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SUBJECT: Transmittal of WCAP-15614, "AP1000 Seismic and Structural Design Activities"

Enclosed is WCAP-15614, "AP1000 Seismic and Structural Design Activities." The report outlines our proposed approach to use design acceptance criteria in the areas of seismic analyses, structural design, and piping design for an AP1000 Design Certification application. We request your review of this report as part of phase 2 of the AP1000 pre-certification review.

Please direct questions on this submittal to Mike Corletti at 412-374-5355.

Very truly yours,

J. W. Winters, Manager
AP600 Engineering
Passive Plant Projects

cc: J. N. Wilson - NRC

D063

Westinghouse Non-Proprietary Class 3

WCAP-15614

AP1000 Seismic and Structural Design Activities

Westinghouse Electric Company LLC

AP1000 DOCUMENT COVER SHEET

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WESTINGHOUSE NON-PROPRIETARY CLASS 3

WCAP-15614

AP1000
Seismic and Structural Design Activities

January 2001

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1 INTRODUCTION

This report has been prepared in support of a pre-application review of the AP1000 standard nuclear plant by the U.S. NRC. Westinghouse has initiated development of the AP1000 standard nuclear reactor design based closely on the AP600 design. The AP1000, with a power output of approximately 1000 MWe (3400 MWt), maintains the AP600 design configuration, use of proven components, design basis and licensing basis by limiting the changes to the AP600 design to as few as possible. The design features of the plant have been selected to preserve key features and performance characteristics embodied in the AP600. The AP1000 design approach has been to retain the AP600 design basis, and therefore retain, to the extent possible, the licensing basis of the AP600. WCAP-15612, "AP1000 Plant Description and Analysis Report" (Reference 1) provides an overview description of the design differences between the AP600 and the AP1000.

A potential difference in the licensing approach for the AP1000 is the use of Design Acceptance Criteria in lieu of detailed design for selected areas of the plant. 10 CFR Part 52(a)(2) requires the following:

"The application must contain a level of design information sufficient to enable the Commission to judge the applicant's proposed means of assuring that construction conforms to the design and to reach a final conclusion on all safety questions associated with the design before the certification is granted. The information submitted for a design certification must include performance requirements and design information sufficiently detailed to permit the preparation of acceptance and inspection requirements by the NRC, and procurement specifications and construction and installation specifications by an applicant. The Commission will require, prior to design certification, that information normally contained in certain procurement specifications and construction and installation specifications be completed and available for audit if such information is necessary for the Commission to make its safety determination."

The AP600 application met this requirement by providing detailed design information in the areas of seismic analyses, structural design, and piping design. Other applicants have taken a different approach in these areas by substituting Design Acceptance Criteria (DAC) for detailed design information. DAC, in conjunction with well-written Inspections, Tests Analysis and Acceptance Criteria (ITAAC) and Combined License (COL) applicant items, were sufficient to permit the NRC staff to judge that an application for Design Certification meets the requirements of 10 CFR Part 52.

In the AP1000 Design Certification application, Westinghouse intends to use DAC in the areas mentioned above. This document outlines Westinghouse's proposed approach to support an AP1000 Design Certification for these selected activities. The AP1000 Design Certification application will include less design detail than that provided in the AP600 Design Certification application. Specifically, Westinghouse will provide less design detail in the following areas:

- Seismic analyses (DCD Chapter 2 and Section 3.7)
- Structural design (DCD Section 3.8)
- Piping design (DCD Section 3.6 and 3.9)

The information to be provided in the AP1000 DCD and that to be provided by the Combined License applicant is summarized in the following table and discussed in subsequent sections.

Scope in Design Certification Document	Scope in Combined License Application	ITAAC/DAC
• Development of stick models for AP1000 • Fixed base seismic analyses of stick models for rock site, including typical results (accelerations, displacements, member forces and floor response spectra) • Overturning and stability for rock site • Preliminary assessment to confirm feasibility of key structural elements with significant increase in load from AP600 • Seismic analysis ITAAC (DAC) at soil sites • Structural design ITAAC (DAC) • Piping design criteria and methodology	• Development of finite element models for AP1000 • If site is not rock, SASSI analysis, including typical results (accelerations, displacements, member forces and floor response spectra) • Overturning and stability • Response spectrum analyses of structures, including soil amplification factor • Structural design, including design reports • Piping analyses for lines qualified for leak before break	• Seismic analyses of soil sites (to be included in Combined License application) • Structural design (to be included in Combined License application) *The following ITAACs are the same as for the AP600* • As-constructed structural and seismic reconciliation • Piping stress reports • Pipe rupture hazard evaluation

The DCD provides design descriptions, design and analysis methodology and design acceptance criteria (ITAAC/DAC). The Combined License applicant includes in his DCD information on implementation of the detail design. This detail design information permits closure of the ITAAC/DAC when the Combined License is issued. The ITAACs verify the as-constructed condition.

2 SEISMIC ANALYSES

2.1 SCOPE IN DESIGN CERTIFICATION

The AP1000 Design Certification Document will include results of seismic analyses for a hard rock site. These analyses use stick models with a fixed base. Two cases will be analyzed, one fixed at the base mat, and one fixed at the base mat and also fixed horizontally up to grade. These two cases will be enveloped for design inputs (member forces, relative deflections, maximum accelerations, and floor response spectra).

The AP1000 Design Certification Document will include criteria and methodology for seismic analyses at soil sites. These criteria and methodology will be consistent with that approved by NRC in the AP600 Design Certification. Detail design implementation will be covered by the addition of the following ITAAC (DAC) in Table 3.3.6.

Table 3.3-6 Inspections, Tests, Analyses, and Acceptance Criteria

Design Commitment	Inspections, Tests, Analyses	Acceptance Criteria
Nuclear island soil-structure interaction seismic analyses provide seismic responses for the analysis and design of building structures and seismic subsystems. *Note:* *These seismic analyses are only required when the AP1000 is to be located at a site where the soil below the underside of the base mat has a shear wave velocity less than 3500 feet per second.*	Soil-structure interaction (SSI) analyses of the nuclear island are performed to generate its soil-structure interaction responses. Results include nodal displacements, nodal accelerations, building structure member forces and floor response spectra.	The results of soil structure interaction analyses are documented in a seismic analysis report and summarized in the Combined License application. *Note:* *The seismic analyses at a soil site will be reviewed and accepted by NRC during the Combined License application.*

Chapter 2 of the AP1000 DCD will include the same site parameters in Table 2-1 as those specified for the AP600. The AP1000 DCD will not include the AP600 DCD Appendices 2A, 2B, and 2C. These appendices gave the results of soil structure interaction parametric analyses and provided the basis for the design soil profiles considered in the AP600 plant design.

Draft text for the AP1000 DCD Sections 2.5, 3.7.1 and 3.7.2 is provided in Appendix A. Typical results of the AP1000 hard rock analyses are compared to the AP600 results in Appendix B. These analyses use methodology similar to that for the AP600 and are described in the draft DCD sections in Appendix A.

The AP1000 stick models included in the AP1000 DCD are developed using data from the AP600 analyses. These models will be reconciled by the Combined License applicant, as required by AP600 DCD subsection 3.7.5.4. This reconciliation will also include comparison against the AP1000 finite element models required for structural design.

2.2 SCOPE IN COMBINED LICENSE APPLICATION

The new ITAAC shown above requires the Combined License applicant to perform seismic analyses and to provide results for NRC review. It is expected that this ITAAC will be complete when the Combined License is issued. Additional detail on these analyses is included in subsection 3.7.5.5 of the draft AP1000 DCD in Appendix A. These requirements are such that the seismic analysis methodology for a soil site will be consistent with the analyses performed at soil sites for the AP600.

2.2.1 Rock site

At a hard rock site, the Combined License applicant may use the seismic results included in AP1000 Design Certification.

2.2.2 Soil or rock site

At a soil site, the Combined License applicant will select the range of soil conditions for which he is requesting approval. He will perform seismic analyses and structural design in accordance with the ITAAC (DAC). The range of soil conditions will be selected at the time of the Combined License submittal and may include one of the following examples:

Option 1

Analyze one case for the best estimate site properties described in Section 2.5 of the Combined License application and upper and lower bound cases to bound the site. This results in a design applicable to a narrow range of sites.

Option 2

Envelope the results from the three soil cases of Option 1 and also envelope the results of the hard rock analyses included in the AP1000 Design Certification. This results in a design that is demonstrated to be acceptable at a single site and has additional margin so that it is applicable to a broader range of sites than in Option 1.

Option 3

Perform analyses for two, three or four of the following soil cases considered for AP600. Demonstrate that these cases bound the range of site specific soil conditions described in Section 2.5 of the Combined License application.

- For the hard rock site, an upper bound case for firm sites using fixed base seismic analysis. The results of this case are provided in the AP1000 Design Certification.

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

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

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

Enveloping the results of all four of the above cases will provide a design satisfying the full range of site parameters identified in DCD Table 2-1.

The Combined License applicant at a rock site may also elect one of these options in order to broaden the applicability of the design.

3 STRUCTURAL DESIGN

Design criteria and methodology are the same for the AP1000 as for the AP600. The changes necessary to the DCD are described for each section in Table 1 and discussed further below. The changes typically are minor changes to the structural descriptions and the elimination of detail design results. Detail design and analysis will be required by the Combined License applicant by the addition of the following ITAACs.

Table 3.3-6 Inspections, Tests, Analyses, and Acceptance Criteria		
Design Commitment	**Inspections, Tests, Analyses**	**Acceptance Criteria**
The nuclear island structures are seismic Category I and are designed and constructed to withstand design basis loads as specified in the Design Description, without loss of structural integrity and the safety-related functions.	The nuclear island structures will be analyzed for the design basis loads.	Report(s) exist which conclude that the nuclear island structures, including the auxiliary and shield building, the containment internal structures and the nuclear island foundation and base mat, conform to the approved design methodology and will withstand the design basis loads specified in the Design Description without loss of structural integrity or the safety-related functions. This report will be summarized in the Combined License application *Note:* *The structural report(s) will be reviewed and accepted by NRC during the Combined License application.*

3.1 CONCRETE CONTAINMENT (DCD SUBSECTION 3.8.1)

This DCD subsection is not applicable for AP600 or AP1000

3.2 STEEL CONTAINMENT (DCD SUBSECTION 3.8.2)

The steel containment vessel will be constructed to ASME. The ASME Design Specification will be complete at the time of AP1000 Design Certification. Westinghouse will use the 1998 edition of the ASME Code, including the 1999 addenda, and will provide justification, as needed, in the DCD. The level of design information available will be sufficient to permit the AP1000 DCD to have the same level of detail as was provided in the AP600 DCD. For AP600, NRC staff audited the containment vessel design report and confirmed that the commitments in the DCD were sufficient and implemented appropriately. The AP1000 containment vessel design will follow the same methodology as that already reviewed by NRC for AP600. The final AP1000 containment vessel design will be documented in the ASME Code Section III design report which will be required by an AP1000 ITAAC identical to that for the AP600 (Table 2.2.1-3).

3.3 CONTAINMENT INTERNAL STRUCTURES (DCD SUBSECTION 3.8.3)

The containment internal structures for the AP1000 are the same as for the AP600 except that the height of the steam generator and pressurizer compartment walls are increased and the steam generator snubbers are raised to the top of the steam generator compartments. This requires only minor revisions to the AP600 DCD. Member forces will increase slightly due to the increased wall height and larger equipment. Hence, all design calculations will be updated by the Combined License applicant following the methodology documented in the DCD. For AP600, NRC staff audited the design report and calculations and confirmed that the commitments in the DCD were sufficient and implemented appropriately. The AP600 DCD provides typical design results in Tables 3.8.3-4 to 3.8.3-7. The equivalent AP1000 results will be provided by the Combined License applicant. The AP1000 containment internal structures report and calculations will not be available at the time of Design Certification. However, they will follow the same methodology as that already reviewed by NRC for AP600. The final AP1000 containment internal structures design report will be required by an AP1000 ITAAC identical to that for the AP600 (Table 3.3-6).

3.4 OTHER CATEGORY I STRUCTURES (DCD SUBSECTION 3.8.4)

The other Category I structures for the AP1000 are the same as for the AP600 except for the height of the shield building (increased by 25′ 6″), the passive containment cooling tank on the shield building roof (increased to 800,000 gallons), and the fuel and cask loading pits (increased in depth by 1′ 6.5″). This requires only minor revisions to the AP600 DCD. Member forces will increase due to the increased shield building height and larger tank. This is shown in the Tables in Attachment #2. All design calculations will be updated by the Combined License applicant following the methodology documented in the DCD. For AP600, NRC staff audited the design report and calculations and confirmed that the commitments in the DCD were sufficient and implemented appropriately. The AP600 DCD provides typical design results in Appendix 3H. AP1000 DCD will describe these critical sections with detailed results to be provided by the Combined License applicant. The AP1000 Category I structures report and calculations will not be available at the time of Design Certification. However, they will follow the same methodology as that already reviewed by NRC for AP600. The final AP1000 Category I structures design report will be required by an AP1000 ITAAC identical to that for the AP600 (Table 3.3-6).

3.5 FOUNDATIONS

The Combined License applicant will use finite element models of the nuclear island to determine soil pressures for design of the auxiliary building base mat and exterior walls, and member forces in the nuclear island and containment internal base mat. Loadings include:

Dead and live loads
Seismic loads
Settlement during construction

The AP1000 nuclear island base mat report and calculations will be provided by the Combined License applicant at the time of his application. They will follow the same methodology as that already reviewed by NRC for AP600. The final AP1000 nuclear island base mat design report will be required by an AP1000 ITAAC identical to that for the AP600 (Table 3.3-6).

Table 3-1	AP1000 DCD Section 3.8 "Design of Category I Structures" Changes from AP600
DCD Subsection	**Discussion of Change from AP600**
3.8.1	**Concrete Containment** **No Change**
3.8.2	**Steel Containment**
3.8.2.1	**Description of Containment** Height, thickness, material and design pressure are changed Design revised to reference ASME 99 Addenda Figures 3.8.2-1 (3 sheets), 2, 4 (sheet 1), 5, 6 revised
3.8.2.2	**Applicable Codes, Standards, and Specifications** Design revised to reference ASME 99 Addenda Need to review status of ASME Code Case N-284 as given in Appendix 3G Need to add Code Case for new material
3.8.2.3	**Loads and Load Combinations** No change
3.8.2.4	**Design and Analysis Procedures** Minor revisions to reflect requirements that are included in the ASME Design Specification ASME sizing calculations will be performed for internal and external pressure to size shell thickness, stiffeners and equipment hatch reinforcement and head thickness. Finite element model will be updated and internal pressure axisymmetric analyses will be performed using ANSYS and described in DCD. Detail analysis and design are to be performed by Containment Vessel supplier and documented in the ASME Design Report. ITAAC will confirm existence of the Design Report. Ultimate pressure capacity will be revised for change in thickness and material. Figures 3.8.2-5, 6 to be revised
3.8.2.5	**Structural Criteria** **No change**
3.8.2.6	**Materials, Quality Control, and Special Construction Techniques** Add any special requirements for new material Minimum service temperature increased Corrosion protection at base to be described
3.8.2.8	**Testing and In-Service Inspection Requirements** **No change**

Table 3-1 (cont.)	AP1000 DCD Section 3.8 "Design of Category I Structures" Changes from AP600
DCD Subsection	**Discussion of Change from AP600**
3.8.3	**Concrete and Steel Internal Structures of Steel Containment**
3.8.3.1	**Description of the Containment Internal Structures** Steam generator and pressurizer compartment walls raised. Steam Generator support system revised Figures 3.8.3-1 (sheets 2 and 3), 4, 5, 6 to be revised Figures 3.8.3-7 (9 sheets) showing detail reinforcement design to be deleted
3.8.3.2	**Applicable Codes, Standards, and Specifications** **No change**
3.8.3.3	**Loads and Load Combinations** IRWST heatup rates will be revised
3.8.3.4	**Analysis Procedures** **No change**
3.8.3.5	**Design Procedures and Acceptance Criteria** Seismic loads for soil sites to be developed by COL applicant Design methodology for critical sections will continue to be described in subsection 3.8.3.5.8. The COL applicant will provide the detailed results of the design in AP600 Tables 3.8.3-3 through 3.8.3-7.
3.8.3.6	**Materials, Quality Control, and Special Construction Techniques** **No change**
3.8.3.7	**Testing and In-Service Inspection Requirements** **No change**
3.8.3.8	**Construction Inspection** **No change**

Table 3-1 (cont.)	AP1000 DCD Section 3.8 "Design of Category I Structures" Changes from AP600
DCD Subsection	**Discussion of Change from AP600**
3.8.4	**Other Category I Structures**
3.8.4.1	**Description of the Structures** Revise description of air baffle design for increase in height Figures 3.8.4-1 (sheet 1), 7, 9 to be revised
3.8.4.2	**Applicable Codes, Standards, and Specifications** **No change**
3.8.4.3	**Loads and Load Combinations** **No change**
3.8.4.4	**Analysis Procedures** Seismic loads for soil sites to be developed by COL applicant Seismic loads for exterior walls (previously in Appendix 2C) to be developed by COL applicant
3.8.4.5	**Design Procedures and Acceptance Criteria** Critical section design descriptions will be included in Appendix 3H. Detailed design results will be provided by Combined License applicant Table 3.8.4-7 results will be provided by Combined License applicant
3.8.4.6	**Materials, Quality Control, and Special Construction Techniques** **No change**
3.8.4.7	**Testing and In-Service Inspection Requirements** **No change**
3.8.4.8	**Construction Inspection** **No change**

Table 3-1 (cont.)	AP1000 DCD Section 3.8 "Design of Category I Structures" Changes from AP600
DCD Subsection	**Discussion of Change from AP600**
3.8.5	**Foundations** AP600 to AP1000 Table 3.8.5-2, 3 results will be provided by the Combined License applicant Figure 3.8.5-3 results will be provided by the Combined License applicant
3.8.5.1	**Description of the Foundations** **No change**
3.8.5.2	**Applicable Codes, Standards, and Specifications** **No change**
3.8.5.3	**Loads and Load Combinations** **No change**
3.8.5.4	**Design and Analysis Procedures** This subsection will be revised. A draft revision follows: The Combined License applicant will describe the design and analysis procedures for the nuclear island basemat and exterior walls. Dead, live, containment pressure and seismic loads will be applied to a finite element model similar to that described for the seismic analysis in subsection 3.7.2.3. The model will include a sufficient portion of the structures above the basemat to consider the effect of openings in the shear walls on the distribution of loads. The model of the basemat will be sufficiently refined that bending moments and shear forces in the base mat can be utilized directly for design. Alternatively, the model of the basemat may be less refined and bending moments and shears in the slab may be obtained from separate calculations using the bearing pressures under the basemat from the finite element analysis. Soils will be represented by spring elements with properties applicable to the site soil conditions. The model will include consideration of uplift of the basemat from the soil, as well as uplift of the containment internal structures from the lower basemat. Two dimensional SASSI models will be used to consider the effect of adjacent structures on the lateral soil pressures on the exterior walls. For soft soil sites, the Combined License applicant will evaluate the effect of differential settlement during construction. The analyses will account for the construction sequence, the associated time varying load and stiffness of the nuclear island structures, and the resulting settlement time history. The effects of settlement during construction will be included as dead loads in each of the post-construction load combinations.
3.8.5.5	**Structural Criteria** Table 3.8.5-2 results for overturning and sliding will show results for hard rock site. Results at soil sites will be provided by the Combined License applicant
3.8.5.6	**Materials, Quality Control, and Special Construction Techniques** **No change**

Table 3-1 (cont.)	AP1000 DCD Section 3.8 "Design of Category I Structures" Changes from AP600
DCD Subsection	**Discussion of Change from AP600**
3.8.5.7	**Testing and In-Service Inspection Requirements** **No change**
3.8.5.8	**Construction Inspection** **No change**
3.8.6	**Combined License Information** New COL information items will be added where indicated in each DCD subsection
3.8.7	**References** **No change**

4 PIPING DESIGN AND PROTECTION FROM POSTULATED RUPTURES OF PIPING

The acceptance criteria and methodology for piping are provided in Sections 3.6 and 3.9 and Appendices 3B, 3C and 3E of the AP600 DCD. These sections will be the same for AP1000, except as shown in Table 4-1. The AP1000 piping layouts are similar to the AP600 layouts. The Combined License applicant will be responsible for completing the detailed piping design, including the leak before break evaluation and the pipe rupture hazard evaluation. This detailed design will be completed by the Combined License applicant in accordance with the AP1000 ITAACs, identical to those for the AP600 illustrated in the table below, plus required information to be provided by the Combined license applicant as described below.

Extract from Tables 2.1.2-4 and 3.3-6 Inspections, Tests, Analyses, and Acceptance Criteria		
Design Commitment	**Inspections, Tests, Analyses**	**Acceptance Criteria**
Table 2.1.2-4 2.b) The piping identified in Table 2.1.2-2 as ASME Code Section III is designed and constructed in accordance with ASME Code Section III requirements.	Inspection will be conducted of the as-built piping as documented in the ASME design reports.	The ASME Code Section III design reports exist for the as-built piping identified in Table 2.1.2-2 as ASME Code Section III.
Table 2.1.2-4 6. Each of the as-built lines identified in Table 2.1.2-2 as designed for LBB meets the LBB criteria, or an evaluation is performed of the protection from the dynamic effects of a rupture of the line.	Inspection will be performed for the existence of an LBB evaluation report or an evaluation report on the protection from dynamic effects of a pipe break. Tier 1 Material, Section 3.3, Nuclear Island Buildings, contains the design descriptions and inspections, tests, analyses, and acceptance criteria for protection from the dynamic effects of pipe rupture.	An LBB evaluation report exists and concludes that the LBB acceptance criteria are met by the as-built RCS piping and piping materials, or a pipe break evaluation report exists and concludes that protection from the dynamic effects of a line break is provided.
Table 3.3-6 8. Equipment labeled as essential targets in Table 3.3-4 and located in rooms identified in Table 3.3-4 are protected from the dynamic effects of postulated pipe breaks.	An inspection will be performed of the as-built high energy pipe break pipe whip restraints features for systems located in rooms identified in Table 3.3-4.	An as-built Pipe Rupture Hazard Analysis Report exists and concludes that equipment labeled as essential targets in Table 3.3-4 and located in rooms identified in Table 3.3-4 can withstand the effects of postulated pipe rupture without loss of required safety function.

For AP600, NRC staff audited the piping analysis reports and confirmed that the commitments in the DCD were sufficient and implemented appropriately. The AP1000 piping analyses will not be available at the time of Design Certification. However, they will follow the same methodology as that already reviewed by NRC for AP600. The final AP1000 piping analyses will be documented in the ASME Code Section III design reports, which will be required by AP1000 ITAACs.

The AP600 design demonstrated applicability of leak-before-break for certain piping. This should be demonstrated prior to start of construction of the AP1000. Appendix 3B of the AP1000 DCD will show bounding curves for the increased diameter of the AP1000 piping covered by leak-before-break. The Combined License applicant will perform piping analyses of the piping covered by leak-before-break and will demonstrate that they satisfy the bounding curves included in Appendix 3B.

The AP1000 layout is substantially the same as that of the AP600. Changes from the AP600 to the AP1000 will be reviewed for their effect on the pipe rupture hazard evaluation. Assessments will be made for those changes affecting critical information in Design Certification. This will include subcompartment pressurization analyses, where the energy release or vent path changes significantly, and where adequate margin may not be available. The Combined License applicant will complete the pipe rupture hazard analysis report using as-built information as required by the current AP600 ITAAC.

Table 4-1	AP1000 DCD Section 3.6 and 3.9 Piping Design and Protection from Postulated Ruptures of Piping
DCD Section	**Discussion of Change from AP600**
3.6	**Protection Against the Dynamic Effects Associated with Postulated Rupture of Piping** Change AP600 to AP1000 throughout 3.6 and Appendices
3.6.1	**Postulated Piping Failures in Fluid Systems Inside and Outside Containment** No changes to text Table 3.6-2 to be revised to increased AP1000 piping diameters Check pipe sizes in Table 3.6-3
3.6.2	**Determination of Break Locations and Dynamic Effects Associated with the Postulated Rupture of Piping** Update FW piping diameter in subsection 3.6.2.5
3.6.3	**Leak-before-Break Evaluation Procedures** Revise 3.6.3.1, main steam velocity to approximately equal to 150 fps, instead of < 150 fps. Revise 3.6.3.4 to reference the seismic input and soil profiles in 3.7.1 and 3.7.5.5
3.6.4	**Combined License Information** **No change**
3.7.3	**Seismic Subsystem Analysis** **No change**
3.9	**Mechanical Systems and Components** Revise AP600 to AP1000 throughout section
3.9.1.1	**Design Transients** Minor changes to be included for AP1000 transients
3.9.1.2	**Computer Programs Used in Analyses** **No change**
3.9.2.1	**Piping Vibration, Thermal Expansion, and Dynamic Effects** **No change**
3.9.3	**ASME Code Classes 1, 2, and 3 Components, Component Supports, and Core Support Structures** Add requirement for Combined License applicant to make results of piping analyses for piping qualified to leak-before-break available for audit during the NRC review of the combined License application.
3.9.3.1	**Loading Combinations, Design Transients, and Stress Limits** **No change**
3.9.3.4	**Component and Piping Supports** **No change**

Table 4-1 (Cont.)	AP1000 DCD Section 3.6 and 3.9 Piping Design and Protection from Postulated Ruptures of Piping
DCD Section	**Discussion of change from AP600**
Appendix 3B	**Leak-Before-Break Evaluation Of The AP1000 Piping** Add bounding analysis curves for new pipe sizes: 38″ Main steam line, 14″ PRHR / ADS 4, 18″ PRHR / ADS 4 Confirm other bounding curves are applicable to AP1000 pressure and temperature conditions.
Appendix 3C	**Reactor Coolant Loop Analysis Methods** **No change**
Appendix 3E	**High Energy Piping In The Nuclear Island** Update pipe sizes on P&IDs

APPENDIX A

AP1000 DESIGN CERTIFICATION DOCUMENT PRELIMINARY DRAFT

This appendix includes a mark up of the AP600 DCD showing changes proposed for the AP1000. It includes Section 2.5 and subsections 3.7.1, 3.7.2, and 3.7.5. Tables and Figures for Chapter 3 are not included in the attachment. They will be revised for AP1000 parameters and results.

As described in Section 2, the AP1000 DCD will include results for a hard rock site and criteria and methodology for a soil site. Subsections 3.7.1 and 3.7.2 are marked up to show only the hard rock analyses. Criteria and methodology for soil sites are provided in subsection 3.7.5.5.

The purpose of this attachment is to show implementation of the level of detail proposed for the AP1000 DCD. At this time Westinghouse is requesting review of the approach and has not requested detail review of the technical changes.

2.5 GEOLOGY, SEISMOLOGY, AND GEOTECHNICAL ENGINEERING

Combined License applicants referencing the ~~AP600~~AP1000 certified design will address site specific information related to basic geological, seismological, and geotechnical engineering of the site and the region, as discussed in the following subsections.

2.5.1 Basic Geological and Seismic Combined License Information

Combined License applicants referencing the ~~AP600~~AP1000 certified design will address the following site-specific geologic and seismic information:

- Regional and site physiography
- Geomorphology
- Stratigraphy
- Lithology
- Structural geology
- Tectonics
- Seismicity

2.5.2 Vibratory Ground Motion

The ~~AP600~~AP1000 is designed for a safe shutdown earthquake (SSE) defined by a peak ground acceleration (PGA) of 0.30g and the design response spectra specified in subsection 3.7.1.1, Figures 3.7.1-1 and 3.7.1-2. The ~~AP600~~AP1000 design response spectra were developed using the Regulatory Guide 1.60 response spectra as the base and modified to address high frequency amplification effects observed in eastern North America earthquakes. The peak ground accelerations in the two horizontal and the vertical directions are equal.

~~The AP600 has been designed using a set of four design soil profiles described in subsection 3.7.1.4.~~

2.5.2.1 Combined License Seismic and Tectonic Characteristics Information

Combined License applicants referencing the ~~AP600~~AP1000 certified design will address the following site-specific information related to seismic and tectonic characteristics of the site and region:

- Correlation of earthquake activity with geologic structure or tectonic provinces
- Maximum earthquake potential
- Seismic wave transmission characteristics of the site
- Safe shutdown earthquake (SSE) ground response spectra

The Combined License applicant must demonstrate that the proposed site meets the following requirements:

- The free field peak ground acceleration at the finished grade level is less than or equal to a 0.30g safe shutdown earthquake.

- The site design response spectra at the finished grade level in the free-field are less than or equal to those given in Figures 3.7.1-1 and 3.7.1-2. The site specific response spectra must be developed at the finished grade elevation considering site specific soil amplification.

- The site specific response spectra at the foundation level in the free field are less than or equal to those given in Figures 3.7.1-18 and 3.7.1-19.

- ~~Foundation material layers are approximately horizontal (dip less than 20 degrees) and the shear wave velocity of the soil is greater than or equal to 1000 feet per second.~~

~~2.5.2.2 Sites With Geoscience Parameters Outside the Certified Design~~

~~If the site specific spectra at plant grade exceed the response spectra in Figures 3.7.1-1 and 3.7.1-2 at any frequency, if the site specific spectra at foundation level exceed the response spectra in Figures 3.7.1-18 and 3.7.1-19 at any frequency, or if soil conditions are outside the range evaluated for AP600 design certification, a site specific evaluation can be performed. This evaluation will consist of a site-specific dynamic analysis and generation of in-structure response spectra to be compared with the floor response spectra of the certified design at 5 percent damping. The site design response spectra at the finished grade level in the free-field given in Figures 3.7.1-1 and 3.7.1-2 were used to develop the floor response spectra. The site is acceptable for construction of the if the floor response spectra from the site-specific evaluation do not exceed the spectra for each of the locations identified below.~~

- ~~Reactor vessel support ———— Figure 3.7.2-17, Sheets 1-3~~
- ~~Containment operating floor ———— Figure 3.7.2-17, Sheets 4-6~~
- ~~Shield building roof ———— Figure 3.7.2-15, Sheets 7-9~~
- ~~Control room floor ———— Figure 3.7.2-15, Sheets 1-3~~
- ~~Coupled auxiliary roof and shield building ———— Figure 3.7.2-15, Sheets 10-12~~
- ~~Steel containment vessel at polar crane support ———— Figure 3.7.2-16, Sheets 1-3~~

~~Lateral earth pressures for a site evaluated using site specific spectra are acceptable if the lateral earth pressures from the site-specific analyses do not exceed the AP600 design values at any location. Lateral earth pressure design values are given in Table 2C-1 through 2C-4.~~

~~Site-specific soil structure interaction analyses must be performed by the Combined License applicant to demonstrate acceptability of sites that have seismic and soil characteristics outside of the site parameters in Table 2-1. These analyses would use the site specific soil conditions (including variation in soil properties in accordance with Standard Review Plan 3.7.2). The three components of the site specific ground motion time history must satisfy the enveloping criteria of Standard Review Plan 3.7.1 for the response spectrum for damping values of 2, 3, 4, 5 and 7 percent and the enveloping criterion for power spectral density function. Floor response spectra and lateral earth pressures determined from the site specific analyses should be~~

~~compared against the design basis of the described above. These evaluations and comparisons will be provided and reviewed as part of the Combined License application.~~

2.5.2.~~3~~2 Site-Specific Seismic Structures

The ~~AP600~~AP1000 includes all seismic Category I structures, systems and components in the scope of the design certification.

2.5.3 Surface Faulting Combined License Information

Combined License applicants referencing the ~~AP600~~AP1000 certified design will address surface and subsurface geological and geophysical information including the potential for surface or near-surface faulting affecting the site.

2.5.4 Stability and Uniformity of Subsurface Materials and Foundations

Combined License applicants referencing the AP1000 certified design will address the following site specific information related to the stability and uniformity of subsurface materials and foundations.

- Excavation
- Bearing Capacity
- Settlement
- Liquefaction
- Subsurface uniformity

Seismic analysis and foundation design for rock sites is described in sections 3.7 and 3.8. Seismic analysis and foundation design for soil sites is described in subsections 3.7.5.5 and 3.8.5.

2.5.4.1 Excavation

Excavation in soil for the nuclear island structures below grade will establish a vertical face with lateral support of the adjoining undisturbed soil or rock. One alternative is to use a soil nailing method. Soil nailing is a method of retaining earth in-situ. As the nuclear island excavation progresses vertically downward, holes are drilled horizontally into the adjoining undisturbed soil, a metal rod is inserted into the hole, and grout is pumped into each hole to fill the hole and to anchor the "nail" rod.

As each increment of the nuclear island excavation is completed, nominal eight to ten inch diameter holes are drilled horizontally through the vertical face of the excavation into adjacent undisturbed soil. These "nail" holes, spaced horizontally and vertically on five to six feet centers, are drilled slightly downward to the horizontal. A "nail", normally a metal bar/rod, is center located for the full length of the hole. The nominal length of soil nails are 60 percent to 70 ercent of the wall height, depending upon soil conditions. The hole is filled with grout to anchor the rod to the soil. A metal face plate is installed on the exposed end of the rod at the excavated wall vertical surface. Welded wire mesh is hung on the wall surface for wall reinforcement and secured to the soil nail face plates for anchorage. A 4,000 psi to 5,000 psi non-expansive pea gravel shotcrete mix is blown onto the wire mesh to form a nominal four to six inch thick soil retaining wall. Installation of the soil retaining wall closely follows the progress of the excavation and is from the top down, with each wire mesh-reinforced, shotcreted wall section being supported by the soil "nails" and the preceding elevations of soil nailed wall placements. The shotcrete contains a crystalline waterproofing material as described in subsection 3.4.1.1.1.

Soil nailing as a method of soil retention has been successfully used on excavations up to 55 feet deep on projects in the U.S. Soils have been retained for up to 90 feet in Europe. The state of California CALTRANS uses soil nailing extensively for excavations and soil retention installations. Soil nailing design and installation has a successful history of application which is evidenced by its excellent safety record.

The soil nailing method produces a vertical surface down to the bottom of the excavation and is used as the outside forms for the exterior walls below grade of the nuclear island. Concrete is placed directly against the vertical concrete surface of the excavation.

For excavation in rock and for methods of soil retention other than soil nailing, four to six inches of shotcrete are blown on to the vertical surface. The concrete for the exterior walls is placed against the shotcrete. The shotcrete contains a crystalline waterproofing material as described in subsection 3.4.1.1.1.

2.5.4.2 Bearing Capacity

The average bearing reaction of the ~~AP600~~AP1000 is about 8,2~~0~~00 pounds per square foot. The minimum average allowable static soil bearing capacity is 8,2~~0~~00 pounds per square foot over the footprint of the nuclear island at its excavation depth (see Table 2-1). ~~Net allowable static bearing capacities have been computed for the design soil profiles as shown in Table 2-2. Capacities are calculated using bearing capacity equations in Terzaghi and Peck (Reference 1), for both cohesive and cohesionless soils (both dry and saturated cases).~~

~~For cohesive soils, an estimate for undrained shear strength~~ $\cancel{(S_u)}$ ~~was made by using the relationship between low-strain shear modulus~~ $\cancel{(G_{max})}$ ~~and undrained shear strengths. The shear modulus was obtained from the shear wave velocity profiles at a depth of approximately 90 feet. This corresponds to a depth of D+B/2 (Depth, D = 40 feet; Width, B = 104 feet, average) which accounts for the zone of influence under the nuclear island basemat. The water table has been shown to have no effect on the bearing capacity of mats on cohesive soils. For cohesionless soils, relative density and friction angle were calculated from their relationships with shear wave velocity and low strain shear modulus. Location of the ground water table significantly influences the bearing strength of cohesionless soils. In determining the bearing strengths, the ground water table was assumed to be at grade. For the rock profiles, the bearing strengths shown are based on the rock quality designation in accordance with Peck et al. (Reference 2).~~

~~In general, higher bearing capacities are associated with more competent soil profiles. The bearing capacities provided in Table 2-2 are preliminary estimates for static loading conditions~~

~~only.~~ 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.

~~Generally, once the static bearing capacity at a given site is adequate, the dynamic bearing demand will also be satisfied. The maximum bearing stress due to the dead load, live load, and safe shutdown earthquake is presented in subsection 3.8.5.5.1 for the worst combination of site and soil conditions. The Combined License applicant may either use these loads to demonstrate soil bearing acceptability or may perform site-specific seismic analyses to develop bearing loads applicable to the site and seismic conditions using the methods outlined in subsection 2.5.2.2.~~

2.5.4.3 Settlement

The Combined License applicant will address ~~S~~short-term (elastic) and long-term (heave and consolidation) settlement for ~~limiting cases of deep soft~~ soil sites ~~are evaluated~~ 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. ~~The basemat and structure are analyzed at various stages of construction as described in subsection 3.8.5.~~

~~The settlement analysis utilizes the one-dimensional consolidation theory in which excess pore pressure is dissipated consistent with the site consolidation parameters such as the initial void ratio, compression and recompression index and the coefficient of consolidation. The limiting cases of deep soft soil sites comprised of compressible soils are represented by subsurface profiles consisting of compressible clay deposits extending down to a depth of 360 feet underlying a 40-foot layer of sand at the surface. The evaluation considers two profiles. One profile has alternate layers of sand and clay and the second profile consists of only clay. Profile 1 maximizes settlements in the early stages of construction while profile 2 maximizes settlement during the later stages of construction and during the operational period of the plant. The elastic properties for the soils are consistent with the minimum shear wave velocity of Table 2-1 and the expected soil strains due to construction loads. The clay is assumed to be normally consolidated and the water table is assumed to be at grade.~~

~~The analysis considers the effects of dewatering and excavation, the history of construction loading, elastic deformation and consolidation of the subsurface soils, and the effect of the progressive stiffness of the structure. For the limiting deep soft soil sites examined, the maximum estimated settlement after placement of first concrete for the basemat is 4.5 inches for the postulated alternating sand and clay site and 14 inches for the all clay site.~~

The ~~AP600~~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.

2.5.4.4 Liquefaction

The Combined License applicant will demonstrate that t~~T~~he potential for liquefaction is negligible. ~~was evaluated for the soft soil and the soft-to-medium parabolic soil profiles. In this evaluation, the profiles were assumed to be of clean sand deposits with the water table at ground level. The cyclic shear stresses generated by the safe shutdown earthquake were evaluated against the cyclic shear strengths calculated in accordance with Seed's liquefaction chart (Reference 4). These strengths were estimated using normalized blow count values representative of the shear wave velocities. The evaluation indicated that the soft profile with clean sand deposits may be susceptible to liquefaction under the generic safe shutdown earthquake. However, other factors, such as the age of the deposit or the silt and clay content, can significantly increase the resistance to liquefaction. Such sites would require detailed site-specific investigation. The soft-to-medium parabolic soil profile and any firmer soil profiles are not susceptible to liquefaction.~~

2.5.4.5 Subsurface Uniformity

The Combined License applicant will address the effects on s~~S~~oil structure interaction and foundation design ~~are a function~~ of the uniformity of the soil or rock below the foundation. ~~Although the AP600 design and analysis of the AP600 is based on soil or rock conditions with uniform properties within horizontal layers, it includes provisions and design margins to accommodate many non-uniform sites. This subsection identifies the requirements for site investigation that may be used to demonstrate that:~~

- ~~A site is "uniform" based on the criteria outlined in subsection 2.5.4.5.3. If the site can be demonstrated to be "uniform" no further site specific analysis is required to qualify the site for the AP600.~~

- ~~A "non-uniform" site is acceptable to locate the AP600 based on the criteria for acceptability outlined in subsection 2.5.4.5.3. Some non-uniform sites are acceptable as described in subsection 2.5.4.5.3 based on evaluation performed as part of design certification. Other non-uniform sites may be shown to be acceptable as described in subsection 2.5.4.5.3.1 using site specific evaluation as part of the Combined License application.~~

Considerations with respect to the materials underlying the nuclear island are the type of site, such as rock or soil, and whether the site can be considered uniform. If the site is nonuniform, the nonuniform soil characteristics such as the location and profiles of soft and hard spots should be considered. These considerations can be assessed with the information developed in response to Regulatory Guides 1.132 and 1.138. The geological investigations of subsections 2.5.1 and 2.5.4.6.1 provide information on the uniformity of the site~~, whether it may be geologically impacted, and whether the bedrock may be sloping or undulatory~~.

~~Appendix 2A presents a~~ A survey of 22 commercial nuclear power plant sites in the United States~~. This survey~~ focused on site parameters that affect the seismic response such as the depth to bedrock, type and characteristic of the soil layers, including the variation of shear wave velocities, the depth to the ground water level, and the embedment depth of the plant structures. Of the 22 sites, 11 are rock sites where competent rock exists at relatively shallow depths. At the other sites, the depth to bedrock varies from about 50 feet (Callaway) to well in excess of 4,000 feet (South Texas). A review of these 11 soil sites, all of which are marine, deltaic, or lacustrine deposits, did not reveal any significant variation of soil characteristics below the nuclear island footprint. There was one possible nonuniform site, Monticello, which is underlain by glacial deposits; the geologic description is such that there might be lateral variability in the foundation parameters within the plan dimension of the plant. The review of the 22 commercial nuclear power plant sites in the United States suggests that the majority of ~~AP600~~AP1000 sites exhibit "uniform" soil properties within the nuclear island footprint.

2.5.4.5.1 Site Investigation for Uniform Sites

For sites that are expected to be uniform, based on the geologic investigation outlined in subsections 2.5.1 and 2.5.4.6.2, Appendix C to Regulatory Guide 1.132 provides guidance on the spacing and depth of borings of the geotechnical investigation for safety-related structures. Specific language in the Regulatory Guide suggests a spacing of 100 feet supplemented with borings on the periphery and at the corners for favorable, uniform geologic conditions.

For foundation engineering purposes, a series of primary borings should be drilled on a grid pattern that encompasses the nuclear island footprint and 40 feet beyond the boundaries of the nuclear island footprint. The 40-foot extension for the grid of borings is established from a Boussinesq analysis of the zone of influence of the foundation mat which shows that the net change in the effective vertical overburden stress is less than seven percent at a distance of 40-feet from the edge of the foundation mat. The grid need not be of equal spacing in the two orthogonal directions, but it should be oriented in accordance with the true dip and strike of the rock in the immediate area of the nuclear island footprint. If geologic conditions are such that true dip and strike are not obvious, or if the dip is practically flat, then the orientation of the grid can be consistent with the major orthogonal lines of the nuclear island. The spacing of the borings on the grid should be on the order of 50 to 60 feet. For example, an acceptable grid could have 5 borings in the short direction and 7 borings in the long direction, resulting in 35 primary borings to cover the nuclear island footprint and 40 feet beyond. The depth of borings should be determined on the basis of the geologic conditions. Borings should be extended to a depth sufficient to define the site geology and to sample materials that may swell during excavation, may consolidate subsequent to construction, may be unstable under earthquake loading, or whose physical properties would affect foundation behavior or stability. At least one-fourth of the primary borings should penetrate sound rock or, for a deep soil site, to a maximum depth of 250 feet below the foundation mat. At this depth of 250 feet the change in the vertical stress during or after construction for the combined foundation loading is less than 10 percent of the in-situ effective overburden stress. Other primary borings may terminate at a depth of 160 feet below the foundation (equal to the width of the structure).

2.5.4.5.2 Site Investigation for Non-uniform Sites

At sites that are determined to be non-uniform or potentially non-uniform during the course of the geological investigations outlined in subsections 2.5.1 and 2.5.4.6.2, the investigation effort is extended to determine if the site is acceptable for an ~~AP600~~AP1000. The following paragraphs identify the site geotechnical investigations required to demonstrate that the site is acceptable.

As the ~~AP600~~AP1000 foundation/structural system is robust, the probability of being able to show compliance for all but the worst of sites is high, unless liquefaction or faulting is prevalent on the site. As stated in Regulatory Guide 1.132, where variable conditions are found, spacing of boreholes should be smaller, as needed, to obtain a clear picture of soil or rock properties and their variability. Where cavities or other discontinuities of engineering significance may occur, the normal exploratory work should be supplemented by secondary borings or soundings at a spacing small enough to detect such features. The depth of the secondary borings is 160 feet below the foundation mat. At this depth, the maximum change in vertical stress during or after construction is about 11 percent of the in-situ effective overburden stress. The depth of borings should be extended beyond 160 feet if the geologic investigation indicates the possible presence of karst conditions, under-consolidated clays, loose sands, intrusive dikes or other forms of geologic impacts at depth greater than 160 feet.

To provide guidance for the site investigation of non-uniform sites, three non-uniform cases are described that might occur for nuclear plants. For each of these cases, the type of site investigation is described.

Sloping Bedrock Site

The sloping bedrock site as shown on Figure 2.5-2 is typical for a river front site where in the geologic past the bedrock has been eroded to a valley slope and then the valley was subsequently filled with alluvium. The bedding in the rock is nearly horizontal, but the surface of the rock is sloping on a strike parallel to the direction of the river. The shear wave velocity of the uniform soil layer overlying rock may vary between 1,000 and 2,500 feet per second. The shear wave velocity of 3,500 feet per second for the bedrock is representative of sites with a sloping rock surface. Sites where the bedrock has much higher shear wave velocities are not likely to exhibit such conditions.

Investigations for a site with a sloping bedrock surface must define the depth to bedrock as a function of plan location and the shear wave velocity of the overlying soil and bedrock. More borings may be necessary than required for a uniform site in order to establish the variation in depth to bedrock within the nuclear island footprint.

Undulatory Bedrock Site

An undulatory bedrock site as shown in Figure 2.5-3 is one where the bedding planes in the bedrock are (or nearly) horizontal but the surface is undulatory. Such a situation may occur if the bedrock surface is an erosion surface in a marine or lake environment. Another example might be a limestone site overlain by saprolite as in the southeast United States. The undulations could be the result of differential weathering or by soft zones associated with solution activity in the limestone.

Investigations for a site with an undulatory bedrock surface associated with weathering or karst condition must define the depth to bedrock as a function of plan location and the shear wave velocity of the overlying soil and bedrock. For cases with the overlying soil layer between the foundation level and the bedrock less than 40 feet, the pattern dimensions of the undulations must be defined with borings, specifically the width and depth of the undulations. Boring spacing on the order of 10 feet may be required for undulations having dimensions on the order of 20 feet in order to establish the variation in depth to bedrock within the nuclear island footprint.

Geologically Impacted Site

A geologically impacted site as shown on Figure 2.5-4 is one where the bedrock has abrupt facies change or has been interrupted either by a fault (shear zone) or by an intrusive such as a dike. This leads to the possibility of lateral variation in the bedrock properties affecting soil structure interaction and bearing pressure. Three subcases are identified. The first type includes an abrupt facies change. The second type has a shear zone of varying width and position. The third case is an intrusive dike of very competent rock compared to the surrounding rock.

Investigations for a geologically impacted site must define the width of the zone of the higher (or lower) shear wave velocity. The location of the zone of higher (or lower) shear wave velocity must be determined in relation to the center of containment. The azimuths of the bounding postulated vertical planes of the higher (or lower) shear wave velocity must be determined.

The zone of the higher (or lower) shear wave velocity is shown in Figure 2.5-4 bounded by non-curvilinear vertical parallel planes. It is recognized that such a situation is highly unlikely in nature. In order to define the width and location of the zone of higher (or lower) shear wave velocity, the spacing of the borings will have to be on the order of 10 feet for a zone with a width of 20 feet. It may be more practical to trench the site to locate and define the dimensions and locations of the intrusive or shear zone, thus eliminating many of the borings that would otherwise be required.

~~**2.5.4.5.3 Site Foundation Material Evaluation Criteria**~~

~~The AP600 is designed for application at a site where the foundation conditions do not have extreme variation within the nuclear island footprint. This subsection provides criteria for evaluation of soil variability.~~

~~The subsurface may consist of layers and these layers may dip with respect to the horizontal. If the dip is less than 20 degrees, the generic analysis using horizontal layers is applicable as described in NUREG CR-0693 (Reference 6). The physical properties of the foundation medium may or may not vary systematically across a horizontal plane. The recommended methodology~~

~~for checking uniformity is to calculate from the boring logs a series of "best estimate" planes beneath the nuclear island footprint that define the top (and bottom) of each layer. The planes could represent stratigraphic boundaries, lithologic changes, unconformities, but most important, they should represent boundaries between layers having different shear wave velocities. Shear wave velocity is the primary property used for defining uniformity of a site.~~

~~The distribution of bearing reactions under the basemat is a function of the subgrade modulus which in turn is a function of the shear wave velocity. The Combined License applicant shall demonstrate that the variation of subgrade modulus or shear wave velocity across the footprint is within the range considered for design of the nuclear island basemat. The farther that the non-uniform layer is located below the foundation, the less influence it has on the bearing pressures at the basemat. Lateral variability of the shear wave velocity at depths greater than 120 feet below grade (80 feet below the foundation) do not significantly affect the subgrade modulus.~~

~~If a site can be classified as uniform, it qualifies for the AP600 based on analyses and evaluations performed to support design certification without additional site specific analyses. For a site to be considered uniform, the variation of shear wave velocity in the material below the foundation to a depth of 120 feet below finished grade within the nuclear island footprint shall meet the criteria outlined below:~~

- ~~The depth to a given layer indicated on each boring log may not fall precisely on the postulated "best estimate" plane. The deviation of the observed layers from the "best-estimate" planes should not exceed 5 percent of the observed depths from the ground surface to the plane. If the deviation is greater than 5 percent, additional planes may be appropriate or additional borings may be required, thereby diminishing the spacing.~~

- ~~For a layer with a low strain shear wave velocity greater than or equal to 2500 feet per second, the layer should have approximately uniform thickness, should have a dip no greater than 20 degrees and the shear wave velocity at any location within any layer should not vary from the average velocity within the layer by more than 20 percent.~~

- ~~For a layer with a low strain shear wave velocity less than 2500 feet per second, the layer should have approximately uniform thickness, should have a dip no greater than 20 degrees and the shear wave velocity at any location within any layer should not vary from the average velocity within the layer by more than 10 percent.~~

~~2.5.4.5.3.1 Site-Specific Subsurface Uniformity Design Basis~~

~~Many sites that do not meet the above criteria for a uniform site are acceptable for the AP600. The key attribute for acceptability of the site for an AP600 is the bearing pressure on the underside of the basemat. A site having local soft or hard spots within a layer or layers does not meet the criteria for a uniform site. Non-uniform soil conditions may also require evaluation of the AP600 seismic response as described in subsection 2.5.2.2.~~

~~As described in subsection 3.8.5 the nuclear island foundation is designed specifically for bearing pressures of 120 percent of those of the uniform soil properties case. Evaluation criteria are defined to evaluate sites that do not satisfy the site parameters directly. The design basis provided below is included to provide a clear specification of the design commitment and evaluation criteria required to demonstrate that a site specific application satisfies AP600 requirements. Application of the AP600 to sites using this site-specific evaluation is not approved as part of the AP600 design certification and the evaluation should be provided and reviewed as part of the Combined License application.~~

~~Rigid Basemat Evaluation~~

~~A site with nonuniform soil properties may be demonstrated to be acceptable by evaluation of the bearing pressures on the underside of a rigid rectangular basemat equivalent to the nuclear island. Bearing pressures are calculated for dead and safe shutdown earthquake loads. The safe shutdown earthquake loads used for the evaluation are associated with one of the AP600 design soil cases evaluated for design certification. The soil case representative of the site-specific soil is used. For the site to be acceptable, the bearing pressures from this analysis need to be less than or equal to 120 percent of the bearing pressures calculated in similar analyses for a site having uniform soil properties.~~

~~Alternatively, the safe shutdown earthquake loads may be determined from a site-specific seismic analysis of the nuclear island using site specific inputs as described in subsection 2.5.2.2. For the site to be acceptable, the bearing pressures from the site-specific analyses need to be less than or equal to 120 percent of the bearing pressures calculated in rigid basemat analyses using the AP600 design ground motion at a site having uniform soil properties.~~

~~This evaluation method shows acceptability for geologically impacted sites where there is a sufficient soil layer between the foundation level and the abrupt stiffness change of the bedrock.~~

~~Flexible Basemat Evaluation~~

~~For sites having bedrock close to the foundation level the assumption of a rigid basemat may be overly conservative because local deformation of the basemat will reduce the effect of local soil variability. For such sites, a site-specific analysis may be performed using the AP600 basemat model and methodology described in subsection 3.8.5. The safe shutdown earthquake loads are those from the AP600 design soil case representative of the site-specific soil. Alternatively, bearing pressures may be determined from a site-specific soil structure interaction analysis using site specific inputs as described in subsection 2.5.2.2. For the site to be acceptable the bearing pressures from the site-specific analyses including static and dynamic loads need to be less than the capacity of each portion of the basemat.~~

2.5.4.6 Combined License Information

Combined License applicants referencing the ~~AP600~~AP1000 design will address the following site specific information related to the geotechnical engineering aspects of the site. No further action is required for sites within the bounds of the site parameters.

2.5.4.6.1 Site and Structures – Site–specific information regarding the underlying site conditions and geologic features will be addressed. This information will include site topographical features, as well as the locations of seismic Category I structures.

2.5.4.6.2 The Combined License applicant will establish the properties of ~~demonstrate that~~ the foundation soils to be ~~are within the range~~ considered for design of the nuclear island basemat. ~~The design basis for sites that require a site specific analysis is defined in subsection 2.5.2.2.~~

Properties of Underlying Materials – A determination of the static and dynamic engineering properties of foundation soils and rocks in the site area will be addressed. This information will include a discussion of the type, quantity, extent, and purpose of field explorations, as well as logs of borings and test pits. Results of field plate load tests, field permeability tests, and other special field tests (e.g., bore–hole extensometer or pressuremeter tests) will also be provided. Results of geophysical surveys will be presented in tables and profiles. Data will be provided pertaining to site–specific soil layers (including their thicknesses, densities, moduli, and Poisson's ratios) between the basemat and the underlying rock stratum. Plot plans and profiles of site explorations will be provided.

Laboratory Investigations of Underlying Materials – Information about the number and type of laboratory tests and the location of samples used to investigate underlying materials will be provided. Discussion of the results of laboratory tests on disturbed and undisturbed soil and rock samples obtained from field investigations will be provided.

2.5.4.6.3 Excavation and Backfill – Information concerning the extent (horizontal and vertical) of seismic Category I excavations, fills, and slopes, if any will be addressed. The sources, quantities, and static and dynamic engineering properties of borrow materials will be described in the site–specific application. The compaction requirements, results of field compaction tests, and fill material properties (such as moisture content, density, permeability, compressibility, and gradation) will also be provided. Information will be provided concerning the specific soil retention system, for example, the soil nailing system, including the length and size of the soil nails, which is based on actual soil conditions and applied construction surcharge loads. Information will also be provided on the waterproofing system along the vertical face and the mudmat.

2.5.4.6.4 Ground Water Conditions – Groundwater conditions will be described relative to the foundation stability of the safety–related structures at the site. The soil properties of the various layers under possible groundwater conditions during the life of the plant will be compared to the range of values assumed in the standard design in Table 2-1.

2.5.4.6.5 Response of Soil and Rock to Dynamic Loading – The Combined license applicant will establish ~~T~~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. ~~The parametric analyses described in Appendices 2A and 2B cover a broad range of dynamic characteristics appropriate for most soil types (sand, silts, clays, gravels, and various combinations). The shear wave velocity (based on low strain best estimate soil properties) must be greater than or equal to 1000 feet per second.~~

2.5.4.6.6 Liquefaction Potential – Soils under and around seismic Category I structures will be evaluated for liquefaction potential for the site specific SSE ground motion. This should include justification of the selection of the soil properties, as well as the magnitude, duration, and number of excitation cycles of the earthquake used in the liquefaction potential evaluation (e.g., laboratory tests, field tests, and published data). Liquefaction potential will also be evaluated to address seismic margin.

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 SSAR. The Combined License applicant will verify that the dynamic site-specific bearing capacity is equal or greater than the seismic bearing demand.

2.5.4.6.8 Earth Pressures – The Combined License applicant will describe the ~~AP600 is~~ design~~ed~~ for static and dynamic lateral earth pressures and hydrostatic groundwater pressures acting on plant safety–related facilities using soil parameters as evaluated in previous subsections. ~~No additional information is required on earth pressures.~~

2.5.4.6.9 Soil Properties for Seismic Analysis of Buried Pipes – The ~~AP600~~AP1000 does not utilize safety related buried piping. No additional information is required on soil properties.

2.5.4.6.10 Static and Dynamic Stability of Facilities – Soil characteristics affecting the stability of the nuclear island will be addressed including foundation rebound, settlement, and differential settlement.

2.5.4.6.11 Subsurface Instrumentation – Data will be provided on instrumentation, if any, proposed for monitoring the performance of the foundations of the nuclear island. This will specify the type, location, and purpose of each instrument, as well as significant details of installation methods. The location and installation procedures for permanent benchmarks and markers for monitoring the settlement will be addressed.

2.5.5 Combined License Information for Stability of Slopes

Combined License applicants referencing the ~~AP600~~AP1000 design will address site-specific information about the static and dynamic stability of soil and rock slopes, the failure of which could adversely affect the Nuclear Island.

2.5.6 Combined License Information for Embankments and Dams

Combined License applicants referencing the ~~AP600~~AP1000 design will address site-specific information about the static and dynamic stability of embankments and dams, the failure of which could adversely affect the Nuclear Island.

2.5.7 References

~~1. Terzaghi, K. and Peck, R.B., "Soil Mechanics in Engineering Practice," 2nd Edition, John Wiley & Sons, New York, 1967.~~

2. ~~Peck, R.B., Hanson, W.E., and Thornburn, T.H., "Foundation Engineering," John Wiley & Sons, New York, 1974.~~

3. ~~Harr, M.E.,"Foundations of Theoretical Soil Mechanics," McGraw-Hill Book Co., New York, 1966.~~

4. ~~Seed, H.B., "Soil Liquefaction and Cyclic Mobility Evaluation for Level Ground During Earthquakes," Journal of Geotechnical Engineering Division, ASCE, Vol. 105, GT2, February 1979.~~

5. ~~NUREG/CR-5956, "Consideration of Uncertainties in Soil-Structure Interaction Computations," December 1992~~

6. ~~NUREG/CR-0693, "Seismic Input and Soil Structure Interaction," February, 1979.~~

Table 2-1 (Sheet 1 of 2)

SITE PARAMETERS

Air Temperature	
Maximum Safety [a]	115 ° F dry bulb/80°F coincident wet bulb 81°F wet bulb (noncoincident)
Minimum Safety [a]	-40°F
Maximum Normal [b]	100°F dry bulb/77°F coincident wet bulb 80°F wet bulb (noncoincident) [d]
Minimum Normal [b]	-10°F
Wind Speed	
Operating Basis	110 mph; importance factor 1.11 (safety), 1.0 (nonsafety)
Tornado	300 mph
Seismic	
SSE	0.30g peak ground acceleration [c]
Fault Displacement Potential	None
Soil	
Average allowable static soil bearing capacity	Greater than or equal to 8,000 pounds per square foot over the footprint of the nuclear island at its excavation depth.
Lateral variability	Soils supporting the nuclear island should not have extreme variations in subgrade stiffness ~~Case 1: For a layer with a low strain shear wave velocity greater than or equal to 2500 feet per second, the layer should have approximately uniform thickness, should have a dip not greater than 20 degrees, and should have less than 20 percent variation in the shear wave velocity from the average velocity within any layer.~~ ~~Case 2: For a layer with a low strain shear wave velocity less than 2500 feet per second, the layer should have approximately uniform thickness, should have a dip not greater than 20 degrees, and should have less than 10 percent variation in the shear wave velocity from the average velocity within any layer.~~ ~~(see subsection 2.5.4.5)~~
Shear Wave Velocity	Greater than or equal to 1000°ft/sec based on low strain best estimate soil properties
Liquefaction Potential	None

Table 2-1 (Sheet 2 of 2)

SITE PARAMETERS

Missiles	
Tornado	4000 - lb automobile at 105 mph horizontal, 74 mph vertical 275 - lb, 8 in. shell at 105 mph horizontal, 74 mph vertical 1 inch diameter steel ball at 105 mph horizontal and vertical
Flood Level	Less than plant elevation 100′
Ground Water Level	Less than plant elevation 98′
Plant Grade Elevation	Less than plant elevation 100′ except for portion at a higher elevation adjacent to the annex building
Precipitation	
Rain	19.4 in./hr (6.3 in./5 min)
Snow/Ice	75 pounds per square foot on ground with exposure factor of 1.0 and importance factors of 1.2 (safety) and 1.0 (non-safety)
Atmospheric Dispersion Values - χ/Q	
Site boundary (0-2 hr)	$\leq 1.0 \times 10^{-3}$ sec/m^3
Site boundary (annual average)	$\leq 2.0 \times 10^{-5}$ sec/m^3
Low population zone boundary	
0 - 8 hr	$\leq 1.35 \times 10^{-4}$ sec/m^3
8 - 24 hr	$\leq 1.0 \times 10^{-4}$ sec/m^3
24 - 96 hr	$\leq 5.4 \times 10^{-5}$ sec/m^3
96 - 720 hr	$\leq 2.2 \times 10^{-5}$ sec/m^3
Population Distribution	
Exclusion area (site)	0.5 mi

Notes:

(a) Maximum and minimum safety values are based on historical data and exclude peaks of less than 2 hours duration.

(b) Maximum and minimum normal values are the 1 percent exceedance magnitudes.

(c) With ground response spectra (at plant grade) less than or equal to those given in Figures 3.7.1-1 and 3.7.1-2, and with ground response spectra at the plant foundation level (40 feet below the plant grade level) less than or equal to those given in Figures 3.7.1-18 and 3.7.1-19.

(d) The noncoincident wet bulb temperature is applicable to the cooling tower only.

~~Table 2-2~~

~~NET ALLOWABLE STATIC BEARING CAPACITIES~~
~~(KIPS PER SQUARE FOOT)~~

~~Soil Shear Wave Velocity Profile~~	~~Cohesive Soil~~		~~Cohesionless Soil~~			
	~~40 feet below grade~~	~~At grade~~	~~40 feet below grade~~		~~At grade~~	
			~~Dry~~	~~Submerged~~	~~Dry~~	~~Submerged~~
~~Soft Soil~~	~~7~~	~~6.8~~	~~70.3~~	~~32.2~~	~~35.1~~	~~16.1~~
~~Soft to Medium—Linear~~	~~18.9~~	~~12~~	~~102~~	~~46.6~~	~~55.8~~	~~25.6~~
~~Soft to Medium—Parabolic~~	~~32~~	~~24~~	~~139~~	~~63.8~~	~~79.7~~	~~36.5~~
~~Upper Bound, Soft to Medium—Parabolic~~	~~60~~	~~50~~	~~265~~	~~121.3~~	~~159.3~~	~~73~~
~~Soft Rock~~				~~>220~~		
~~Hard Rock~~				~~>450~~		

Figure 2.5-1 DELETED

<u>Revise Figures 2.5-2, 2.5-3 and 2.5-4 for increase in height of AP1000</u>

3.7 SEISMIC DESIGN

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

Each plant structure, system, equipment, and component is classified in an applicable seismic category depending on its function. A three-level seismic classification system is used for the ~~AP600~~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 ~~AP600~~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 ~~AP600~~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 ~~generic~~ plant site are discussed in Section 2.5. ~~Qualification of a site where the soil characteristics are outside the range of the generic site interface is discussed in subsection 2.5.4.~~

The peak ground acceleration of the safe shutdown earthquake has been established as 0.30g for the ~~AP600~~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 ~~AP600~~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 ~~AP600~~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 ~~AP600~~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 ~~AP600~~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 ~~AP600~~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 finished grade in the free field.

3.7.1.2 Design Time History

A "single" set of three mutually orthogonal, statistically independent, synthetic acceleration time histories is used as the input in the dynamic analysis of seismic Category I structures. Site specific time histories may be used as defined in subsection 2.5.4.5.5. The synthetic time histories were generated by modifying a set of actual recorded "TAFT" earthquake time histories. The design time histories include a total time duration equal to 20 seconds and a corresponding stationary phase, strong motion duration greater than 6 seconds. The acceleration, velocity, and displacement time-history plots for the three orthogonal earthquake components, "H1," "H2," and "V," are presented in Figures 3.7.1-3, 3.7.1-4, and 3.7.1-5. Design horizontal time history, H1, is applied in the north-south (Global X or 1) direction; design horizontal time history, H2, is applied in the east-west (global Y or 2) direction; and design vertical time history is applied in the vertical (global Z or 3) direction. The cross-correlation coefficients between the three components of the design time histories are as follows:

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

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

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

The ground motion time histories (H1, H2, and V) are generated with time step size of 0.010 second for applications in soil structure interaction analyses. For applications in the fixed-base mode superposition time-history analyses, the time step size is reduced to 0.005 second by linear interpolation. The cutoff frequency used in the horizontal and vertical seismic analysis of the nuclear island for the hard rock site is 33 hertz. The cutoff frequencies used in the soil structure interaction analyses are 33 hertz for the soft rock site, and 15 hertz horizontal and 21 hertz vertical for the soft-to-medium soil site and 20 hertz horizontal and 33 hertz vertical for the upper bound soft-to-medium soil site. The maximum "cut-off" frequency for the soil structure interaction analyses and the fixed-base analyses is well within the Nyquist frequency limit.

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

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

The NUREG/CR-5347 procedures involve ad hoc hybridization of two earlier power spectral density envelopes. Since the modification to the RG 1.60 design spectra adopted for ~~AP600~~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 ~~AP600~~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 ~~AP600~~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 ~~AP600~~AP1000 Minimum Power Spectral Density is presented in Figure 3.7.1-9. This ~~AP600~~AP1000 target power spectral density is compatible with the ~~AP600~~AP1000 horizontal design response spectra and envelops a target power spectral density compatible with the ~~AP600~~AP1000 vertical design response spectra. This ~~AP600~~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 ~~AP600~~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 4-86 (Reference 3), and 5 percent damping for piping, except for the damping value of the primary coolant loop piping, which is based on Reference 22, and conduits, cable trays and their related supports.

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

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

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

where:

β_n	= ratio of critical damping for mode n
nc	= number of elements
$\{\phi_n\}$	= mode n (eigenvector)
$[K_t]_i$	= stiffness matrix of element i
β_i	= ratio of critical damping associated with element i
$[K_t]$	= total system stiffness matrix

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

3.7.1.4 Supporting Media for Seismic Category I Structures

~~The seismic design basis for the AP600~~AP1000 ~~is to provide design coverage for as many plant sites as practical.~~ The supporting media will be described by the Combined License applicant. Seismic analyses for a rock site are described in Section 3.7.2. Seismic analyses for soil sites are described in subsection 3.7.5.5. ~~For the design of seismic Category I structures, a set of four design soil profiles of various shear wave velocities is established in Appendices 2A and 2B. The four design soil profiles include a hard rock site, a soft rock site, an upper bound soft-to-medium soil site and a soft-to-medium soil site. The shear wave velocity profiles and related governing parameters of the four sites considered are the following:~~

- ~~For the hard rock site, an upper bound case for firm sites using fixed base seismic analysis.~~
- ~~For the soft rock site, a shear wave velocity of 2400 feet per second at the ground surface, increasing linearly to 3200 feet per second at a depth of 240 feet, and base rock at the depth of 120 feet.~~
- ~~For the soft-to-medium soil site, a shear wave velocity of 1000 feet per second at ground surface, increasing parabolically to 2400 feet per second at 240 feet, base rock at the depth of 120 feet, and ground water is assumed at grade level.~~
- ~~For the upper bound soft-to-medium soil site, a shear wave velocity of 1414 feet per second at ground surface, increasing parabolically to 3394 feet per second at 240 feet, base rock at the depth of 120 feet, and ground water is assumed at grade level. The initial soil shear modulus profile is twice that of the soft-to-medium soil site.~~

The ~~AP600~~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.

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

3.7.2 Seismic System Analysis

Seismic Category I structures, systems, and components are classified according to Regulatory Guide 1.29. Seismic Category I building structures of ~~AP600~~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.

~~Separate~~Fixed base seismic analyses are performed for the nuclear island at a rock site~~, one for each of the four design soil profiles defined in subsection 3.7.1.4~~. The analyses generate ~~one~~ a set of in-structure responses~~ for each of the design soil profiles. The four sets of in-structure seismic responses are enveloped to obtain the seismic design envelope~~ (design member forces, nodal accelerations, nodal displacements, and floor response spectra) 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 ~~soil-structure interaction~~ fixed base analyses, and the results thereof. A separate report provides the floor response spectra for the nuclear island.

3.7.2.1 Seismic Analysis Methods

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

Seismic analyses, using the response spectrum method, and the mode superposition time-history method, ~~and the complex frequency response analysis method,~~are performed for the safe shutdown earthquake to determine the seismic force distribution for use in the design of the nuclear island structures, and to develop in-structure seismic responses (accelerations, displacements, and floor response spectra) for use in the analysis and design of seismic subsystems.

3.7.2.1.1 Response Spectrum Analysis

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

Coupled Shield and Auxiliary Buildings on Fixed Base

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

Response spectrum analyses are performed for the hard rock site~~ where the soil-structure interaction effect is negligible, as described in Appendix 2B. Response spectrum analyses are performed~~ using the fixed-base, three-dimensional, finite element models. The support provided by the embedment below grade is not considered in these response spectrum analyses.

~~**Coupled Shield and Auxiliary Buildings on Flexible Base**~~

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

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

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

Containment Internal Structures

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

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

3.7.2.1.2 Time-History Analysis ~~and Complex Frequency Response Analysis~~

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

The three-dimensional, lumped-mass stick models of the nuclear island structures developed as described in subsection 3.7.2.3 are used ~~in conjunction with the design soil profiles presented in subsection 3.7.1.4~~ to obtain the in-structure responses. The lumped-mass stick models of the nuclear island structures are presented in Figure 3.7.2-4 for the coupled shield and auxiliary buildings, in Figure 3.7.2-5 for the steel containment vessel, in Figure 3.7.2-6 for the containment internal structures, and in Figure 3.7.2-7 for the reactor coolant loop model. The individual building lumped-mass stick models are interconnected with rigid links~~stiff beam elements~~ to form the overall dynamic model of the nuclear island. ~~The nuclear island basemat and the periphery walls of the embedded portion of the nuclear island are represented by a three-dimensional, finite element model, as shown in Figure 3.7.2-8.~~

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

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

~~Seismic responses of the nuclear island structures for the various design soil profiles are enveloped and the resulting response spectra are used in the design and analysis for most of the seismic subsystems. Certain subsystems, as described in subsection 3.7.3.6, are analyzed using~~

~~the time histories obtained from a series of soil-specific analyses for the design soil profiles presented in subsection 3.7.1.4.~~

3.7.2.2 Natural Frequencies and Response Loads

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

The time history seismic analysis of the nuclear island considers 200~~70~~ vibration modes, extending up to a~~the~~ frequency ~~limit~~ of 118.6~~33~~ hertz, shown in Table 3.7.2-4. The total cumulative mass participating in the seismic response constitute more than 97~~88, 89, and 88~~ percent of the total mass, excluding the building mass within the embedded portion of the nuclear island.

~~Table 3.7.2-3, sheet 1, demonstrates the large stiffness of the containment internal structures. The table shows, for frequencies up to 33 hertz, a total cumulative mass of 32 percent in the north-south direction, 29 percent in the east-west direction, and negligible amount in the vertical direction. For frequencies up to 60 hertz, the table shows the total cumulative mass increased to 83, 83, and 41 percent in the three respective directions. Because of the high frequency modal participation, the seismic force and moment responses of the containment internal structures are determined from a response spectrum analysis of the fixed-base nuclear island lumped-mass stick model. The response spectrum analysis considers 70 vibration modes, up to 33 hertz, using the double sum modal combination method and, above 33 hertz, non-participating mass are considered using the procedure given in Appendix A to SRP 3.7.2, Revision 2.~~

Figures 3.7.2-9 through 3.7.2-11 show the vibration mode shapes for the combined lumped-mass stick model consisting of the coupled shield and auxiliary buildings, the steel containment vessel and the containment internal structures.

Maximum absolute acceleration (ZPA) responses ~~of the design soil profiles~~ at selected locations on the coupled shield and auxiliary buildings, the steel containment vessel, and the containment internal structures are summarized in Tables 3.7.2-5, 3.7.2-6, and 3.7.2-7, respectively. Similarly, maximum displacement responses relative to the base of the lumped-mass nuclear island stick model at top of basemat~~, for the design soil profiles,~~ are summarized in Tables 3.7.2-8 through 3.7.2-10, respectively, for the coupled shield and auxiliary buildings, the steel containment vessel, and the containment internal structures. Maximum seismic response forces and moments determined in the lumped-mass stick model ~~for the design soil profiles~~ are summarized in Tables 3.7.2-11 through 3.7.2-13, respectively, for the coupled shield and auxiliary buildings, the steel containment vessel, and the containment internal structures.

~~3.7.2.2.1 Seismic Model Modifications~~

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

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

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

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

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

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

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

3.7.2.3 Procedure Used for Modeling

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

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

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

3.7.2.3.1 Coupled Shield and Auxiliary Buildings and Containment Internal Structures

The finite element models of the coupled shield and auxiliary buildings and the reinforced concrete portions of the containment internal structures are based on the gross concrete section with the modulus based on the specified compressive strength of concrete of contributing structural walls and slabs. The properties of the concrete-filled structural modules are computed using the combined gross concrete section and the transformed steel face plates of the structural modules. Furthermore, the weight density of concrete plus the uniformly distributed miscellaneous dead weights are considered by adjusting the material mass density of the structural elements. An equivalent tributary slab area load of 50 pounds per square foot is considered to represent miscellaneous deadweight such as minor equipment, piping and raceways. 25 percent of the floor live load or 75 percent of the roof snow load, whichever is applicable, is considered as mass in the global seismic models ~~(these masses are only included in the modified model described in subsection 3.72.2.1)~~. Major equipment weights are included as concentrated lumped masses at the equipment locations. Figures 3.7.2-1 and 3.7.2-2 show, respectively, the finite element models of the coupled shield and auxiliary buildings and the containment internal structures. A lumped-mass stick model of the shield building roof structure is coupled with the finite element model and the stick model of the coupled auxiliary and shield buildings. The stick model of the shield building roof structure is included in the seismic analyses. The lumped-mass stick model of the shield building roof is not shown in Figure 3.7.2-1 to maintain visual clarity of the finite element model.

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

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

The shield building roof including the passive containment cooling system water storage tank is represented by a lumped-mass stick model simulating the dynamic behavior of this portion of the roof structure. The member properties of the stick model are selected to match the frequencies and mode shapes from the finite element model. The portion of the roof from the bottom of the air inlets to the bottom of the passive containment cooling system tank is modelled by an equivalent beam. This lumped-mass stick model is combined with the lumped-mass stick model representing the lower portion of the shield building. In the three-dimensional finite element model, the lumped-mass stick model of the shield building roof is located at the center of the shield building represented using cylindrical shell elements. The lumped-mass stick model of the shield building roof is connected to the three-dimensional shell elements using 18 horizontal stiff beams.

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

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

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

3.7.2.3.2 Steel Containment Vessel

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

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

This method used to construct a stick model from the axisymmetric shell model of the containment vessel is verified by comparison of the natural frequencies determined from the stick model and the shell of revolution model as shown in Table 3.7.2-15. The shell of revolution vertical model (n = 0 harmonic) has a series of local shell modes of the top head above elevation 24~~0~~65′ 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 22~~0~~9′-0″. It is modelled as a single degree of freedom system attached to the steel containment shell as shown in Figure 3.7.2-5. The polar crane model includes the flexibility of the crane bridge girders and truck assembly, and the containment shell's local flexibility.

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

3.7.2.3.3 Nuclear Island Seismic Model

The various building lumped-mass stick models are interconnected with rigid links ~~stiff beam elements~~ to form the overall dynamic model of the nuclear island as shown in Figure 3.7.2-18. For the fixed-base analysis, the nuclear island seismic model consists of 312~~71~~ mass points and 1532~~178~~ dynamic degrees of freedom. The mass properties of the lumped-mass stick models include all tributary mass expected to be present during plant operating conditions. This includes the dead weight of walls and slabs, weight of major equipment, and equivalent tributary slab area loads representing miscellaneous equipment, piping and raceways.

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

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

3.7.2.4 Soil-Structure Interaction

Seismic analyses to be performed for soil sites are described in subsection 3.7.5.5.

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

~~The nuclear island soil-structure interaction analyses using three-dimensional models are performed for the design soil profiles described in subsection 3.7.1.4, except for the hard rock site condition, where the possibility of soil-structure interaction is negligible. Furthermore, the effects of the adjacent structures (turbine, annex, and radwaste buildings) on the seismic~~

~~response of the nuclear island are negligible. Therefore, the adjacent structures are not included in the soil-structure interaction analyses using three-dimensional models. The effect of the adjacent structures is included in the two-dimensional models analyzedsis for lateral earth pressure as described in Appendix 2C.~~

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

~~Soil-structure interaction analyses are performed using the three-dimensional model of the soil profiles coupled with the nuclear island lumped-mass stick model developed in subsection 3.7.2.3. The nuclear island lumped-mass stick model consists of (1) vertical elastic beam elements between floor elevations to represent wall stiffness and (2) lumped masses at the center of mass of each floor elevation. At each floor elevation, these vertical beam elements are connected with the lumped masses through horizontal stiff beam elements. For the soil-structure interaction analyses using program SASSI, these horizontal stiff beams have the following properties:~~

- ~~The area to length ratio of the stiff beam element is within the range of 10^3 to 10^5 times the largest area to length ratio of its connecting elastic structural elements.~~

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

~~Furthermore, the stiffness and mass contributed by the periphery walls in the embedded portion of the nuclear island are subtracted from the model properties of the lumped-mass stick model. The mass and stiffness properties adjustment is accomplished by recalculating the properties of the embedded portion of the three-dimensional lumped-mass stick model based on the finite element model without the periphery walls. To form the soil-structure interaction model, the lumped-mass stick models are coupled to the three-dimensional, finite element foundation model through stiff beams at elevations 82'-6" and 100'-0" (see Figure 3.7.2-13). The stiffness of each of these stiff beams is based on the lower stiffness of the connecting members.~~

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

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

3.7.2.5 Development of Floor Response Spectra

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

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

The floor response spectra for the design of subsystems and components are generated by enveloping the nodal response spectra determined for the two cases at a rock site~~different design soil profiles~~. One case is fixed at the base mat only (elevation 66′ 6″); the second case is fixed both at the base mat (elevation 66′ 6″) and horizontally at the floors at and below grade (elevations 82′ 6″ and 100′ 0″). The envelopes of the floor response spectra ~~for the four design soil profiles~~ are developed as follows:

- The spectral acceleration is calculated at the same frequencies for ~~all four of the design soil~~ both profiles
- The maximum spectral acceleration at each frequency from either ~~any of the four design soil~~ profiles is then selected for the envelope
- The enveloped floor response spectra is then broadened by ±15 percent

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

The safe shutdown earthquake floor response spectra for 5 percent damping, at representative locations of the coupled auxiliary and shield buildings, the steel containment vessel, and the containment internal structures are presented in Figures 3.7.2-15 through 3.7.2-17. ~~representative response spectra figures includinge the acceleration response spectra computed for the individual design soil profiles and the corresponding enveloped and widened floor response spectrum.~~

~~The broadened floor response spectra for the shield building roof in the vertical direction are based on soil structure interaction analyses which include added inventory in the Passive Containment Cooling System tank. These analyses use the modified nuclear island seismic model described in subsection 3.7.2.2.1. The peak of the vertical broadened spectra for the~~

~~shield building roof are increased to envelope the results of the additional analysis as shown in sheet 9 of Figure 3.7.2-15.~~

3.7.2.6 Three Components of Earthquake Motion

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

In mode superposition time-history analyses using computer program ~~BSAP~~ANSYS, the three components of earthquake are applied either simultaneously or separately. In the ~~BSAP~~ 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 analyses, the effect of the three components of earthquake motion are combined using one of the following methods:

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

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

- The peak responses due to the three earthquake components are combined directly, using the assumption that when the peak response from one component occurs, the responses from the other two components are 40 percent of the peak (100 percent-40 percent-40 percent method). Combinations of seismic responses from the three earthquake components, together with variations in sign (plus or minus), are considered. This method is used in the nuclear island basemat analyses and in the containment vessel stability analyses.

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

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

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

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

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

3.7.2.7 Combination of Modal Responses

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

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

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

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

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

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

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

The structures adjacent to the nuclear island are the annex building, the radwaste building, and the turbine building.

3.7.2.8.1 Annex Building

The annex building is classified as seismic Category II. The structural configuration is shown in Figure 3.7.2-19. The annex building is analyzed for the safe shutdown earthquake for the four sites described in subsection 3.7.1.4. Seismic input is defined by response spectra applied at the base of a dynamic model of the annex building. The horizontal spectra are obtained from the 2D SASSI analyses described in Appendix 2C and account for soil-structure and structure-soil-structure interaction. Input in the east-west direction uses the response spectra obtained from the two dimensional analyses for the annex building mat. Input in the north-south direction uses the response spectra obtained from the two dimensional analyses for the turbine building mat. Vertical input is obtained from 2D FLUSH finite element soil-structure interaction analyses. The seismic response spectra input at the base of the annex building are the envelopes of the four sites and also envelope the ~~AP600~~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 1991 Uniform Building Code provisions for Zone 3 with an Importance Factor of 1.0. For an eccentrically braced structure the resistance modification factor is 10 (UBC-91, reference 1) using allowable stress design. When using allowable stress design, the allowable stresses are not increased by one third for seismic loads. The resistance modification factor is reduced to 7 for load and resistance factor design (ASCE 7-93, reference 35).

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

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

3.7.2.9 Effects of Parameter Variations on Floor Response Spectra

Seismic model uncertainties due to, among other things, uncertainties in material properties, mass properties, damping values, the effect of concrete cracking, and the modeling techniques are accounted for in the widening of floor response spectra, as described in subsection 3.7.2.5. Stresses in the concrete structural elements due to the safe shutdown earthquake are below the tensile strength of the concrete. The effect of cracking of the concrete-filled structural modules inside containment due to thermal loads is discussed in subsection 3.8.3.4.2.

3.7.2.10 Use of Constant Vertical Static Factors

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

3.7.2.11 Method Used to Account for Torsional Effects

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

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

3.7.2.12 Comparison of Responses

The three-dimensional lumped mass fixed base stick model of the nuclear island was analyzed by mode superposition time history analysis and by the response spectrum analysis method for the hard rock site condition. Tables 3.7.2-17, 3.7.2-18, and 3.7.2-19 compare the maximum absolute nodal accelerations, member forces, and moments, respectively. The time history ~~Both~~ analyses considered vibration modes up to 118.6~~33~~ hertz. In the response spectrum analyses, the combination of modal responses used the double sum method for vibration modes up to 33 hertz, and included high frequency effects as discussed in subsection 3.7.2.7 and summarized in Table 3.7.2-16. The two methods of analysis give similar results with the response spectrum analysis being generally more conservative. Investigations of the two analyses showed that the conservatism in the response spectrum analyses is due to cross coupling of the directions in the multistick model. The double sum modal combination method used in the response spectrum analysis is very conservative when there are closely spaced modes some of which are out-of-phase.

3.7.2.13 Methods for Seismic Analysis of Dams

Seismic analysis of dams is site specific design.

3.7.2.14 Determination of Seismic Category I Structure Overturning Moments

Subsection 3.8.5.5.4 describes the effects of seismic overturning moments.

3.7.2.15 Analysis Procedure for Damping

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

3.7.5 Combined License Information

3.7.5.1 Seismic Analysis of Dams

Combined License applicants referencing the ~~AP600~~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 ~~AP600~~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 applicants referencing the AP1000 certified design will prepare the finite element models described in subsections 3.7.2.3 and 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. ~~If it is necessary to update the soil structure interaction analyses, these analyses should be performed with site specific soil properties using seismic input defined by the response spectra given in Figures 3.7.1-1 and 3.7.1-2.~~

3.7.5.5 Seismic Analyses of Nuclear Island Structures at Soil Sites

Combined License applicants referencing the AP1000 certified design at soil sites will perform soil structure interaction analyses for the nuclear island. These additional seismic analyses are to be performed when the AP1000 is to be located at a site where the soil below the underside of the base mat has a shear wave velocity less than 3500 feet per second. The results of these analyses will be documented in the Combined License application.

The three-dimensional, lumped-mass stick models of the nuclear island structures developed as described in subsections 3.7.2.3 and 3.7.5.4 will be used in conjunction with the design soil profiles presented by the Combined License applicant to obtain the in-structure responses. The three-dimensional, nuclear island stick model will be coupled with the foundation media to form a soil-structure interaction model to account for the effects of embedment and foundation rocking, torsion, and translation. The seismic soil-structure interaction analysis of the coupled nuclear island and soil foundation model will be performed using computer program SASSI.

3.7.5.5.1 Supporting Media for Seismic Category I structures

Soil structure interaction analyses will be performed for a range of soil properties specified by the Combined License applicant. The range of soil conditions will be selected at the time of the Combined License submittal. Examples of acceptable options are:

Option 1

Analyze one case for the best estimate site properties described in Section 2.5 of the Combined License application and upper and lower bound cases to bound the site. This results in a design applicable to a narrow range of sites.

Option 2

Envelope the results from the three soil cases of Option 1 and also envelope the results of the hard rock analyses included in the AP1000 Design Certification. This results in a design that is demonstrated to be acceptable at a single site and has additional margin so that it is applicable to a broader range of sites than in Option 1.

Option 3

Perform analyses for two, three or four of the following soil cases considered for AP600. Demonstrate that these cases bound the range of site specific soil conditions described in Section 2.5 of the Combined License application.

- For the hard rock site, an upper bound case for firm sites using fixed base seismic analysis. The results of this case are provided in the AP1000 Design Certification.

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

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

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

Enveloping the results of all four of the above cases will provide a design satisfying the full range of sites with shear wave velocity greater than 1000 feet per second identified in DCD Table 2-1.

The strain-dependent shear modulus curves for the foundation materials, together with the corresponding damping curves, are shown in Figures 3.7.1-14 and 3.7.1-15 for rock material and soil material, respectively.

The Combined License applicant at a rock site may also elect one of these options in order to broaden the applicability of the design.

3.7.5.5.2 Seismic Analysis Models

The soil structure interaction analyses will be performed using the AP1000 stick models described in subsection 3.7.2.3. Minor changes will be implemented by the Combined License applicant to adjust these models for the SASSI computer program.

The nuclear island lumped-mass stick model consists of vertical elastic beam elements between floor elevations to represent wall stiffness and lumped masses at the center of mass of each floor elevation. At each floor elevation, these vertical beam elements are connected with the lumped masses through horizontal rigid links in the ANSYS analyses. For the soil-structure interaction analyses using program SASSI, these rigid links will be replaced by horizontal stiff beams with properties as follows:

- The area to length ratio of the stiff beam element will be within the range of 10^3 to 10^5 times the largest area to length ratio of its connecting elastic structural elements.

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

To form the soil-structure interaction model, the lumped-mass stick models will be coupled to the three-dimensional, finite element foundation model through stiff beams at elevations 82′-6″ and 100′-0″ (see Figure 3.7.2-13). The nuclear island basemat and the periphery walls of the embedded portion of the nuclear island will be represented by a three-dimensional, finite element model, as shown in Figure 3.7.2-8. The stiffness and mass contributed by the periphery walls in the embedded portion of the nuclear island will be subtracted from the model properties of the lumped-mass stick model used in the ANSYS hard rock analyses. The mass and stiffness properties will be adjusted by recalculating the properties of the embedded portion of the three-dimensional lumped-mass stick model based on the finite element model without the periphery walls.

3.7.5.5.3 Soil Structure Interaction

Soil-structure interaction (SSI) analyses of the nuclear island will be performed to generate its soil-structure interaction responses, including nodal displacements, nodal accelerations, and floor response spectra. The modeling of the soil medium will consider effects on the seismic Category I structures due to embedment of the nuclear island, the ground water, and the layering of soil profiles.

Soil-structure interaction analyses will be performed using the complex frequency-response method with computer program SASSI (Reference 8). This program is capable of handling two- and three-dimensional soil-structure interaction problems involving multiple structures with rigid or flexible embedded foundations of arbitrary shape.

Computer program SHAKE (Reference 9) will be used to compute the safe shutdown earthquake dynamic strain compatible soil properties, such as shear modulus and damping. The material (hysteretic) damping ratio for soil in the soil-structure interaction analyses will be limited not to exceed 15 percent.

The nuclear island soil-structure interaction analyses using three-dimensional models are performed for the soil profiles described in subsection 3.7.1.4 of the Safety Analysis Report in the Combined License application. The effects of the adjacent structures (turbine, annex, and radwaste buildings) on the overall seismic response of the nuclear island are negligible. Therefore, the adjacent structures will not be included in the soil-structure interaction analyses using three-dimensional models. However, the effect of the adjacent structures will be included in two-dimensional models to determine lateral earth pressure for design of the exterior walls of the nuclear island below grade.

The cutoff frequencies used in the soil structure interaction analyses will be dependent on the soil properties. Typically the cut-off frequency will be about 33 hertz for a soft rock site, and 20 hertz horizontal and 33 hertz vertical for a stiff soil site, and 15 hertz horizontal and 21 hertz vertical for a soft-to-medium soil site.

The soil-structure interaction analyses will be performed with the three statistically independent acceleration time histories described in subsection 3.7.1.2 applied separately. The total seismic response is then obtained by combining the responses of the three components of earthquake algebraically in each time step.

3.7.5.5.4 Floor Response Spectra

The floor response spectra will be smoothed, and the spectral peaks associated with the structural frequencies 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.

3.7.5.5.5 Structural Design Loads

The seismic analyses will provide design loads for structural design. The member forces in the stick models from the soil structure interaction analyses will be compared to those from the fixed base hard rock analyses. In cases where the soil structure interaction analyses give higher element forces than the hard rock profile, the forces obtained from the response spectrum analyses of the finite element models for the hard rock site are increased by a scaling factor. The scaling factor, at a given plant elevation, is equal to the ratio of the largest three-dimensional stick model element force over the three-dimensional stick model element force for the hard rock profile.

The response spectrum analysis performed for the hard rock assumes that the foundation below the basemat remains rigid. Soil flexibility will be considered in separate static analyses of the nuclear island base mat and superstructure. This will provide the relative distribution of loads to the various shear walls when the plant is located at a soil site.

As an alternate to use of the fixed base hard rock results increased by factors to account for the soil flexibility, the Combined License applicant may perform equivalent static analyses using the finite element models. These analyses would apply the maximum seismic acceleration response obtained from the stick models and would include the base mat on soil springs.

3.7.5.5.6 Combined License Information

The Combined License applicant at a soil site will provide representative response spectra figures including the acceleration response spectra computed for his design soil profiles and the corresponding enveloped and widened floor response spectrum.

The Combined License applicant will describe the design loads for the structures.

APPENDIX B

AP1000 SEISMIC RESPONSE

This appendix provides preliminary results of the hard rock seismic analyses for the AP1000 and compares them against the AP600 results documented in the AP600 DCD tables. The AP1000 results will be included in the corresponding tables in the AP1000 DCD. At this time Westinghouse is requesting review of the approach and has not requested detail review of the technical changes.

Comparison of AP600 and AP1000 Seismic Results for Hard Rock Site

This document provides a comparison of the results of the fixed base seismic analyses of the AP1000 and the AP600. The AP1000 model includes the following differences from the AP600 configuration:

- Shield building raised by 25′6″
- PCS tank capacity increased to 800,000 gallons
- Containment vessel raised by two courses (25′6″) and increased in thickness from 1.625″ to 1.75″
- Polar crane raised and capacity increased.
- Reactor coolant system equipment increased in size
- Steam generator upper support snubbers raised
- Steam generator and pressurizer compartment walls raised
- 25 percent of live load and 75 percent of snow load added as mass.

The AP1000 seismic analyses use stick models with properties modified from the AP600 stick models to account for the AP1000 changes. The AP600 stick models were created from finite element models. The AP600 properties are applied to the corresponding portions of the AP1000 auxiliary building and containment internal structures. Elements were added or extended to reflect the increased height of certain walls. New stick models were developed for the containment vessel, the shield building roof and the reactor coolant loop. The resulting AP1000 stick models will be reconciled by the Combined License applicant when he develops the AP1000 finite element models required for structural design.

Table 1 provides an overall summary of results at a few key locations extracted from the subsequent tables. Tables 3.7.2-5 through 3.7.2-12 contain the same information as the corresponding table in the AP600 Design Certification document. The results for the AP1000 are shown below the AP600 results. Figures 1 and 2 show the absolute response acceleration and relative deflection of the auxiliary and shield building versus height for the two plants.

Figures 3 to 10 show the floor response spectra at 5 percent damping for each plant. The locations shown are those included in Figures 3.7.2-15, 16 and 17 of the AP600 Design certification document. The AP1000 results are those for the fixed base analysis; the AP600 results are the design spectra which have been broadened to envelope the four design soil cases.

The fundamental frequencies of the auxiliary and shield building decrease by about 19%, from 4.78 hertz to 3.87 hertz in the north-south direction and from 4.35 hertz to 3.57 hertz in the east-west direction.

Table B-1 Summary of Seismic Responses

Maximum Absolute Nodal Acceleration, ZPA (g)						
	AP600			AP1000		
Elevation	N-S	E-W	VERT	N-S	E-W	VERT
Top of shield building	1.44	1.47	0.90	1.44	1.54	0.89
Shield building air inlet	0.82	0.78	0.55	0.86	0.86	0.53
Top of containment vessel	0.94	1.21	1.49	0.96	1.03	1.42
CIS pressurizer compartment	0.79	0.65	0.30	0.83	0.77	0.36
CIS operating floor	0.61	0.52	0.30	0.52	0.48	0.32
Maximum Relative Displacement (in.)						
	AP600			AP1000		
Elevation	N-S	E-W	VERT	N-S	E-W	VERT
Top of shield building	0.54	0.64	0.19	0.95	1.10	0.25
Air inlet	0.33	0.40	0.04	0.56	0.63	0.06
Top of containment vessel	0.21	0.22	0.05	0.33	0.33	0.06
CIS pressurizer compartment	0.04	0.05	0.01	0.05	0.06	0.01
CIS operating floor	0.03	0.03	0.00	0.03	0.03	0.01
Maximum Forces ($x10^3$ Kips)						
	AP600			AP1000		
Elevation	Axial	N-S Shear	E-W Shear	Axial	N-S Shear	E-W Shear
Shield building air inlet	11.54	12.52	10.57	14.83	14.71	15.75
Aux. building - El. 100′	34.96	37.54	37.59	41.61	46.8	38.69
Containment vessel El. 100′	4.60	3.93	4.49	5.26	5.11	4.79
Cont. Int. Struc. – El 103′	4.07	7.02	6.90	5.97	9.35	8.13
Maximum Moment ($x10^3$ K-ft)						
	AP600			AP1000		
Elevation	Torque	about N-S Axis	about E-W Axis	Torque	about N-S Axis	about E-W Axis
Shield building air inlet	46	747	746	36	891	804
Aux. building - El. 100′	1396	4188	4045	1640	5564	6048
Containment vessel El. 100′	11.23	489.70	429.50	37.81	628.59	651.72
Cont. Int. Struct. – El 103′	321.90	244.30	225.90	264.60	277.60	242.10

Table B-3.7.2-5 Maximum Absolute Nodal Acceleration (ZPA) Coupled Auxiliary & Shield Buildings Hard Rock Site Condition

AP600						
Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g)					
	N-S Direction		E-W Direction		Vertical Direction	
306.25	1.44		1.47		0.90	
297.08	1.32		1.27		0.90	
284.42	1.20		0.98		0.89	
272.42	1.09		0.94		0.88	
241.00	0.82		0.78		0.55	
220.00	0.73	(0.75)	0.69	(0.73)	0.53	(0.65)
200.00	0.63	(0.64)	0.67	(0.69)	0.49	(0.63)
180.20	0.51	(0.51)	0.60	(0.63)	0.45	(0.59)
161.50	0.44	(0.45)	0.54	(0.56)	0.42	(0.53)
153.50	0.42	(0.43)	0.51	(0.55)	0.40	(0.50)
135.25	0.38	(0.40)	0.41	(0.45)	0.37	(0.45)
117.50	0.34	(0.35)	0.34	(0.37)	0.35	(0.40)
100.00	0.30	(0.30)	0.30	(0.30)	0.32	(0.35)
82.50	0.30	(0.30)	0.30	(0.30)	0.30	(0.32)
66.50	0.30		0.30		0.30	
AP1000						
Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g)					
	N-S Direction		E-W Direction		Vertical Direction	
333.12	1.44		1.54		0.89	
295.23	1.07		1.15		0.88	
265.00	0.86		0.86		0.53	
245.50	0.77		0.83		0.50	
222.75	0.71		0.77		0.47	
200.00	0.65		0.70		0.44	
180.20	0.58		0.58		0.39	
161.50	0.52		0.52		0.37	
153.50	0.49		0.50		0.37	
135.25	0.41		0.42		0.35	
117.50	0.36		0.35		0.33	
100.00	0.30		0.30		0.31	
82.50	0.30		0.30		0.30	
66.50	0.30		0.30		0.30	

Note:

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

Table B-3.7.2-6 Maximum Absolute Nodal Acceleration (ZPA) Steel Containment Vessel Hard Rock Site Condition						
AP600						
Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g)					
	N-S Direction		E-W Direction		Vertical Direction	
256.33	0.94		1.21		1.49	
248.33	0.90		1.17		1.20	
240.33	0.87	(0.88)	1.13	(1.14)	1.04	(1.15)
229.52	0.83		1.07		0.84	
218.71	0.78		1.01		0.77	
205.33	0.72	(0.73)	0.93	(0.94)	0.75	(0.85)
205.33 (Polar Crane)	1.82		1.09		1.14	
190.00	0.65		0.82		0.70	
170.00	0.56		0.68		0.64	
162.00	0.51	(0.52)	0.62	(0.63)	0.60	(0.68)
144.50	0.41		0.48		0.53	
138.58	0.38		0.44		0.50	
132.25	0.36		0.39		0.48	
116.86	0.33	(0.33)	0.34	(0.34)	0.41	(0.46)
112.50	0.32		0.33		0.39	
110.50	0.32		0.33		0.36	
104.13	0.31		0.31		0.36	
100.00	0.30		0.30		0.31	
AP1000						
281.83	0.96		1.03		1.42	
273.83	0.93		1.00		1.16	
265.83	0.90		0.96		0.98	
255.02	0.86		0.92		0.83	
244.21	0.81		0.86		0.78	
225.33	0.73		0.77		0.74	
225.33 P.C.	1.82		1.95		1.15	
200.00	0.59		0.64		0.68	
169.93	0.46		0.48		0.59	
162.00	0.44		0.45		0.56	
141.50	0.39		0.40		0.49	
131.68	0.37		0.38		0.46	
112.50	0.31		0.32		0.38	
104.13	0.30		0.30		0.35	
100.00	0.30		0.30		0.31	

Note:

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

Table B-3.7.2-7 Maximum Absolute Nodal Acceleration (ZPA) Containment Internal Structure Hard Rock Site Condition

AP600						
Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g)					
	N-S Direction		E-W Direction		Vertical Direction	
158.00 (PRZ Compartment)	0.79		0.65		0.30	
148.00 (SG-West Compartment)	0.73		0.58		0.31	
148.00 (SG-East Compartment)	0.69		0.54		0.32	
135.25	0.61	(0.73)	0.52	(0.71)	0.30	(0.34)
107.17	0.32	(0.32)	0.30	(0.31)	0.30	(0.32)
103.00	0.31		0.30		0.30	
98.10	0.30		0.30		0.30	
87.50	0.30		0.30		0.30	
82.50	0.30		0.30		0.30	
AP1000						
Elevation (ft)	Maximum Absolute Nodal Acceleration, ZPA (g)					
	N-S Direction		E-W Direction		Vertical Direction	
169.00 (PRZ Compartment)	0.83		0.77		0.36	
155.00 (SG-West Compartment)	0.71		0.61		0.35	
155.00 (SG-East Compartment)	0.64		0.51		0.31	
135.25	0.52		0.48		0.32	
107.17	0.31		0.30		0.30	
103.00	0.30		0.30		0.30	
98.10	0.30		0.30		0.30	
87.50	0.30		0.30		0.30	
82.50	0.30		0.30		0.30	

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

Table B-3.7.2-8 Maximum Displacement Relative to Top of Basemat Coupled Auxiliary & Shield Buildings Hard Rock Site Condition						
AP600						
Elevation (ft)	Maximum Relative Displacement (in.)					
	N-S Direction		E-W Direction		Vertical Direction	
306.25	0.54		0.64		0.19	
297.08	0.50		0.61		0.19	
284.42	0.46		0.56		0.19	
272.42	0.42		0.51		0.19	
241.00	0.33		0.40		0.04	
220.00	0.26	(0.28)	0.32	(0.34)	0.04	(0.15)
200.00	0.19	(0.21)	0.25	(0.27)	0.04	(0.12)
180.20	0.13	(0.15)	0.17	(0.20)	0.02	(0.10)
161.50	0.09	(0.10)	0.12	(0.14)	0.01	(0.08)
153.50	0.07	(0.09)	0.11	(0.12)	0.01	(0.07)
135.25	0.04	(0.05)	0.06	(0.08)	0.01	(0.06)
117.50	0.02	(0.03)	0.03	(0.04)	0.01	(0.04)
100.00	0.	(0.)	0.	(0.)	0.	(0.02)
82.50	0.	(0.)	0.	(0.)	0.	(0.01)
66.50	0.		0.		0.	
AP1000						
Elevation (ft)	Maximum Relative Displacement (in.)					
	N-S Direction		E-W Direction		Vertical Direction	
333.13	0.95		1.10		0.25	
295.23	0.70		0.82		0.25	
265.00	0.56		0.63		0.06	
242.50	0.48		0.53		0.06	
220.00	0.39		0.42		0.06	
200.00	0.32		0.33		0.05	
180.20	0.24		0.22		0.03	
161.50	0.19		0.17		0.03	
153.50	0.15		0.16		0.03	
135.25	0.09		0.09		0.02	
117.50	0.04		0.04		0.01	
100.00	0.00		0.00		0.01	
82.50	0.00		0.00		0.00	
66.50	0.00		0.00		0.00	

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

Table B-3.7.2-9 Maximum Displacement Relative to Top of Basemat Steel Containment Vessel Hard Rock Site Condition

Elevation (ft)	Maximum Relative Displacement (in.)					
	N-S Direction		E-W Direction		Vertical Direction	
AP600						
256.33	0.21		0.22		0.05	
248.33	0.20		0.22		0.04	
240.33	0.19	(0.19)	0.21	(0.21)	0.04	(0.06)
229.52	0.18		0.20		0.03	
218.71	0.17		0.18		0.03	
205.33	0.15	(0.15)	0.17	(0.17)	0.02	(0.05)
205.33 (Polar Crane)	0.59		2.20		0.54	
190.00	0.13		0.14		0.02	
170.00	0.10		0.11		0.02	
162.00	0.09	(0.09)	0.10	(0.10)	0.02	(0.05)
144.50	0.06		0.07		0.01	
138.58	0.05		0.06		0.04	
132.25	0.04		0.05		0.01	
116.86	0.02	(0.02)	0.02	(0.02)	0.01	(0.03)
112.50	0.02		0.02		0.01	
110.50	0.01		0.01		0.02	
104.13	0.01		0.01		0.01	
100.00	0.		0.		0.01	
AP1000						
281.83	0.33		0.33		0.06	
273.83	0.32		0.32		0.05	
265.83	0.31		0.31		0.04	
255.02	0.30		0.29		0.03	
244.21	0.28		0.27		0.03	
225.33	0.25		0.24		0.03	
225.33 P.C.	0.58		0.79		0.33	
200.00	0.20		0.19		0.02	
169.93	0.13		0.13		0.02	
162.00	0.12		0.12		0.02	
141.50	0.07		0.07		0.01	
131.68	0.06		0.06		0.01	
112.50	0.02		0.02		0.01	
104.13	0.01		0.01		0.01	
100.00	0.00		0.00		0.01	

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

Table B-3.7.2-10 Maximum Displacement Relative to Top of Basemat Containment Internal Structure Hard Rock Site Condition

AP600						
Elevation (ft)	**Maximum Relative Displacement (in.)**					
	N-S Direction		**E-W Direction**		**Vertical Direction**	
158.00 (PRZ Compartment)	0.04		0.05		0.01	
148.00 (SG-West Compartment	0.04		0.04		0.01	
148.00 (SG-East Compartment)	0.02		0.04		0.	
135.25	0.03	(0.04)	0.03	(0.05)	0.	(0.01)
107.17	0.	0.	0.	0.	0.	(0.01)
103.00	0.		0.		0.	
98.10	0.		0.		0.	
87.50	0.		0.		0.	
82.50	0.		0.		0.	

AP1000						
Elevation (ft)	**Maximum Relative Displacement (in.)**					
	N-S Direction		**E-W Direction**		**Vertical Direction**	
169.00 (PRZ Compartment)	0.05		0.06		0.01	
155.00 (SG-West Compartment	0.05		0.05		0.01	
155.00 (SG-East Compartment)	0.04		0.05		0.01	
135.25	0.03		0.03		0.01	
107.17	0.01		0.00		0.00	
103.00	0.01		0.00		0.00	
98.10	0.00		0.00		0.00	
87.50	0.00		0.00		0.00	
82.50	0.00		0.00		0.00	

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

Table B-3.7.2-11 (Sheet 1 of 2) Maximum Member Forces and Moments Coupled Auxiliary & Shield Buildings Hard Rock Site Condition

AP600						
Elevation (ft)	Maximum Forces (x10³ Kips)			Maximum Moment (x10³ K-ft)		
	Axial	N-S Shear	E-W Shear	Torque	about N-S Axis	about E-W Axis
306.25					18.20	18.20
	1.45	2.46	2.43	3.88		
297.08					59.80	59.10
	3.40	4.47	4.36	9.17		
284.42					181.90	178.70
	7.65	8.30	7.67	25.50		
272.42					274.00	266.00
	11.54	12.52	10.57	46.08		
241.00					747.50	746.00
	15.44	16.43	15.68	81.10		
220.00					1072.00	1109.00
	18.05	18.72	18.32	109.50		
200.00					1402.00	1488.00
	20.43	20.68	20.32	134.50		
180.20					1835.00	2140.00
	23.40	23.28	23.03	923.80		
161.50					2243.00	2483.00
	25.45	25.51	25.17	911.40		
153.50					2389.00	2482.00
	28.14	28.82	28.40	716.10		
135.25					2896.00	2972.00
	31.92	34.03	33.57	1157.00		
117.50					3539.00	3417.00
	34.96	37.54	37.59	1396.00		
100.00					4188.00	4045.00

Table B-3.7.2-11 (Sheet 2 of 2) Maximum Member Forces and Moments Coupled Auxiliary & Shield Buildings Hard Rock Site Condition						
AP1000						
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
333.12						
	2.60	6.12	6.71	10		
295.23					281	254
	14.83	14.70	15.74	35		
265.00					891	804
	17.27	19.09	19.94	79		
242.50					1327	1229
	19.35	22.46	22.89	119		
220.00					1881	1756
	21.40	25.21	25.03	151		
200.00					2415	2280
	23.30	27.44	26.51	175		
180.20					2966	2837
	25.94	30.57	27.96	1252		
161.50					3536	3518
	27.78	32.87	28.76	1138		
153.50					3809	3547
	30.60	36.16	31.27	873		
135.25					4387	4363
	35.71	41.54	35.32	1334		
117.50					4983	5129
	41.62	46.80	38.69	1640		
100.00					5565	6049

Table B-3.7.2-12 (Sheet 1 of 2) Maximum Member Forces and Moments Steel Containment Vessel Hard Rock Site Condition						
AP600						
Elevation (ft)	Maximum Forces ($x10^3$ Kips)			Maximum Moment ($x10^3$ K-ft)		
	Axial	N-S Shear	E-W Shear	Torque	about N-S Axis	about E-W Axis
256.33					0.00	0.00
	0.28	0.17	0.22	0.00		
248.33					2.73	2.34
	0.61	0.46	0.59	0.16		
240.33					9.36	7.79
	0.99	0.77	0.99	0.47		
229.52					22.70	18.44
	1.33	1.06	1.37	0.90		
218.71					40.57	32.52
	1.66	1.36	1.76	1.37		
205.33					70.98	58.59
	2.79	2.60	2.78	10.26		
190.00					118.10	101.30
	3.23	3.00	3.29	10.59		
170.00					187.40	162.80
	3.58	3.29	3.67	10.81		
162.00					219.60	191.10
	3.91	3.52	3.96	10.97		
144.50					291.40	254.30
	4.19	3.72	4.22	11.99		
138.58					316.40	276.40
	4.21	3.73	4.24	11.43		
132.25					345.30	302.10
	4.41	3.86	4.40	11.52		
116.86					413.10	362.40
	4.49	3.89	4.44	11.55		
112.50					433.10	380.00
	4.55	3.92	4.47	11.36		
110.50					442.30	387.90
	4.57	3.92	4.48	11.22		
104.13					471.20	413.80
	4.60	3.93	4.49	11.23		
100					489.70	429.50

Table B-3.7.2-12 (Sheet 2 of 2) Maximum Member Forces and Moments Steel Containment Vessel Hard Rock Site Condition						
AP1000						
Elevation (ft)	Maximum Forces (x10³ Kips)			Maximum Moment (x10³ K-ft)		
	Axial	N-S Shear	E-W Shear	Torque	about N-S Axis	about E-W Axis
	0.30	0.19	0.20	0.00		
273.83					1.62	1.52
	0.73	0.51	0.55	4.02		
265.83					9.99	10.23
	1.10	0.84	0.89	6.65		
255.02					22.38	21.56
	1.45	1.17	1.24	9.48		
244.21					38.96	36.94
	1.87	1.58	1.68	13.02		
225.33					74.81	70.39
	2.90	3.62	3.32	21.46		
200.00					168.59	168.27
	3.54	4.16	3.89	26.61		
169.93					291.70	297.84
	4.09	4.54	4.27	30.50		
162.00					330.36	338.74
	4.43	4.76	4.49	32.73		
141.50					425.49	438.33
	4.72	4.91	4.63	35.24		
138.58					441.71	454.45
	4.72	4.91	4.63	35.24		
131.68					473.67	488.08
	5.00	5.04	4.74	36.68		
112.50					567.51	586.58
	5.19	5.10	4.78	37.74		
110.50					578.00	597.82
	5.19	5.10	4.78	37.74		
104.13					608.47	630.36
	5.26	5.11	4.79	37.81		
100.00					628.59	651.72

Table B-3.7.2-13 (Sheet 1 of 2) Maximum Member Forces and Moments Containment Internal Structures Hard Rock Site Condition						
AP600						
Elevation (ft)	Maximum Forces (x10³ Kips)			Maximum Moment (x10³ K-ft)		
	Axial	N-S Shear	E-W Shear	Torque	about N-S Axis	about E-W Axis
Above Elevation 135.25′, West SG Compartment						
158.00					0.07	0.07
	0.05	0.16	0.15	0.25		
153.56					0.71	0.74
	0.05	0.29	0.28	0.25		
148.00					2.65	3.05
	0.24	0.81	0.76	6.69		
135.25					12.25	13.23
Above Elevation 135.25′, East SG Compartment						
148.00					0.56	0.16
	0.13	0.31	0.27	2.20		
135.25					3.79	4.10
Below Elevation 135.25′						
135.25					40.40	35.70
	1.99	5.73	5.98	245.70		
121.50					117.40	108.60
	1.99	5.83	6.07	247.50		
107.17					219.60	196.10
	4.07	7.02	6.90	321.90		
103.00					244.30	225.90

Table B-3.7.2-13 Maximum Member Forces and Moments Containment Internal Structures
(Sheet 2 of 2) Hard Rock Site Condition

AP1000						
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
Above Elevation 135.25′, West SG Compartment						
169.00						
	0.10	0.22	0.20	0.20		
163.79					1.08	1.23
	0.10	0.33	0.29	0.18		
155.00					3.62	4.10
	0.56	1.13	1.66	11.05		
135.25					38.23	26.81
Above Elevation 135.25′, East SG Compartment						
155.00						
	0.17	0.45	1.45	1.98		
135.25					32.57	9.00
Below Elevation 135.25′						
135.25						
	2.87	5.93	5.15	121.40		
121.50					124.90	105.10
	2.87	5.93	5.15	121.40		
107.17					195.30	178.10
	5.97	9.35	8.13	264.60		
103.00					277.60	242.10

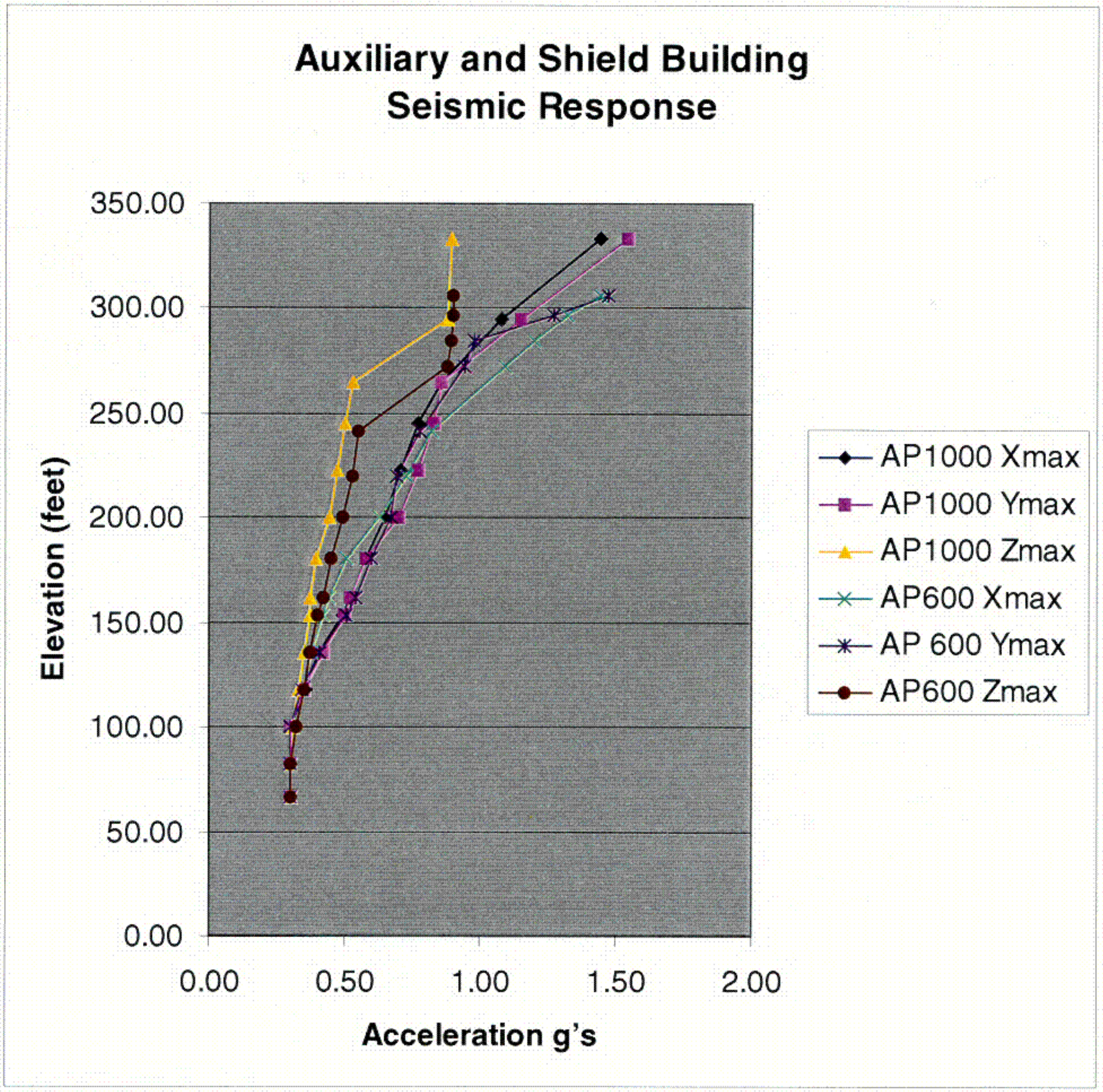


Figure B-1 Auxiliary and Shield Building Maximum Acceleration Response

C01

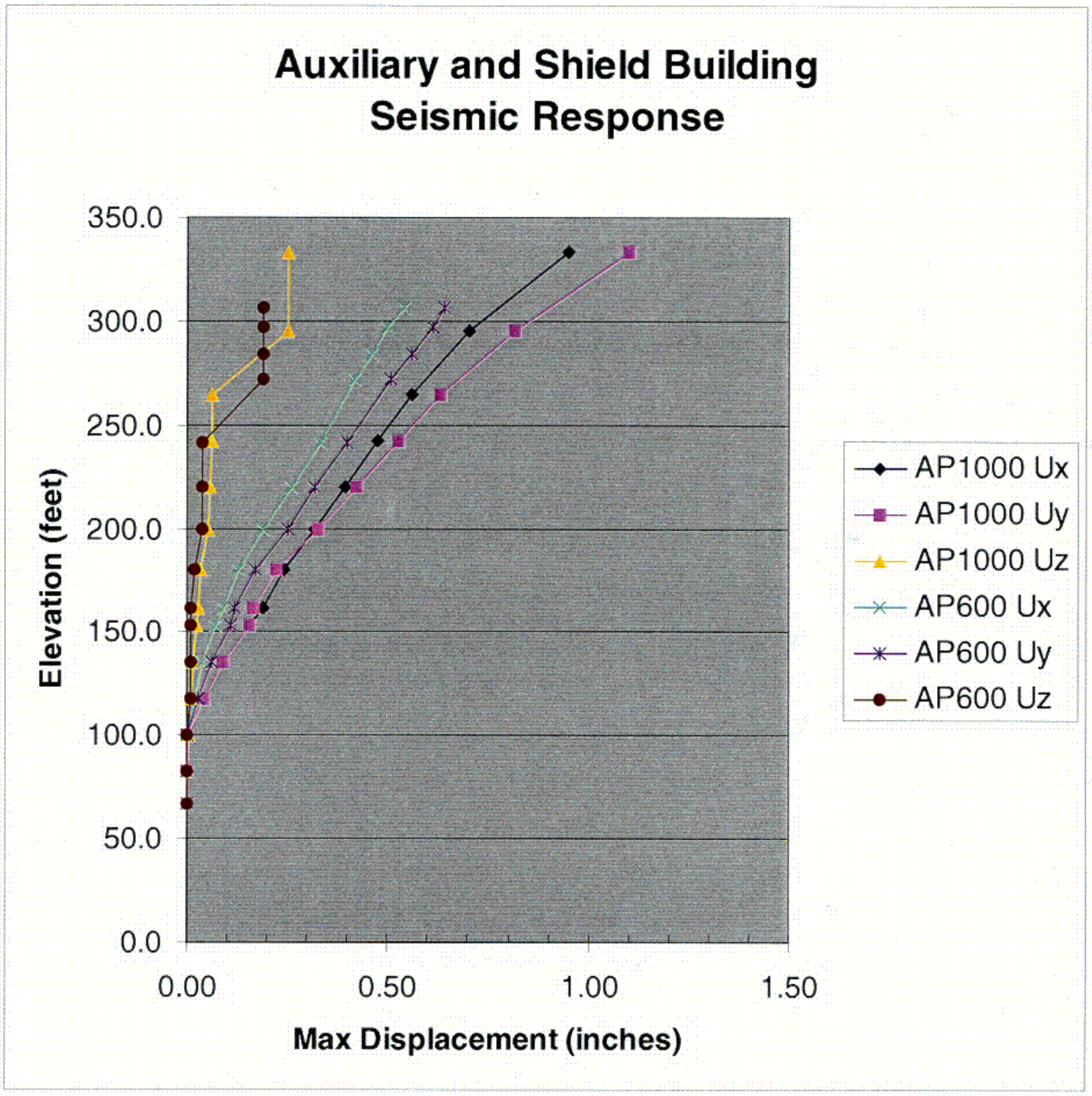
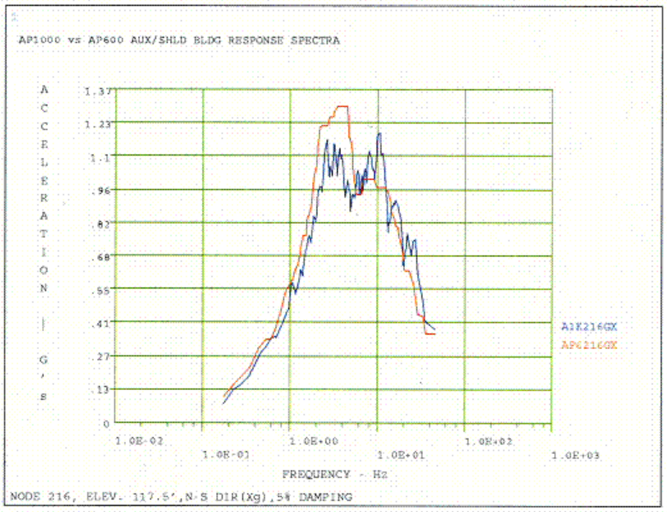
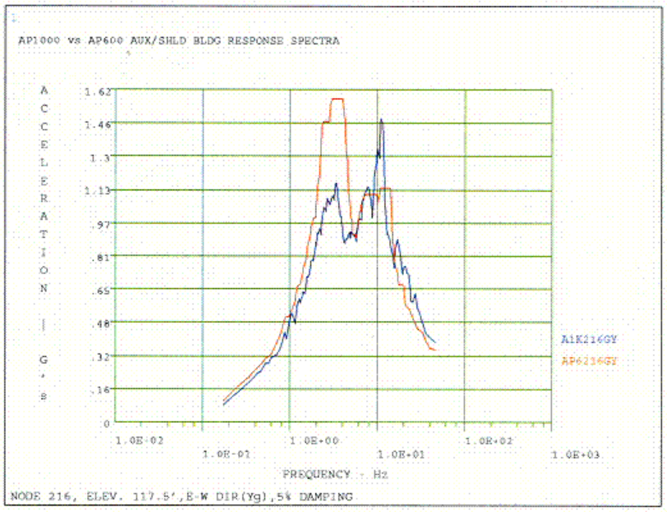


Figure B-2 Auxiliary and Shield Building Maximum Relative Displacement Response

C02





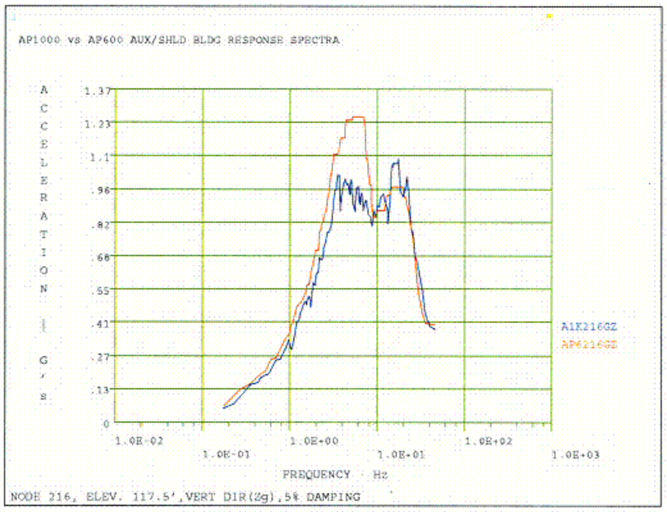
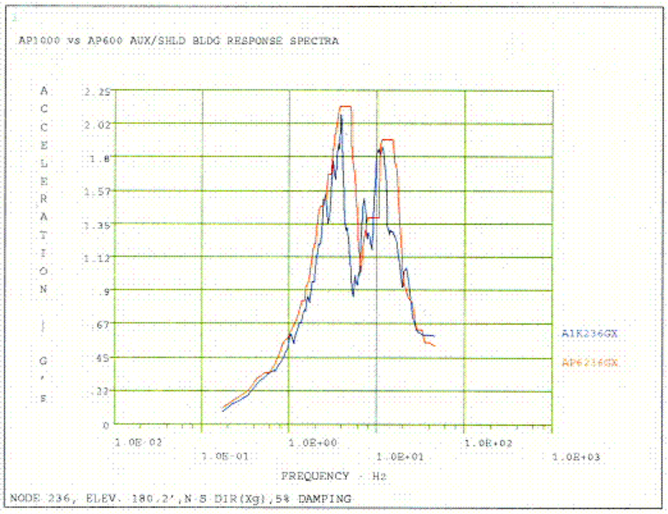
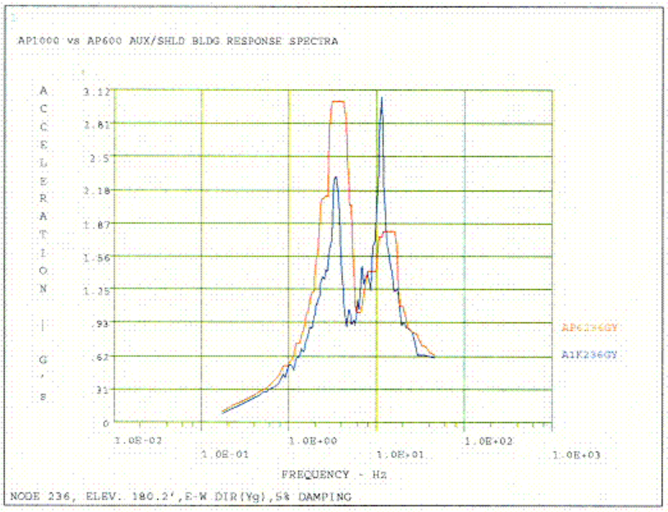


Figure B-3 Floor Response Spectra - Auxiliary Building Elevation 117 6"

C03





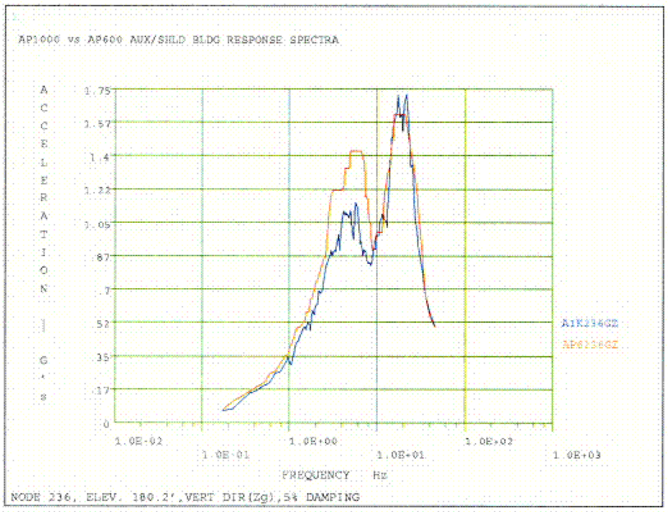
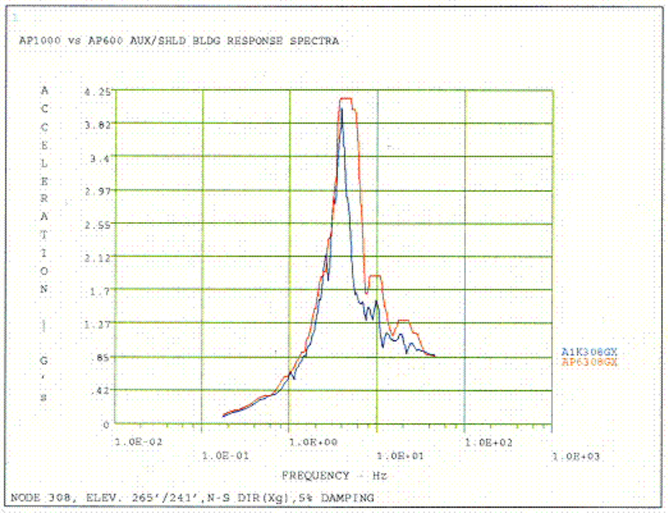
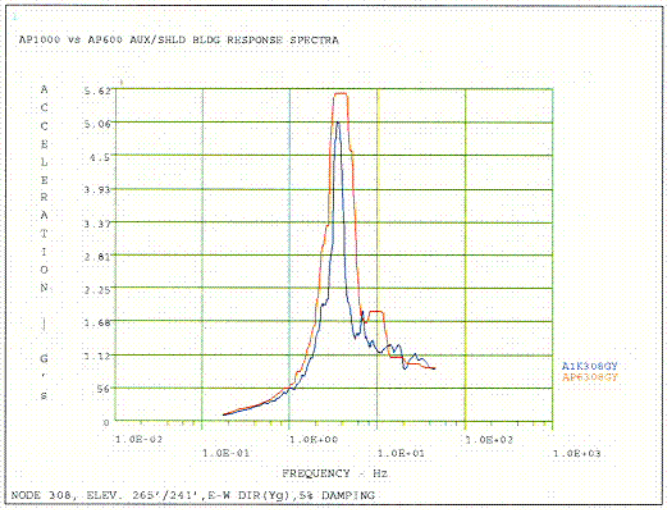


Figure B-4 Floor Response Spectra - Auxiliary Building Elevation 180'

C04





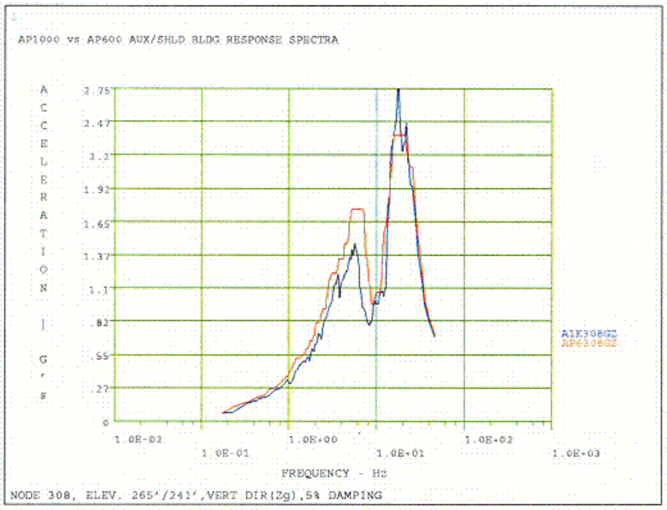


Figure B-5 Floor Response Spectra - Shield Building - Bottom of Air Inlet

C05

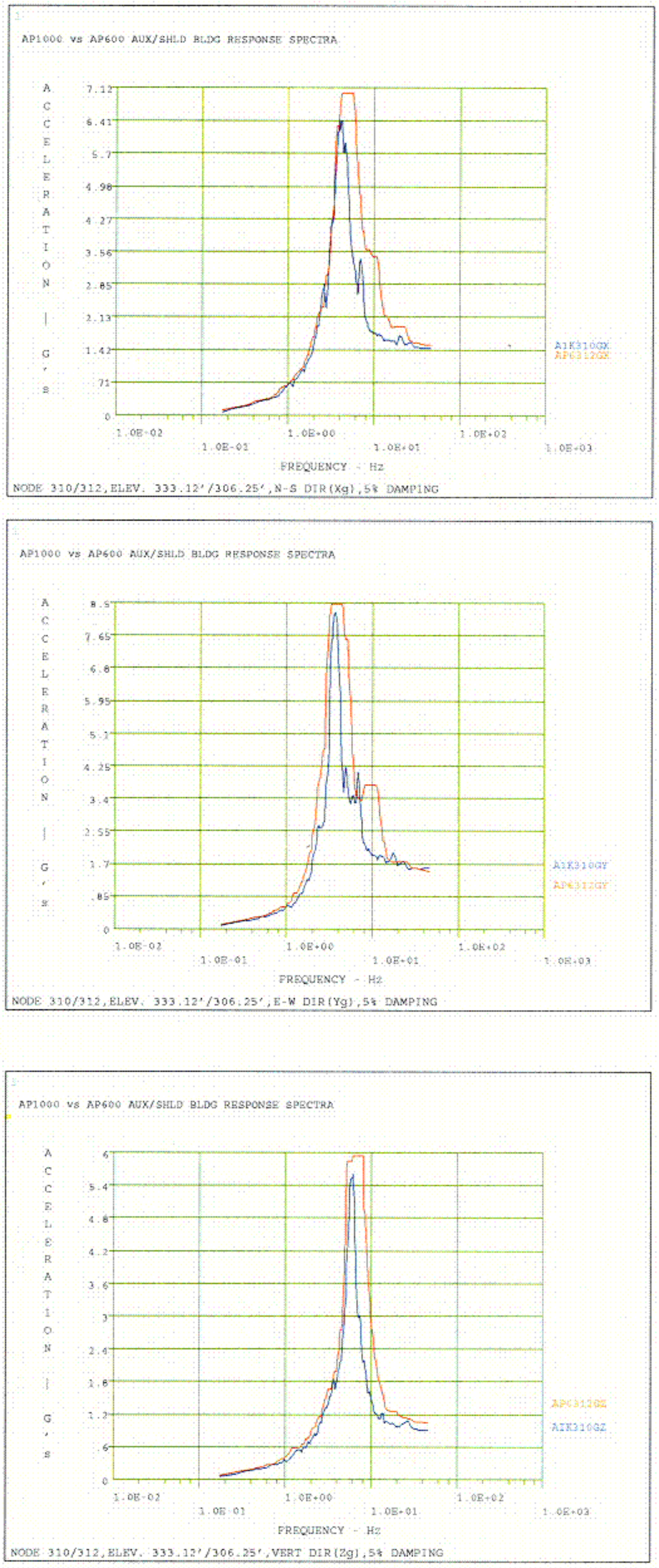


Figure B-6 Floor Response Spectra - Top of Shield Building Roof

c06

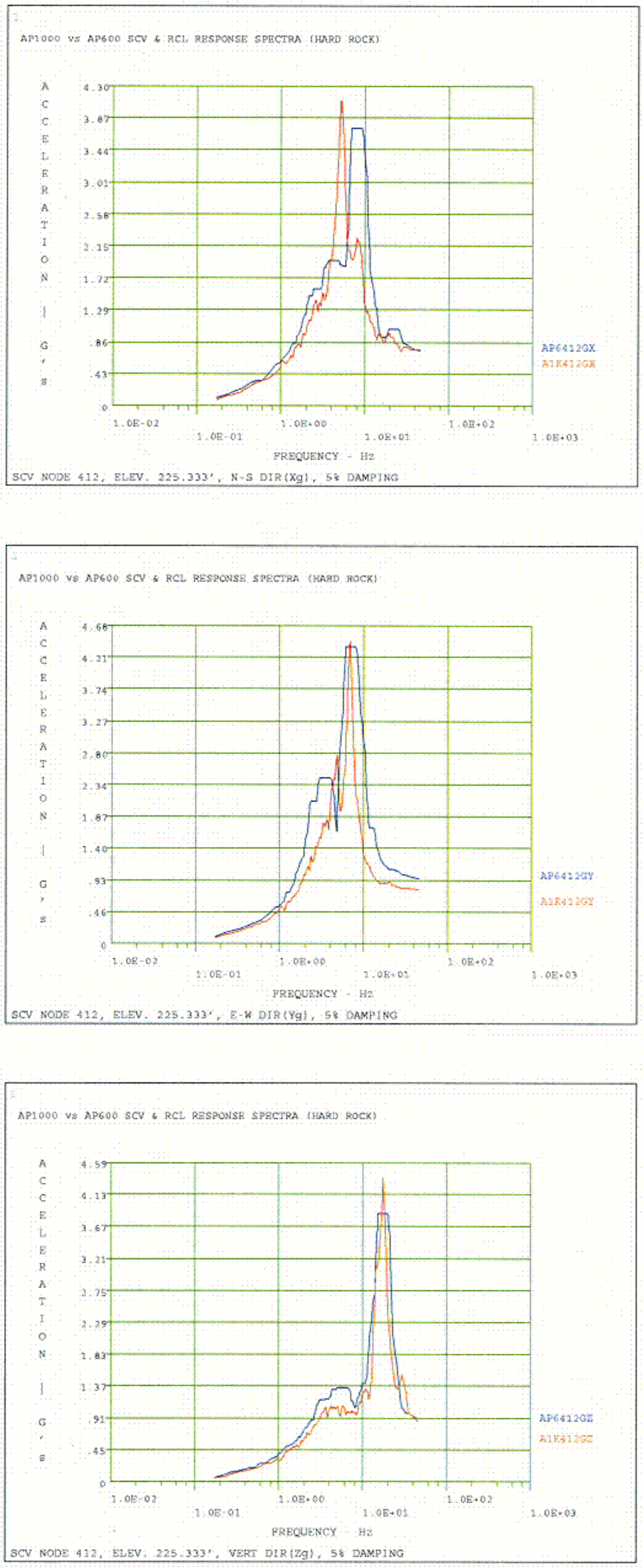


Figure B-7 Floor Response Spectra - Containment Vessel at Polar Crane Elevation

C07

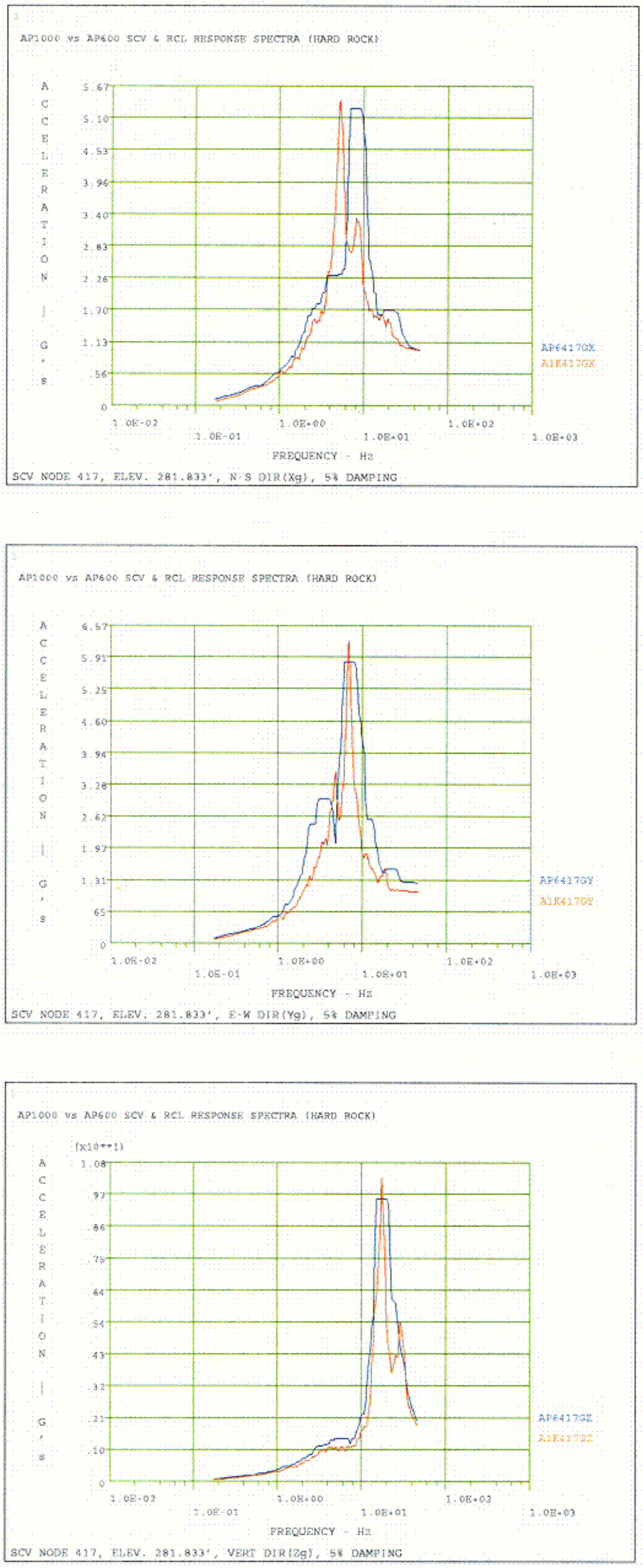


Figure B-8 Floor Response Spectra - Top of Containment Vessel

C08

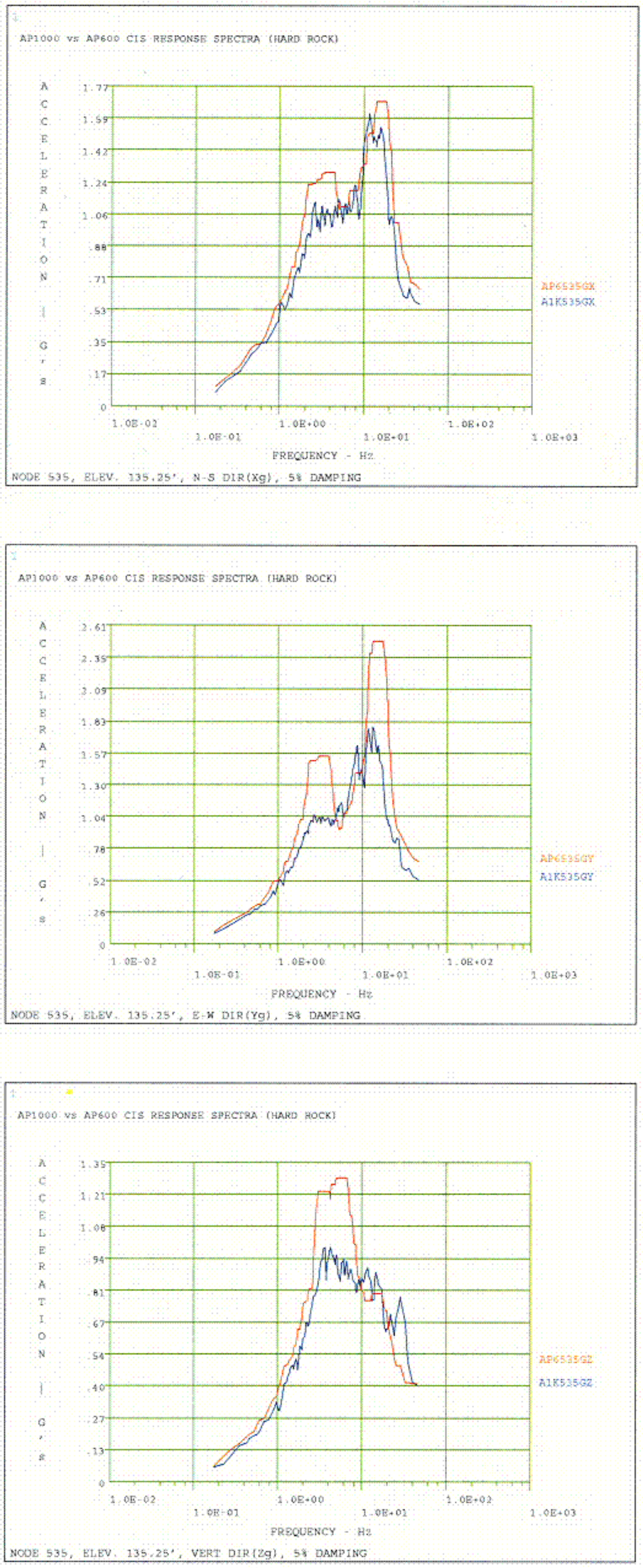


Figure B-9 Floor Response Spectra - Containment Internal Structures at Operating Floor

C09

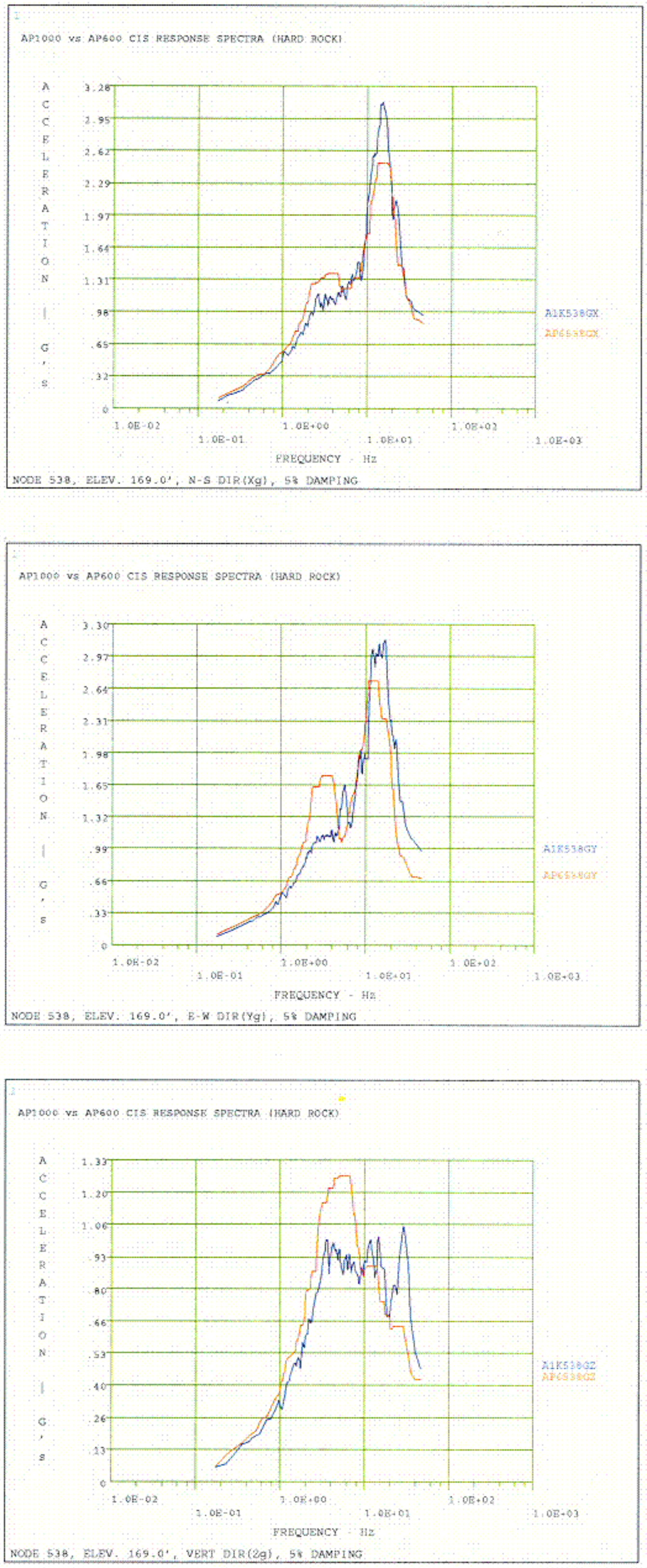


Figure B-10 Floor Response Spectra Containment Internal Structures – Top of Pressurizer Compartment

C10