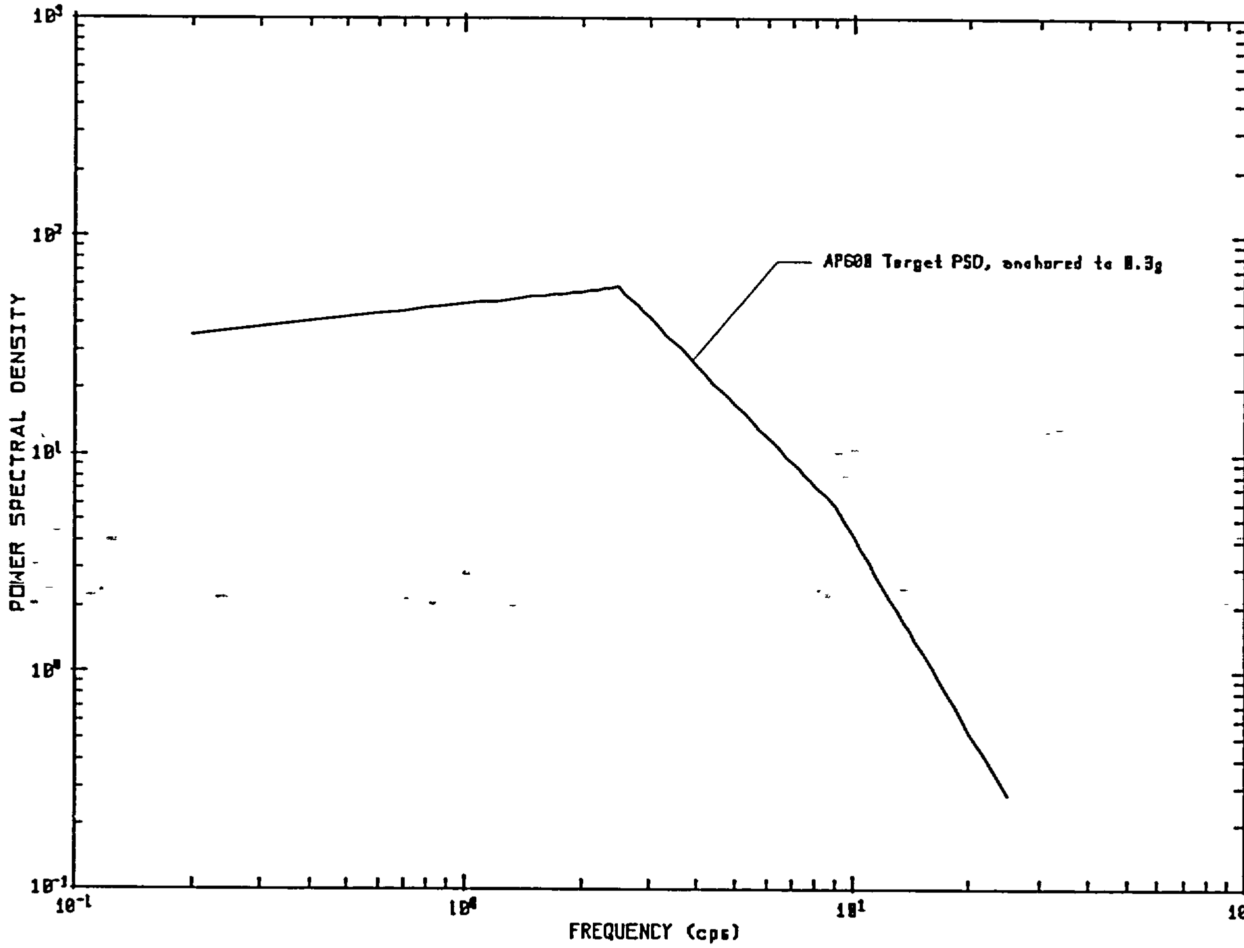


Figure 3.7.1-9

Minimum Power Spectral Density Curve (Normalized to 0.3g)

Note: AP600 and AP1000 seismic inputs are identical

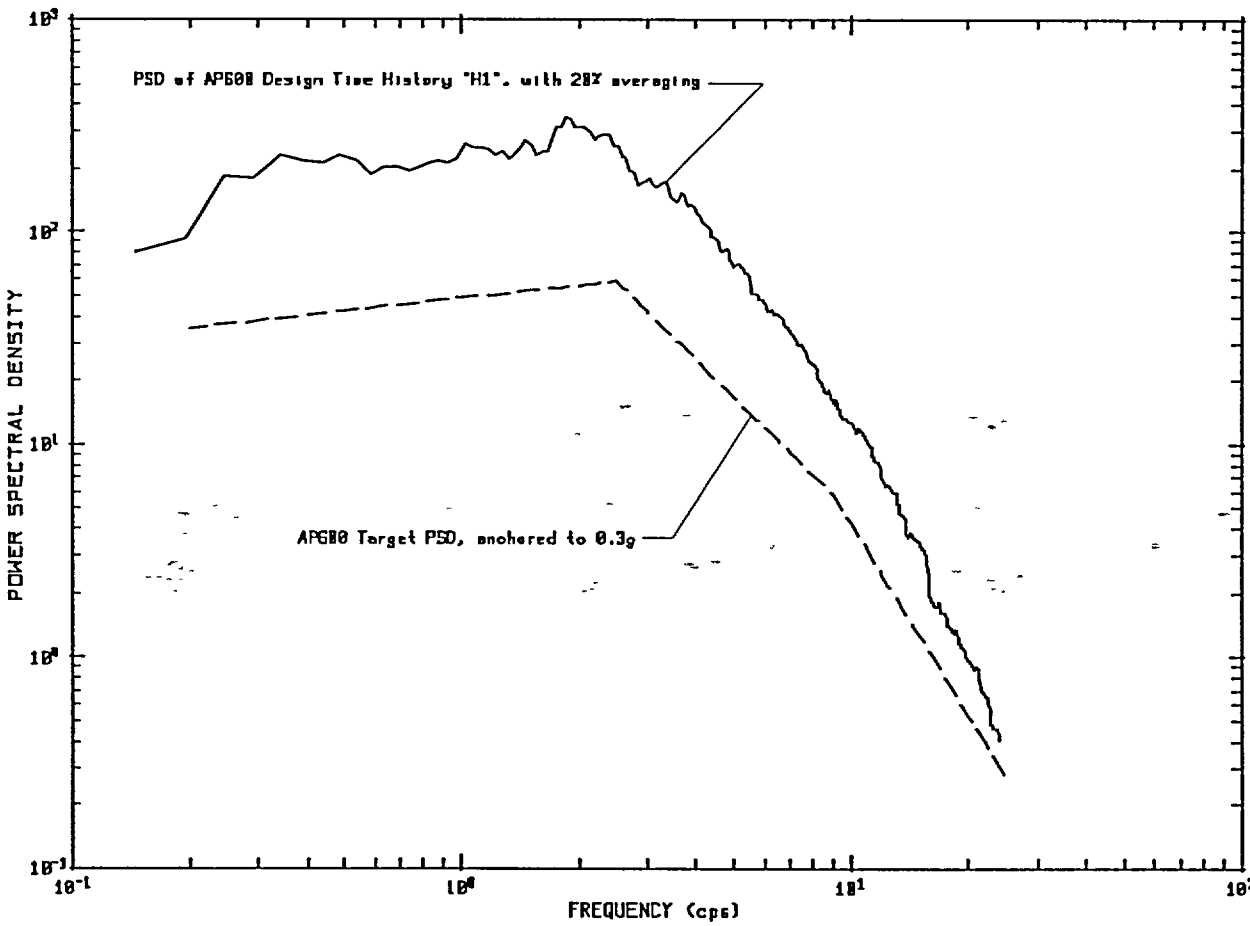


Figure 3.7.1-10

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

Note: AP600 and AP1000 seismic inputs are identical

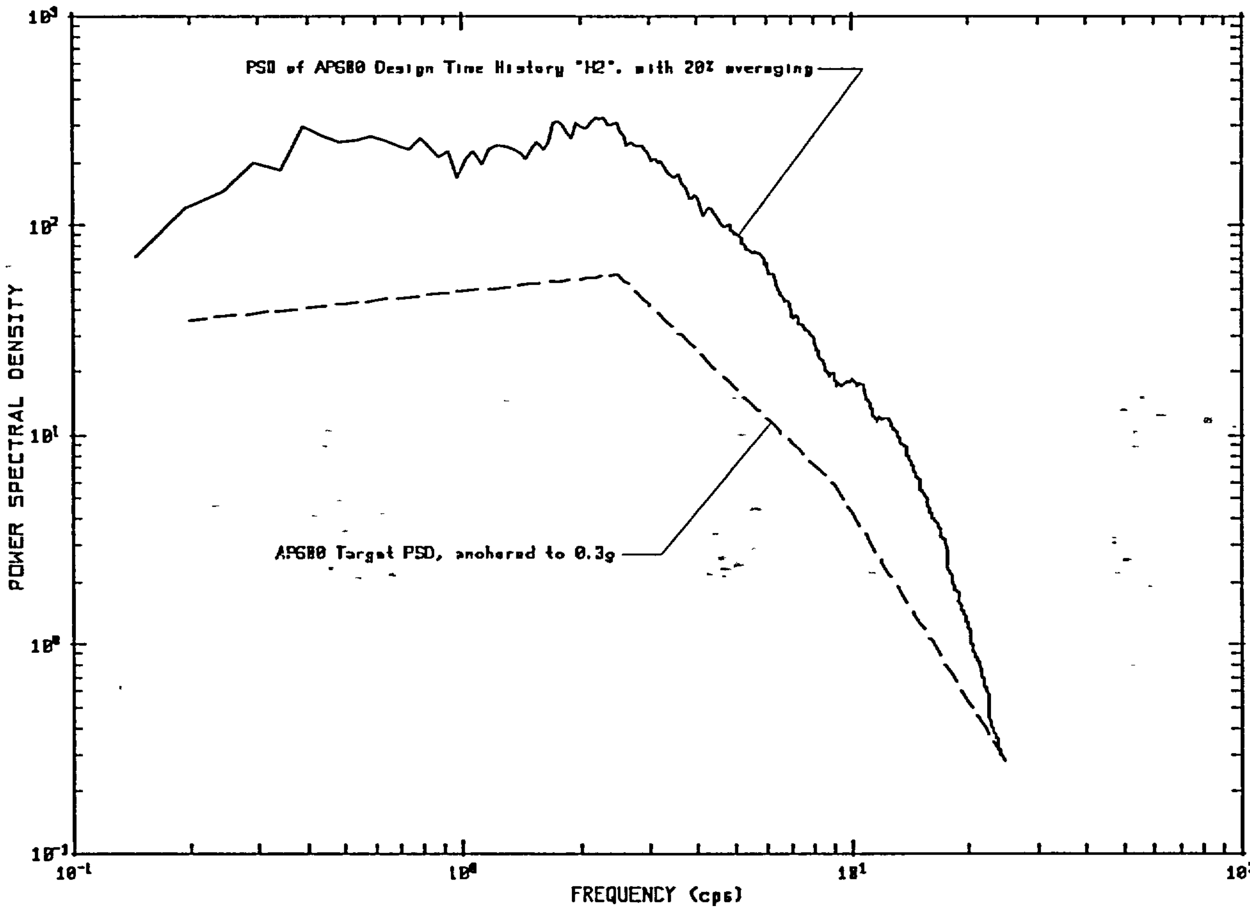


Figure 3.7.1-11

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

Note: AP600 and AP1000 seismic inputs are identical

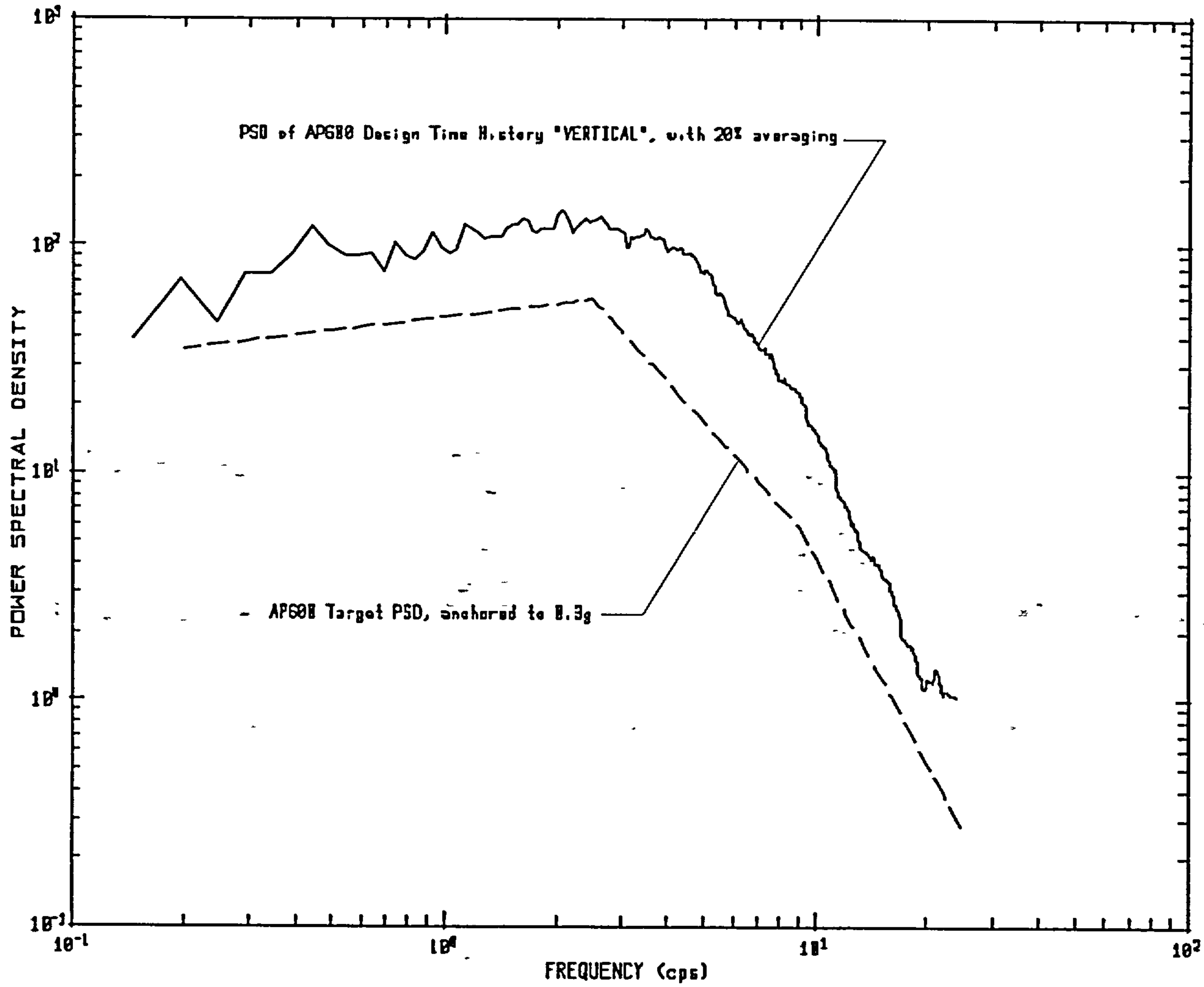
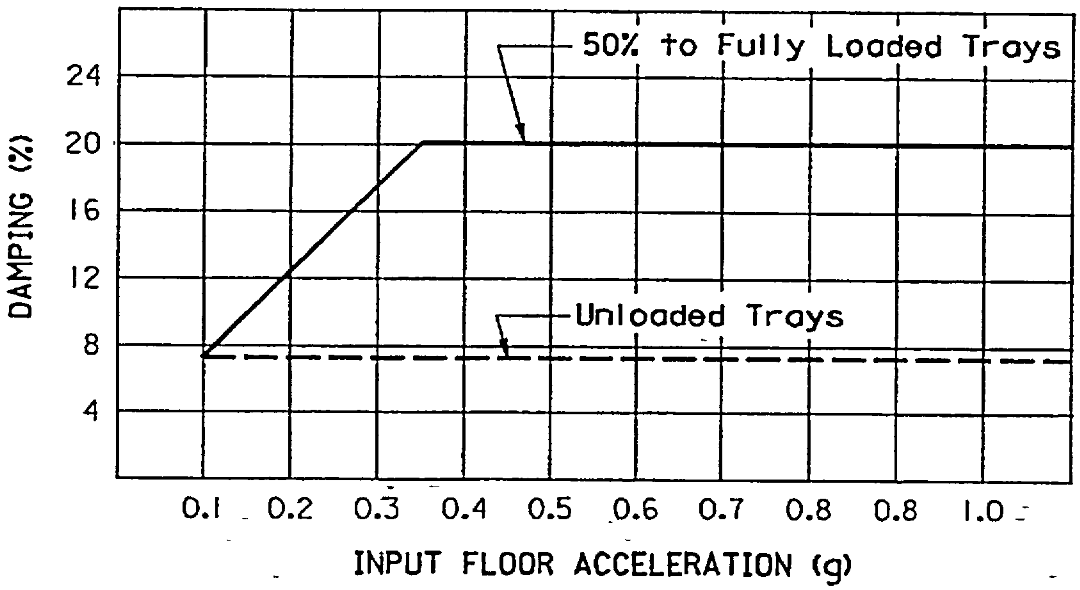


Figure 3.7.1-12

Note: AP600 and AP1000
seismic inputs are identical

Power Spectral Density of
Design Vertical Time History



Notes:

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

Figure 3.7.1-13

Damping Values for Cable Trays & Supports