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Energy Systems

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NTD-NRC-94-4087
DCP/NRC0024
Docket No.: STN-52-003

March 24, 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 letter January 26, 1994. In addition, revised responses for a number of previously provided 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.

Brian A. McIntyre for

Nicholas J. Liparulo, Manager
Nuclear Safety & Regulatory Activities

/nja

Enclosure

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

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NTD-NRC-94-4087
ATTACHMENT A
AP600 RAI RESPONSES
SUBMITTED MARCH 24,1994

RAI No.	Issue
220.025	Containment seals at transition region
220.030	Justification for factor of safety of 1.67
220.032	Justification for factor of safety of 2.5
220.034	Nonmetalic items under SA conditions
220.036	Containment shell stress analysis results
220.038	Axisymmetric model vs. Sandia criteria
220.039	Strains at discontinuities vs. Sandia criteria
220.042	Design criteria for severe weather phenomena
220.043	Stability evaluations for safety-related structure
220.044	Methodology for seismic load calculations
220.046	Use of epoxy-coated reinforcing steel
220.050	Factor of safety for sliding & overturning
230.024	Difference between non-Cat I & non-seismic
230.025	Non-Cat I & seismic Cat II clarification
230.027	Frequency intervals in response spectra
230.028	Ground motion cross correlation coefficients
230.029	Basis for damping ratio
230.030	Basis for hard-rock, soft-rock damping values
230.031	Shear wave velocity profile for base rock
230.032	Location of input ground motion
230.033	Justification for envelope of potential sites
230.034	Use of "time history analysis"
230.036	SASSI code validation pachage
230.040	Modeling of steel containment shell
230.042	Structural member forces used for design
230.043	Discrepancy between Sections 3.7.2.5 & 3.7.2.1.2

NTD-NRC-94-4087
ATTACHMENT A
AP600 RAI RESPONSES
SUBMITTED MARCH 24,1994

RAI No.	Issue
230.044 :	Application of 3 components of earthquake motion
230.045 :	Analyses for fixed base structural model
440.002R01:	CMT testing
440.004R01:	CMT testing
440.005R01:	CMT testing
440.006R01:	CMT testing
440.010R01:	CMT testing
440.050 :	Impact of ADS design change on OSU & SPES
450.008R01:	ESF atmospheric cleanup

Question 220.25

At the transition region between the free-standing part and the encased portion of the steel containment, seals are provided at the top of the concrete at elevation 108 ft inside the vessel and at elevation 100 ft outside the vessel so that moisture is not trapped next to the steel vessel just below the top of concrete. The seal on the inside accommodates radial growth of the vessel due to pressurization and heatup. The staff is concerned about the mechanical properties of this seal material and the stress conditions and buckling potential of the steel containment in this region. No information is provided in the SSAR concerning (1) composition of the seal material, (2) the method used to obtain these material properties, (3) the uncertainties associated with these material properties, (4) the accessibility to perform periodic inspection, and (5) the behavior under the severe accident conditions. Address the issues associated with (1) the uncertainty of the mechanical properties of this seal material and the environmental qualification as well as age related degradation management for the proposed 60-year design life for this seal material, and (2) the measures to be implemented to prevent collection of moisture in the transition region (Section 3.8.2 of the SSAR).

Response:

To prevent collection of moisture in the transition region between the free-standing and the encased portion of the steel containment, at Elevation 108'-2" inside the vessel and at Elevation 100'-0" outside the vessel, the following measures are provided:

- The concrete curb inside the containment vessel at Elevation 108'-2" and the concrete slab outside the containment vessel at Elevation 100' are sloped away from the steel containment vessel to prevent water ponding adjacent to the vessel.

- Silicone seals are provided at these two locations. The configuration of the silicone seal is shown in the attached Figure 3.8.2-7. These seals are not required to function under design basis or SSE conditions. They are provided to enhance corrosion protection of the vessel and are designed for the transient conditions anticipated during normal operation.

  The sealant material being considered is a neutral methanol cured silicone adhesive sealant designed for concrete and masonry substrates. The sealant material adheres to both the steel and the concrete surfaces to form a water tight barrier. The silicone seals, because of their design configuration and placement location, can be inspected periodically and replaced if necessary.

The silicone adhesive sealant is an ultra-low modulus material with the following as-cured properties:

- Durometer Hardness (Shore A, points - ASTM D2240) 15
- Ultimate Tensile Strength (psi) @ maximum elongation (ASTM D412) 100
- Elongation, percent maximum (ASTM D412) 1600
- Peel Strength (#/in, MIL-S-8802) 25
- Tensile Adhesion with 25% extension (ASTM C1135) 15
- Tensile Adhesion with 50% extension (ASTM C1135) 20
- Joint Movement Capabilities, extension 100%
- Joint Movement Capabilities, compression 50%
- Stay Rubbery from -45 to 300°F

At present, aging test data for the nuclear environment (radiation and temperature) are not available for this material. Aging tests for radiation and temperature, followed by pressure test will be performed at the time of procurement to demonstrate both design function as well as design life. Based on the test results of the planned aging test, the silicone seal replacement criteria will be established.

SSAR Revision:

Revise the last paragraph of Section 3.8.2.1.2 as shown below:

Vertical and lateral loads on the containment vessel and internal structures are transferred to the basemat below the vessel by friction and bearing. Seals are provided at the top of the concrete on the inside and outside of the vessel to prevent moisture between the vessel and concrete. A typical cross section design of the seal is presented in Figure 3.8.2-7, sheets 1 and 2. Furthermore, the concrete floor area and curb inside containment near Elevation 108' and the concrete slab outside containment at Elevation 100' are designed to slope away from the steel containment vessel to prevent water ponding adjacent to the vessel.

Add the attached Figure 3.8.2-7, sheets 1 and 2, into Section 3.8.2.

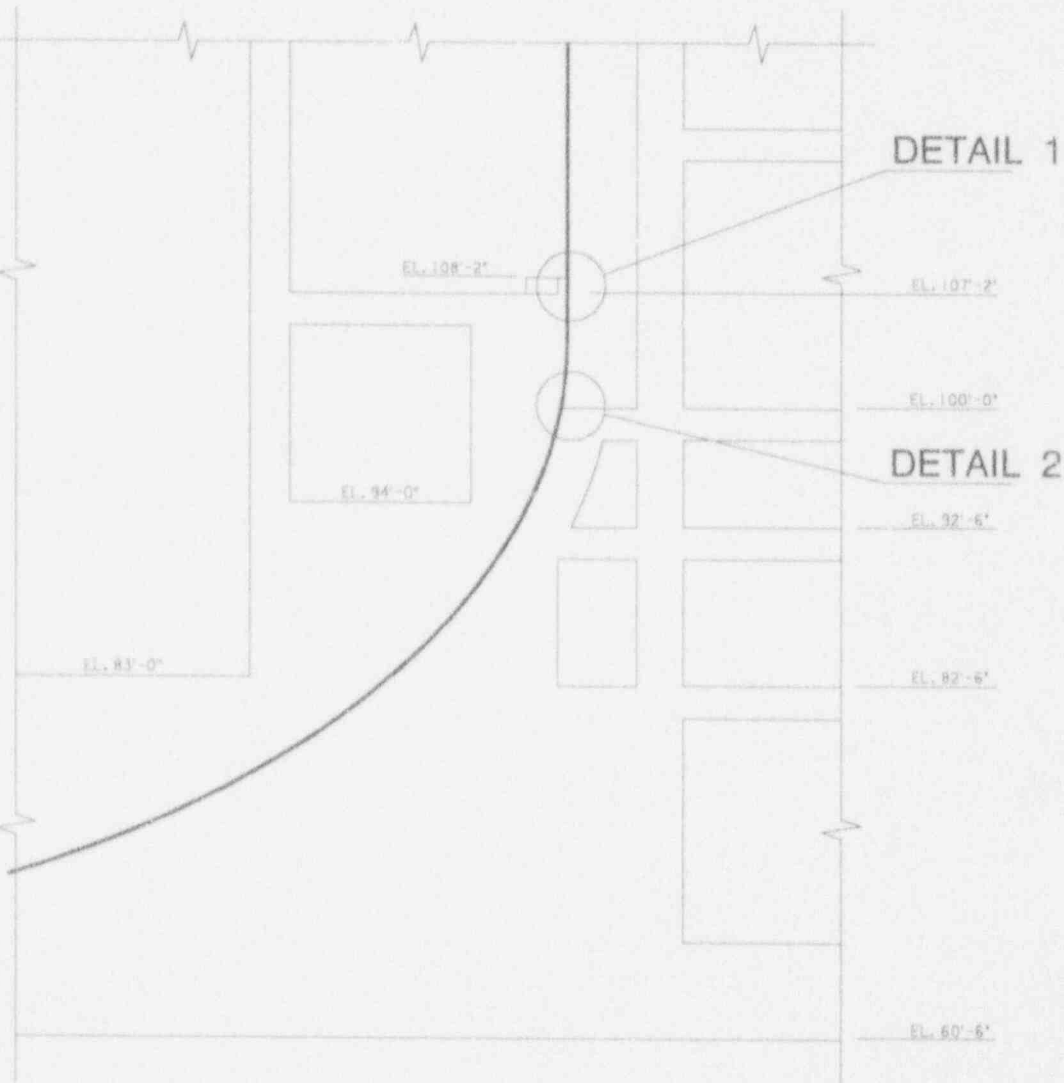
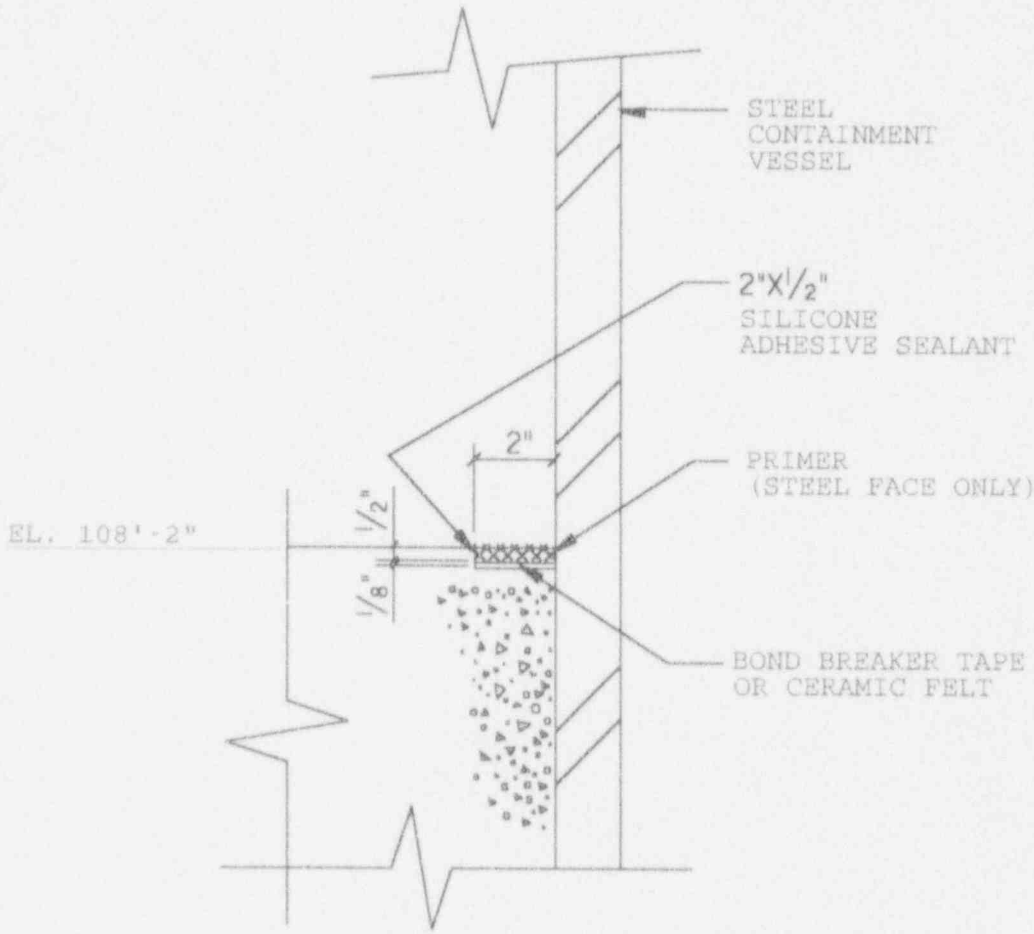


Figure 3.8.2-7 (Sheet 1 of 2)
Location of Containment Seal



DETAIL 1

(DETAIL 2 SIMILAR)

Figure 3.8.2-7 (Sheet 2 of 2)

Seal Sections and Details

Question 220.30

Westinghouse estimates the maximum pressure at ambient temperature corresponding to the following stress and buckling criteria: (1) deterministic severe accident pressure capacity corresponding to ASME Level C Service Limit on stress intensity, Code Case N-284 for buckling of the equipment hatch covers, and two-thirds of critical buckling for the top head, and (2) best estimate capacity corresponding to gross membrane yield at the ASME-specified minimum yield stress (SA 537, Class 2, yield stress = 60 ksi, ultimate stress = 80 ksi), and critical buckling for the equipment hatch covers and top head. However, neither the Code Case N-284 for buckling of the equipment hatch covers (see Q220.32) nor the two-thirds of critical buckling for the top head is acceptable. The factor of safety due to the internal pressure (see Appendix A to this enclosure) is 1.67 for the Level C Service Limit as specified in the Code Case N-284. Note (1) in Table 3.8.2-2 should be revised to reflect that the factor of safety is 1.67, or acceptable justification should be provided for not doing so.

In addition, Westinghouse analyzed the steel containment vessel for the theoretical buckling capacity using the BOSOR-5 computer code, which uses both large displacement and nonlinear material properties. The yielding started at a pressure of 144 psig for the cylinder, at 146 psig for the top of crown, and at 152 psig for the knuckle region, using elastic-plastic material properties, a yield stress of 60 ksi, and the von Mises yield criterion. Provide the bases for the use of von Mises criterion instead of ASME stress intensity criterion to establish yield.

Westinghouse determined that the theoretical plastic buckling pressure is 174 psig. At the pressure of 174 psig, Westinghouse calculated the maximum effective pre-buckling strain of 0.23 percent in the knuckle region where buckling eventually occurred, and 2.5 percent at the crown. However, it is not clear how these strains were derived. For the SA 537 Class 2 material, it is reported that the stress-strain curve has the strain plateau from 0.2 percent to 0.6 percent without pressure increase and strain hardening after 0.6 percent (see Section 3.8.2.4.2.6 of the SSAR). At the knuckle region, Westinghouse states that it started to yield at the pressure of 152 psig, which will go to 0.6 percent strain with no further pressure increase. At the top of the head, the expected stress at 174 psig is 72 ksi and the corresponding strain is about 8 percent. Explain how a value of only 2.5 percent strain was obtained and provide justification for the ultimate capacity of the containment (Section 3.8.2 of the SSAR).

Response:

The calculation of the deterministic severe accident pressure capacity was revised in Revision 1 of the response to RAI 220.9. This revised information has been included in Revision 1 of the SSAR. The capacity of the top head under internal pressure is now calculated in accordance with the staff position and utilizes a safety factor of 1.67. The capacity of the equipment hatch covers is discussed in the response to RAI 220.32.

The theoretical buckling capacity of the top head is calculated by BOSOR-5 using an elastic-perfectly plastic material model based on the ASME specified minimum yield of 60 ksi. The analysis is primarily intended for the calculation of ultimate capacity and uses the von Mises yield criterion and not the stress intensity criterion specified in ASME for stress intensity evaluation. ASME does not specify a yield criterion to be used in buckling evaluations. Tests of ductile steel materials, such as the SA537 steel in the AP600 containment vessel, support use of the von Mises criterion and show that the ASME stress intensity criterion is conservative. The von Mises criterion has been used in many previous estimates of containment ultimate capacity (References 220.30-1 and 220.30-2). The Sandia strain criteria referred to in RAI 220.38 and RAI 220.39 also utilize the von Mises yield criterion. The difference between the two criteria is 15% when both principal stresses are in tension with the larger equal to twice the smaller, such as in the cylinder subject to internal pressure (see SSAR Subsection 3.8.2.4.2.1). There is no difference when both principal stresses are in tension with equal magnitudes, such as at the crown of the top head under internal pressure.

The maximum effective pre-buckling strain of 0.23 percent in the knuckle region and of 2.5 percent at the crown are the results of the BOSOR-5 analysis using an elastic-perfectly plastic material model based on the ASME specified minimum yield of 60 ksi. The actual stress strain curve used in BOSOR-5 had slight strain hardening such that the tangent modulus of the "perfectly plastic" portion was 0.001 times the elastic portion. This gives a stress of 60.68 ksi at a strain of 2.5% . This increase was incorporated to improve the numerical analysis and has no significant effect on the results. Once yield initiates, the vessel deflections increase substantially; to a magnitude of 15.9 inches as reported in SSAR Subsection 3.8.2.4.2.2. The increase in strains during the incremental loading in the elasto-plastic range results in large deflections which change the geometry of the head and reduce the radius of curvature. This allows an increased pressure without an increase in stress.

As described above the BOSOR-5 analyses used an elastic-perfectly plastic material model based on the ASME specified minimum yield of 60 ksi. The analyses did not use the typical material properties reported in SSAR Subsection 3.8.2.4.2.6. If these properties had been used, the pressure capacity would be expected to increase in proportion to the actual to specified minimum yield.

References:

220.30-1 Ahl, T.J., Mokhtarian, K. and Horacek, D.R., "Analysis of a Mark I Containment Vessel for Severe Accident Conditions," NUREG/CP-0095, pp 551-570.

220.30-2 Miller, J.D., and Clauss, D.B., "Evaluation of the Performance of the Sequoyah Unit 1 Containment under Conditions of Severe Accident Loading," NUREG/CP-0095, Paper No. SAND88-1631C, pp 571-590.

SSAR Revisions: NONE

Question 220.32

Westinghouse estimates critical buckling pressures for equipment hatches as 196 psig for a 22-foot-diameter hatch and 161 psig for a 16-foot-diameter hatch. The corresponding ASME Level C Service Limits are 117 psig and 96 psig using the Code Case N-284, respectively. From Figure 3.8.2-2, the equipment hatch covers appear convex to the center line of the containment. Therefore, the use of the Code Case N-284 (i.e., the factor of safety of 1.67 for the Level C Service Limit) is not acceptable because the internal pressure of the containment acts as the external pressure to the spherical cap covers and subjects the cap covers to compression. In the case of external pressure, ASME NE-3222 (i.e., the factor of safety of 2.5 for the Level C Service Limit) should be used for the compressive stresses. Note (1) in Table 3.8.2-2 should be revised to reflect the factor of safety of 2.5, or acceptable justification should be provided for not doing so (Section 3.8.2 of the SSAR).

Response:

Code Case N-284 provides criteria for evaluation of unstiffened spherical caps subjected to compressive stress due to pressure loading. In the Code Case, the theoretical buckling value is given in paragraph 1712.1.3, the capacity reduction factor is given in paragraph 1512 (b), and the plasticity reduction factor is given in paragraph 1620 and 1610 (a). The capacities of the hatch covers as described in the SSAR are in accordance with this code case.

ASME Code Case N-284 was developed by the code committee based on detailed review and evaluation of test data. Figure 220.32-1 shows test results from references 220.32-1 and 220.32-2 for fabricated steel hemispherical shells and spherical segments. The ratio of test buckling stress to theoretical buckling stress (α) is shown as a function of the non-dimensional unsupported length along the shell ($M = L_i/\sqrt{Rt}$, where L_i is the unsupported length along the spherical shell, R is the radius of the shell and t is the thickness of the shell). The lower bound curve to these data points, as shown in the figure, is used in Code Case N-284. For the AP600 16 foot diameter equipment hatch, M = 14.5, and the capacity reduction factor, corresponding to α in the figure, is 0.167. The stresses in the hatch cover are well below yield and the plasticity reduction factor is unity. The test data for shell lengths of 10 to 20 show capacities significantly above those of the Code Case. The capacity of the hatch covers, as calculated by the ASME Code Case, corresponds to the lower bound of the test data. As a result the 1.67 factor of safety specified in paragraph 1400 of the ASME Code Case is considered appropriate for calculating the Service Level C pressure capacities of the hatch covers.

SSAR Revision: NONE

References:

220.32-1 Kiernan, T.J. and Nishida, K., "The Buckling Strength of fabricated HY-80 Steel Spherical Shells," DTMB 1721, July 1966.

220.32-2 Arne, C., "Stiffened Spherical Shell Tests, " CBI Contract C-1752, 1959

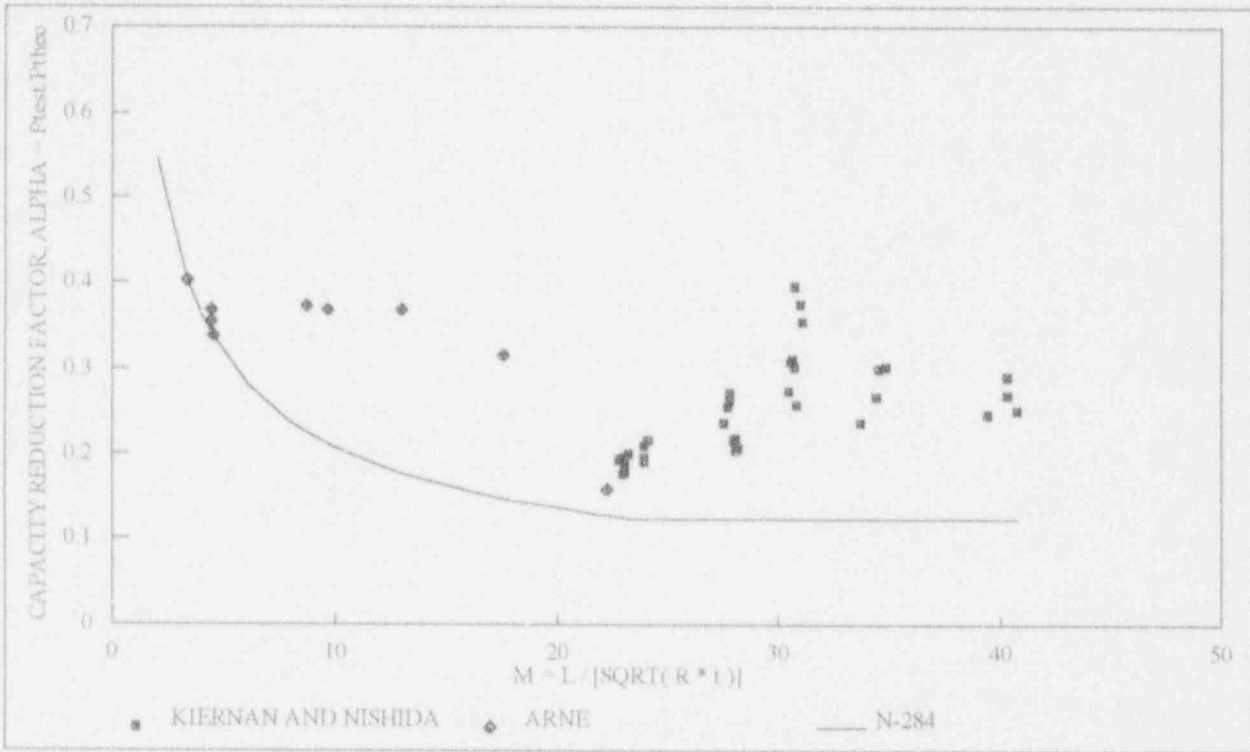


Figure 220.32-1 Comparison of Capacity Reduction Factors for Tests with Code Case N-284

Question 220.34

Nonmetallic items, such as gaskets, are qualified to function at the design temperature. The SSAR should provide the functionality of such items under the severe accident conditions (Section 3.8.2 of the SSAR).

Response:

The containment vessel includes nonmetallic gaskets for the equipment hatches and the personnel airlock. The functionality of the personnel airlocks is discussed in SSAR Subsection 3.8.2.4.2.4. The functionality of the gaskets for the equipment hatches is addressed in the proposed revision to the SSAR identified below.

SSAR Revision:

Revise SSAR Subsection 3.8.2.4.2.3 and 3.8.6 as follows:

3.8.2.4.2.3 Equipment Hatches

The equipment hatch covers were evaluated for buckling according to ASME Code Case N-284. The critical buckling capacity is based on classical buckling capacities reduced by capacity reduction factors to account for the effects of imperfections and plasticity. These capacity reduction factors are based on test data and are generally lower-bound values for the tolerances specified in the ASME code.

The critical buckling pressures are 196 psig for the 22 foot diameter hatch and 161 psig for the 16 foot diameter hatch at ambient temperature. For the Service Level C limits a safety factor of 1.67 is specified, resulting in capabilities of 117 psig (22' dia) and 96 psig (16' dia).

Typical gaskets have been tested for severe accident conditions as described in NUREG/CR-5096 (Reference 25). The gaskets for the AP600 would be similar to those already tested with material such as Presray EPDM E 603. For such gaskets the onset of leakage occurred at a temperature of about 600°F.

3.8.6 References

16. NUREG/CR-5096 SAND88-7016, Evaluation of Seals for Mechanical Penetrations of Containment Buildings.

Question 220.36

Submit the stress analysis results for the most highly stressed portions of the containment shell in both meridian and circumferential directions (Section 3.8.2 of the SSAR).

Response:

Detailed stress analysis results for the containment shell are available for staff review in the design calculations for the containment vessel. Representative results are provided in the SSAR as described below.

Design of the containment shell is primarily controlled by the internal pressure of 45 psig. The meridional and circumferential stresses for the internal pressure case are snown in SSAR Figure 3.8.2-5. The most highly stressed regions for this load case are the portions of the shell away from the hoop stiffeners and the knuckle region of the top head. In these regions the stress intensity is close to the allowable for the design condition.

Location and sizing of the circumferential stiffeners are controlled by the external pressure. Meridional and circumferential stresses for this case can be obtained by factoring the results for the internal pressure.

Seismic member forces for the stick model are shown in SSAR Table 3.7.2-12. The axial and overturning moments result in meridional stresses in the shell. The interaction equation value using these safe shutdown earthquake stresses in combination with those from dead load are about one half of the allowable value of 1.0 specified in the ASME Code Case N-284.

SSAR Revisions: NONE

Question 220.38

Discuss whether all strains in the axisymmetric analysis model are comparable to the Sandia** strain criteria (Section 3.8.2 of the SSAR).

** Miller, J.D. and Clauss, D.B., "Evaluation of the Performance of the Sequoyah Unit 1 Containment Under Conditions of Severe Accident Loading," NUREG/CP-0095, Paper No. SAND88-1631C, pp 571-588, 1988.

Response:

As described in SSAR Subsection 3.8.2.4.2.8, the ultimate pressure capacity for containment function is defined as the pressure at which excessive radial deflections of the cylinder occur. This pressure is calculated based on general membrane yield of the cylinder. Other portions of the vessel are below yield at this pressure. Strains are significantly lower than those permitted by the mean values of the Sandia strain criteria.

Two axisymmetric analysis models were used in the evaluation of containment capacity. One model was the elastic analysis for the design internal pressure as referenced in SSAR Subsection 3.8.2.4.2.1. All strains in this model are below yield. The second model was the BOSOR-5 analysis of the top head as described in SSAR Subsection 3.8.2.4.2.2. The maximum strain in this model of 2.5%, as given in the SSAR subsection, is less than the mean value of the Sandia strain criteria. This maximum strain occurs in the top crown at a pressure of 174 psig, which is substantially higher than the 144 psig corresponding to ultimate capacity of the vessel based on yield of the cylinder. The BOSOR-5 analyses show that the top head is below yield at a pressure of 144 psig.

SSAR Revisions: NONE

Question 220.39

Discuss whether the strains at all discontinuities (i.e., around penetrations and penetration reinforcements) are comparable to the Sandia strain criteria** (Section 3.8.2 of the SSAR).

** Miller, J.D. and Clauss, D.B., "Evaluation of the Performance of the Sequoyah Unit 1 Containment Under Conditions of Severe Accident Loading," NUREG/CP-0095, Paper No. SAND88-1631C, pp 571-588, 1988.

Response:

As described in SSAR Subsection 3.8.2.4.2.8, the ultimate pressure capacity for containment function is defined as the pressure at which excessive radial deflections of the cylinder occur. This pressure is calculated based on general membrane yield of the cylinder (see also the response to RAI 220.38). As described in SSAR Subsection 3.8.2.4.2.5, penetration reinforcement is designed following the area replacement method of the ASME Code. The insert plates and sleeves are thick enough to develop hoop tensile yield stresses in the cylinder before membrane yield occurs in the insert plate or sleeve. Strains do not exceed the mean values of the Sandia strain criteria for pressures up to the predicted ultimate capacity.

SSAR Revisions: NONE

Question 220.42

Westinghouse states that for all safety-related structures, the design rainfall is 493 mm/km}/hr (19.4 in/mi}/hr). The roof of the seismic Category I Structures should be designed to have parapets with scuppers to supplement roof drains or be designed without parapets so that excessive ponding of water cannot occur. Provide detailed design criteria against severe weather phenomena, such as heavy rainfall and snow loadings (Section 3.8.4 of the SSAR).

Response:

The roofs are designed for snow loads in accordance with American Society of Civil Engineers, "Minimum Design Loads for Buildings and Other Structures," ASCE 7-88 (formerly ANSI A58.1-82)(SSAR Reference 3.3-1). The ground snow load is 75 pounds per square foot. The exposure and importance factors are 1.0 and 1.2, respectively.

The roofs do not have drains or parapets. The roofs are sloped such that rainfall is directed towards gutters located along the edges of the roofs. Therefore, ponding of water on the roofs is precluded.

SSAR Revision:

Revise SSAR Table 2.0-1 as follows:

Table 2.0-1 (Sheet 2 of 2)

Site Interface Parameters

Precipitation	
Rain	19.4 in./hr (6.2 in./5 min)
Snow/Ice	75 ~~50~~ pounds per square foot on ground with exposure and importance factors of 1.0 and 1.2, respectively ~~static load~~.

Question 220.43

The applicant for an combined construction/operating license (COL) will need to ensure that the settlement of adjacent buildings will be such that the integrity of underground piping or tunnel will not be jeopardized. The SSAR should contain a statement that the COL applicant should perform stability evaluations of all safety-related facilities, including foundation rebound, settlement, differential settlement, and bearing capacity. Provide that statement (Section 3.8.4 of the SSAR).

Response:

There are no safety-related underground piping or tunnels. The Combined License applicant will perform stability evaluations of the nuclear island structures, including foundation rebound, settlement, differential settlement, and bearing capacity. The requirement to perform evaluation of bearing capacity and foundation settlement are already identified as Items No. 2.13 and 3.7 of SSAR Table 1.8-1.

SSAR Revision:

Revise the Interface description for Item No. 3.7 of Table 1.8-1 to read as follows:

Table 1.8-1

Summary of AP600 Plant Interfaces With Remainder of Plant

Item No.	Interface	Interface Type	Matching Interface Item	Section or Sub-section
3.7	Foundation Rebound and Settlement Monitoring	Requirement of AP600	Combined License applicant coordination	3.8.5

Question 220.44

State which methodology [SRSS or (1.0, 0.4, 0.4) method] is used for the seismic loads calculation. For the computation of global seismic loads, indicate whether the inertial properties include all tributary mass expected to be present in operating conditions at the time of earthquakes. This mass should include the dead load, stationary equipment, piping, and appropriate part of the live load (Section 3.8.4 of the SSAR).

Response:

In the computation of global seismic loads for the nuclear island structures, the mass properties of the seismic model included all tributary mass expected to be present during plant operating condition. This included dead weight of walls and slabs, weight of major equipment, and equivalent tributary slab area loads representing miscellaneous equipment, piping and raceways.

For the seismic load responses presented in Section 3.7.2.2, the effect of three components of earthquake were considered as follows:

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

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

SSAR Revision:

Revise the first paragraph of Section 3.7.2.3.3 as shown below:

The various building lumped-mass stick models are interconnected with rigid linking elements to form the overall dynamic model of the nuclear island. The nuclear island seismic model consists of 80 mass points and 249 dynamic degrees of freedom. The mass properties of the lumped-mass stick models include all tributary mass expected to be present during plant operating condition. This includes dead weight of walls and slabs, weight of major equipment, and equivalent tributary slab area loads representing miscellaneous equipment, piping and raceways.

Add the following to the end of Section 3.7.2.6:

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

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

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

Question 220.46

Specify whether epoxy-coated reinforcing steel is used for areas where a corrosive environment is encountered (Section 3.8.4 of the SSAR).

Response:

As stated in SSAR Subsection 3.4.1.1.1, seismic Category I structures which are located below grade elevation are protected against flooding by waterproofing membranes and waterstops. This, in conjunction with 2 inches of concrete cover for the reinforcing steel provides sufficient protection to the reinforcing steel. Therefore, epoxy coated reinforcing is not required.

SSAR Revision: NONE

Question 220.50

The factor of safety against sliding and overturning the nuclear island due to tornado and wind should be provided. In Table 3.8.5-1, provide the rationale for the buoyancy force criterion for the submerged structure (Section 3.8.5 of the SSAR).

Response:

The factors of safety (F.S.) against sliding and overturning of the nuclear island due to tornado and design wind loads are as follows:

- F.S. due to Tornado Load:

  Sliding, N-S direction = 6.8 , E-W direction = 6.0

  Overturning, N-S direction = 19.6 , E-W direction = 8.0

- F. S. due to Design Wind Load:

  Sliding, N-S direction = 10.2 , E-W direction = 9.3

  Overturning, N-S direction = 47.3 , E-W direction = 22.8

The buoyant force on the submerged structures used in the flotation evaluation is that due to the maximum high ground water level specified in Table 2.0-1 of the SSAR. The design condition for high ground water table is a severe environmental condition. A minimum factor of safety equal to 1.5 is applied in the evaluation of buoyancy force on the submerged structure.

SSAR Revision:

Revise Section 3.8.5.5.2 as shown below:

The factor of safety against flotation of the nuclear island is calculated as follows:

$F.S._1 = W/F$ and $F.S._2 = W/B$

Where:

$F.S._1$ = factor of safety against flotation from design basis flood
$F.S._2$ = factor of safety against flotation from high ground water table

W = total weight of structures and foundation

F = buoyant force due to the design basis flood
B = buoyant force on submerged structure from high ground water table

The factors of safety against flotation for the nuclear island are ~~is~~ 3.1 and 3.3, respectively, for design basis flood and high ground water table. As shown in Table 3.8.5-1, the minimum required factor of safety against flotation for the two plant conditions are 1.1 and 1.5, respectively ~~is 1.1~~.

Revise the last note in Table 3.8.5-1 to read as follows:

B = buoyant force on submerged structure due to high ground water table

Revise Section 3.8.5.5.3 as shown below:

The factor of safety against sliding of the nuclear island during a seismic event is calculated as follows:

where:

$$F.S. = \frac{F_s + F_p}{F_D + F_H}$$

F.S. = *factor of safety against sliding*
F_s = *shearing and sliding resistance*
F_p = *passive pressure resistance, including surcharge*
F_D = *maximum dynamic lateral force, including dynamic active earth pressures*
F_H = *maximum lateral force due to all loads except seismic loads*

The factor of safety against sliding of the nuclear island during a safe shutdown earthquake is 1.34 in the north-south direction and 1.67 in the east-west direction. As shown in Table 3.8.5-1, the minimum required factor of safety against sliding during a safe shutdown earthquake is 1.1."

The factor of safety against sliding of the nuclear island during a tornado and a design wind is calculated as follows:

where:

$$F.S. = \frac{F_s + F_p}{F_H}$$

F.S. = *factor of safety against sliding*
F_s = *shearing and sliding resistance*
F_p = *passive pressure resistance*

F_H = *maximum lateral force due active pressure, including surcharge, and tornado or design wind*

The factor of safety against sliding of the nuclear island during a tornado is 6.8 in the north-south direction and 6.0 in the east-west direction. The factor of safety against sliding of the nuclear island during a design wind is 10.2 in the north-south direction and 9.3 in the east-west direction. As shown in Table 3.8.5-1, the minimum required factor of safety against sliding during a tornado and a design wind are 1.1 and 1.5, respectively.

Revise the last paragraph of Section 3.8.5.5.4 as shown below:

The factor of safety against overturning of the nuclear island during a safe shutdown earthquake is 724 in the north-south direction and 235 in the east-west direction. As shown in Table 3.8.5-1, the minimum required factor of safety against overturning ~~sliding~~ during an SSE is 1.1.

The factor of safety against overturning of the nuclear island during a tornado and a design wind is calculated as follows:

where:

$$F.S. = \frac{M_R}{M_w}$$

$F.S.$ = *factor of safety against overturning*
M_R = *Resisting Moment*
M_w = *Overturning moment of tornado or design wind*

The factor of safety against overturning of the nuclear island during a tornado is 19.6 in the north-south direction and 8.0 in the east-west direction. The factor of safety against overturning of the nuclear island during a design wind is 47.3 in the north-south direction and 22.8 in the east-west direction. As shown in Table 3.8.5-1, the minimum required factor of safety against overturning during a tornado and a design wind are 1.1 and 1.5, respectively.

Question 230.24

Section 3.7 of the SSAR states that a three-level seismic classification system is used for the AP600; seismic Category I, seismic Category II, and non-Category I. However, Section 3.2.1 (Seismic Classification) of the SSAR states that the methodology classifies structures, systems and components into three categories: seismic Category I (C-I), seismic Category II (C-II) and non-seismic (NS). Clarify the difference between non-Category I and non-seismic.

Response:

SSAR Section 3.7 was changed in Revision 1 (01/13/94) to make the seismic classifications consistent. The AP600 classification is seismic Category I (C-I), seismic Category II (C-II) and non-seismic (NS). This revision has generally eliminated the terminology "non-Category I" or "non-seismic Category I". Where retained, the term covers those items that are not classified as seismic Category I, i.e. it includes both seismic category II and non-seismic.

SSAR Revisions: Already incorporated in Revision 1

Question 230.25

Section 3.7 of the SSAR states that non-Category I structures are designed or physically arranged (or both) so that the safe shutdown earthquake (SSE) could not cause unacceptable structural interaction with or failure of seismic Category I structures, systems and components. However, Section 3.7.2 of the SSAR states that seismic Category II structures are designed and/or physically arranged so that the SSE could not cause unacceptable structural interaction with or failure of seismic Category I structures, systems and components. These two statements imply that classifications for non-Category I and seismic-Category II are the same. Clarify these statements.

Response:

SSAR Section 3.7 was changed in Revision 1 (01/13/94) to make the seismic classifications consistent. The AP600 classification is seismic Category I (C-I), seismic Category II (C-II) and non-seismic (NS). This revision has generally eliminated the terminology "non-Category I" or "non-seismic Category I". Where retained, the term covers those items that are not classified as seismic Category I, i.e. it includes both seismic category II and non-seismic.

The evaluation of non-seismic Category I items for interaction with seismic Category I structures, systems and components is described in SSAR Subsections 3.7.2.8 and 3.7.3.13. The definition for the seismic categories is given in SSAR Subsection 3.7.2.1.1. Seismic Category II applies to structures, systems and components that may include specific structural design provisions such that they will not fail during the safe shutdown earthquake.

SSAR Revisions: NONE

Question 230.27

On Page 3.7-1 of Section 3.7.1.2, the last paragraph states that SRP Section 3.7.1 contains the provision of frequency intervals used in the computation of the response spectra. Was this SRP provision satisfied in the computation of the response spectra?

Response:

The SRP Section 3.7.1 provision of frequency intervals was satisfied in the computation of the response spectra.

SSAR Revision:

See SSAR revision for Section 3.7.1.2 provided in response to RAI 230.28.

Question 230.28

a. In Section 3.7.1.2 of the SSAR, the cross-correlation coefficients between the three components of the ground motion time history should be specified to demonstrate that these three components are statistically independent. Provide that information.

b. Provide the procedures for the development of the vertical target PSD in Section 3.7.1.2 of the SSAR.

c. Explain the meaning of "with 20% averaging," as shown in Figures 3.7-10 through 3.7-12 of the SSAR.

Response:

a. The cross-correlation coefficients between the three components of the ground motion time histories are as follows:

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

where: 1, 2, and 3 represent the north-south, the east-west, and the vertical components of input motions, respectively.

Since all of the three coefficients are less than 0.16 as recommended in Revision 1 of NRC Regulatory Guide 1.92, it is concluded that these three components are statistically independent.

b. The target power spectral density (PSD) for horizontal input ground motion is as specified in Appendix A of the Standard Review Plan, Section 3.7.1, Revision 2 while the target PSD for the vertical direction is not given. Since the target acceleration response spectra for the vertical direction are similar to those for the horizontal direction, the target PSD for the horizontal direction is also used for the vertical direction in AP600.

c. The PSD functions of the input ground motions are calculated at uniform frequency steps of 0.0489 cycles per second. The PSDs presented in Figures 3.7.1-10 through 3.7.1-12 are the averaged PSD obtained over a moving frequency band of $\pm 20\%$ centered at each frequency. The PSD amplitude at frequency F has the averaged PSD amplitude between the frequency range of 0.8 F and 1.2 F as stated in Standard Review Plan, Section 3.7.1, Appendix A, Revision 2.

SSAR Revision:

Revise Section 3.7.1.2 as shown below:

A "single" set of three mutually orthogonal, statistically independent, synthetic acceleration time histories is used as the input in the dynamic analysis of seismic Category I structures. The synthetic time histories were generated by modifying a set of actual recorded "TAFT" earthquake time histories. The design time histories include a total time duration equal to 20 seconds and a corresponding stationary phase, strong motion duration greater than 6 seconds. The acceleration, velocity, and displacement time-history plots for the three orthogonal earthquake components 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 all of the three coefficients are less than 0.16 as recommended in of 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 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, ~~contains the~~ 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. An AP600-compatible target power spectral density curve was developed according to Standard Review Plan, Section 3.7.1 Appendix A, Revision 2 and is shown in Figure 3.7.1-9. This target power spectral density curve is used in both the horizontal and vertical directions.

~~An AP600-compatible target power spectral density curve was developed according to Appendix A of SRP 3.7.1, and is shown in Figure 3.7.1-9. The average power spectral density curve of the AP600 time histories conservatively enveloped the target power spectral density curve instead of 80 percent of the target power spectral density curve.~~ The comparison plots of the power spectral density curve of the AP600 acceleration time histories versus the target power spectral density curve are presented in Figures 3.7.1-10, 3.7.1-11, and 3.7.1-12. The PSD functions of the design time histories are calculated at uniform frequency steps of 0.0489 cps. The PSDs presented in Figures 3.7.1-10 through 3.7.1-12 are the averaged PSD obtained over a moving frequency band of $\pm 20\%$ centered at each frequency. The PSD amplitude at frequency F has the averaged PSD 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.

Question 230.29

On Page 3.7-2 and in Table 3.7-1 of the SSAR, the damping ratios assigned for HVAC ductwork, cable trays and fuel assemblies are 7%, 20% and 20%, respectively. Provide the bases for these parameters to justify the adequacy of using high damping ratios for the analyses of the welded ductworks, cable trays and fuel assemblies.

Response:

HVAC Ductwork:

The AP600 ductwork are bolted with [illegible]ed flange construction. The damping value used for heating, ventilation, and air conditioning (HVAC) systems, including ducts and the related supports, is equal to 7 percent of critical damping in conformance with guidance for bolted structures in Regulatory Guide 1.61.

Cable Trays:

The damping value used for electrical raceways systems, including cable trays and the related supports, is established based on the Bechtel/ANCO test results (Reference 19 of SSAR Section 3.7 Revision 1) for a variety of raceway configurations. The damping value depends on the magnitude of the input motion and the amount of cable fill within the cable tray as shown in SSAR Figure 3.7.1-13. Within the AP600 design range of acceleration, the damping value is equal to 7 percent for empty cable trays and up to 20 percent for greater than 50 percent filled cable trays.

Fuel assemblies:

The fuel assembly damping values are based on measured values from mechanical tests in both air and water environments. The fuel assembly damping value increases as vibrational amplitude increases. The fuel assembly damping under flowing water conditions exhibit very high values. Plant in-core neutron detector data indicate that a PWR fuel assembly is a highly damped structural system. The assembly damping is a result of combined inter-assembly rubbing and scraping, frictional forces and constraint of relative motion between the fuel rods and supports within an assembly, and fluid/structure interactions in a closely packed reactor core.

In analyses of a safe shutdown earthquake or of a LOCA transient, a fuel assembly is usually predicted to deflect to the physical limit of accumulated inter-fuel assembly gaps. To assess the fuel assembly dynamic responses under a postulated faulted condition transient, a 20 percent damping value is used to account for the mechanical and hydrodynamic effects for the assembly fundamental mode. This 20 percent fuel assembly damping value used in the analysis is conservative relative to the data from in-core neutron detectors. The fundamental mode of a fuel assembly is identified as the predominant mode for fuel dynamic analysis. Thus, the 20 percent damping ratio is applied to all fuel assembly vibrational modes.

Question 230.30

Figures 3.7.1-14 and 3.7.1-15 of the SSAR provide the damping values for rock material and soil material, respectively.

a. Clarify what damping values are to be used for hard-rock material and soft-rock material.

b. Provide the basis and source of these two figures.

Response:

a. The strain-dependent damping property for rock material shown in Figure 3.7.1-14 was used in the free-field SHAKE analysis for the hard rock profile and the soft rock profile.

The strain-compatible properties of the soft rock profile, obtained from the SHAKE analysis results, are shown in Table 2A-9. These material properties, shear modulus and damping values, are used in the soil-structure interaction analysis for the soft rock profile.

As stated in Section 3.7.2.1.2, the nuclear island is analyzed as a fixed-base structure without soil-structure interaction effect such that the hard rock material properties are not used in the analysis for the hard rock profile.

b. The strain-dependent soil properties shown in Figure 3.7.1-15 are obtained from the average soil curves shown in Figures 5 and 9 of Reference 7 of Appendix 2A (see list of references in Subsection 2A.7). The damping curve for soil material was adjusted to observe the limiting value of 15% per SRP Section 3.7.2 Rev. 2 criteria. The strain-dependent rock properties shown in Figure 3.7.1-14 are obtained from Figure 4 of Reference 8 of Appendix 2A.

SSAR Revision:

Revise the last paragraph of Section 3.7.1.3 as shown below:

Strain-dependent damping values are used for the foundation material in accordance with References 5 and 6. The damping curves for soil and rock materials are presented in Figures 3.7.1-14 and 3.7.1-15 ~~for rock material and soil material, respectively~~. These figures are same as those described in Subsection 2A.4 of Appendix 2A, Figures 2A-8 and 2A-9. The strain-dependent soil material damping is limited to 15 percent of critical damping.

Question 230.31

Section 3.7.1.4 of the SSAR describes the shear wave velocity profile for the supporting media from ground surface to a depth of 240 feet for both soft-rock site condition and soft-to-medium stiff soil site condition, and states that the base rock is at a depth of 120 feet.

a. What is the shear wave velocity profile for the base rock?

b. Because the base rock is at the depth of 120 feet, how significant will it be to specify the shear wave velocity profile from the depth of 120 feet to 240 feet for both soft-rock and soft-to- medium stiff soil sites? Provide such a profile or provide justification for not doing so.

c. Because the location of the base rock is not shown in Figure 2A-7 of SSAR, provide a complete plot for the shear wave velocity profile for hard-rock, soft-rock and soft-to-medium stiff soil sites.

Response:

a. The soil velocity profiles are defined in their general form in Section 3.7.1.4 by specifying the end points of each profile at ground surface and at 240 feet depth. The parametric soil-structure interaction (SSI) study presented in the Appendix 2A showed that for each profile, the location of base rock at a depth of 120 feet results in larger seismic responses. In the 3D SSI cases, each profile was modeled with its respective velocity profile up to the depth of 120 feet. Beyond this depth, the base rock was modeled as a uniform halfspace with the shear wave velocity of 8000 ft/sec.

b. As shown in the Appendix 2A both the soft-rock and soft-to-medium profiles were analyzed for depth to base rock ranging from very deep to 120 feet. In all of these cases the base rock was modeled with rock velocity of 8000 feet/sec. The parametric study in effect includes soft rock and soft-to-medium stiff soil profiles with depths ranging from 120 feet to very deep profiles. Based on the results of the parametric study, Appendix 2A, Subsection 2A.5 SSI analysis results, the case with depth to base rock equal to 120 feet is the governing site condition for both the soft rock and the soft-to-medium soil profiles. Therefore, the effect of increasing the depth to base rock from 120 feet to 240 feet is expected to be insignificant.

c. The shear wave velocity profile for the design soil profiles, hard-rock, soft rock and soft-to-medium stiff soil sites, with variation of depth to base rock, are shown in the attached Figure 3.7.1-17.

SSAR Revision:

Add the attached Figure 3.7.1-17 to Section 3.7.1

Revise the first paragraph of Section 3.7.1.4 as shown below:

The seismic design basis for the AP600 is to provide design coverage for as many plant sites as practical. For the design of seismic Category I structures, a set of three design soil profiles of various shear wave velocities is established in Appendix 2A. The three design soil profiles include a hard rock site, a soft rock site, and a soft-to-medium stiff soil site. The shear wave velocity profiles and related governing parameters of the three sites considered are the following:

- For the hard rock site, a uniform shear wave velocity of 8000 feet per second
- For the soft rock site, a shear wave velocity of 2400 feet per second at the ground surface, increasing 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 stiff soil site, a shear wave velocity of 1000 feet per second at ground surface, increasing to 2400 feet per second at 240 feet, base rock at the depth of 120 feet, and ground water at grade level.

The shear wave velocity profile for the design soil profiles, with variation of depth to base rock, is shown in Figure 3.7.1-17.

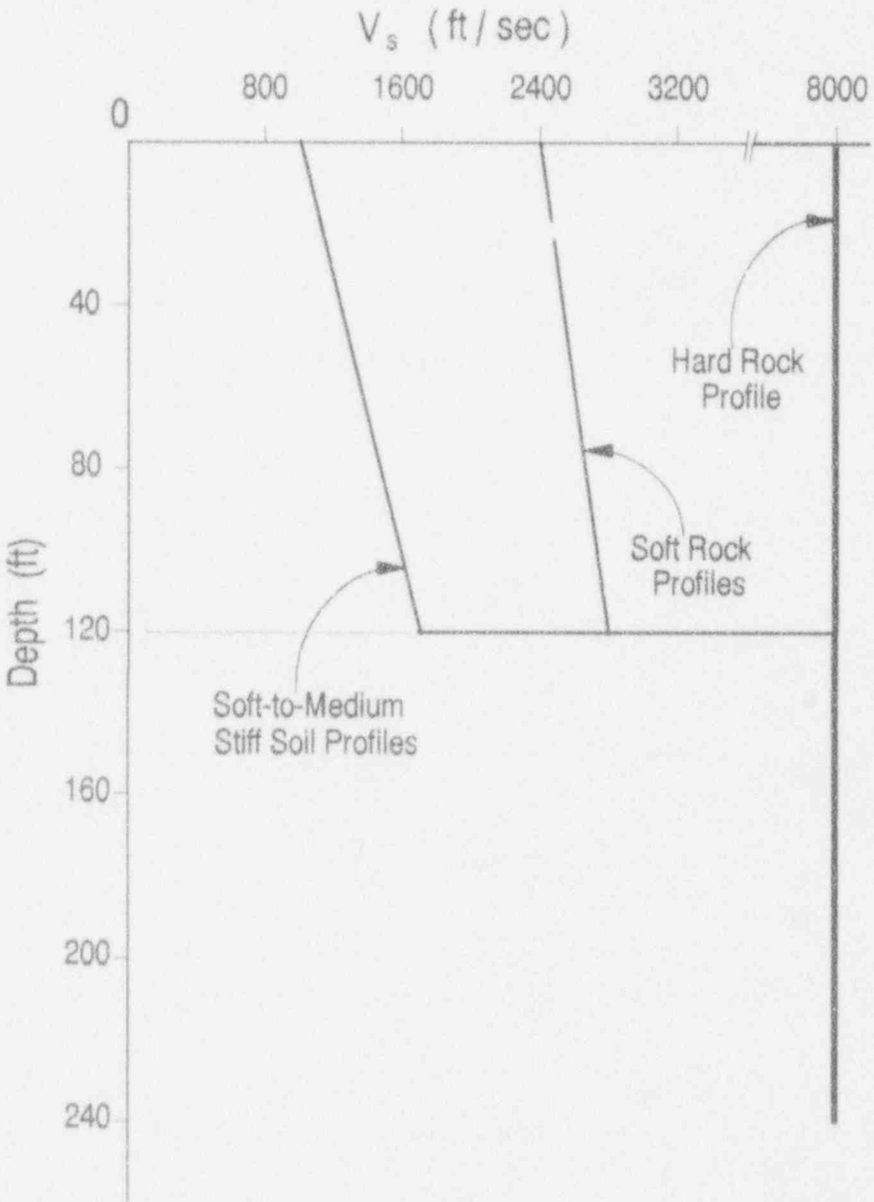


Figure 3.7.1-17
Shear Wave Velocity of Design Soil Profiles

Question 230.32

In Section 3.7.1 of the SSAR, the location of the input ground motion to be specified is not shown for the site conditions selected. Provide this information.

Response:

The input ground motion is applied at the finished grade in the free field as discussed in Subsections 2A.3 and 3.7.1.1.

SSAR Revision:

See SSAR revision shown in responses to RAI 230.28.

Question 230.33

From the staff's review of Section 3.7.1.4 and Appendix 2A of the SSAR, it appears that only three design soil profiles are required for the design of AP600 seismic Category I structures, and that some potential governing site conditions, such as a shallow soil site and a deep soil site, were not considered. Provide justification to demonstrate that the design of the seismic Category I structures and subsystems based on these three site conditions can envelop the design of the structures and subsystems founded on other potential sites in the United States.

Response:

The 3D analysis of the AP600 seismic Category I structures were based on three design soil profiles. These design profiles were identified as a result of a series of 2D soil-structure interaction (SSI) analysis which includes 5 velocity profiles along with variation of depth to base rock and depth to water table. Both deep and shallow soil profiles were considered in the 2D SSI analysis (see Table 2A-16). The 2D analysis results identified the critical combination of soil property and soil profile configuration for 3D analysis. Thus, the three selected design soil profiles bracket a wide variation of the soil properties and profiles considered in the 2D analysis to develop the governing seismic responses. The soil property variation considered envelopes the potential sites for nuclear power plants that are within the site interface parameters specified in SSAR Section 2.5.

SSAR Revision: NONE

Question 230.34

The term "time history analysis" appears to be inconsistently used throughout Section 3.7.2 of the SSAR. In some cases, it is mixed with the term "complex frequency response analysis." From the staff's review of Section 3.7.2.1.2 of the SSAR, it is the staff's understanding that the (modal) time history analysis method was used for the fixed base structural model (hard-rock site condition) to generate floor response spectra, and the complex frequency response analysis method was used for the soil-structure interaction analysis when the structures are founded on soft-rock site and soft-to-medium stiff soil site. Is this correct? Clarify any inconsistency.

Response:

Mode superposition time history analysis, using the computer program "BSAP", was used for the fixed-base structural model founded on hard-rock site. Complex frequency response analysis method, using program "SASSI", was used for the soil-structure interaction analysis when the structures are founded on soft-rock and soft-to-medium stiff soil sites. SSAR Section 3.7.2 is revised to clarify the description inconsistency as noted below. See also SSAR revisions identified in responses to RAI 230.43 for Section 3.7.2.5 and in responses to RAI 230.45 for Section 3.7.2.7

SSAR Revision:

Revise the second paragraph of Section 3.7.2.1 as shown below:

Seismic analyses, using the response spectrum method ~~and the~~**, the mode superposition** time-history method, **and the complex frequency response analysis method,** are performed for the SSE 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.

Revise the title of Section 3.7.2.1.2 as shown below:

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

Revise the third paragraph of Section 3.7.2.4 as shown below:

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

Revise the second and third paragraphs of Section 3.7.2.6 as shown below:

In mode superposition time-history analyses using computer program BSAP ~~seismic analyses using the time-history method~~, 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 BSAP and SASSI ~~time history~~ analyses with the earthquake components applied separately, and in the response spectrum analyses, the effect of the three components of earthquake 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.
- The peak responses due to each of the earthquake component from either the response spectrum analyses or the BSAP and SASSI ~~time history~~ analyses are combined using the square root of the sum of squares (SRSS) method.
- The peak responses due to each of the earthquake component 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.

Revise Section 3.7.2.12 as shown below:

In the seismic analyses, the response spectrum analysis method is used in conjunction with the finite element models, while the mode superposition time-history and the complex frequency response method ~~is~~ are applied to the lumped-mass stick model of the nuclear island. Therefore, a comparison of responses calculated by alternative methods is not necessary.

Question 230.36

The following request for additional information pertains to Section 3.7.2.1.2 of the SSAR:

a. Provide the validation package of computer code SASSI for review.

b. Explain the difference between the two phrases "applied simultaneously (time history in the seismic analysis)" and "applied separately."

Response:

a. The validation package of the computer program SASSI used for AP600 analysis is available for review in Bechtel's San Francisco office.

b. The input time histories are applied simultaneously in the mode superposition time history seismic analysis using the computer program BSAP for the hard rock site. In the BSAP analysis, the three time history components of earthquake are input together in a single computer run, and the seismic response output from this single computer run include the effects of three components of earthquakes.

For the soft-rock and soft-to-medium soil sites, soil-structure interaction (SSI) is considered using the computer program SASSI. In the complex frequency response analysis using the computer program SASSI, separate analysis is performed for each of the three global directions. Response time histories, in terms of accelerations and member forces, from each of these three analyses are computed. These response time histories are then added algebraically to obtain the "total" response time histories which simulate responses from 3 simultaneously applied motions.

SSAR Revision: NONE

Question 230.40

Section 3.7.2.3.2 of the SSAR states that the three-dimensional, lumped-mass stick model of the steel containment vessel is developed based on the axisymmetric shell model. This implies that the steel containment shell model is axisymmetric. However, Section 6.3.2.2.3 of the SSAR states that the in-containment refueling water storage tank (IRWST) is constructed as an integral part of the containment structure. In addition, when the polar crane is included in the dynamic model, the trolley should be assumed to be parked at the end of the crane girder. Explain how the steel containment shell can be modeled from an axisymmetric shell model.

Response:

The steel containment shell is modelled as an axisymmetric model because of the following:

The structural design of the steel containment vessel is described in Section 3.8.2.1, and its configuration is shown in Figure 3.8.2-1. The structural design of the IRWST is described in Section 3.8.3.1.7, and its configuration is shown in sheets 2 and 7 of Figure 3A-4. The IRWST is designed and constructed as an integral part of the containment internal structures and is isolated from the steel containment vessel as shown in Figure 3A-4. Hence, the IRWST is not constructed as an integral part of the containment structure as stated in Section 6.3.2.2.3.

The polar crane is supported on a ring girder which is an integral part of the steel containment vessel at Elevation 209'-0", see sheet 3 of Figure 3.8.2-1. The polar crane 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 considers the flexibility of the crane bridge girders and truck assembly, and the containment shell's local flexibility.

During plant operating condition, 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 following is used:

- The slight offset of the polar crane is neglected by assuming the crane bridge spanning in the north-south direction.
- The trolley is conservatively assumed to be located at the midspan of the crane bridge girders. It is judged that this trolley configuration would induce the maximum polar crane and containment shell seismic response while neglecting only the minor torsional effect on the containment vessel which has a large torsional capacity.

SSAR Revision:

Revise the first paragraph of Section 6.3.2.2.3 as shown below:

The in-containment refueling water storage tank is a large, stainless-steel lined tank located underneath the operating deck inside the containment. The in-containment refueling water storage tank is AP600 Equipment Class C and is designed to meet seismic Category I requirements. The tank is constructed as an integral part of the containment internal structures, and is isolated from the steel containment vessel. See Subsection 3.8.3 for additional information.

Add the following paragraphs to the end of Section 3.7.2.3.2:

The polar crane is supported on a ring girder which is an integral part of the steel containment vessel at Elevation 209'-0". It is modelled as a single degree of freedom system attached to the steel containment shell as shown in Figure 3.7.2-5. The polar crane model includes the flexibility of the crane bridge girders and truck assembly, and the containment shell's local flexibility.

During plant operating condition, 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 following is used:

- The slight offset of the polar crane is neglected by assuming the crane bridge spanning in the north-south direction.

- The trolley is conservatively assumed to be located at the midspan of the crane bridge girders. It is judged that this trolley configuration would induce the maximum polar crane and containment shell seismic responses while neglecting only the minor torsional effect on the containment vessel which has a large torsional capacity.

Question 230.42

The following request for additional information pertains to Section 3.7.2.4 of the SSAR:

a. The first paragraph of Section 3.7.2.4 states that the nuclear island SSI responses generated for the analysis and design of seismic subsystems include nodal displacements, nodal accelerations and floor response spectra (FRS). Explain how the structural member forces (forces and moments) used for the structural design were generated for a soil site condition.

b. The last paragraph of Section 3.7.2.4 (Page 3.7-7) states that the selected soil conditions envelop the potential variation of soil properties and, therefore, the guidelines of SRP Section 3.7.2 for the variation of soil properties were not considered. Justify this statement, especially, when structures are founded on soft soil site for which the variation (uncertainty) in soil properties should be carefully considered.

c. Explain the differences between the two phrases "the time-history SSI analysis using the program SASSI" and "the complex frequency response analysis using the program SASSI."

d. When the computer code SHAKE was applied, which soil degradation curve was used?

Response:

a. The structural member forces used for the structural design for the soil site condition are generated as described in the last paragraph of subsection 3.7.2.1.1.

b. The SRP requirement for variation of soil properties is not considered because of the following:

- Sensitivity studies have been performed for a broad range of soil and rock site conditions (see Appendix 2A of the SSAR) to assess the impact of site parametric uncertainties because the seismic design basis is to provide design coverage for as many plant sites as practical.

- A suitable set of design soil profiles, covering sites with shear velocities from 1000 fps to 8000 fps, have been established for the plant seismic design based on the above evaluation of the generic sites.

- The site interface requirements established in Section 2.5 require the proposed sites to be within the generic site sensitivity analyses, such as:

  - The shear wave velocity (based on low strain best estimate soil properties) is greater than or equal to 1000 fps.
  - There is no potential for fault displacement at plant site.

c. As discussed in responses to RAI 230.34, Section 3.7.2.4 will be revised and the statements are clarified to read as follows:

- The soil-structure interaction (SSI) analyses of the nuclear island are performed using the program SASSI, and

- SSI analyses are performed using the complex frequency response method with computer program SASSI.

d. As discussed in Section 2A.4, the strain-dependent shear modulus and damping curves used in the free-field SHAKE analysis are presented in Figures 2A-8 for soil materials and in Figure 2A-9 for rock materials. These curves are obtained from references 6, 7 and 8 shown in Subsection 2A.7.

SSAR Revision:

See SSAR revision identified in responses to RAI 230.34.

Question 230.43

a. The second paragraph of Section 3.7.2.5 states that seismic floor response spectra are computed using the nodal time-history responses determined from the nuclear island seismic time-history analyses with the various design soil profiles. As stated in Section 3.7.2.1.2, the complex frequency response analysis using the computer program SASSI was applied for the soil site conditions to generated the FRS. Clarify this statement.

b. The second paragraph of Section 3.7.2.5 states that Figures 3.7.2-24 through 3.7.2-26 present the s shutdown earthquake FRS for the hard rock site condition at selected locations of the coupled model of the shield and auxiliary building, the steel containment vessel and the containment internal structures. Provide the FRS for the other site conditions.

Response:

a The statement is revised to read as follows:

"Seismic floor response spectra are computed using the time-history responses determined from the nuclear island seismic analyses with the various design soil profiles. The time-history responses for the hard rock condition are determined from a mode superposition time history analysis using computer program BSAP. The time history responses for the soft rock and the soft-to-medium soil cases are obtained from a complex frequency response analysis using computer program SASSI."

b The requested SSE floor response spectra, including acceleration response spectra for the three design site conditions and the corresponding enveloped and widened spectra, are presented in SSAR Revision 1 (1/13/94) of Figures 3.7.2-25 through 3.7.2-27.

SSAR Revision:

Revise the second paragraph of Section 3.7.2.5 as shown below:

Seismic floor response spectra are computed using the ~~nodal~~ time-history responses determined from the nuclear island seismic ~~time-history~~ analyses with the various design soil profiles. The time-history responses for the hard rock condition are determined from a mode superposition time history analysis using computer program BSAP. The time history responses for the soft rock and the soft-to-medium soil cases are obtained from a complex frequency response analysis using computer program SASSI. Floor response spectra for ASME Code Case N411 damping, and for damping values equal to 2, 3, 4, 5, 7, 10, and 20 percent of critical damping are computed at the required locations."

Question 230.44

When the complex frequency response analysis method was used, were the three components of the earthquake motion applied simultaneously or separately (Section 3.7.2.6 of the SSAR)?

Response:

In complex frequency response analysis, separate analysis is performed for each of the three global directions. Response time histories, in terms of accelerations and member forces, from each of these three analyses are computed. These response time histories are then added algebraically to obtain the "total" response time histories which simulate responses from 3 simultaneously applied motions.

SSAR Revision: NONE

Question 230.45

The second paragraph of Section 3.7.2.7 of the SSAR states that in the time-history analyses, combination of modal responses is not necessary. It is not clear how the modal time-history analyses (using the computer program BSAP) for a fixed base structural model (structures founded on hard rock site) were performed. Clarify this statement.

Response:

The last sentence of Section 3.7.2.7 should be stated as follows:

> "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."

SSAR Revision:

Revise the last sentence of Section 3.7.2.7 as shown below:

The modal responses of the response spectrum system structural analysis are combined using the square root of the sum of squares method. When closely spaced modes are present, these modes are considered using either the grouping method, the 10 percent method or the double sum method shown in Section C of Regulatory Guide 1.92, Revision 1. 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. ~~In the time history analyses, combination of modal responses is not necessary.~~

Question 440.2

WCAP-13345, Rev. 2, "AP600 Core Makeup Tank Test Specification," is dated November 1991. Since that time, substantial changes have been made in both the test program and the design of the test article; these have been communicated to the staff during meeting presentations. A revised and updated copy of the test specification, with current information on test article design, instrumentation, test matrix, should be submitted for staff review.

Response (Revision 1):

WCAP-13345, "AP600 Core Makeup Tank Test Specification," Revision 0, dated November 1991 has been updated. The current revision, Revision 2, was provided to the NRC via Westinghouse letter NTD-NRC-94-4068, dated February 22, 1994.

SSAR Revision: NONE

Question 440.4

The test matrix in WCAP-13345, Rev. 2 indicates that the maximum pressure to be tested in the CMT facility is approximately 1500 psia. Discuss why this is adequate, in view of the fact that the CMT will be approximately at the normal primary system pressure of 2250 psia when the safety systems are actuated. If the upper limit for the tests has been changed, this should be indicated in the updated test specification requested in Q440.2.

Response (Revision 1):

The CMT test facility has been designed and fabricated to operate at 2250 psig. The test matrix provided in Revision 0 of WCAP-13345, to which the question refers, has been revised to include tests at the higher pressure. This revised matrix is included in WCAP-13345, "AP600 Core Makeup Tank Test Specification," Revision 2 which was provided to the NRC via Westinghouse letter NTD-NRC-94-4068, dated February 22, 1994.

SSAR Revision: NONE

Question 440.5

The test numbers referenced in Section 8.0 of Revision 2 to WCAP-13345, "Test Operation", do not correspond to those shown in Table 8.1, "AP600 Core Makeup Tank Test Matrix." There also appears to be a similar inconsistency between the tests referenced in the "Comments" column of Table 8.1 and the test numbers listed in the left-most column. These inconsistencies should be corrected in the updated Test Specification requested in Q440.2.

Response (Revision 1):

The inconsistencies within the CMT test specification have been corrected. WCAP-13345, "AP600 Core Makeup Tank Test Specification," Revision 0 has been updated and the current revision, Revision 2, was provided to the NRC via Westinghouse letter NTD-NRC-94-4068, dated February 22, 1994.

SSAR Revision: NONE

Question 440.6

The test descriptions in Section 8.0 of Revision 2 to WCAP-13345 are general in scope, and do not include detailed test procedures. In some cases, temperatures, pressures, liquid levels, and other test facility conditions are not specified, nor are detailed data acquisition procedures discussed. The updated Test Specification requested in Q440.2 should include sufficient detail on test methods, facility conditions, and data acquisition, including step-by-step procedures, for the staff to determine if an adequate range of data on component performance will be provided.

Response (Revision 1):

When Westinghouse prepares a test specification, the detailed test procedures are not included since they are the responsibility of the testing organization. The testing organization develops the test procedures which are then reviewed and approved by Westinghouse. The procedures are operator instructions on how to operate the facility to obtain the test conditions which are specified in the test specification. The test specification (WCAP-13345, Revision 2) provides adequate information to determine the adequacy of the tests including: information on the facility initial conditions, data acquisition, range of conditions, expected data, and instrumentation and method of testing.

SSAR Revision: NONE

Question 440.10

While the tests in the matrix of WCAP-13345 (Table 8.1) include individual experiments to study condensation, recirculation, depressurization, and draining behavior, there does not appear to be any test that takes the CMT through the entire sequence of events that would be expected to occur in the plant. The behavior of the CMT as it goes through the transitions that occur during such a sequence is of substantial interest. At least one test that captures the entire series of states and transitions expected in the CMT should be included in the test matrix.

Response (Revision 1):

The test matrix has been revised. The revised matrix includes tests which capture the entire series of states and transitions expected in the CMT. WCAP-13345, "AP600 Core Makeup Tank Test Specification," has been updated to include this revised test matrix. The current revision, Revision 2, was provided to the NRC via Westinghouse letter NTD-NRC-94-4068, dated February 22, 1994.

SSAR Revision: NONE

Question 440.50

The new configuration of the ADS announced in late 1993 appears to have implications regarding the test matrices for the test programs for the OSU/APEX and SPES-2 facilities. While the design change in stages 2 and 3 appears to be relatively easy to account for in the two integral facilities, the new design of the 4th stage of the ADS means that the single failure behavior of the AP600 (i.e., failure of one 4th stage ADS valve) is substantially different, in that such a failure no longer completely eliminates the venting capacity of one train of the 4th stage, but simply reduces it. Furthermore, this may completely change the limiting single failure for the AP600 design over a range of design basis accidents involving depressurization (see Q440.51).

How does Westinghouse plan to account for the change in the ADS design and potential changes in limiting single failures for those tests in the SPES-2 and OSU/APEX facilities in which the limiting single failure is to be simulated?

Response:

The SPES-2 (Simulazione PWR per Esperienze di Sicurezza) tests are integral systems tests to obtain thermal-hydraulic data for computer code validation and to investigate systems interactions during high pressure transients. The impact of the revised changes have been evaluated with respect to the configuration of the automatic depressurization system (ADS) and the following modifications to the SPES-2 facility have been made:

The changes required for Stages 1, 2 and 3 are not significant. A single valve is used in each stage to initiate and control the blowdown and an orifice is used to represent the second valve. These orifices have been sized to represent the minimal valve area, or maximum pressure drop in each stage. This represents the minimum flow capability of the ADS for each stage.

The revised design of the fourth stage of ADS incorporates an additional flow path and valve in both fourth stages discharging from each of the hot legs. When a single failure of a fourth stage valve is assumed, flow can occur through each fourth stage. SPES-2 has been reconfigured to allow both fourth stages to discharge into a single header and flow measurement system. The total flow out of both fourth stages will be measured simultaneously.

Orifices installed in each ADS fourth stage discharge line will be sized to simulate the total pressure drop and flow area in each line. For a single failure of a fourth stage valve, an orifice representing the pressure drop of a single flow path will be installed.

The Oregon State University (OSU) tests are integral systems tests to obtain thermal-hydraulic data for computer code validation and to investigate long term cooling behavior. The impact of the revised changes have been evaluated with respect to the configuration of the automatic depressurization system and the following modifications to the OSU facility are planned:

The same changes described above for SPES-2 for Stages 1, 2 and 3 will be implemented. For tests that assume a single failure in any of the first three stages of ADS, the installed orifices will be modified to represent the minimum flow area, and therefore, minimum venting capability in each flow path.

The Break and ADS Measurement System (BAMS) at the OSU facility will be reconfigured to measure the flows out from a simulated single ended pipe break, the first three stages of ADS, and each of the fourth stages. During a test of a double ended break, the fourth stage will be headered into a single break measurement system and the total flow out of both fourth stages will be measured. The orifices in each fourth stage will be sized to represent the appropriate flow area, i.e., in the case of a single failure of a fourth stage valve, an orifice representing the pressure drop of a single flow path will be used.

SSAR Revision: NONE

Question 450.8

Section 6.5 of the SSAR indicates that the AP600 does not have ESF filter systems, a containment spray system, and secondary containment for the fission product control. The only fission product control system is the primary containment.

GDC 41 specifies the requirements of containment atmosphere cleanup systems. Sections 6.2.3, 6.5.1, 6.5.2, and 6.5.3 of the SRP provide guidelines on fission product leakage control through secondary containment functional design, ESF atmosphere cleanup systems, containment spray as a fission product cleanup system, and fission product control systems and structures. The function of the fission product control systems and structures is to limit the potential release of radioactive materials that would result from accidents.

Section 15.6.5.3.9 of the SSAR states that the calculated dose consequences at the site boundary and control room meet the regulatory requirements. The staff is reviewing the methodology of these calculations separately and has not reached a conclusion on i s acceptability. However, the staff concludes that there is a reduction of the fission product control systems in the design of AP600 compared to the design of current operating plants. It results in a lack of redundancy and reduction in safety margin. The staff has not found any testing program in the SSAR to demonstrate the adequacy of the overall fission product control systems of AP600. Based on the above discussion, the staff is concerned that the fission product control systems may not be determined to be adequate, even if the calculated dose consequences are found to be acceptable. Provide any additional information or testing results to address the above staff concern.

Response (Revision 1):

As with current operating plants, the primary containment for the AP600 is the most significant system for limiting release of radioactivity to the environment in the event of a core-damage event. The effectiveness of the AP600 containment is enhanced relative to the design of current operating plants by significant reduction in the number and size of containment penetrations; by the simple, reliable passive containment cooling system; and by design features addressing potential containment challenges in severe accident scenarios. The effectiveness of the AP600 containment is indicated by the low probability of significant offsite releases discussed in the AP600 PRA. Fission product control safety margin for the AP600 is enhanced relative to current operating plants~~, contrary to the staff conclusion stated in the RAI~~.

The radiological consequences analyses discussed in Subsection 15.6.5.3 of the SSAR provides the licensing design basis evaluation of the AP600 containment function. This conservative, deterministic evaluation uses the ALWR physically based source term, as discussed in SSAR Subsection 15.6.5.3.1.2, to define the fission product release transient to the AP600 containment atmosphere. This source term is similar to the new source term being developed by the NRC. The AP600 analysis accounts for the natural processes for removal of fission products from the containment atmosphere during the event. The elemental iodine removal coefficients are calculated using the model provided in Revision 2 of Section 6.5.2 of the Standard Review Plan. The particulate removal coefficients are based on ~~analytical results. Both~~ sectional aerosol codes and empirical correlations which have compared well with experimental results of particulate deposition. ~~A report discussing the experimental basis for particulate removal is being compiled and will be provided for NRC review by May 21, 1993.~~

Reference 450.8-1 provides the basis for the determination of post-LOCA particulate removal coefficients and was transmitted by EPRI to the NRC on April 30, 1993. The document reports the following removal coefficients for the AP600:

0 - 10.3 hours	0.49 hr^{-1}
10.3 - 11.0 hours	0.72 hr^{-1}
> 11.0 hours	0.52 hr^{-1}

These values supersede those that were used in the LOCA dose analysis reported in the SSAR:

0 - 4.0 hours	0.35 hr^{-1}
4.0 - 4.5 hours	1.3 hr^{-1}
> 4.5 hours	0.5 hr^{-1}

The limiting dose for the LOCA is the site boundary thyroid dose which is calculated over the first two hours of the accident. With the set of removal coefficients from Reference 450.8-1, the doses reported in the SSAR would be reduced from currently reported values.

In Revision 1 to the response to RAI 470.9, an analysis has been provided of the LOCA doses based on the NRC source term. This analysis utilizes a particulate removal coefficient of 0.5 hr^{-1} for all time periods and is thus consistent with the set of values presented in Reference 450.8-1.

Reference:

450.8-1 Passive ALWR Containment Natural Aerosol Removal," April 29, 1993 was prepared by David E. Leaver (Polestar Applied Technology, Inc.), Jun Li (TENERA, L.P.), and Rudolph Sher (Rudolph Sher Associates).

SSAR Revision: NONE