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Docket No.: STN-52-003

May 20, 1994

Document Control Desk
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

ATTENTION: R. W. BORCHARDT

SUBJECT: WESTINGHOUSE RESPONSES TO NRC REQUESTS FOR ADDITIONAL INFORMATION ON THE AP600

Dear Mr. Borchardt:

Enclosed are three copies of the Westinghouse responses to NRC requests for additional information on the AP600 from your letters of March 16, 1994, April 19, 1994 and April 29, 1994. In addition, revisions of previous responses are included.

A listing of the NRC requests for additional information responded to in this letter is contained in Attachment A.

These responses are also provided as electronic files in WordPerfect 5.1 format with Mr. Hasselberg's copy.

If you have any questions on this material, please contact Mr. Brian A. McIntyre at 412-374-4334.

Nicholas J. Liparulo

Nicholas J. Liparulo, Manager
Nuclear Safety & Regulatory Activities

/nja

Enclosure

cc: B. A. McIntyre - Westinghouse
F. Hasselberg - NRR

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NTD-NRC-94-4142
ATTACHMENT A
AP600 RAI RESPONSES
SUBMITTED MAY 19,1994

RAI No.	Issue
210.008R01	Piping analysis
210.016R01	Reactor vessel internals integration
210.018R01	Reactor vessel internals testing
210.030	Acceptability of EPRI NP-6628
210.046	SSAR section 3.7.3.1
220.041R01	Soil pressure effects on embedded wall section
220.056	Finite element analysis model for basemat
220.057	Basis for use of uniform Winkler spring
220.064	Pre-operational SIT of containment in SSAR
220.065	LIst of missile sources
220.069	Computer codes used for nonaxisymmetrical loads
220.071	Structural modules
220.077	Modular construction design information in SAR
220.086	Applying seismic loads to finite element model
230.048R01	Descritpion of "design by rule" analysis
230.056	Structure to structure interaction
230.059	Comparison between SRSS and 1,.4,.4 method
230.064	Adequacy of M-O method
230.065	Lateral earth pressures on NI structure walls
230.071	Use of seismic responses for soft rock site
230.076	Use of damping ratio for cable tray systems
230.086	Effects of energy feedback
230.087	Method of ground motion combination
231.021	SRP 3.7.2 guidance
231.023	Acceptable ranges of backfill properties
231.026	Properties in SSAR Table 2A-6

NTD-NRC-94-4142
ATTACHMENT A
AP600 RAI RESPONSES
SUBMITTED MAY 19,1994

RAI No.		Issue
231.029	:	Effects of assumed Poisson ratio
920.004	:	SRP compliance reference

Question 210.8

Section 3.7.3.8.2.2 of the SSAR states that for ASME Class 1 piping equal to or less than one inch nominal pipe size and ASME Class 2 and 3 piping equal to or less than two inch nominal pipe size, one of the following three methods of analysis may be used:

a. The method for large diameter pipe described in Section 3.7.3.8.2.1 of the SSAR.

b. Equivalent static analysis.

c. Seismic qualification by experience based on the guidelines in EPRI Report NP-6628, "Procedure for Seismic Evaluation and Design of Small Bore Piping."

If the procedure for use of the equivalent static analysis as noted in Item b above is different from that described in Section 3.7.3.5 of the SSAR, revise Section 3.7.3.8.2.2 to provide a detailed description of the methodology to be used.

The staff is currently reviewing EPRI NP-6628 as a topical report, which was submitted to the staff by the Nuclear Management and Resources Council in a letter dated March 19, 1991. Pending completion of this review, the staff's position is that the methodology in this report is not acceptable. Revise Section 3.7.3.8.2.2 to remove the reference to EPRI NP-6628.

Response: (Revision 1)

There are no differences between the equivalent static analyses described in Subsections 3.7.3.5 and 3.7.3.8.2.2. The SSAR will be revised to delete the option to use NP-6628. See RAI 210.46 for additional information. ~~We believe the methodology presented in EPRI NP 6628 will be found acceptable by the NRC and should be included in the AP600 review and approval process.~~

SSAR Revision: ~~NONE~~

See the SSAR revision for RAI 210.46 for the SSAR changes.

Question 210.16

As indicated in Section 3.9.2.3 of the SSAR, the reactor vessel internals in the AP600 are similar in size and configuration to the 3-loop reactor at the H.B. Robinson plant with additional design changes from several reference reactors. However, the AP600 is not a 3-loop reactor, and effects of those design changes, although their acceptability were individually verified by separate tests in different reactors or lab conditions, may interact and result in unacceptable dynamic response. Since flow-induced excitations are complex and sensitive to a simultaneous effect of several parameters, such as configuration of flow path, pressure, temperature, flow velocity, etc., provide details of the evaluation to show how a combination of analysis, testing, and comparison to the results in several reference plants was used to verify the acceptability of flow-induced vibrations of the internals under operational transients and steady-state conditions. In addition, describe acceptance criteria and verify that the above stated evaluation, including detail drawings and calculations, was properly documented.

Response (Revision 1):

H. B. Robinson is the original Westinghouse designed three-loop reactor internals ~~plant~~ prototype. As indicated in the RAI, the three-loop plant internals are similar in size and configuration to the AP600 internals. ~~Therefore,~~ The long, successful operation of these internals provides one part of the basis for verification of the adequacy of the AP600 internals. ~~However,~~ Vibration assessments have been performed on numerous internals configurations, including those shown in Subsection 3.9.2.3 of the SSAR, providing a broad data base for prediction of the AP600 internals flow-induced vibration behavior. These data and analytical models of the AP600 internals configurations have been ~~will be~~ used to demonstrate the adequacy of the AP600 internals.

The approaches ~~to be~~ used for this program are summarized in the following. Turbulence and reactor coolant pump excitations are discussed.

- **Turbulence Excitation**

Lower Internals Response To Inlet Nozzle And Downcomer Turbulence

The dominant excitation of the lower internals is flow turbulence generated at the reactor vessel inlet nozzles and in the downcomer annulus.

Data from plant preoperational vibration measurement data and scale model flow test data for lower internals designs having neither a circular thermal shield nor neutron pads in the core barrel-reactor vessel downcomer annulus characterize the forcing functions due to inlet nozzle and downcomer annulus turbulence. These data, which are for four inlet nozzles as in the AP600 design, are ~~will be~~ used to establish a forcing function for four-loop-size internals and ~~will be~~ scaled to the lower velocity, small size of the AP600 internals. Known behavior of response variation with flow from scale model tests ~~will~~ assist in the scaling evaluation. The resulting forcing function is ~~will be~~ applied to a model of the AP600 reactor vessel core barrel, reflector, and core. The resulting responses are ~~will be~~ compared to allowable high-cycle fatigue limits at key areas on the lower internals and to design interface loads at the reactor vessel/core barrel lower restraints. The small variation of the excitation with temperature during heatup and cooldown are ~~will also be~~ evaluated in the analyses.

The vibration of the core barrel due to inlet nozzle and downcomer turbulence is proportional to flow velocity raised to a power greater than 2. Since the velocities are significantly lower than those of previous plants, the vibration levels of the AP600 are expected to be lower, providing a basis for the ~~expectation that the~~ analyses that ~~will~~ show ~~that~~ the high-cycle fatigue stresses to ~~will~~ be acceptable for the AP600.

The reflector is ~~will be~~ modeled in the analysis so that vibration of the lower internals in modes that include motion of the reflector and reflector core barrel interface loads can be calculated. These results are ~~will be~~ used to demonstrate that the lower internals design with the reflector has ~~will have~~ adequate margins against flow-induced vibration. The preoperational vibration measurement program for the first AP600 will include transducers to confirm the vibrating response and adequacy of the reflector for flow-induced vibration.

The vortex suppression ring has been designed and tested so that fluctuations in the flow patterns in the lower reactor vessel head plenum that have been observed in some previous plant designs will not be significant in the AP600. Analytical estimates of the response of the vortex suppression ring and its supporting structure ~~will~~ consider turbulence excitation, base excitation due to vibration of the core barrel, and the potential for vortex shedding.

Upper Internals Components

Coolant flow exiting the core outlet converges on the reactor vessel outlet nozzles so that the highest velocities and flow turbulence excitation levels occur on the guide tubes and support columns located near the outlet nozzles. The UPPLEN code is used to calculate the velocities and flow forces due to the coolant flowing across these components. The integrated effects of these crossflows produce beam deflections and end reactions that will be compared to similar results for upper internals components for which vibration measurements have been made. Since the guide tube and support column designs for the AP600 plant are identical to previous designs, the adequacy of the AP600 components can be verified.

~~As noted in the initial submittal,~~ The outlet nozzle velocities in the AP600 design are lower than the corresponding velocities in previously tested plants. Additionally, the outermost components of the AP600 design are more distant from the outlet nozzles than in previously tested designs. This provides a high confidence that the AP600 upper internals components ~~will~~ have adequate margins.

- **Reactor Coolant Pump-Related Excitation**

Plant data shows that internals vibration responses include contributions at reactor coolant pump rotating speed and impeller blade-passing-related frequencies. Laboratory and plant test data have been used to develop an analytical computer model (ACSTIC) to estimate the pump-related excitation forces on the reactor vessel internals. The calculated vibratory loads are added to turbulence-induced loads to determine the net vibration levels and high-cycle fatigue margins.

Summary

In summary, an extensive assessment of the adequacy of the AP600 reactor vessel internals against flow-induced vibration has been completed. ~~is planned to be finalized in the first quarter of 1994.~~ This analysis ~~will~~ utilized plant and scale model vibrations measurement results to verify the adequacy of the internals for high cycle fatigue. As discussed in the response to RAI 210.18, the lower internals of the first plant will be instrumented so that preoperational vibration measurements can be obtained to confirm the adequacy of the core barrel with the reflector. Acceptance criteria for the analysis are the ASME Code allowable high-cycle fatigue stresses and design loads at interfaces calculated directly or inferred by comparison of the AP600 results to previously analyzed/tested designs.

SSAR Revision: NONE

Question 210.18

Since the AP600 design has different coolant loop configuration from the design of H.B. Robinson plant (see Q210.16), and it also has incorporated additional design changes from several reference plants, it is difficult to visualize the assertions that the reactor internals of the H.B. Robinson design is the valid representative for the AP600 internals. A vibration measurement program should be implemented per RG 1.20 during the preoperational test for either the first AP600 internals or the internals similar to the AP600 but with some design modifications (the Non-prototype Category II). Provide detailed information regarding the vibration measurement program, including numbers, types and locations of sensors, the basis of sensor selection and analyses for predicting levels of response of individual sensors. In addition, acceptance criteria of vibration measurements should also be described (Section 3.9.2.4).

Response (Revision 1):

As indicated in the response to RAI 210.16, in addition to the H. B. Robinson experience, data for internals responses are available for geometries similar to the AP600 geometry for verification of the adequacy of the AP600 internals.

Consistent with the guidelines of Regulatory Guide 1.20, a preoperational vibration measurement program will be conducted on the first AP600 plant to confirm the adequacy of the core barrel-reflector configuration used in this design.

Westinghouse has successfully completed preoperational vibration measurement programs on seven plants. These programs included strain gages and accelerometers mounted on the internals to measure structural responses during hot functional test heatup, steady operation of several combinations of reactor coolant pumps, and startup and shutdown of reactor coolant pumps.

A preoperational measurement program is planned ~~intended~~ for the initial AP600 plant. The test plan~~s will be~~ has been developed in conjunction with the analysis described under Response 210.16. This plan ~~These plans will~~ includes:

- The locations and types of transducers to be installed
- The bases used to establish expected and acceptable vibration levels and expected natural frequencies. The final values established for expected and acceptable levels will be established prior to the start of testing.
- The conditions at which data are to be acquired

SSAR Revision: NONE

Question 210.30

The response to Q210.8 dated December 22, 1992 is not completely acceptable. At the request of the Nuclear Management and Resources Council (NUMARC), the staff's review of EPRI NP-6628 has been put on hold pending a decision by NUMARC relative to the continuation of this review. To date, the staff has not accepted an experience-based approach for the seismic design of safety-related piping systems. Therefore, the staff's position remains that EPRI NP-6628 is currently not acceptable. Revise Section 3.7.3.8.2.2 of the SSAR and the response to Q210.8 to remove the reference to this report.

Response:

The SSAR will be revised to delete the reference to EPRI NP-6628 and to the design by rule option. See RAI 210.46 for additional information. The response to RAI 210.8 will also be revised.

SSAR Revision:

See the SSAR revision for RAI 210.46 for the SSAR changes.

Question 210.46

Section 3.7.3.1 of the SSAR states that one of the methods used for seismic analysis is "design by rule." Revise this section to define this term and to reference those sections of the SSAR which contain design by rule methods.

Response:

Design by rule refers to use of EPRI Report NP-6628, "Procedure for Seismic Evaluation and Design of Small Bore Piping." The Nuclear Energy Institute has withdrawn the request to have the NRC approve use of the methods in this report. Design by rule will be removed from the AP600 SSAR as a method for seismic analysis of subsystems. Industry efforts are continuing to include the methods of NP-6628 in the ASME Code, Section III.

SSAR Revision:

Revise the paragraph under Subsection 3.7.3.1 as follows:

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

Revise the paragraph under Subsection 3.7.3.8.2.2 as follows:

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

- Same method as described in Subsection 3.7.3.8.2.1; or,
- Equivalent static analysis based on appropriate load factors applied to the response spectra acceleration values. ~~; or,~~
- ~~Seismic qualification by experience is based on the guidelines provided by EPRI Report NP6628 (Reference 12). Since there are no calculated values of seismic pipe stress, this method is not applied to systems requiring a load combination with seismic (for example, loads due to relief valve discharge combined with safe shutdown earthquake loads).~~

Revise Reference 12 of Subsection 3.7.5 as follows:

12. Deleted. ~~"Procedure for Seismic Evaluation and Design of Small Bore Piping," NCIG-14 EPRI NP6628, April 1990.~~

Question 220.41

Discuss the design of the embedded portion of the exterior walls of the nuclear island of seismic Category I structure and the methods for the consideration of static soil pressure and the soil pressure induced by the earthquake. Westinghouse should follow the guidelines documented in the staff position for the embedded wall and retaining wall design. Evaluate the potential local soil failure around the embedded walls during the design seismic event (Section 3.8.4 of the SSAR).

Response: (Revision 1)

The embedded portions of the exterior walls of the nuclear island are designed for dead loads, live loads, SSE loads, hydrostatic loads due to groundwater and probable maximum flood, static soil pressure loads, surcharge loads, and soil pressure induced by the SSE.

The walls are designed according to ACI 349 with the load combinations given in Table 3.8.4-2.

The static soil pressure is based on at-rest soil pressure. ~~The soil pressure induced by the SSE is based on the Mononobe-Okabe formula. Since the exterior walls are assumed to be non-yielding, the forces obtained by the Mononobe-Okabe formula are multiplied by two.~~ Two-dimensional SSI analysis results are being used to establish the soil pressure induced by the SSE and to verify the structural integrity of the walls. This methodology follows the guidelines documented in the staff position for the embedded wall and retaining wall design. These SSI analyses include consideration of surcharge and energy feedback from the adjacent structures. The potential for local soil failure is also considered in the design.

Results of this evaluation will be submitted by July 30, 1994.

SSAR Revision :

Add the following paragraph after the second paragraph of Subsection 3.8.4.4.1:

The embedded portions of the exterior walls of the nuclear island are designed for dead loads, live loads, safe shutdown earthquake loads, hydrostatic loads due to groundwater and probable maximum flood, static soil pressure loads, surcharge loads, and soil pressures induced by the safe shutdown earthquake. The static soil pressure is based on at-rest soil pressure. Two dimensional soil structure interaction analyses are used to establish the soil pressure induced by the safe shutdown earthquake. These two dimensional soil structure interaction analyses include consideration of surcharge and energy feedback from the adjacent structures.

Question 220.56

Provide the following information pertaining to the finite element analysis model for the basemat: (a) the input seismic loads at the various nodes, (b) the spring connecting the internal structure to the basemat, and (c) the soil springs attached to the basemat.

Response:

Please see response to RAI 220.86.

SSAR Revision: NONE

Question 220.57

Provide the basis for using a uniform Winkler spring in the foundation analyses instead of the expected variable stiffness from edge to center of foundation mat.

Response:

The soil elastic stiffness of the foundation has been expressed in terms of a uniform Winkler spring. The spring coefficient was taken as the average at the edge and center of the basemat. Deflections of the basemat were calculated for a uniform pressure applied to an equivalent rectangular flexible basemat. For a flexible basemat the deflections at the corners are twice the deflections at the center. This would lead to an expected variation in stiffness such that the stiffness at the center is twice that at the corners. The basis for use of the average value considered the following:

- the nuclear island structure with the shear walls and floors of the auxiliary building integrally connected to the shield building provides a stiff basemat
- increase in the stiffness at the edges will tend to increase the bending moments in the basemat

SSAR Revision: NONE

Question 220.64

Provide, in the SSAR, the critical locations for taking measurements during the pre-operational structural integrity test (SIT) of the steel containment and describe how this information is to be used to demonstrate the consistency between the observed and predicted responses.

Response:

See response to RAI 220.26

SSAR Revision: NONE

Question 220.65

Provide a list of potential sources of missiles and sources of high pressure resulting from a high energy line break between (a) the containment and operating floor and refueling cavity walls, (b) the secondary shield walls and the containment, and (c) the containment and the shield building.

Response:

See response to RAI 220.27

SSAR Revision: NONE

Question 220.69

Describe how the containment was modeled and what computer code was used when the containment shell was analyzed for the non-axisymmetrical loads due to earthquake and crane loads.

Response:

The containment vessel was modelled as an axisymmetric shell. Asymmetric loads were represented by Fourier harmonics. The analyses used CBI computer codes, which are summarized below.

General Shell of Revolution Stress Analysis (CBI Computer Program 0781)

This program calculates displacements and stresses in thin walled, elastic, shells of revolution when subjected to static edge, surface, and/or temperature loads with arbitrary distribution over the surface of the shell. This program was originally developed by Arturs Kalnins at Yale University, and is based on his method of analysis presented in the Journal of Applied Mechanics, Volume 31, September 1964. Since 1966, CBI's version of the program has been extensively modified and enhanced.

The structure to be analyzed is modelled in parts which end at logical changes in geometry, thickness, or loading. Each part is further subdivided into segments that are internally equivalent to finite elements. However, each segment does not have an assumed displacement function. Instead, the thin shell differential equa.ions are integrated within each segment to determine an influence matrix describing the relationship between forces and deflections at each end of the segment.

In particular, the program solves the H. Reissner-Meissner equations by reducing these equations to eight ordinary differential equations in eight unknowns. The eight unknowns are chosen as those which appear on the boundaries of the axially symmetric shell so that the entire problem can be expressed in terms of these fundamental variables. A very accurate 4th order Runge-Kutta numerical integration scheme is then used to integrate the differential equations. Finally, the remaining solution uses a Gaussian elimination technique to solve the resulting set of simultaneous equations for the unknown displacements and forces.

Special features include the use of stiffness matrices at model boundaries and part junctions, the use of materials that may be orthotropic, and the representation of nonaxisymmetric loads with Fourier series. The model can include up to nine branches and seven material layers through the thickness. Both material properties and layer thicknesses can vary using separate linear functions along the length of a part. In addition, two model boundaries can be connected to form a closed loop.

General Shell of Revolution Stress Analysis (Dynamic Version) (CBI Program 1374)

This program calculates the stress and displacements in thin walled elastic shells of revolution when subjected to either static or dynamic loading over the surface of the shell. The program is based on a program originally developed by Arturs Kalnins at Yale University and on his multi-segment, numerical integration technique that was presented in the Journal of Applied Mechanics, Volume 31, September 1964. Each problem is broken down into segments small enough such that the governing differential equations can be accurately integrated using numerical integration techniques to determine the segment stiffness. The eight ordinary differential equations of thin shell are those derived by H. Reissner. The equations are derived such that the eight variables are chosen which appear on the boundaries of the axially symmetric shell so that the entire problem can be expressed in these fundamental variables. Three different types of analyses can be performed:

1) Static analysis can be performed for any arbitrary loading distribution. Longitudinally, concentrated loads may be applied at panel ends and distributed loads may be applied varying linearly between specified points within each panel. The circumferential distribution is obtained through the use of Fourier Series.

2) Natural frequencies, mode shapes (displacements and forces), and participation factors for any loading that can be handled statically can be calculated and output to a file for use in either a spectral analysis, when a response spectrum is available, or in a modal superposition analysis, when transient forcing functions of the form $g(s,\theta)\ f(t)$ are available. The program can handle extra concentrated and distributed mass acting in any or all directions plus fluid structure interaction.

3) Direct time integration analysis for general transient problems can be calculated where the forcing function cannot readily be separated into separate spatial and temporal functions. As in natural frequency analysis, additional concentrated and distributed masses acting in any and all directions can be applied. Pressures can vary in an arbitrary fashion versus time and damping may be included.

The geometry of the shell is made up of spheres, torispheres, ellipsoids, plates, and cylinders, with or without stiffeners. Generally, problems are modeled as isotropic, with the cross section and geometric configuration varying only at discrete points. Using a special geometry option, however, linear variation in cross sectional properties and non-isotropic properties can be varied along the length of a panel. In addition, spring matrices may be applied at model boundaries as well as at panel end points.

SSAR Revisions: NONE

Question 220.71

Describe plans, criteria, or specifications for fabrication, storage, transportation, handling, assembly, inspection, and QA/QC related to structural modules. This information, including goodness of fit, inspection and hold points, and sequence of construction, should be included in the SSAR.

Response:

Fabrication, assembly, and inspection of the structural modules in the nuclear island is the same as for structures constructed of structural steel and will follow the guidelines from AISC-N690 and AWS D 1.1. The response to RAI 220.77 discusses the inspection in greater detail.

Structural module packaging, transportation, receiving, storage, and handling will be in accordance with ASME NQA-2, Part 2.2, 1989 Edition.

Proper fit up between sub-modules will be provided by the following:

- Use of horizontal and vertical datum lines
- Sub-modules will be cut-to-fit to exact size including allowance for shrinkage
- Use of erection stock on corner sub-modules

Inspection and sequence of construction are addressed in the response to RAI 220.73.

Quality assurance and quality control for the structural modules will be in accordance with ANSI/ASME NQA-1.

SSAR Revision:

Revise SSAR Subsection 3.8.3.6.1 as follows:

3.8.3.6.1 Special Construction Techniques

Modular construction techniques are used extensively in the containment internal structures. The modular construction approach uses both off-site and on-site module pre-fabrication. Subassemblies, sized for commercial rail shipment, are assembled off-site and transported to the site. On-site fabrication consists of combining the subassemblies in structural modules, which are then installed in the plant.

Structural modules are used in the construction of the secondary shield walls, in-containment refueling water storage tank, refueling cavity, operating floor, and other miscellaneous areas.

The use of concrete filled steel structures is a proven construction method and has been used successfully in the nuclear industry for years. It has been used for reactor vessel pedestals and shield walls for boiling water reactors, spent fuel pools and refueling cavity walls, the liner plate and stiffeners on prestressed containment walls, and as left in place steel forms in various locations. Construction techniques are similar to those of conventional concrete and steel structures as described below:

- Fabrication, assembly, and inspection is in accordance with AISC-N690 and AWS D 1.1. Inspection includes verification for conformance to the drawings, inspection of workmanship, plumb, and square, and visual and nondestructive examination (NDE) of welds.

- Structural module packaging, transportation, receiving, storage, and handling is in accordance with ASME NQA-2, Part 2.2.

- Concrete placement is in accordance with American Concrete Institute (ACI) standards. Inspection includes cleanliness, temperature, protection against inclement weather, method of concrete placement to avoid segregation, depths of concrete placement, and adequate vibration.

Question 220.77

For the modular construction design, provide detailed information in the SSAR regarding (a) the construction sequence, (b) the plan for inspection during fabrication, (c) the inspection plan for pouring concrete, (d) the measurements for controlling curing and corrosion, (e) the connection joint details, (f) the details at intersection of modular walls, (g) the connection between two modules, and (h) the connections between the modules and pour-in-place concrete elements.

Response:

a. The construction plan is not required for the NRC safety determination and is not part of the AP600 licensing documentation. The plan is availble for review by the NRC at Westinghouse's Rockville office.

b. Inspection during fabrication of the structural modules will be the same as for structures constructed of structural steel and will follow the guidelines from AISC-N690 and AWS D 1.1. Inspection will include verification for conformance to the drawings, inspection of workmanship, plumbness, squareness, and visual and nondestructive examination (NDE) of welds.

c. The inspection plan for concrete placement will be similar to reinforced concrete work and will follow the guidelines from American Concrete Institute (ACI) standards. Inspection will include cleanliness, temperature, protection against inelement weather, method of concrete placement to avoid segregation, depths of concrete placement, and adequate vibration.

d. Following standard ACI procedures for maintaining the proper placing temperatures and use of low slump concrete to minimize the amount of water in the concrete will control curing and corrosion.

e. See respoase to RAI 220.73.

f. See response to RAI 220.73.

g. See response to RAI 220.73.

h. See response to RAI 220.73.

See response to RAI 220.71 for SSAR revisions

SSAR Revision: NONE

Question 220.86

For the design of the NI foundation, provide the detailed procedures for applying: (a) the structural seismic loads to the finite element foundation analysis model, (b) the springs attached to the bottom nodes of the basemat, and (c) the springs connecting the internal structures to the basemat.

Response:

a. The basemat of the nuclear island is analyzed using a finite element model with the computer program ANSYS. The finite element model of the basemat extends to elevation 100' for the auxiliary building and to elevation 236' for the shield building. The basemat, walls, and slabs have been simulated by shell type elements. The soil below the basemat is represented by springs attached to nodes of the basemat model. Figure 3.8.5-2 in the SSAR shows representative features of the model. The containment internal structures have been simulated with tetrahedral elements. The containment internal structures are connected to the basemat through spring elements normal to the surface of the containment vessel.

   The SSE forces and moments (axial force, N-S shear force, E-W shear force, torque, maximum moment about N-S axis, maximum moment about E-W axis), obtained from the 3D lumped mass stick model analyses, are used in the analysis and design of the nuclear island basemat. The forces and moments are applied to the finite element model as follows:

   1. The forces and moments at elevation 100' are distributed to the walls at elevation 100' based on the stiffness and area of the walls. The torque is converted to shear forces and the moments are converted to tension and compression forces.

   2. The forces are applied as static concentrated loads to the nodes of the finite element model at elevation 100'.

   3. An equivalent static acceleration is applied to the finite element below elevation 100'.

   The seismic loads from the various structures in each direction on the nuclear island were considered to be in phase. The responses due to seismic loads in the three directions were combined using the 1.0, 0.4, 0.4 method as described in the response to RAI 230.87.

b. The foundation is simulated by a soil elastic foundation stiffness capability included in the basemat shell type elements, as well as a horizontal (in north-south and east-west directions) spring system attaching to 175 nodes uniformly distributed over the basemat. A uniform vertical stiffness of 518 kips per cubic foot was used as described in the response to RAI 220.57. This vertical stiffness was based on the elastic deflection of a rectangular foundation on a semi-infinite half space. Horizontal springs were used having a stiffness equal to one half of the vertical stiffness in order to distribute the horizontal reaction uniformly.

c. The springs between the internal structures and the basemat represent the thickness of the steel vessel and are oriented normal to the surface of the containment vessel.

SSAR Revision: NONE

Question 230.48

Provide a detailed description regarding the "design by rule" analysis method in the SSAR and discuss what activities are underway for adoption of this method by a consensus code or standard (Section 3.7.3.1 of the SSAR).

Response: (Revision 1)

The "design by rule" method for small bore piping is based on EPRI Report NP-6628 as described in SSAR, Revision 1, subsection 3.7.3.8.2.2. The SSAR will be revised to delete the design by rule option. See RAI 210.46 for additional information.

SSAR Revision: ~~NONE~~

See the SSAR revision for RAI 210.46 for the SSAR changes.

Question 230.56

Regarding the structure-to-structure interaction:

a. evaluate the potential pounding between the NI structures and the non-seismic Category I structures, and

b. evaluate the potential of structure-to-structure interaction through soil to ensure the integrity of both Category I and Category II structures.

Response:

a. The structural separation between the nuclear island and the adjacent structures is established to prevent pounding between the structures. The second paragraph of Subsection 3.8.5.1 of the SSAR states that the structural separation is as follows:

- At and below plant grade, the adjoining buildings are separated from the nuclear island by a two inch gap.
- Above plant grade, the adjoining buildings are separated from the nuclear island by a four inch minimum gap.

SSAR Subsection 3.7.2.8 states that the minimum space required between structures to avoid contact is obtained by performing either a time history or a response spectrum analysis for each structure. If these analyses indicate that separation greater than 4 inches is required, then the separation will be increased. It is concluded that there is no potential pounding between the nuclear island and the non-seismic Category I adjoining buildings.

b. As discussed in the response to RAI 230.17, the surface founded, relatively light weight adjoining structures are expected to induce negligible effect on the soil structure interaction response of the massive nuclear island.

Floor response spectra for the grade elevation of the nuclear island are shown in Sheet 1 of Figures 2A-29, 2A-30 and 2A-31. The spectra for hard rock correspond to the ground input motion. The spectra for the soil sites show little amplification above the hard rock spectra. This indicates that the adjacent buildings will not be significantly driven by the nuclear island. Hence design of the adjacent buildings for seismic input without consideration of the effects of the nuclear island is adequate. It is concluded that the potential of structure-to-structure interaction through soil is negligible in the design of both the nuclear island and the adjacent buildings.

SSAR Revision: NONE

Question 230.59

Provide, in the SSAR, a comparison between the SRSS method and the 1.0, 0.4, 0.4 method, or the bases for use of only the 1.0, 0.4, 0.4 method for the combination of seismic loads. Also Q220.67.

Response:

Note: The reference to Q220.67 should be to Q220.66

In the AP600 two methods are permitted for the combination of the effects due to three spatial components of an earthquake using response spectrum methods. The SRSS method is identified in Regulatory Guide 1.92 as an acceptable method to combine maximum structural response values associated with each of the three components of earthquake motion. Co-directional structural responses of interest (eg., stress, deflection, strain, seismic anchor motion) are calculated for each of the three components of earthquake motion. The term "co-directional response" indicates that it is a unidirectional response with contributions from each of the three directions of seismic input. The co-directional responses due to the three directions of seismic input are combined by the SRSS method in order to obtain the estimated maximum response. This is appropriate when the design methods are based on allowable stresses or deflections for a single direction of response. Certain formulations (eg., principal stress) may become overly conservative when using the SRSS method since stresses in two directions are each taken at their estimated maximum response. For these cases the 40% method is considered appropriate.

The 1.0, 0.4, 0.4 method, referred herein as the 40% method, is appropriate for nuclear plant applications. An example of two references that allow its use are given below:

NUREG/CR-0098, "Development of Criteria for Seismic Review of Selected Nuclear Power Plants," Newmark and Hall, May 1978, Prepared for U.S. Nuclear Regulatory Commission, p. 30.

> "It is conservative, simpler, and much more readily defined and calculated to take the combined effects as 100 percent of the effects due to motion in one particular direction and 40 percent of the effects corresponding to the two directions of motion at right angles to the principal motion considered. It is this combination that is recommended for general use, especially in nuclear power plant design."

ASCE Standard, "Seismic Analysis of Safety-Related Nuclear Structures and Commentary on Standard for Seismic Analysis of Safety Related Nuclear Structures," ASCE 4-86, American Society of Civil Engineers, September 1986, Section 3.2.7.1.2, pp. 24 and 25.

> "Alternatively, the responses may be combined directly, using the assumption that, when the maximum response from one component occurs, the responses from the other two components are 40% of the maximum. In this method, all possible combinations of the three components, ..., including variations in sign (plus or minus), shall be considered. ... ."

To further support the use of the 40% seismic criteria method, comparisons between the SRSS method and the 40% method are given below. Combinations of maximum co-directional component responses and principal stresses are considered. In order to compare the results from the SRSS and 40% methods, the results obtained from these two methods are compared to those obtained from a time history analysis. A comparison of the two methods is also provided in the response to RAI 220.29 for the containment vessel.

1. Combination of maximum co-directional component responses

   A representative set of co-directional responses are assumed having different relationships between these responses. Figure 230.59-1 shows these relationships for each of one hundred cases. The co-directional response for the X shock, Y shock, and Z shock have all been normalized by the maximum response. There are various cases that include two of the components of equal magnitude, three of equal magnitude, and many cases in between with one component being dominant. Figure 230.59-2 shows the formulation and results of SRSS and 40% combination methods. In only one case does the 40% combination method yield results that are lower (only 1%, 1.414 versus 1.4) than the SRSS method. This is when two of the components are equal, and the other is zero.

2. Principal Stresses

   Principal stresses in a plate were studied along with the maximum shear stress and stress intensity. The sum of σ_Y and τ is also included in the study since it is representative of design for tangential shear. It was assumed that there was a shear stress (τ) directly proportional to the X seismic input, that one membrane stress was zero, and that the other membrane stress (σ_Y) was a combination of the Y and Z input. This would be representative of the seismic response of a shear wall or the containment vessel. The magnitudes of the X, Y and Z responses were those shown in Figure 230.59-1.

   Two of the cases are shown as examples in Table 230.59-1. Case 1 has the response components X and Y equal with Z zero. Case 100 has the response components X, Y, and Z all equal. The ratio between the 40% method and the SRSS results range from 0.75 to 0.91 for the various principal stress combinations, and are 0.70 to 0.75 for the combination of σ_Y and τ. The 40% method results are lower than the SRSS methods by as much as 30%. The reason for the difference is that the SRSS method does not reflect the statistical independence of the individual co-directional responses.

   The results, along with the associated SRSS and 40% method formulations, are shown in Figures 230.59-3. Two combinations were studied so as to reflect the effect of sign of the components on the results. One combination considered all of the co-directional responses X, Y, and Z as positive, while the other considered Y and Z as negative, and X positive. The results were similar with Sigma 1 (σ_1) and 2 (σ_2) reversing themselves. The results for the 40% method and the SRSS method are similar to those given in Table 230.59-1, recognizing that the SRSS method tends to reflect the absolute summation of responses in complex motions.

3. Time History Comparison Results

   The 40% method and the SRSS method were compared against results using two sets of time histories. The first set of time histories were the seismic input time histories as described in SSAR Subsection 3.7.1, which are of equal magnitude (0.3g) and are statistically independent. In addition, arbitrary time histories were developed as shown in Figure 230.59-4. For these time histories no attempt was made to assure that each component resulted from statistically independent motions. These time histories were considered as the component responses (X, Y, and Z) for the same examples of co-directional component response and principal stresses discussed previously. As in the first section that discussed co-directional component response cases, the maximum co-directional responses associated with the X, Y, and Z components represent the stresses as used in the respective formulations as shown on Figure 230.59-2 and 230.59-3. The results are shown in Table 230.59-2. For the co-directional resultant response, the 40% method produced results equal to 89% of the time history method and the SRSS method gave results equal to 85% of the time history method. For the principal stresses, the results obtained using the SRSS combination method are the more conservative. The results obtained for the 40% combination method are close to the time history results with the smallest result being smaller by only 12 percent. Note that these examples were selected specifically to maximize the difference between the various methods and more practical cases would not show as much difference.

In conclusion, the 40% combination method provides realistic results that are not overly conservative. The 40% method is a valid method for combining multiple directional seismic responses. This method provides a margin for those design cases involving combinations of multi-directional responses that is consistent with the margin obtained by use of the SRSS combination for a co-directional response.

SSAR Revision: NONE

Table 230.59-1 Principal Stress Example

Stress Component	Seismic Response Due to X, Y, Z Input			SRSS	40% Method				Ratio 40% to SRSS
Case 1	X	Y	Z		1, .4, .4	.4, 1, .4	.4, .4, 1	Max	
σ_Y	0	1	0	1.0	0.40	1.0	0.40	1.0	1.00
τ	1	0	0	1.0	1.0	0.40	0.40	1.0	1.00
τ_{max}				1.118	1.020	0.640	0.447	1.020	0.91
σ_1				1.618	1.220	1.140	0.647	1.220	
σ_2				-0.618	-0.820	-0.140	-0.227	-0820	
Max. Abs σ_1, σ_2				1.618	1.220	1.140	0.647	1.220	0.75
SI				2.236	2.040	1.280	0.874	2.040	0.91
$\sigma_Y + \tau$				2.0	1.40	1.40	0.80	1.40	0.70
Case 100	X	Y	Z		1, .4, .4	.4, 1, .4	.4, .4, 1	Max	
σ_Y	0	1	1	1.414	0.80	1.40	1.40	1.40	0.99
τ	1	0	0	1.000	1.0	0.40	0.40	1.0	1.0
τ_{max}				1.225	1.077	0.806	0.806	1.077	0.88
σ_1				1.932	1.477	1.506	1.506	1.506	
σ_2				-0.518	-0.677	-0.106	-0.106	-0.677	
Max. Abs σ_1, σ_2				1.932	1.477	1.506	1.506	1.506	0.78
SI				2.449	2.154	1.612	1.612	2.154	0.88
$\sigma_Y + \tau$				2.414	1.80	1.80	1.80	1.80	0.75

Table 230.59-2 - Time History Comparisons

	AP600 Time Histories			Arbitrary Time Histories Figure 230.59-4		
Stress State	Time History	SRSS	40%	Time History	SRSS	40%
Co-directional Resultant Response	0.61	0.52	0.54	2.31	2.72	3.01
Principal Stresses						
Max Shear Stress	0.31	0.37	0.32	1.98	2.17	2.02
Max Principal Stress	0.49	0.58	0.45	2.89	3.11	2.55
Stress Intensity	0.63	0.73	0.65	3.96	4.33	4.04
Sigma Y +Shear Stress	0.61	0.72	0.54	2.64	3.84	3.01

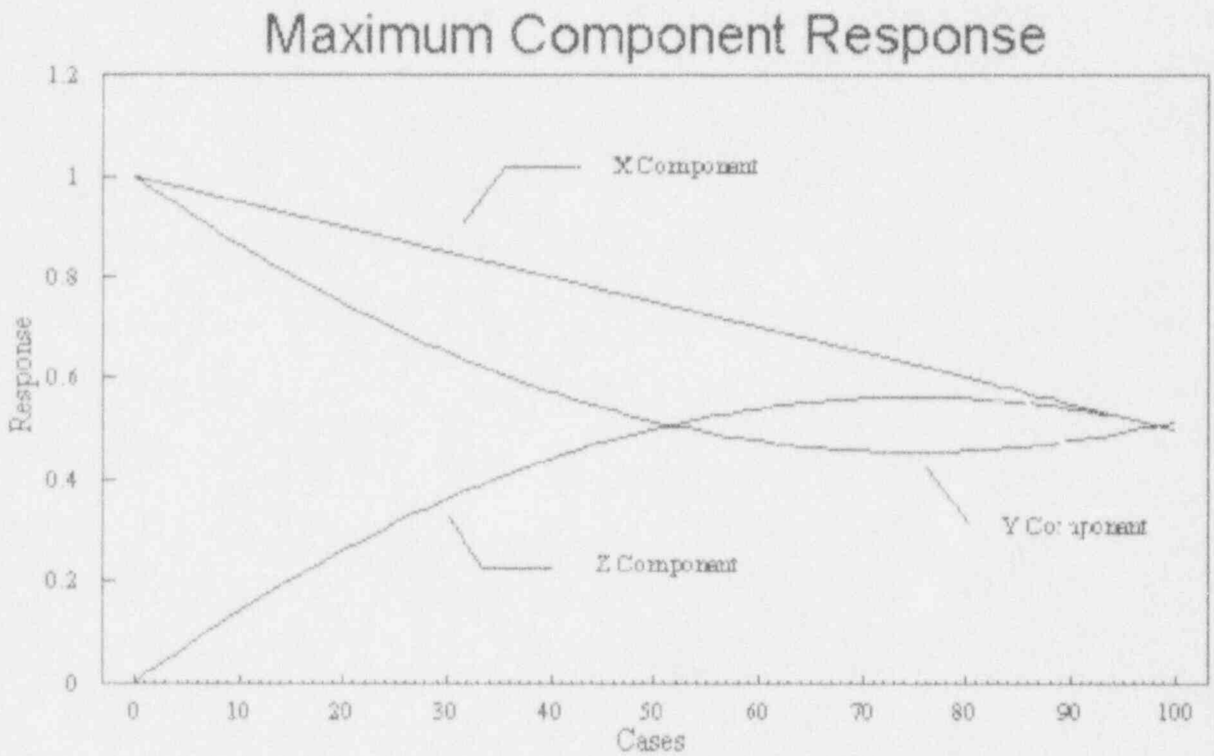
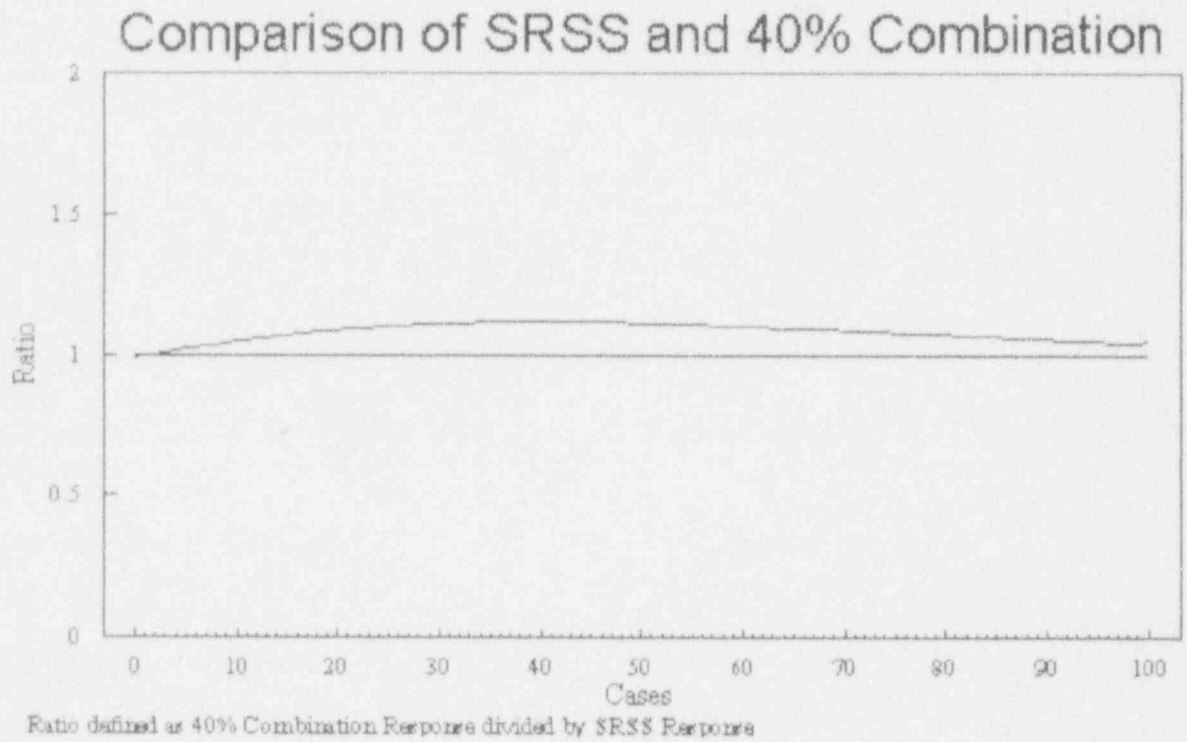


Figure 230.59-1 - Relationships between Maximum Component Responses for X, Y, Z

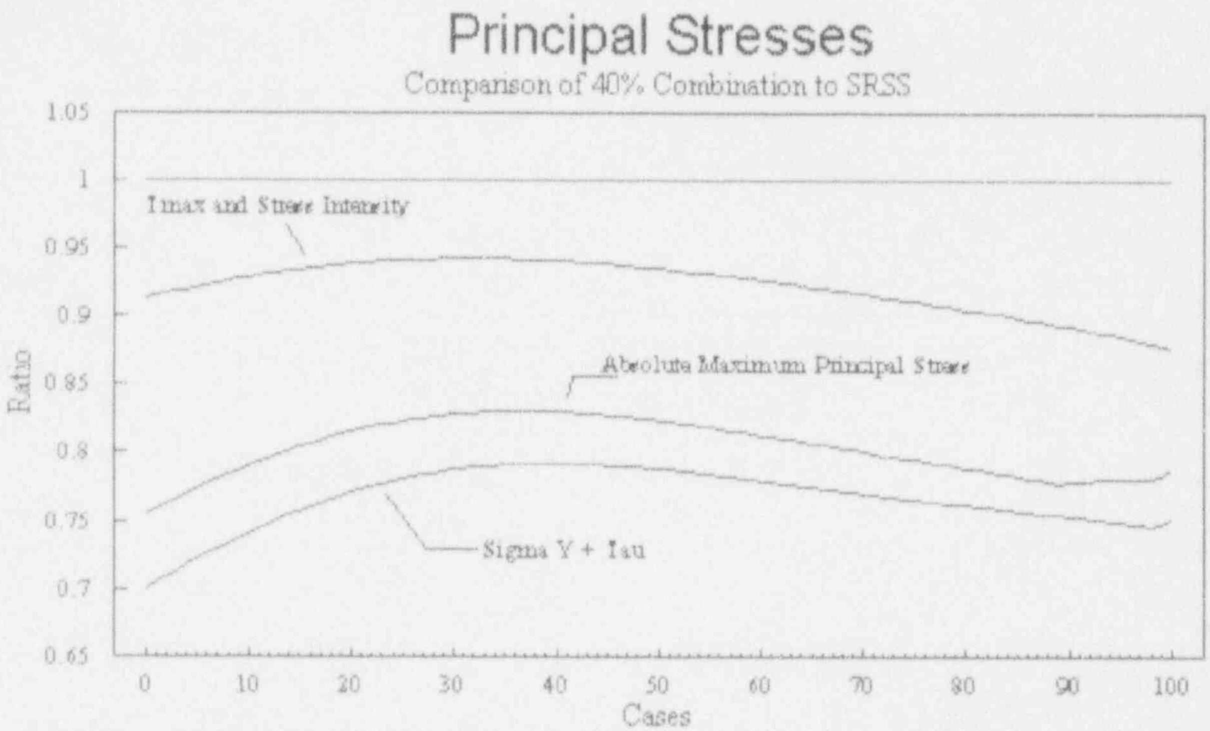


Formulations

$$SRSS_{Response} = (X^2 + Y^2 + Z^2)^{1/2}$$

$$40\%_{Response} = Max\ [\ (X + 0.4(Y + Z));\ (Y + 0.4(X + Z));\ (Z + 0.4(X + Y))]$$

Figure 230.59-2 - Resultant Response Comparisons



Ratio defined by 40% Combination value divided by SRSS value

Formulations

General

$\sigma_X = 0$; $\sigma_Y = f(Y,Z)$; $\tau = g(X)$

$\tau_{max} = [\sigma_Y^2 + 4\tau^2)^{1/2}]/2$

$\sigma_1 = (\sigma_Y/2) + \tau_{max}$

$\sigma_2 = (\sigma_Y/2) - \tau_{max}$

Stress Intensity = Max Absolute Value of $[(\sigma_1 - \sigma_2), \sigma_1, \sigma_2]$

SRSS

$\sigma_Y = (Y^2 + Z^2)^{1/2}$, note that σ_Y retains the sign of Y and Z; $\tau = X$

τ_{max}, σ_1, σ_2 = As Shown Above

40% Combination

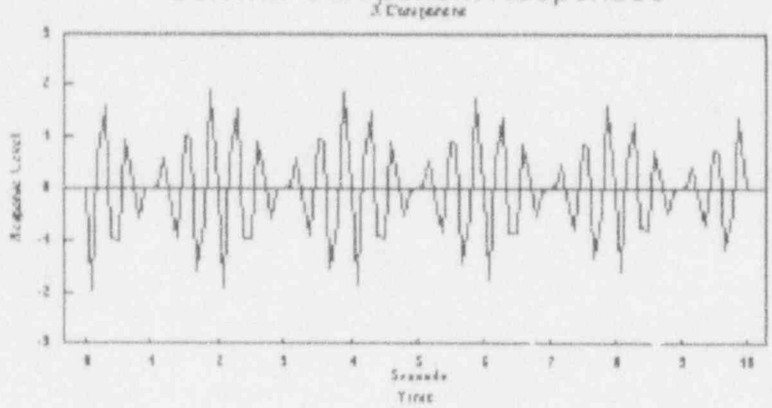
$\sigma_Y = \beta Y + \gamma Z$

$\tau = \alpha X$

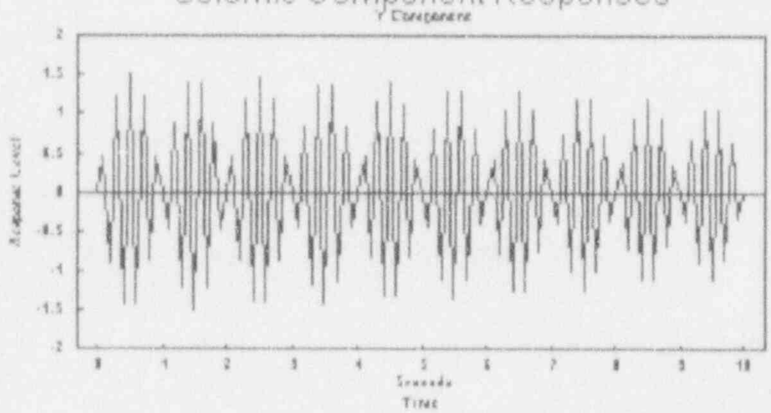
τ_{max}, $(\sigma_1)_{max}$, $(\sigma_2)_{max}$, Stress Intensity defined as the absolute max value (note sign retained) from results for three sets of (α, β, γ). Where the three sets of $(\alpha, \beta, \gamma) = [(1., .4, .4); (.4, 1., .4); (.4, .4, 1.)]$

Figure 230.59-3 - Principal Stress Comparison

Seismic Component Responses



Seismic Component Responses



Seismic Component Responses

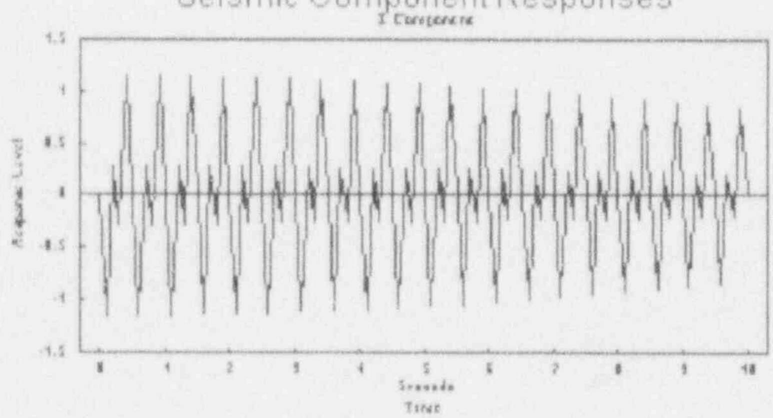


Figure 230.59-4 - Time History Response Characteristics for X, Y, and Z Components

Question 230.64

As discussed during the January 20 and 21, 1994 meeting, the lateral soil pressure on the embedded walls of the NI structures are being calculated using the Mononobe-Okabe (M-O) method, which is considered appropriate for computing soil loads developed on simple retaining walls. Provide a discussion on the adequacy of using the M-O method to compute soil pressures the embedded walls of the NI structures where wall movement relative to the surrounding soil may not develop failure strains in the soil.

Response:

Please see responses to RAIs 220.41 and 220.84.

SSAR Revision: NONE

Question 230.65

For calculating the lateral earth pressures on the embedded NI structure walls, provide justification for not considering the energy feedback between the nuclear island and immediately adjacent structures.

Response:

Energy feedback between the nuclear island and immediately adjacent structures is considered in calculating the lateral earth pressures on the embedded nuclear island walls. See revision 1 of the response to RAI 220.41.

SSAR Revision: NONE

Question 230.71

Justify the adequacy of using the seismic responses (force, shear and moment) corresponding to the soft rock site condition instead of the seismic response envelopes for the foundation design for all site conditions.

Response:

The maximum member forces in the stick models due to the safe shutdown earthquake are shown in SSAR Tables 3.7.2- 11 through 3.7.2-13 and Figures 3.7.2-18 through 3.7.2-18. The maximum member forces at the base of the three stick models (coupled auxiliary and shield building, containment vessel, and containment internal structures) are shown in Tables 230.71-1, 2 and 3. The adequacy of using the seismic responses (force, shear and moment) corresponding to the soft rock site condition instead of the seismic response envelopes for the foundation design for all site conditions is based on the following considerations:

- The member forces in the coupled auxiliary and shield building model at elevation 100' for the soft rock site envelope those for the soft-to-medium stiff soil site and for the hard rock site.
- The member forces in the containment vessel model at elevation 100' for the soft rock site envelope those for the soft-to-medium stiff soil site. The member forces are higher for the hard rock site. However, these forces are small in comparison with those from the coupled auxiliary and shield buildings and the containment internal structures. In addition, basemat forces and moments are lower for the hard rock case since vertical reactions are resisted directly by the hard rock and do not result in overall bending of the basemat.
- The member forces in the containment internal structures at elevation 82'6" for the soft rock site envelope those for the soft-to-medium stiff soil site except for the vertical axial force which is 10% higher. This difference is small in comparison with the total vertical load. The member forces are higher for the hard rock site. However, these forces are small in comparison with those from the coupled auxiliary and shield buildings. In addition, basemat forces and moments are lower for the hard rock case since vertical reactions are resisted directly by the hard rock and do not result in overall bending of the basemat.

SSAR Revision:

Revise the 6th paragraph of SSAR Subsection 3.8.5.4 as follows:

Normal and extreme environmental loads are considered in the analysis. The normal loads include dead loads and live loads. Extreme environmental loads include the safe shutdown earthquake. Safe shutdown earthquake loads for the soft rock case, in combination with the properties of soft-to-medium **stiff** ~~soft~~ soil, are used in the analysis since the soft rock case produces higher applied seismic forces to the structure than the soft to medium soft soil case. **Loads applied to the basemat are slightly higher for the hard rock case, but would not govern the design since they are resisted directly by the hard rock foundation and do not require significant load transfer by bending of the basemat.** Hence, the approach is conservative.

AP600

Table 230.71-1

Maximum Member Forces and Moments At Elevation 100'
Coupled Auxiliary & Shield Buildings

Site Condition	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
Hard Rock	29.94	36.62	41.13	1173.00		
					4200.00	3937.00
Soft Rock	34.80	40.30	42.90	1200.00		
					4740.00	4640.00
Soft-to-Medium Stiff	33.30	31.00	32.20	848.00		
					3380.00	3470.00

Table 230.71-2

Maximum Member Forces and Moments At Elevation 100'
Steel Containment Vessel

Site Condition	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
Hard Rock	4.40	3.68	4.10	14.57		
					449.00	402.20
Soft Rock	3.23	3.08	3.11	5.41		
					368.00	341.00
Soft-to-Medium Stiff	2.97	2.32	2.76	5.84		
					332.00	214.00

Table 231.71-3

Maximum Member Forces and Moments At Elevation 82.50'
Containment Internal Structures

Site Condition	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
Hard Rock	10.85	13.22	13.50	418.70		
					582.20	475.80
Soft Rock	12.60	13.10	13.90	58.60		
					416.00	407.00
Soft-to-Medium Stiff	13.90	10.50	11.70	54.20		
					381.00	316.00

Question 230.76

Table 3.7.1-1 (pg. 3.7-28) of the SSAR specified that a constant damping of 20% was used for the SSE seismic analysis of the cable tray systems (including supports). Figure 3.7.1-13 (pg. 3.7-75) of the SSAR indicated that the damping ratio to be used for the SSE seismic analysis of cable tray systems depends on the amount of cable fill and the damping ratio of 20% specified in Table 3.7.1-1 is the maximum value for trays with cable fill of 50% to 100%. It is the staff's understanding, based on the cable tray tests previously performed by Bechtel Power Corporation (1978) and URS/John A. Blume & Associates, Engineers (1983), that the damping ratios of the cable tray systems depend on a number of factors such as cable tray type, percent of cable fill, hanger type, tray span, hanger length, cable ties, hanger and tray connections, number of trays, fittings, spray for fire protection, etc. Among these factors, lower percent of cable fill, cable ties and spray for fire protection will significantly reduce the resulted damping ratios of the cable tray systems. Based on the above, justify the use of the maximum constant damping ratio of 20% for the SSE seismic analysis of the cable tray systems.

Response:

As stated in SSAR Section 3.7.1.3, the damping ratio used for the AP600 cable tray systems is based on test results presented in Reference 19 (SSAR Section 3.7.5). The cable tray test program conducted by ANCO Engineers Inc. include more than 2000 dynamic tests of representative cable tray system design and construction. The test configurations included items such as various tray types on rigid supports, various tray hanger systems, effects of tray types, effects of strut connections and effects of bracing spacing, unbraced and braced tray systems. Cable ties were also used during the test program. Based on observations during the tests, the high damping values within the cable tray system are provided mainly by the movement, sliding or bouncing of the cables within the tray. The tests show that for unloaded trays the damping ratio closely approximates the 7 percent used for bolted structures, and a minimum damping value of 20% is maintained with cable ties at spacing greater than or equal to four feet. The tests show that for loaded trays, the damping ratio increases with increased cable loading, reaching a value of 30% at cable fill ratio of 50% to 100%. Therefore, the major factors which affect significantly the damping ratio of the cable tray systems are the input acceleration level, cable fill ratio, and the ability of the cables to move within the trays during a safe shutdown earthquake.

The AP600 cable tray system design requires no sprayed on material for fire protection. Cable ties are provided at spacing greater than four feet, thereby permitting cable movement within the trays. The damping ratio used for the cable tray system, therefore, is dependent mainly on the level of seismic input and the amount of cable fill within the trays. As shown in SSAR Figure 3.7.1-13, the 20% constant damping ratio is used only for trays loaded to more than 50% and subjected to input acceleration greater than 0.35g. For cable trays loaded to less than 50% and lower than 0.35g input acceleration, linearly interpolated lower damping values are used.

SSAR Revision: NONE

Question 230.86

In the SSAR, (a) provide justification for not considering the effects of energy feedback between the NI and the surrounding non-seismic Category I structures in the computation of soil pressures on the NI embedded walls, and (b) demonstrate that based on current plant layout, the physical interaction between the NI structures and other non-seismic Category I structures, if any, is negligible. (Section 3.7.2.4)

Response:

a. The effects of energy feedback between the nuclear island and the surrounding non-seismic Category I structures are considered in calculating the lateral pressures on the embedded nuclear island walls. See also response to RAI 220.41.

b. See response to RAI 230.56.

SSAR Revision: NONE

Question 230.87

For the case of the three components of ground motion time histories applied separately in the analyses, it is stated in Section 3.7.2.6 of the SSAR that one of the three methods is used to combine the resulted responses from the three components. Method 1 combines the responses algebraically at each time step. Method 2 combines themaximum responses by the SSRS method. Method 3 combines the maximum responses linearly with the coefficients of 1.0, 0.4 and 0.4. Specify, in the SSAR, when and under what circumstance each of the three methods is to be applied.

Response:

Subsection 3.7.2.6 of the SSAR is revised as shown below to provide the requested information.

SSAR Revision:

Revise the second and third paragraphs of Subsection 3.7.2.6 as shown below. The following text includes (and modifies) the revisions previously identified in the response to RAI 230.34.

In ~~seismic analyses using the time history method~~ mode superposition time-history analyses using computer program BSAP, the three components of earthquake are applied either simultaneously or separately. In the BSAP ~~time history~~ 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 ~~the time history~~ 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 ~~time history~~ analyses with the statistically independent earthquake components applied separately, the time-history responses from the three earthquake components are combined algebraically at each time step to obtain the combined response time-history. This method is used in the BSAP time-history and SASSI analyses.

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

- The peak responses due to ~~each of~~ 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%-40%-40% 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.

Question 231.21

SRP 3.7.2 guidance is that the spectral amplitude of the acceleration response spectra at the foundation level in the free field shall be not less than 60% of the corresponding design response spectra at the finished grade in the free field. However, the spectral amplitudes of the acceleration response spectra shown in Figures 2A-21 through 2A-24 show that the spectral amplitudes at the foundation depth do not satisfy this criterion. Section 2A.4 of the SSAR states that the dip in the amplitude of the response spectrum corresponds to the fundamental soil column frequencies at the depth where the response is calculated. However, the dip is very wide and deep over a frequency range from about 3 Hz to about 6.5 Hz.

a. In view of the above phenomenon, and referring to the response to Q231.10, justify not specifying the control motion at an actual or hypothetical rock outcrop in the above cases as well as other sites with one or more thin soil layers overlying rock.

b. The response spectral curves in the above mentioned figures do not match the legends given in the figures. Clarify the figures.

Response:

a. The AP600 seismic design motion is based on the site-independent Regulatory Guide 1.60 spectra scaled to 0.30g and defined at the finished grade level in the free-field for all sites including shallow soil sites. As part of site interface conditions, the Combined License applicant should demonstrate that the site-specific ground acceleration response spectrum at the grade level for the candidate site, whether it is shallow or deep, soil site is less than or equal to the Regulatory Guide 1.60 spectra scaled to 0.30g maximum acceleration.

   As to the reduction of motion with depth in the free-field, the AP600 design is based on enveloping seismic responses of all soil and rock cases considered. For hard rock profile, the reduction of motion with depth is insignificant (see SSAR Figure 2A-20). Thus, the enveloping ground motion at the foundation level in the free-field for all soil and rock cases meets SRP 3.7.2, Revision 2 criteria with respect to reduction of motion with depth.

b. The response spectra plots were reviewed and match the legends. Attached are larger size plots of SSAR Figures 2A-20 through 2A-24 which may be easier to read.

SSAR Revision: NONE

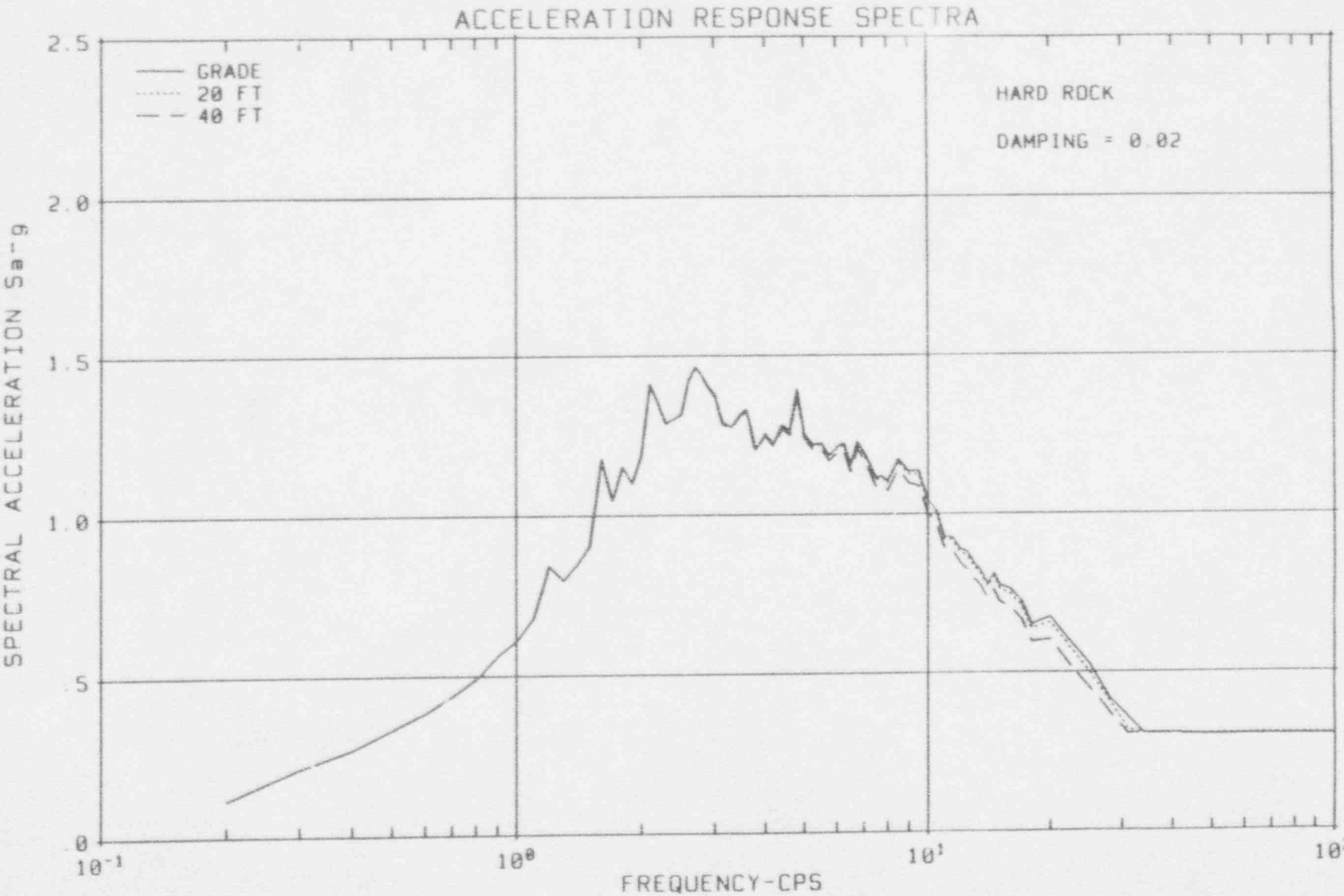

ACCELERATION RESPONSE SPECTRA
GRADE
20 FT
40 FT
HARD ROCK
DAMPING = 0.02
SPECTRAL ACCELERATION Sa-g
2.5
2.0
1.5
1.0
.5
0
10⁻¹
10⁰
10¹
10²
FREQUENCY-CPS

FIGURE 2A-20

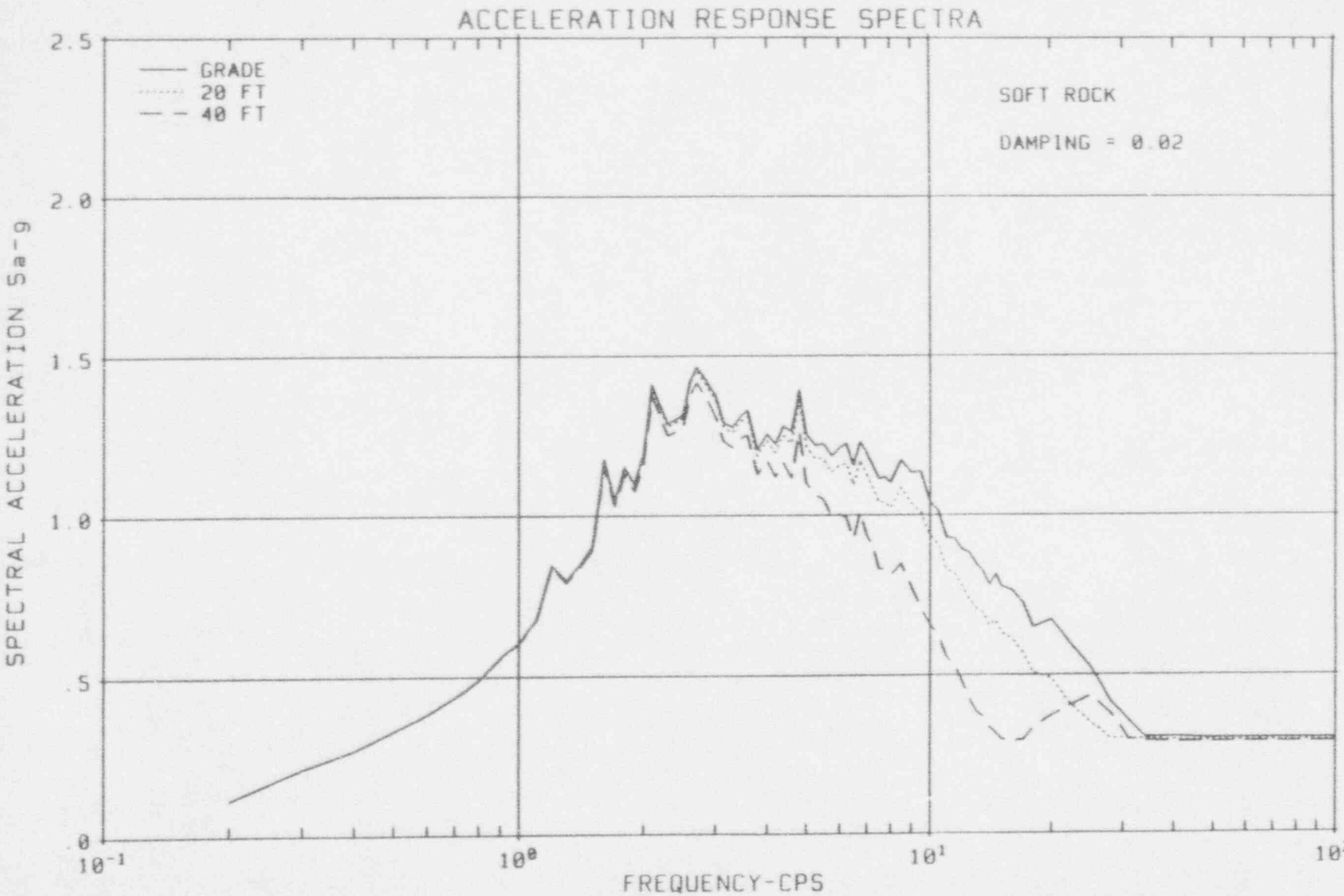
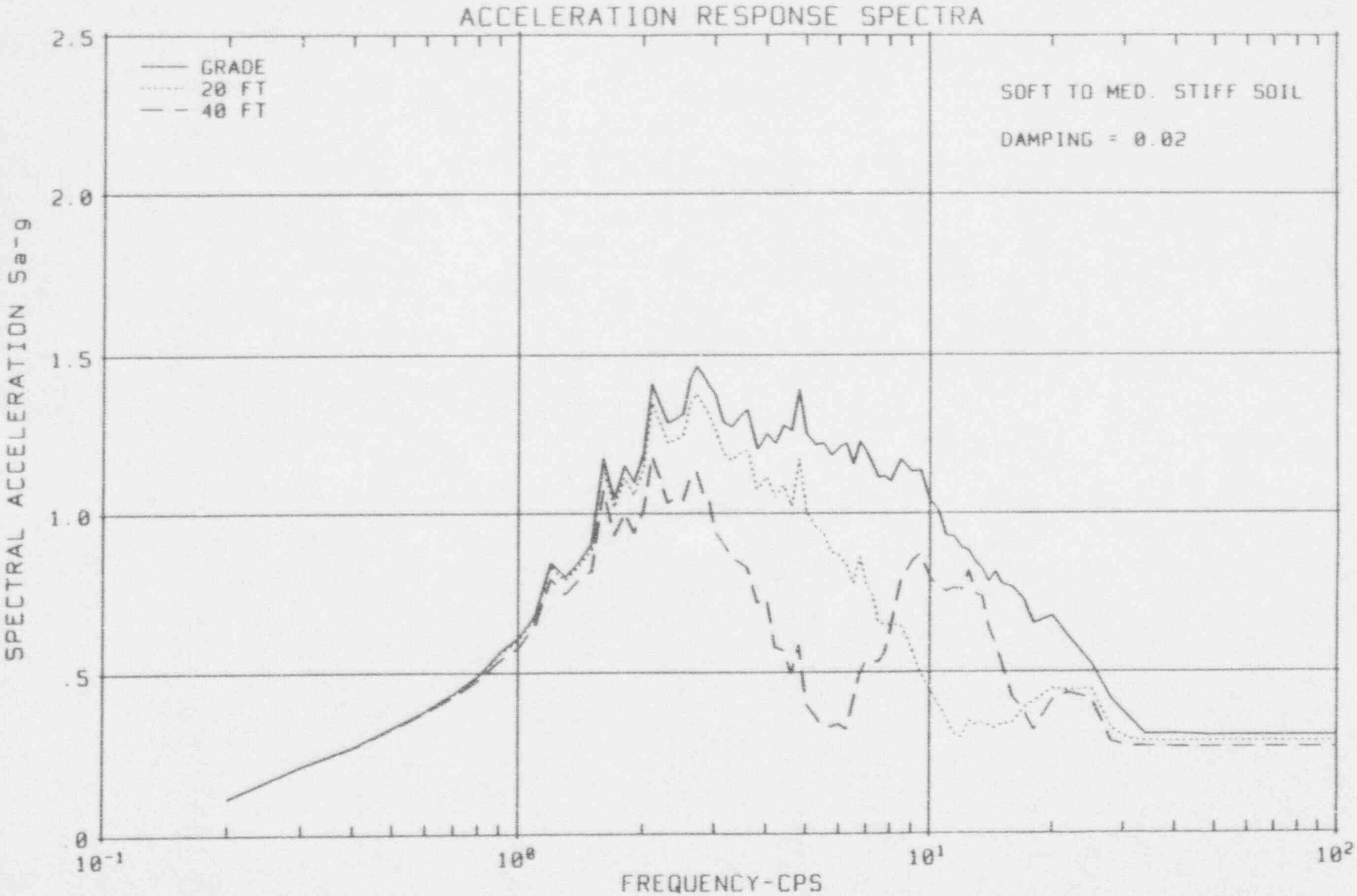

ACCELERATION RESPONSE SPECTRA
GRADE
20 FT
40 FT
SOFT ROCK
DAMPING = 0.02
SPECTRAL ACCELERATION Sa-g
2.5
2.0
1.5
1.0
.5
.0
10^{-1}
10^{0}
10^{1}
10^{2}
FREQUENCY-CPS

FIGURE 2A-21



ACCELERATION RESPONSE SPECTRA
GRADE
20 FT
40 FT
SOFT TO MED. STIFF SOIL
DAMPING = 0.02
SPECTRAL ACCELERATION Sa-g
2.5
2.0
1.5
1.0
.5
0
10^{-1}
10^{0}
10^{1}
10^{2}
FREQUENCY-CPS

FIGURE 2A-22

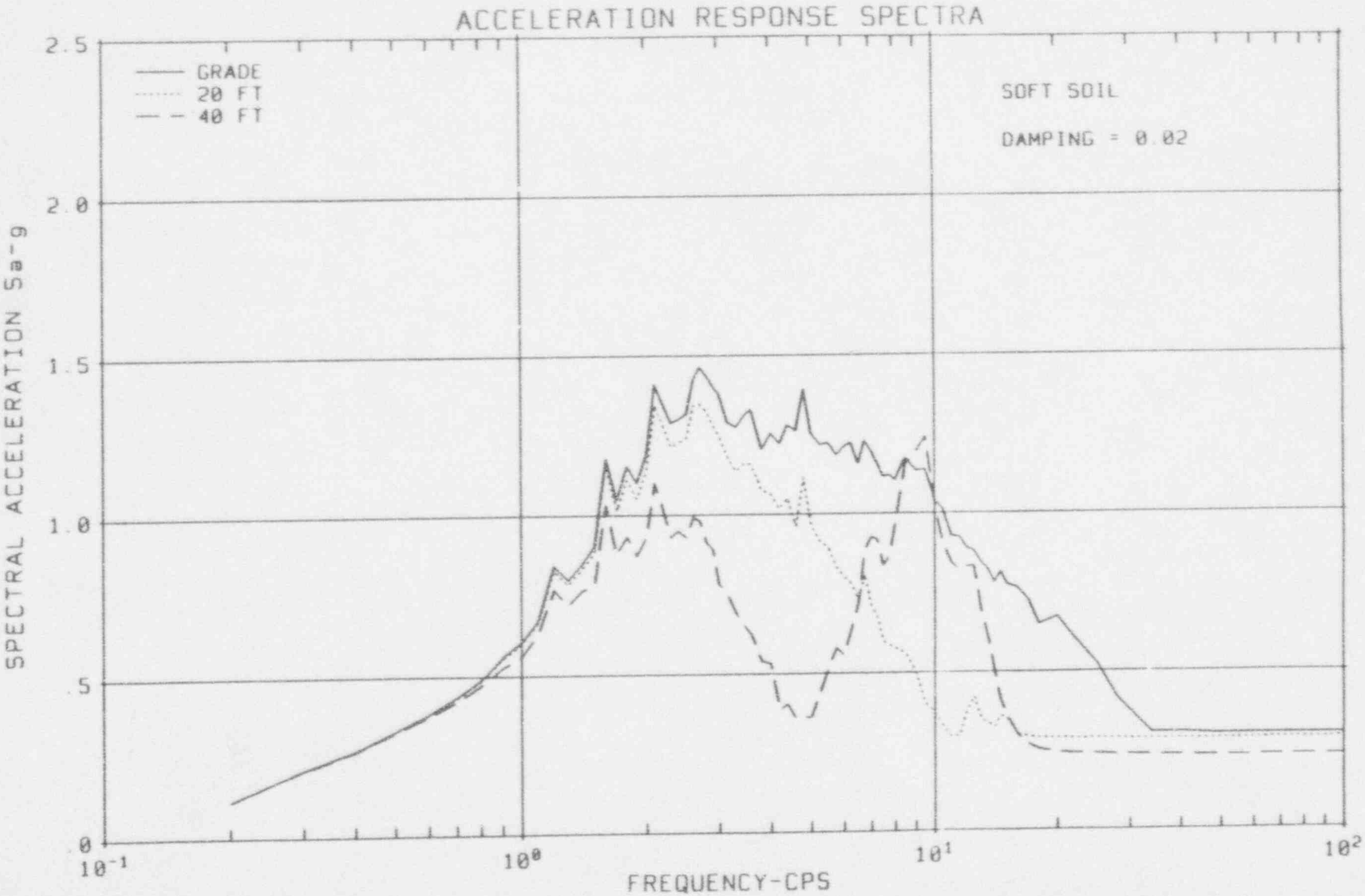

ACCELERATION RESPONSE SPECTRA
GRADE
20 FT
40 FT
SOFT SOIL
DAMPING = 0.02
SPECTRAL ACCELERATION Sa-g
FREQUENCY-CPS
2.5
2.0
1.5
1.0
.5
0
10^{-1}
10^{0}
10^{1}
10^{2}

FIGURE 2A-23

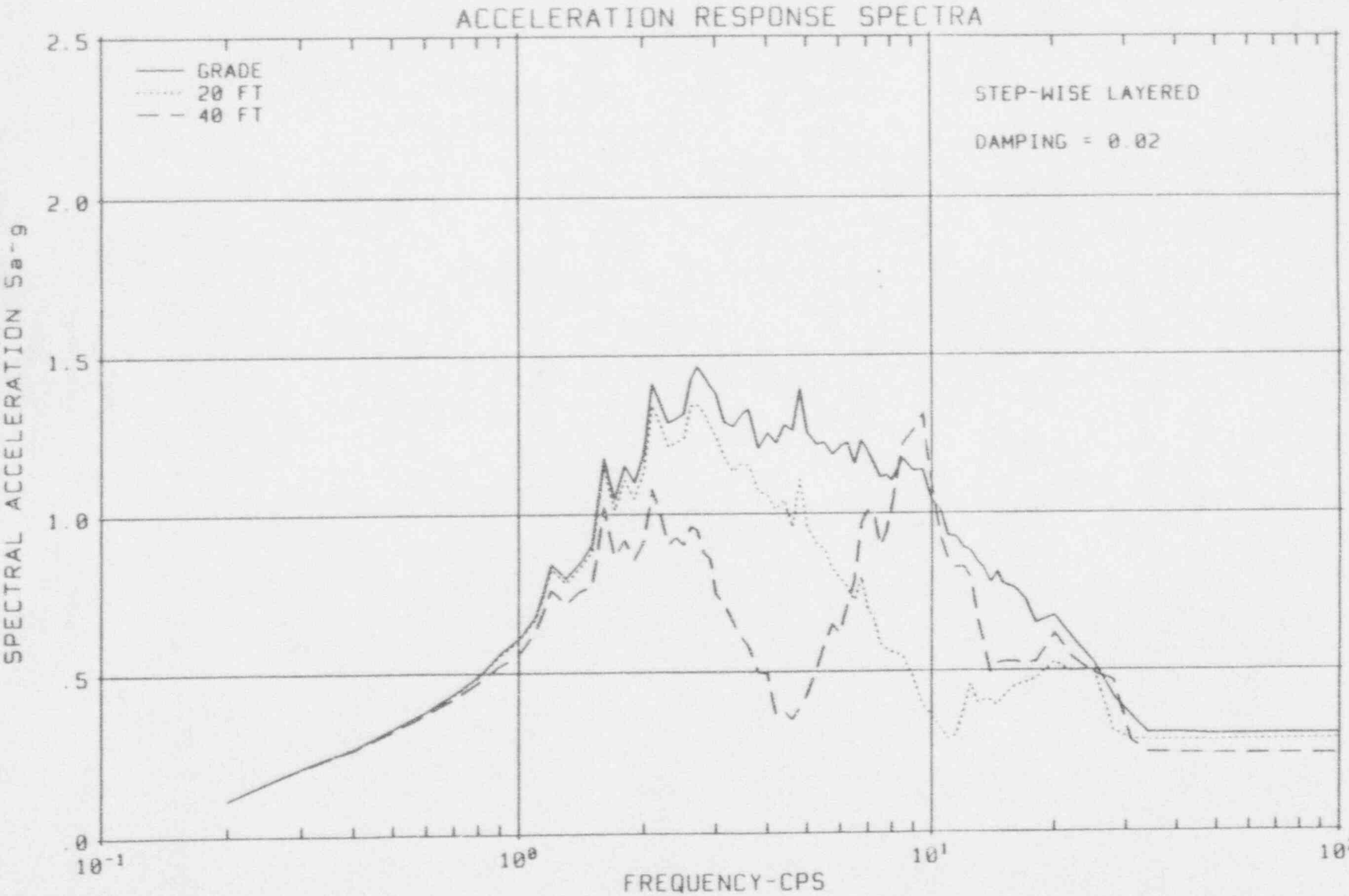

ACCELERATION RESPONSE SPECTRA
GRADE
20 FT
40 FT
STEP-WISE LAYERED
DAMPING = 0.02
SPECTRAL ACCELERATION Sa-g
FREQUENCY-CPS
2.5
2.0
1.5
1.0
.5
.0
10^{-1}
10^{0}
10^{1}
10^{2}

FIGURE 2A-24

Question 231.23

In the November 30, 1992 response to Q231.6 regarding the lateral earth pressure loads, Westinghouse states that the seismic Category I retaining structures and below grade exterior walls are designed for the worst case enveloping the lateral earth pressure, and that the SSAR will be suitably revised. Westinghouse's response does not clearly address the fact that the lateral earth pressures along the walls of the NI are a function of the lateral extent and character of the backfill soils. Based on the above,

a. Specify, in the SSAR, acceptable ranges of backfill properties (such as compacted soil density, minimum acceptable degree of compaction, range of sizes, etc.) for backfill soils to ensure that the design is adequate, and

b. Justify the use of the Mononobe-Okabe (MO) method for calculating the lateral soil loads on walls of the NI where wall movements relative to the surrounding soil may not develop failure strains in the soil.

Response:

a. The design of the nuclear island is not influenced by backfill properties. Backfill material will not be used against the exterior walls of the nuclear island structures. The excavation will have a vertical face as described in the following revision to the SSAR.

b. Please see the response to RAI 220.41for a discussion of the method for calculating the lateral soil loads.

SSAR Revision:

Add the following Subsection to the SSAR:

2.5.1 Excavation and Backfill

Excavation in soil for the nuclear island structures below grade will 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 approximately each five feet depth 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 at fifteen degrees to the horizontal. A "nail", normally a one inch diameter metal bar/rod, is center located for the full length of the hole. The nominal length of soil nails are 60% to 70% 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.

Soil nailing as a method of soil retention has been successfully used on excavations up to 55' deep on projects in the US. Soils have been retained for up to 90' 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, four to six inches of shotcrete are blown on to the rock 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.

Revise Subsection 3.4.1.1.1 as shown below:

3.4.1.1.1 Protection from External Flooding

The probable maximum flood for the AP600 has been established at less than the finished grade as discussed previously in Section 2.4. The probable maximum flood results from site specific events, such as river flooding, upstream dam failure, or other natural causes.

Flooding does not occur from the probable maximum precipitation. Water from roof drains and/or scuppers, as well as runoff from the plant site and adjacent areas, is conveyed to catch basins, underground pipes, or directly to open ditches by sloping the tributary surface area. The site is graded to offer protection to the seismic Category I structures.

The high ground water table interface is at two feet below the grade elevation, as discussed previously in Section 2.4.

~~The seismic Category I structures which are located below grade elevation are protected against flooding by waterproofing membranes and waterstops. Waterproofing membranes are installed on horizontal and vertical exterior surfaces below grade. Waterstops are installed in exterior construction joints below grade.~~

~~Performance criteria for the waterproofing membranes and waterstops are based on the following considerations:~~

- ~~Interaction with the concrete throughout the lifetime of the plant~~
- ~~Ability to withstand the maximum hydrostatic pressure~~
- ~~Ease of installation, having minimum interference during the construction operations~~
- ~~Resistance to attack by soil bacteria~~
- ~~Weathering action~~
- ~~Low permeability~~

- ~~Capability of withstanding movements under seismic conditions~~

- ~~Integrity when subjected to plant radiation.~~

The seismic category I structures below grade are protected against flooding by waterstops and a waterproofing system. The waterproofing system is provided by the introduction of a cementitious crystalline waterproofing additive to the nailed soil retention wall shotcrete or to the shotcrete applied to the rock surface as described in Subsection 2.5.1. For the horizontal surface under the basemat, the cementitious crystalline waterproofing additive is added to the mud mat. The waterproofing additive is a unique chemical treatment added to the concrete at the time of batching and consists of portland cement, very fine silica sand, and various active proprietary chemicals. The active chemicals react with the moisture in fresh concrete, and the byproducts of cement hydration cause a catalytic reaction generating a non-soluble crystalline formation of dendritic fibers throughout the pores and capillary tracts of the concrete. The concrete is thus sealed against penetration of water or liquid.

Question 231.26

The properties of the soft-to-medium soil column given in Table 2A-6 of the SSAR show the shear wave velocity varying linearly from 1000 fps to 2400 fps. Typical variations at sandy soil sites are expected to be curvilinear, with most of the increase in soil stiffness occurring near the upper one-third part of the soil layer due to the nonlinear effects of depth of burial on stiffness. Because such variations may lead to significant differences in soil pressures over the depth of embedment of the NI, as well as changes in free-field ground motions at the foundation mat, provide a comparison of free-field motions at the foundation level obtained from SHAKE deconvolution analysis to indicate the sensitivity of response to this assumption.

Response:

The comparison will be provided by July 30, 1994.

SSAR Revision: NONE

Question 231.29

It appears that the Poisson ratio values selected for soils above the water table may not be consistent with values normally expected for silty sands of densities high enough to support a shear wave velocity of 1000 fps. Evaluate and discuss the effect of the assumed Poisson ratio values on the SSI responses.

Response:

The effect of the assumed Poisson's ratio values on the SSI responses will be submitted by July 30, 1994.

SSAR Revision: NONE

Question 920.4

Reference 1, (SSAR Section 13.7) WCAP-13056, "AP600 Compliance with SRP Acceptance Criteria," August 1991, appears to be an incorrect reference. Update the reference list for this section.

Response:

Reference 1 in SSAR Section 13.7 is incorrect. As noted in WCAP-13056, the AP600 plant security system is in compliance with the applicable requirements of 10 CFR Part 73. In addition, the AP600 plant security system is designed to conform to the applicable portions of the following standard:

- ANS 3.3, " Security for Nuclear Power Plants"

SSAR Revision :

Revise the last paragraph of Subsection 13.6.3.2 as follows:

In addition to the applicable requirements of 10 CFR Part 73, the AP600 plant security system conforms to the applicable portions of Reference 1. ~~The Codes and Standards for the systems are specified in Reference 1.~~

Revise Section 13.7 as follows:

1. ANS 3.3-1988, "Security for Nuclear Power Plants." ~~WCAP-13056, "AP600 Compliance with SRP Acceptance Criteria," August 1991.~~