Confirmatory Soil-Structure Interaction Analysis of the Westinghouse AP600 Nuclear Island Structures

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Enclosure 1

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1.0 Introduction

In revision 2 of AP600 Standard Safety Analysis Report (SSAR), Westinghouse Electric Corporation (Westinghouse), the AP600 License Applicant, used two three-dimensional (3-D) soil-structure interaction (SSI) analysis cases and one fixed-base 3-D analysis case to calculate the design forces and moments and floor response spectra for the Nuclear Island Structures. These 3-D analysis cases were selected on the basis of a number of two dimensional (2-D) SSI parametric analyses in which several soil and foundation condition parameters (e.g. soil depth, soil stiffness properties, depth of water table, etc.) were varied. The Nuclear Regulatory Commission (NRC) staff and its consultants (hereafter called the Staff Review Team) had reviewed these SSI analysis results (both 3-D design-bases analyses and 2-D parametric analyses). Even though the applicant's overall approach of determining seismic design loads and responses was found to be appropriate, the staff had major concerns regarding the following seven issues:

- (i) the ranges of soil conditions covered in the parametric 2-D SSI cases
- (ii) the effects of soil layer thickness between basemat and bedrock
- (iii) the effects of variations in assumed seismic input directions
- (iv) the effects of soil element mesh size on the seismic response
- (v) the adequacy of the response spectra at the foundation with respect to NRC Standard Review Plan (SRP) requirements (hereafter called the Sixty-Percent SRP requirements)
- (vi) the adequacy of applicant's 3-D design basis seismic analyses, and
- (vii) the lateral earth pressure on below-grade exterior walls of the nuclear island

To further assess and resolve these concerns, the staff had issued several Requests for Additional Information (RAIs). While Westinghouse has been performing additional SSI analyses to respond to staff's RAIs, it was decided by the staff to undertake some independent SSI analyses in order to ensure that AP-600 design basis seismic loads and response spectra are adequate. These independent analyses and their results are presented in this report.

[Since these independent analyses have been undertaken, Westinghouse has submitted an updated version of AP600 SSAR (Revision 9, dated August 9, 1996) in which, Westinghouse, based on the results of staff's independent analyses (that are being reported here), has increased the number of 3-D design basis SSI analysis cases from two to three. The conclusions presented in Section 10 of this report are based on a comparison of staff's independent analysis results reported here and the results presented in Revision 9 of AP600 SSAR]

2.0 Scope

The overall scope of work reported here is to verify if the seismic soil-structure interaction (SSI) analyses performed by the applicant to determine the design basis forces and moments and in-structure or floor response spectra for the Nuclear Island are adequate in accordance with SRP requirements and practices. For the purpose of this verification, the work scope was divided by the Staff Review Team into seven tasks listed below:

Task 1: Evaluation of the adequacy of the ranges of soil conditions covered in applicants 2D parametric SSI analyses.

Task 2: Evaluation of the effects of soil layer thickness between basemat and bedrock.

Task 3: Evaluation of the effects of variations in assumed seismic input directions.

Task 4: Evaluation of the effects of soil element mesh sizes on the seismic response.

Task 5: Evaluation of the adequacy of SSI response spectra at the foundation level in the context of sixty-percent SRP requirement.

Task 6: Evaluation of the adequacy of applicant's 3-D design basis seismic analyses.

Task 7: Evaluation of the adequacy of design basis lateral earth pressure on below-grade exterior walls of the Nuclear Island.

These seven evaluations and their results are presented and discussed in the following sections.

3.0 Task 1: Evaluation of Ranges of Soil Conditions Considered

The applicant had originally performed two 3-D SSI analyses and one fixed-base analysis to determine the design basis forces, moments, and in-structure spectra. Based on a number of 2-D SSI analysis case results, the applicant claimed that these three 3-D design basis analysis cases (two SSI and one fixed-base) adequately cover the entire range of sites for which the license is being sought. To verify this claim, the Staff Review Team selected and analyzed the site conditions listed in Table 3-1 using the 2-D SASSI computer program. A summary of these analysis cases are presented below; the details are provided in Appendix A.

The 2-D structural model of the Nuclear Island used in analyzing the cases listed in Table 3-1 was provided by the applicant. The free-field seismic input motion acceleration time histories were also provided by the applicant. In order to evaluate if the applicants 2-D parametric analysis cases are adequate for the specified range of soil sites, the in-structure response spectra at certain Nuclear Island elevations resulting from the Table 3-1 parametric cases were compared with the enveloping spectra at the corresponding locations from the applicant's parametric analysis cases. The spectra were compared at the following locations:

(i) Shield Building Roof (Elevation 307.25 ft)
(ii) Polar Crane (Elevation 205.33 ft)
(iii) Containment Internal Structure Operating Floor (Elevation 135.25 ft)

The comparison of response spectra at these locations showed that applicant's enveloping spectra enveloped most of those generated from Table 3-1 parametric cases. However, a close inspection of the results indicated that the soft rock case (i.e. Case H) produced in structure spectra at some locations that, at certain frequency ranges, exceeded applicant's enveloping spectra. The Staff Review Team judged that the significance of this finding can be high and so it was decided that this finding should be confirmed through a 3-D SASSI analysis so that the conclusions are not contentious. Accordingly, in Task 6, the 3-D SSI analysis case used case H soft rock properties.

Table 3-1
List of 2-D SASSI SSI Analysis Cases (1)

Case	Soil/Rock Type	Assumed Low Strain Shear Wave Velocity	Subcase	Soil Moisture Condition	Depth to(2) Bedrock (feet)
A	Soft to Medium Stiff	1000 fps at grade varying linearly to 2400 fps at 240' (Seed-Idriss degradation properties)	A1 A2 A3 A4	Dry Dry Wet Saturated	80 120 120 120
B	Stiff Rock	11,000 fps	B	N/A	N/A
C	Medium Rock	8,000 fps	C	N/A	N/A
D	Soft to Medium Stiff	1000 fps at grade varying parabolically to 2400 fps at 240' (Seed-Idriss degradation properties)	D	Dry	240
F	Soft to Medium Stiff	1000 fps at grade varying parabolically to 2400 fps at 240' varying parabolically (lower bound degradation properties)	F	Dry	240
G	Uniform Site	1500 fps uniform soil site with Seed-Idriss degradation properties	G	Dry	N/A
H	Soft Rock	3,500 fps	H	N/A	N/A
I	Soft Site	1000 fps uniform soil site with Seed-Idriss degradation properties	I1 I2	Dry Dry	40 45
J	Specific Deep Site	Savannah River deep soil site with variable velocity profile and lower bound degradation properties		Dry	N/A
K	Soft to Medium Stiff	Same as Case A: • Vertical Wave Incidence • 30° Wave Incidence	K1 K2	Dry Dry	120 120

Notes: (1) For cases A,D,F, I, and K, bedrock shear wave velocity of 8,000 fps was used
(2) Depth of bedrock from finished grade level. Foundation depth is 40ft.

4.0 Task 2: Evaluation of the Effects of Soil Layer Thickness Between Basemat and Bedrock

The effects of variation of soil layer thickness between basemat and bedrock was evaluated first by performing four parametric 2-D SASSI analysis cases and then by performing seven 3-D parametric cases using the computer code CARES (Computer Analysis for Rapid Evaluation of Structures).

The four cases analyzed by the 2-D SASSI computer program assumed soft soil sites (shear wave velocity = 1000 FPS) and are characterized as follows:

(i) Case I1-40: Depth of soil form the finished grade level to the top of the bedrock is 40 ft. Thus, the thickness of soil medium between the bottom of the basemat and the bedrock is zero, and the basemat rests directly on the bedrock. The seismic input motion is applied at the finished grade level.

(ii) Case I1-45: Same as I1-40, except that the depth of soil to bedrock is 45 ft. and the thickness of soil medium between the bottom of the basemat and the bedrock is 5 ft.

(iii) Case I2-40: Same as I1-40, except that the seismic input motion is applied at the top of the bedrock.

(iv) Case I2-45: Same as I1-45 except that the seismic input motion is applied at the top of the bedrock.

The results from these four parametric cases were compared with the applicant's response spectrum envelopes ~~enveloping spectra~~ and the results from the other parametric cases listed in Table 3-1 (see Appendix A). The comparison showed that the building response is significantly affected by the variation of the thickness (t) of soil medium between the basemat and the bedrock, especially when the input motion is applied at the top of bedrock and when 't' is small.

The Staff Review Team judged that the finding noted above may increase the design basis seismic loads significantly, and so this finding should be confirmed by additional analyses. As such, seven more confirmatory analyses were performed in which the 3-D computer program CARES was used and the thickness 't' of the soil medium between the basemat and the bedrock was varied from 10 ft to 100 ft (at 10 ft. intervals). The results of these analyses showed that, as 't' decreases, the floor response spectra peaks shift and accentuates the lower frequency structural modes. It was further observed that, when 't' is less than 20 ft., the lower frequency peak of the response spectra increases markedly (see Appendix C). Thus, it was concluded that either AP600 design basis must include such shallow soil sites and recompute the seismic responses (forces, moments, and floor response spectra) or exclude such sites from the licensing basis.

5.0 Task 3: Evaluation of the Effects of Seismic Input Directions

The effects of seismic input motion directions on both the in-plane and out-of-plane response of structures were evaluated first by performing two parametric 2-D SASSI analyses and then by performing several 3-D CARES analyses. The 2-D SASSI analysis consisted of cases K1 and K2 in Table 3-1 (see Appendix A). Case K1 used vertical incidence waves and case K2 used 30° inclined (from vertical) waves. Comparison of the responses resulting from cases K1 and K2 showed that the effects of angle of incidence are insignificant. But, since the 2-D SASSI analysis cannot account for the coupling between two orthogonal direction motions and out-of-plane structural behavior, it was decided to perform additional 3-D CARES analyses. These 3-D CARES analyses also, however, showed that the effects of angle of incidence on the out-of-plane or in-plane response of the structure are negligible (see Appendix C).

6.0 Task 4: Evaluation of Mesh Size Effects

The effects of finite element mesh size representing the soil medium adjacent to the below grade exterior walls were evaluated to determine if the mesh size used by the applicant to calculate the floor response spectra are adequate. For this purpose, three 2-D SASSI analyses were performed for a soft soil site using three different mesh sizes to represent the side soil. The first analysis was the base case analysis that used a fine mesh with 8 layers of elements to represent the side soil; the second case used 5 layers, and the third case used a coarse mesh with only 3 layers of elements (see Appendix A).

A comparison of the resulting floor response spectra showed that the use of 3 soil layers of elements to represent the side soil is unconservative compared to the base case that uses 8 layers; the 3 layer case results in a significant reduction of the spectral peak at a frequency of about 10 fps. Even though the applicant used 8 layers in its 2-D parametric analyses, AP600 design basis floor spectra and member forces and moments (except those for the design of exterior walls) are based either on fixed-base analyses or on 3-D SSI analyses that use only 3 layers of elements to represent the side soil. As such, the applicant must demonstrate that, if its 3-D SSI analyses had used 8 layers instead of 3 layers of soil elements, the resulting floor spectra and member forces and moment in the Nuclear Island would still be enveloped by the AP600 design-basis values.

7.0 Task 5: Evaluation of the "Sixty-Percent" SRP Requirement

In Section 3.7.2-II.4 of NRC's Standard Review Plan (NUREG-0800), it is stated that, "the spectral amplitude of the acceleration response spectra (horizontal component of motion) in the free field at the foundation depth shall not be less than 60 percent of the corresponding design response spectra at the finished grade in the free field." It further states that for a given site under consideration, if three SSI analyses are performed (the first one using the best-estimate site-specific solid shear moduli (G), the second one using 2G, and the third one using 0.5G), then the above 60 percent limitation may be satisfied using an envelope of the three spectra corresponding to the three soil properties.

The applicant has performed several 2-D SSI parametric analyses and only two 3-D SSI analyses (subsequent to the study reported here, the applicant performed a third 3-D SSI analysis). But, in none of these analyses, the compliance with the "sixty-percent" SRP requirement has been demonstrated on an individual case basis. In its interaction with the Staff Review Team on this issue, the applicant indicated that, since the envelope of the 2-D and 3-D case free-field spectra at the foundation level envelops the 60 percent of the design spectra at the finished grade, this may be considered equivalent to satisfying the requirement on the basis of three SSI analyses in which the soil properties are varied as discussed above. This justification may not be appropriate for the following two reasons:

(i) Since the applicant did not vary the soil properties for all cases in accordance with the SRP guidelines discussed above, if a particular site under consideration has soil shear modules value close to the lowest value considered by the applicant (i.e., shear wave velocity of 1000 fps), the site cannot be considered to have complied with the "sixty percent" SRP requirement, because for that site the 0.5G case has not been considered.

(ii) The SRP states, "the 60 percent limitation may be satisfied USING AN ENVELOPE of the three spectra....". The use of enveloped spectra at the foundation level eliminates the possibility of underestimating the in-structure response that may result from various combinations of soil column frequencies and dominant structural frequencies. Since the applicant did not actually use "enveloped" spectra at the foundation level, it is not possible to rule out with certainty that the in-structure response have not been underestimated at some frequencies. This is explained below with an illustrated example; additional explanation is provided in Appendix D.

Let Figure 7-1 show the free-field spectra for a soil site as labeled. Let f_2, f_1, and f_3 be the fundamental soil column frequencies corresponding to best estimate soil property (G), 0.5G, and 2G, respectively at which the free-field spectrum at the foundation level dips below the 60% spectrum (i.e., below Spectrum No. 3 in Figure 7-1). Let Spectrum No. 7 in Figure 7-2 be the enveloped spectra at the foundation level that results from the three different soil properties. If, according to the SRP guidelines, this enveloped spectrum is used as input for the SSI analysis, the "sixty-percent" requirement is considered satisfied, because at every frequency point, the input motion is more than 60% of the grade level motion.

Let a structure on this soil site have three dominant modes with frequencies f_1, f_2, and f_3. Then, if the SSI analysis uses, as input, a motion compatible with the enveloping Spectrum No. 7, structural response will be based on acceleration responses corresponding to frequencies f_1, f_2, and f_3 of the enveloping spectrum, i.e., accelerations A_1, A_2, and A_3, respectively. But, if three separate SSI analyses are performed for the three soil cases, the structural response will be based on accelerations a_1, a_2, and a_3. Thus, even though the envelope of the three foundation level spectra results in a spectrum that exceeds the 60% spectrum, unless this enveloped spectrum is actually used in the SSI analysis, or unless the input motion used for the individual soil property cases are perturbed to satisfy the "sixty-percent" requirement, the in-structure responses from the SSI analysis will be underestimated. Since the applicant performed neither, it is concluded that the "sixty-percent" SRP requirement has not been satisfied, and the in-structure responses (both response spectra and member forces and moments) from the SSI analyses may have been underestimated. Even though the 3-D fixed base case dominates structural response at many locations and for many components, unless the effects of the above noncompliance is ascertained quantitatively, it is not possible to evaluate if the applicant's design basis structural responses (at all frequencies of interest) are conservative (when compared to those from SSI cases where the sixty percent SRP requirement has been complied with).

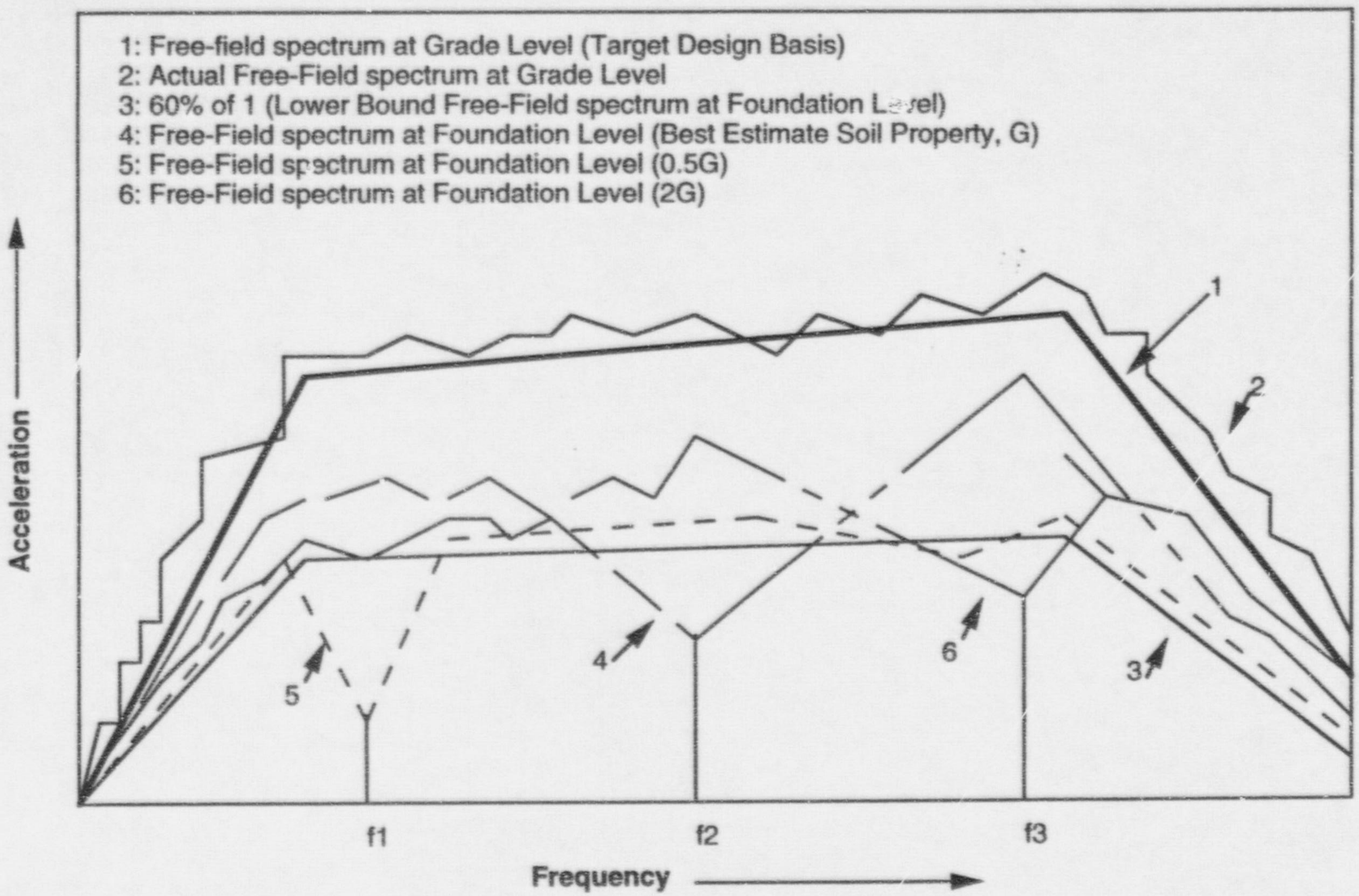

Figure 7-1 Example of Free-Field Spectra Comparison at Foundation Level

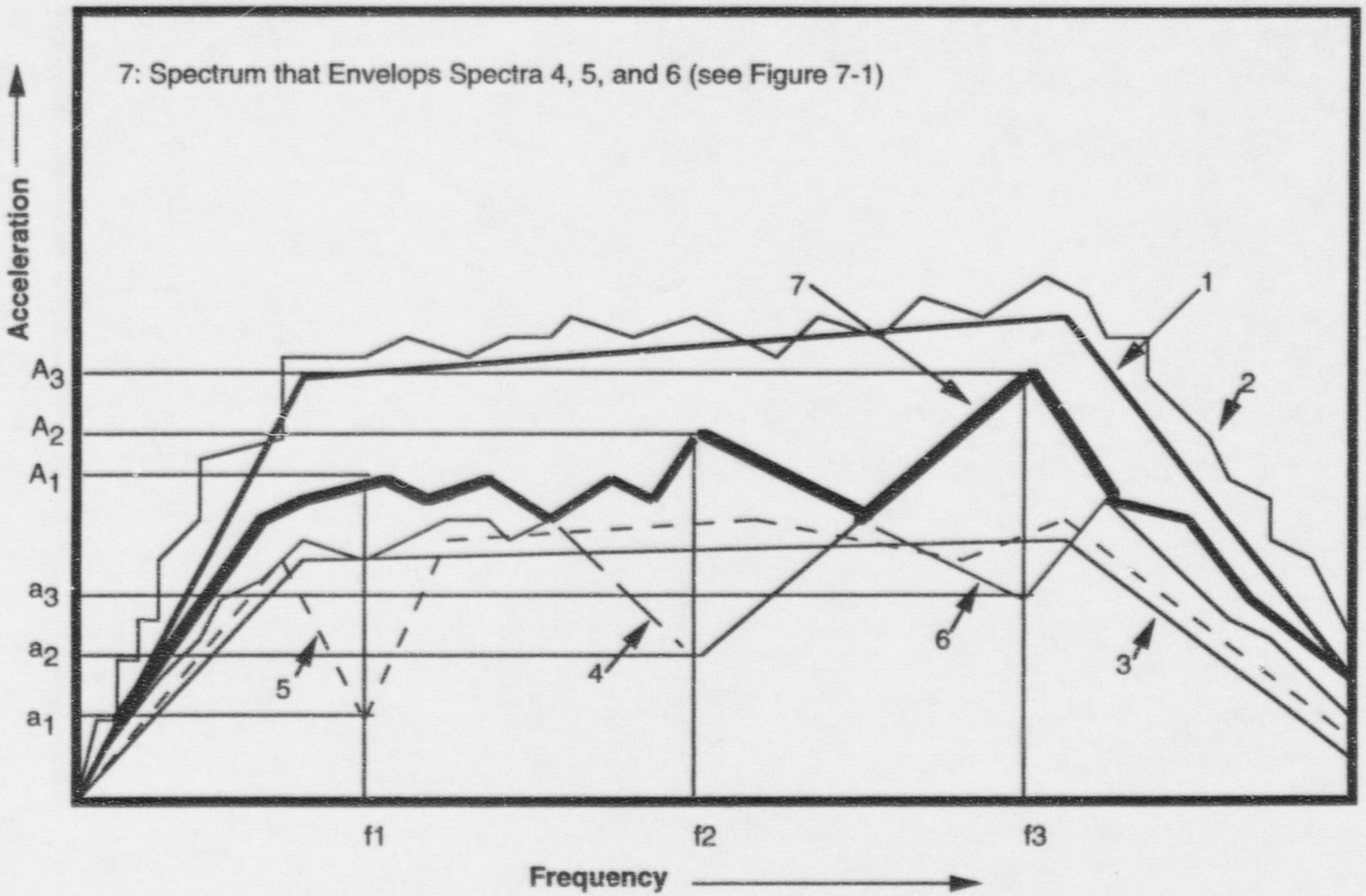

Figure 7-2 Example Use of Enveloped Spectrum

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8.0 Task 6: Evaluation of 3-D Design Basis Cases Considered

In Task 1 (see Section 3.0), the adequacy of the ranges of soil properties considered by the applicant was evaluated by performing 2-D SASSI analysis for the cases listed in Table 3-1. It was concluded from these analysis cases that the enveloping spectra generated from applicant's analysis cases reported in Revision 2 of AP600 SSAR do not envelop those from the soft rock cases (i.e. Case H) listed in Table 3-1. To confirm this finding, this soft rock case was reanalyzed using a SASSI 3-D model.

This 3-D SASSI analysis has been described in detail in Appendix E. This analysis used a Nuclear Island structural model generated by the applicant which, even though very similar, is not identical to the structural model used in Table 3-1 analysis cases; however, the model is identical to the later design-basis analytical model that the applicant used in Revision 9 of AP600 SSAR.

In-structure response spectra at few representative locations that resulted from this 3-D SASSI analysis case were compared with the corresponding design basis enveloping spectra from applicants 3-D SASSI and fixed-base analysis cases as reported in Revision 2 of AP600s SAR. The comparison confirmed the findings of the 2-D SASSI analysis case H (see Table 3-1), i.e., the in-structure response at some locations from the 3-D SASSI analysis of soil case H exceeds, at certain frequencies, the responses from applicant's earlier design basis enveloping spectra (i.e., spectra provided in Revision 2 of AP600 SSAR). As such, it was concluded that the soil property range covered in applicant's 3-D SSI analysis cases is inadequate without the soft rock case H. [Subsequently, the applicant updated AP600 SAR incorporating one such case in SAR Revision 9].

9.0 Task 7: Evaluation of Lateral Earth Pressures on Below-Grade Exterior Walls

The lateral earth pressures on below-grade exterior walls of the Nuclear Island was evaluated from the following four considerations (see also Appendix B):

a) The effect of mesh size (representing the side soil) on the lateral earth pressure determined by the 3-D SASSI analyses.

b) The effect of lateral pressure from rock on the wall design.

c) The effect of ground water on the lateral earth pressure during a seismic event.

d) The effect of adjacent buildings (Turbine Buildings, Radwaste Building, and Annex Buildings) on the lateral earth pressure.

To determine the effect of mesh size on lateral pressure, the Staff Review Team performed three 2-D parametric analyses as described in Section 6.0 by varying the sizes of the elements that represent the side soil. A comparison of the results of these three analyses showed that the coarse representation of the side soil that uses 3 layers is inadequate, and a finer representation using 8 layers in necessary. If, however, from cost considerations, 3-D SASSI analysis cases use only 3 soil layers (as was done by the applicant), it would be preferable to compute the lateral design pressure from the 2-D SASSI analyses that use 8 soil layers. But, in such cases, the additional loads (i.e. loads in addition to those predicted by the 2-D SASSI analysis) on exterior walls near the corners because of "Box Effects" must be considered.

The results of 2-D SSI cases for rock foundation, i.e., cases B, C, and H (see Table 3-1) were inspected to evaluate if, in determining the wall design loads, these cases should be considered. The inspection revealed that, at some locations of the exterior walls, rock cases may produce peak lateral pressures that are higher than soil cases. However, since these pressures are localized, their effect on the design is uncertain and geometry dependent. These pressures may or may not control design shear or moments, but, it was clear that rock cases should also be considered in determining the critical design basis lateral loads on walls.

Determining the effect of ground water on the lateral pressure is difficult because, SSI computer codes like SASSI, that are otherwise sophisticated, have some limitations to properly account for the two-phase interaction between solids and fluids. For example, SASSI solutions tend to be unstable for Poisson ratio values close to 0.5. Thus, instead of explicit or parametric determination of the ground water effect on the lateral wall pressure, the Staff Review Team studied the effect approximately by using a simple one dimensional constrained rod analysis assuming that wall pressure is influenced only by compressional waves moving through the solid and water fractions of the soil. Even though the results of this simplified study was not conclusive, it was judged that, if SASSI analysis model uses a Poisson's ratio of about 0.48, the resulting wall pressure is likely to be conservative, especially at or near the ground surface where computed pressures are the largest.

The effect of adjacent buildings on the lateral pressure was considered only qualitatively, and it was concluded that the applicant should evaluate structure-to-structure interaction through the soil to determine lateral pressures on exterior below-grade Nuclear Island walls.

10.0 Summary and Conclusions

A. In order to evaluate the adequacy of Westinghouse Electric Corporation's seismic soil structure Interaction analyses and determination of design basis seismic forces, moments, and floor response spectra for AP600 Nuclear Island Structures, the NRC Staff Review Team, consisting of NRC staff and its consultants, reviewed revision 2 of AP600 SSAR. This review raised concerns on seven technical issues (see Sections 1 and 2). To further assess and resolve these issues, the NRC Staff Review Team performed independent 2-D and 3-D SSI analyses using computer programs SASSI and CARES. The results of these analyses were studied to draw the following conclusions that are considered pertinent and significant for AP600 license review:

a) The ranges of soil and rock properties considered by the applicant in its 2-D parametric analysis cases and 3-D design basis cases are not adequate, and the SSI effects from a soft rock case (shear wave velocity equal to about 3500 fps) must be included in determining the design basis forces, moments, and floor response spectra (see also the discussion in Paragraph B of this section).

b) The SSI effects of a thin soil layer (less than 20 ft) between the basemat and the bedrock can be significant, and the ranges of site conditions considered by the applicant do not include such sites. As such, either such sites must be excluded from the license basis, or AP600 design basis must include such sites and recompute the design basis seismic forces, moments, and floor response spectra.

c) The use of only 3 layers of finite elements to represent the side soil may not be adequate for generating the floor response spectra for the 3-D SSI analysis cases. The applicant must demonstrate that the design basis floor response spectra, that were generated by enveloping the fixed-base case and the SSI cases that used only 3 layers of side soil elements, would not be exceeded even if the SSI cases had used more (say about 8) layers of side soil elements.

d) The applicant has not explicitly demonstrated that AP600 seismic design complies with the "Sixty Percent" SRP requirement on the free-field seismic motion at the basemat level (see also Paragraph C of this section).

e) The design basis seismic forces and moments for the below-grade walls must be determined either from the 2-D SSI analysis cases that use about 8 side soil elements and considers the 3-D "Box Effects" properly, or from 3-D SSI analysis cases that use about 8 side soil elements. Also, these design basis forces and moments should include the effects of localized peak pressures on the walls from the rock cases, and the effects of through-the-soil structure-to-structure interaction from the buildings adjacent to the Nuclear Island.

B. In response to conclusion "a" above, the applicant has since revised the AP600 seismic design basis and has included an additional soft rock 3-D SSI analysis case and has reported the results in revision 9 of AP600 SSAR. The revised enveloping floor spectra at a few representative locations were compared with those from the Staff Review Team's analysis. The comparison showed that the revision 9 SSAR spectra satisfactorily envelope the SSI effects of the soft rock case.

C. In response to conclusion "d" above, the applicant has since submitted the results of an additional evaluation in which it has been claimed/demonstrated that the free-field soil column frequencies (at which "dips" occur at the free-field basemat level spectra) are not close to the dominant structural frequencies of Nuclear Island, and so the likelihood of underestimating the structural responses due to the non-compliance of the "Sixty

Appendix A

Parametric Studies using 2-D SASSI Analyses

A.1. INTRODUCTION

In performing the seismic assessment of the AP600 nuclear island (NI), the Applicant used the approach of performing three dimensional (3-D) seismic response analyses of the NI for a few critical site conditions. These critical site conditions were, in turn, defined on the basis of the results obtained from a series of simpler two dimensional (2-D) calculations performed for a wider variation in assumed site conditions. The site conditions which led to the critical or bounding response conditions in the 2-D calculations were used to define the critical 3-D site calculational models. These 3-D models were then used for design evaluation of the structure and equipment of the NI, defined in terms of both response spectra at critical points of the NI and stresses in structural elements. To evaluate the adequacy of this design approach, a series of two dimensional (2-D) seismic response calculations were undertaken by the Staff Review Team, with the objective of assessing the completeness of the SSI (soil-structure interaction) calculations performed by the Applicant and the degree of conservatism inherent in their design calculations.

The 2-D structural model of the NI used in this evaluation was provided by the Applicant and consists of a rigid box embedded to a depth of 39.5' below the ground surface. The model is based on an average cross-section through the short direction of the NI (YZ direction of Figure A1) and has a width of approximately 127'. Three lumped mass structural sticks, representing the coupled shield and auxiliary building, the steel containment vessel and the containment internal structure, are attached to the buried box by means of stiff links as indicated in Figure A2. The seismic response calculations were performed using the SASSI (System for Analysis of Soil-Structure Interaction) Computer Code, as were those performed by the Applicant. The free-field seismic input motion used in all cases for these 2-D calculations is defined by a time history generated by the Applicant and labeled as the H2 motion. This time history envelopes the Reg. Guide 1.60 spectra scaled to 0.3 gs and is amplified somewhat in the frequency range from 9 hz to 25 hz to account for currently described Eastern United States(EUS) potential seismic motions. The time history provided for these evaluations is 20 seconds long, is digitized at 0.010 seconds and has a Nyquist frequency of 50 hz. The 5% damped response spectrum for this free-field motion is shown in Figure A3 along with the Reg. Guide 1.60 and AP600 amplified target spectra.

In general, the conclusions reached from these 2-D SASSI numerical studies have been based upon a comparison of the in-structure spectra developed at several key locations on the structural model from the various SSI cases investigated. The particular node locations at which much output was generated are Node 3016 (the top of the Shield Building), Node 3116 (the polar crane), Node 3204 (the operating deck), and Node 3110 (the polar crane support).

A2. SITE CONDITIONS EVALUATED

A number of SASSI calculations were performed by the Staff Review Team to determine the sensitivity of the computed responses to the various parameters of the assumed site conditions and to determine if enough cases were considered by the Applicant in determining their critical site conditions used for defining the more complete 3-D model evaluations. The site conditions considered in the Staff's evaluations are listed in Tables A1 and A2 and include variations in both soil and rock site conditions. The letters assigned to these site cases bear no relation to the site conditions assumed but are merely problem identifiers. The primary parameters considered in the evaluation consist of the depth of soil overburden to bedrock (which defines the depth of rock to the basemat of the NI), the stiffness of the soil overburden, the stiffness of bedrock, the format of the soil degradation models assumed to obtain degraded soil properties and the location of the free-field ground motion input with respect to the NI. A number of additional runs were conducted to investigate the influence of element sizes used in the SASSI computations on the accuracy of the computed responses.

The soil site conditions considered are presented in Table A1. Three of the site conditions (labeled Cases A, D and F) assume values of low strain shear wave velocity ranging from 1000 fps at the ground surface and increasing with depth to a value of 2400 fps at a depth of 240 feet below the ground surface. Such a soil profile was initially selected by the Applicant from studies of a number of existing nuclear plant sites and is labeled as "soft to medium stiff" soil. Several variations on their configuration were used in this evaluation, including the assumed depth to bedrock, the assumed form of the variation of shear wave velocity with depth, assumed soil degradation properties and soil density. Another site (Case G) uses a low strain shear wave velocity of 1500 fps held constant throughout the depth of soil, with no bedrock considered in this case. One site is labeled as a soft soil site (Case I) and has an assumed low strain shear wave velocity of 1000 fps throughout the soil overburden. The final soil case (Case J) has a low strain shear wave velocity taken from the data available to represent the Savannah River site located in South Carolina. This site is a deep soil site with a shear wave velocity which varies with depth in the range of 1200 fps to 1500 fps. The variation of the low strain or initial shear wave velocity is shown in Figure A4 for these soil cases.

In each of these soil cases, nonlinear soil effects were accounted for in the free-field convolution studies, leading to final or iterated values of shear wave velocity, which are then used as input to the SASSI calculations. These final velocities are somewhat lower than the low strain shear wave velocities. The soft soil cases (Case I) had final shear wave velocities as low as 500 fps. This site configuration clearly falls into the class of soft soil site as defined in the SRP and requires special consideration.

For the three rock cases (Cases B, C and H) considered, the NI was considered fully embedded within the rock halfspace, which extends to the ground surface. The purpose of these cases was to determine the influence of rock shear wave velocity on site response since the Applicant used only a single rock velocity of 8,000 fps in all its calculations. It should also be noted that a comparison of results from Cases B, C, H and G also allows for an evaluation of the adequacy of the fixed base evaluations performed by the Applicant.A plot of the shear wave velocity with depth for these cases in the upper 300' of the profile is shown in Figure A5.

A3. TYPICAL RESULTS

From the output generated from these various site conditions, horizontal response spectra were generated at the key output points selected(see Section A1, above) for the purpose of reaching at some allowing for general conclusions. Some typical results are presented in this report to illustrate these conclusions. For example, Figure A6 presents a comparison of spectra developed at the top of the Shield Building (Node 3016) for the cases with relatively deep layers of soft to medium stiff soil above bedrock, together with an envelope or bounding spectrum generated from the results of all the 2-D SASSI runs. This envelope spectrum results from the soft soil, soft to medium stiff soil and rock cases. The results indicate that Case A using a linear variation in shear velocity with depth and the original Seed-Idriss degradation properties, as originally postulated by the Applicant, almost never governs the soil cases. Using a parabolic variation with depth together with more modern approaches to soil degradation (Cases F) lead to higher responses in the frequency range from 2 hz to about 7 hz. In addition, using a deeper depth to bedrock of 240' again increases the peak spectral values. However, these soil cases govern (yield enveloping spectral values) in only a narrow frequency range.

Figure A7, on the other hand, presents the results of the soft thin soil layer above bedrock (Cases I), together with the envelope spectrum. Cases I1 (I1-40 and I1-45) use the H2 criteria motion input at the ground surface. Case I1-40 has the bedrock situated directly under the basemat of the NI, while I1-45 has the bedrock located 45' below the ground surface, or only 5' below the basemat. On the other hand, Cases I2 use the H2 motion input at the top of bedrock, as recommended by the SRP for soft, thin soil sites. Again, I2-40 and I2-45 situate the bedrock either directly under the basemat or only 5' below the basemat. As can be noted from Figure A7, the open dots associated with Cases I1 do not generally reach the envelope values. The NI essentially moves with the bedrock which is assumed to be attached directly to the basemat. The free-field ground motions at the depth of the bottom of the basemat are generally significantly lower than the surface criteria motions, as indicated in Figure A8. Some minor exceedances in free-field motions occur at the higher frequency range above about 6 hz.

However, defining the criteria ground motion as an outcrop motion at the top of bedrock has a major influence on response of the NI and leads to the largest values throughout the frequency range of interest. A comparison of the free-field motions at the level of the basemat, which tends to control plant response, is shown in Figure A8. The ground motions associated with the top of rock are significantly higher than the deconvolved free-field motions. In addition, placing a thin soft soil zone only 5' thick below the basemat (Cases I2-45 of Figure A7) produces an additional peak in the low frequency range (around 2 hz). Thus, the treatment of the soft thin soil case can dominate the plant response calculation and is considered a controlling configuration for plant design.

Figure A9 presents a similar comparison of the envelope spectrum with the results from the uniform site cases. As can be noted, the sites with shear wave velocities exceeding 3,500 fps produce essentially uniform response and can be considered as the lower bound of the definition of rock for the AP600 plant. Similarly, Figure A10 presents a comparison of the calculated envelope response with the original proposed design spectrum for this node as well as the results obtained from the fixed base calculations. As expected, in the high frequency range, the fixed base results exceed even the results from the hard rock cases, although they are significantly below the envelope and the design spectra in the lower frequency range.

A4. SENSITIVITY OF RESULTS TO MESH SIZES

In addition to the various site conditions, a number of variations in mesh sizes used in the finite element representation of the SASSI model were evaluated. In the 2-D model, the primary element discretization made use of 8 elements through the depth of the NI model, as indicated schematically in Figure A2, together with 26 elements along the length of the basemat. In addition, most of the 2-D SASSI models made use of soil elements beneath the concrete basemat and to the sides of the NI to determine the contact pressures developed between the soil and the NI. The question of sensitivity of the computed responses to element sizes became significant since it was known that the 3-D models which were to be used to compute design responses would use larger element sizes to keep the size of the 3-D element meshes within reasonable bounds.

Three additional calculations were run with the 2-D SASSI model using the same general site conditions (Case I1-40), with the criteria motion input at the foundation depth as an outcrop motion, with different soil layer discretizations to the side of the NI. One problem was run with only 3 soil layers provided (coarse layer mesh), one with 5 layers (median mesh) and the third using 8 soil layers (fine mesh) to the side of the NI. This configuration was evaluated since 3 soil layers were indicated to be used in the Applicant's 3-D SASSI calculations. Two of the meshes used are indicated schematically in Figure A11. The number of elements along the basemat was similarly coarsened for these runs. In addition, SASSI runs were made with elements under the basemat either included in the model or removed, with the NI then resting directly on the rock halfspace. It was found that this latter variation did not play a significant role on the computed responses.

However, it was found that the mesh size used for the side walls had a more significant impact on computed response of the NI. Figures A12 and A13 present typical effects of sidewall element number on response spectra. As can be seen, significant differences can occur, particularly at the peaks of the computed spectra. Such behavior indicates that the coarser mesh generates more effective damping leading to reduced peaks of the nodal responses. The 3 layer element discretization for the soft soil site is at the limit of the wave transmission guidance provided for the SASSI calculations at about 10 hz. Some additional responses are noted to be generated at the higher frequencies associated with the second mode response of the structural model. This may indicate that the coarser meshes are somewhat stiffer inducing response at the higher frequencies. In addition, the wall pressures computed from the coarser meshes are significantly influenced by element size effects.

A5. CONCLUSIONS

Based on a review of the numerical responses evaluated, several conclusions have been reached concerning the completeness of the design evaluation performed by the Applicant. It should be noted that these conclusions have been arrived at by comparing the computed response spectra at several locations on the stick models. The conclusions reached can be summarized as follows.

1. By comparing spectra at given locations developed from Cases B, C, H and G and comparing these to similar results obtained from the fixed base case, it was determined that for rock shear wave velocities of 3,500 fps and greater, the fixed base analysis produces conservative results. At frequencies above 10 hz, the fixed base analysis is very conservative since the SSI effects eliminate the second mode noted in some of the fixed base spectra. The spectra from the fixed base analysis fall below the SSI results at the lower

frequencies corresponding to the SSI modes of the system; that is, SSI causes spectral amplifications at the coupled SSI/structure frequencies.

2. The depth to bedrock was found to be a controlling parameter. For the soft to medium stiff soil case (Case A), the Applicant indicated in the SSAR that they considered two depths to bedrock in their 2-D evaluations, 120 feet and 240 feet, with the 120 foot depth found to be more critical. The results of the Staff Review Team evaluation indicates that the results at deeper depths may lead to increased, particularly when combined with more recent degradation properties and more realistic variations in shear wave velocities.

3. For the soft soil site (Case I), the final iterated strains lead to shear wave velocities as low as 500 fps. For the cases considered, with depths to bedrock of either 40 feet or 45 feet, such sites qualify as thin soft soil sites according to the SRP definitions. For such cases, the broad banded RG 1.60 type spectra are considered more appropriate as input as outcrop motions at the top of bedrock. Each of the two cases were then run with the H2 motions input at the ground surface and at the rock interface as an outcrop.

   For Case I1, with the depth to bedrock at the basemat level, the spectral responses computed for rock outcrop input were not significantly different from those computed from the rock cases, since the motion input at the foundation level for the rock cases is similar to that input in the rock outcrop case. The loadings applied by the soil to the sidewalls of the NI, however, can be expected to be significantly higher than those from Case I1 with the motion input at the ground surface, since the peak soil displacements throughout the soil layer above the bedrock are greater than the motions computed with the input at the ground surface. These soil loads can also be expected to be different from the results for the rock cases, although the peak values for all these cases have yet to be evaluated.

   For Case I2, however, the spectral responses were found to significantly exceed the spectra for the fixed base case as well as the rock cases at the horizontal SSI frequency associated with the thin layer of soil between the basemat and the bedrock. This exceedance will change with both frequency and magnitude as the layer of soil remaining below the basemat gets thicker. Therefore, the allowable depth to bedrock is an important parameter controlling response of the NI which is felt to have been incompletely evaluated by the Applicant.

4. Three calculations were run with SASSI using the same general site conditions (Case I1) with different soil layer discretizations to the side of the NI. The results of the calculations

indicate that the use of 3 soil layers for this soft soil condition leads to a reduction in the spectral peak (as compared to the 8 layer case) of about 25% at a frequency of near 10 hz and an increase in the computed spectra at higher frequencies. The adequacy of the 3D SASSI models to be used by the Applicant for the design basis computations should therefore be checked to ensure that they exceed the wave transmission guidance provided in the SASSI users manual. In addition, the computation of wall loadings from such 3D calculations are not recommended.

5. For Case K, which is the same site condition as Case A2, two computations were made using two different angles of incidence of the incoming S-wave, one of 0° (vertical incidence) and one of 30° from the vertical. It was found that the effects of angle of incidence of the incoming wave had negligible effects on computed spectra.

6. The evaluation of the effects of soil saturation on computed responses and on soil pressures acting on the walls of the NI have not been completely evaluated. The Staff Review Team SASSI computations were made assuming dry soil conditions. The Applicant's computations assumed pseudo-saturated conditions, that is, the P-wave velocity of the soil was set equal to 5,000 fps (approximately that of water) while the shear wave velocity and soil unit weights were kept at their dry values. Additional 2D computations made by the Applicant using an increased value of soil unit weight (from 120 pcf to 135 pcf) to account for typical saturated soil effects showed negligible effects on computed responses.

7. An additional SASSI calculation was performed by the Staff Review Team for the soft to medium stiff soil site (Case A) but using an enhanced H2 free-field ground motion for input to the site at the ground surface. In this evaluation, a typical soil variation in the free-field deconvolution calculation was performed, varying the shear moduli from 1/2 to 2 times their best estimate values defined by the Case A configuration. The free-field spectrum at the foundation level of 40 feet was then computed for each case and found to fall below the 60% criteria indicated in the SRP over a frequency range of from about 1.5 hz to about 10 hz. The foundation level spectrum in this frequency range was then amplified and convolved to the surface using the best estimate soil column to arrive at an enhanced surface H2 motion which presumably would satisfy the intent of the SRP guidance. This enhanced motion was then used in the SASSI computation to arrive at response spectra which show significant exceedances over the corresponding fixed base spectra typically used for comparison purposes.

8. Combining the effects of parabolic variations in shear wave velocity with more modern

degradation models leads to somewhat greater responses than those soil cases investigated by the Applicant.

TABLE A.1
SUMMARY OF 2-D SASSI ANALYSIS CASES FOR SOIL SITES (ROCK VELOCITY = 8000 FPS)

CASE	SOIL TYPE	ASSUMED LOW STRAIN SHEAR WAVE VELOCITY	SUBCASE	DESCRIPTOR	DEPTH TO ROCK (feet)
A	Soft to	1000 fps to 2400 fps @ 240'	A1	dry soil	80
	Medium	varying linearly with depth with	A2	dry soil	120
	Stiff	Seed-Idriss degradation properties	A3	wet	120
			A4	saturated	120
D	Soft to	1000 fps to 2400 fps @ 240'		dry soil	240
	Medium	varying parabolically with depth with			
	Stiff	Seed-Idriss degradation properties			
F	Soft to	1000 fps to 2400 fps @ 240'		dry soil	240
	Medium	varying parabolically with depth with			
	Stiff	lower bound degradation properties			
G	Uniform	1500 fps uniform soil site with		dry soil	none
	Site	Seed-Idriss degradation properties			
I	Soft	1000 fps uniform soil site with	I1	dry soil	40
	Site	Seed-Idriss degradation properties	I2	dry soil	45
J	Specific	Savannah River deep soil site with		dry soil	none
	Deep	variable velocity profile and lower			
	Site	bound degradation properties			
K	Soft to	Same as Case A:			
	Medium	Vertical Wave Incidence	K1	dry soil	120
	Stiff	30° Wave Incidence	K2	dry soil	120

TABLE A.2
SUMMARY OF 2-D SASSI PROBLEMS FOR ROCK SITES

CASE	ASSUMED LOW STRAIN ROCK SHEAR WAVE VELOCITY (fps)
B	11,000
C	8,000
H	3,500

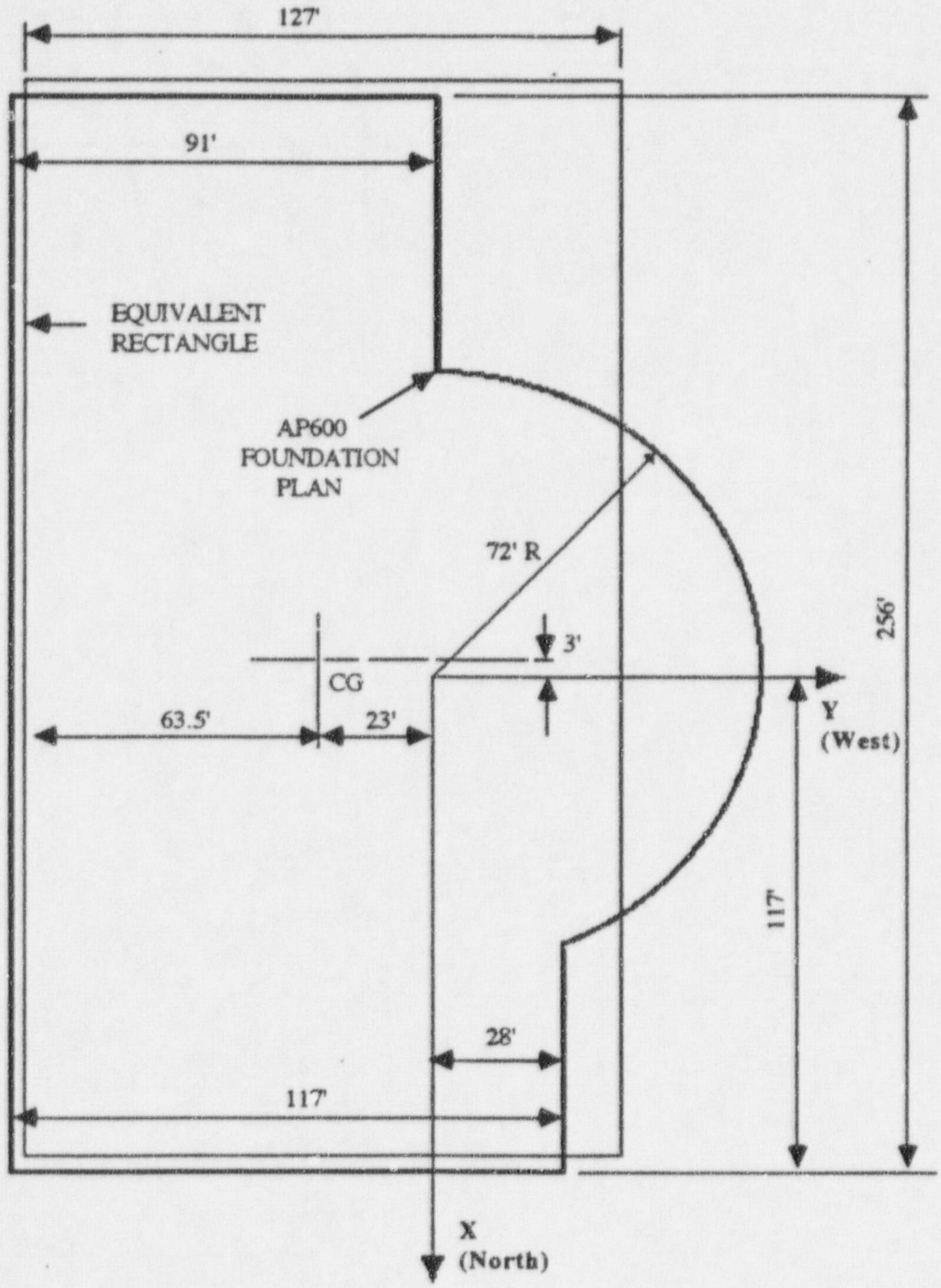

FIGURE A1 AP600 FOUNDATION PLAN

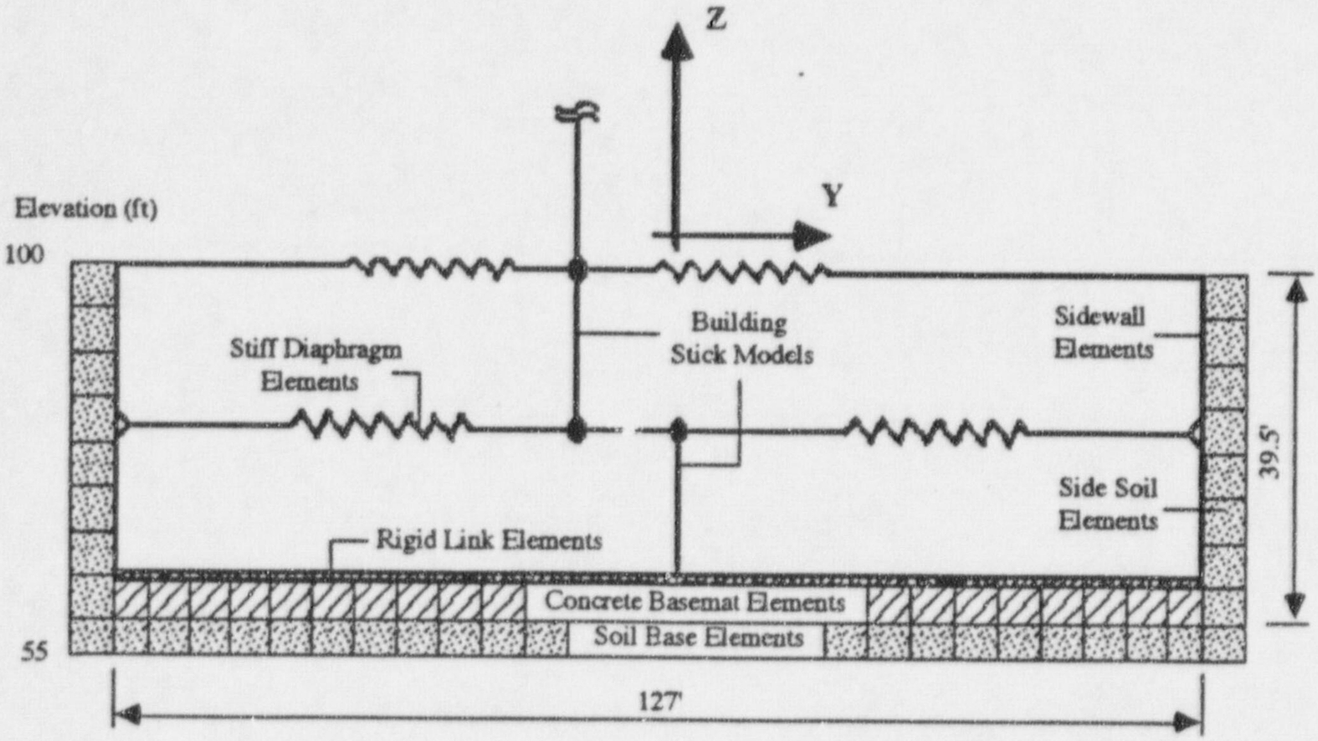

FIGURE A2 SASSI NUCLEAR ISLAND MODEL
IN THE YZ PLANE

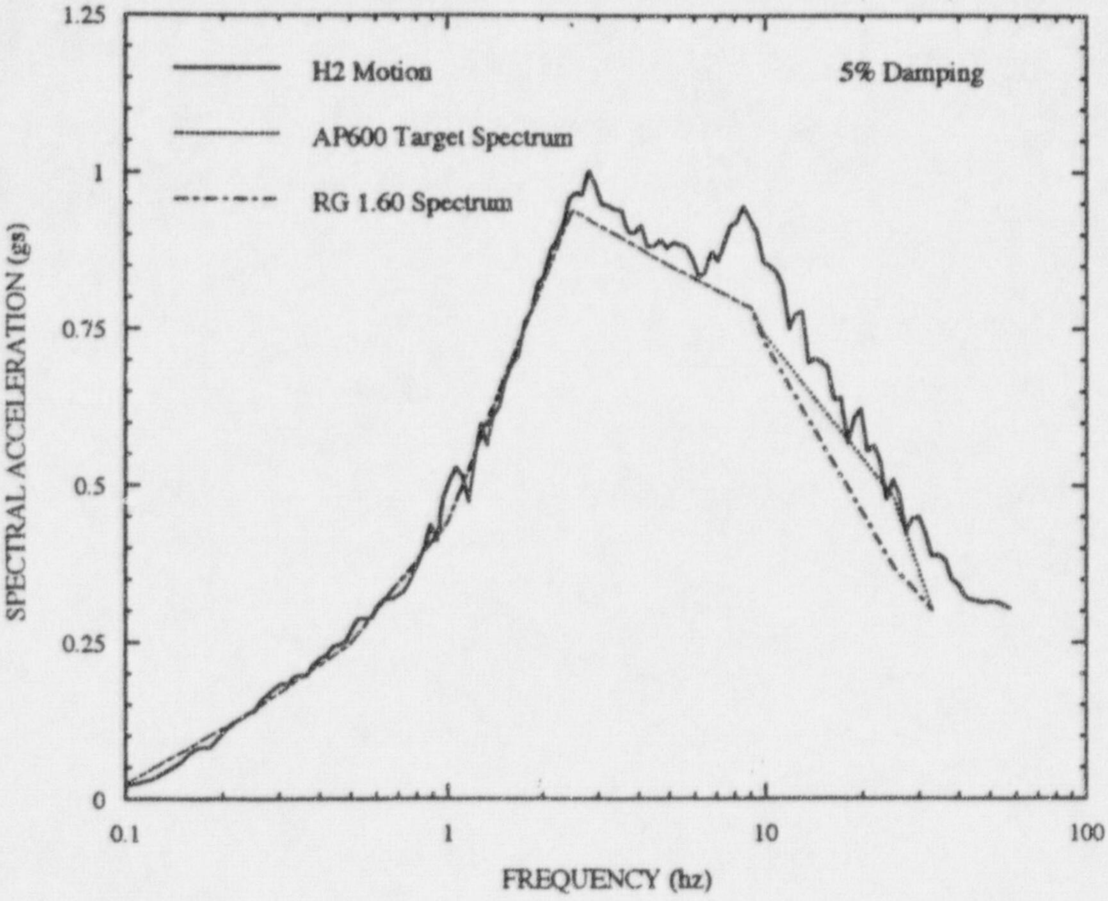

FIGURE A3 COMPARISON OF H2 MOTION SPECTRUM WITH TARGET AND RG 1.60 SPECTRA

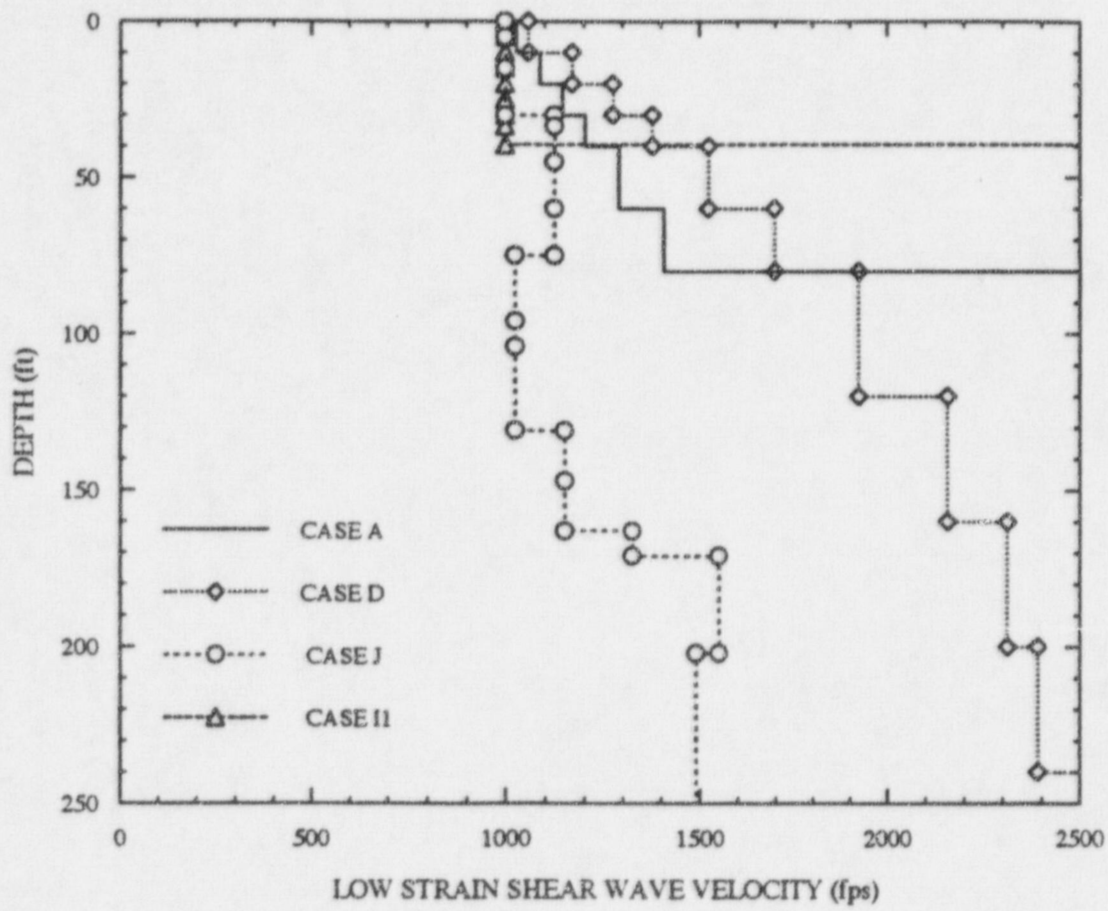

FIGURE A4 VARIATIONS IN LOW STRAIN SHEAR WAVE VELOCITY FOR SOIL SITE MODELS

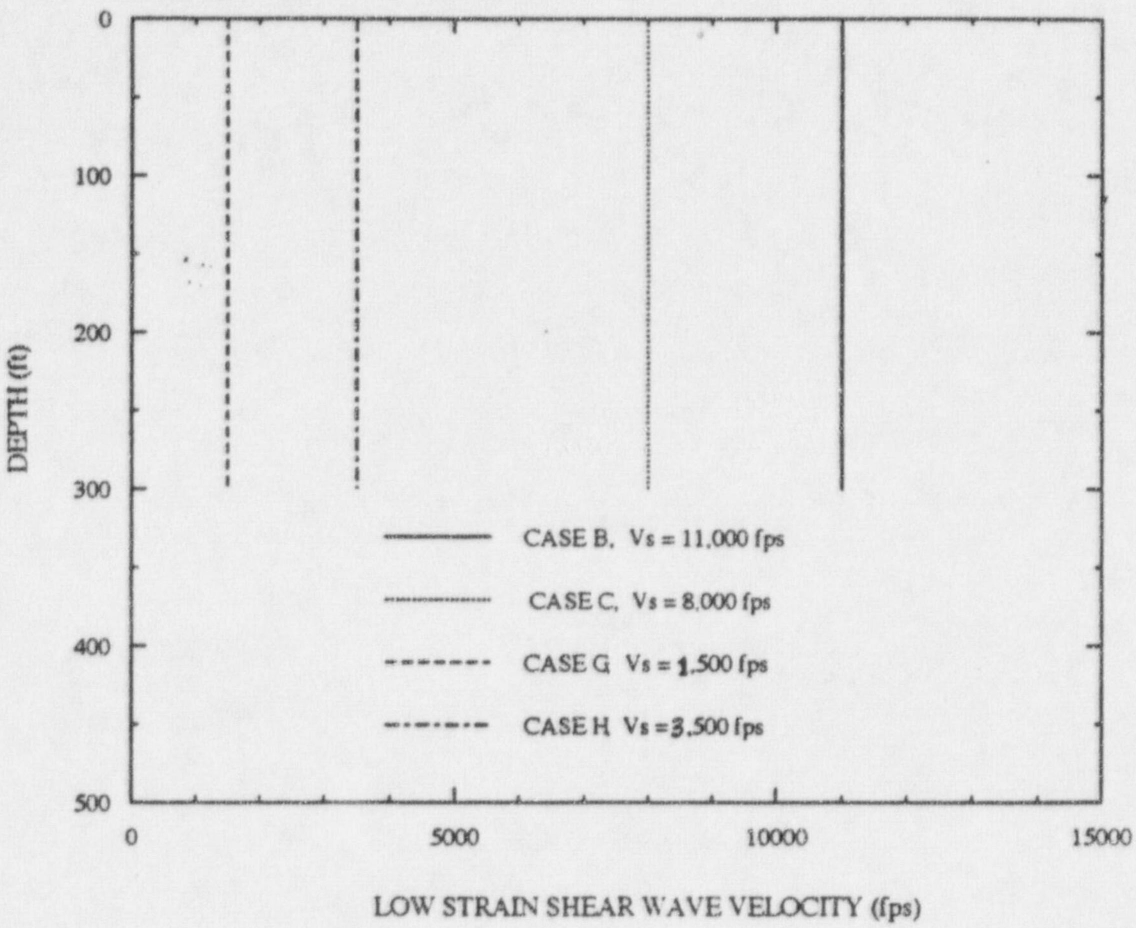

FIGURE A5 VARIATIONS IN LOW STRAIN SHEAR WAVE VELOCITY FOR ROCK SITE MODELS

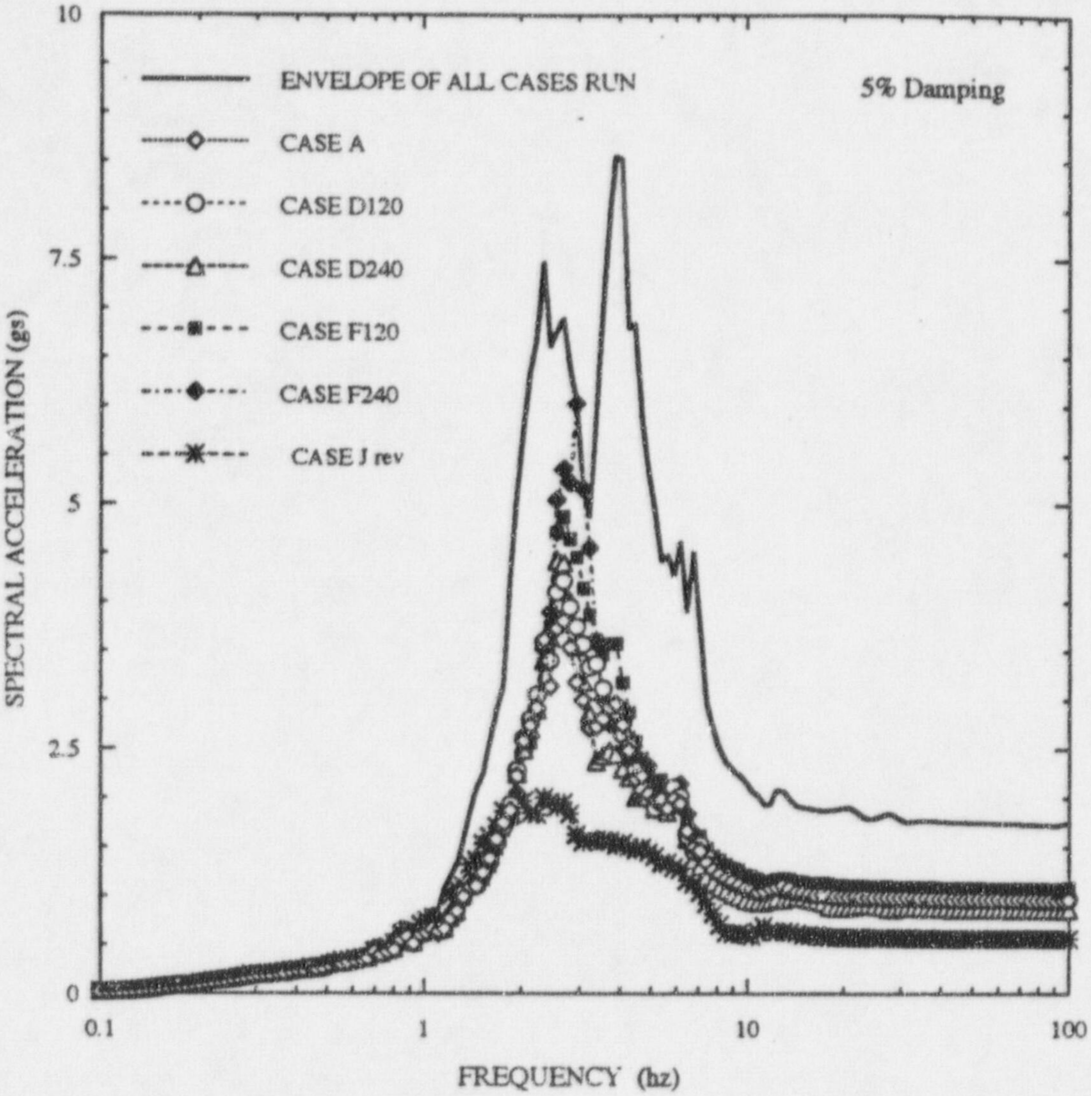

FIGURE A6 RESPONSE SPECTRA AT NODE 3016, SHIELD BUILDING ROOF, AT DEEPER SOIL SITES

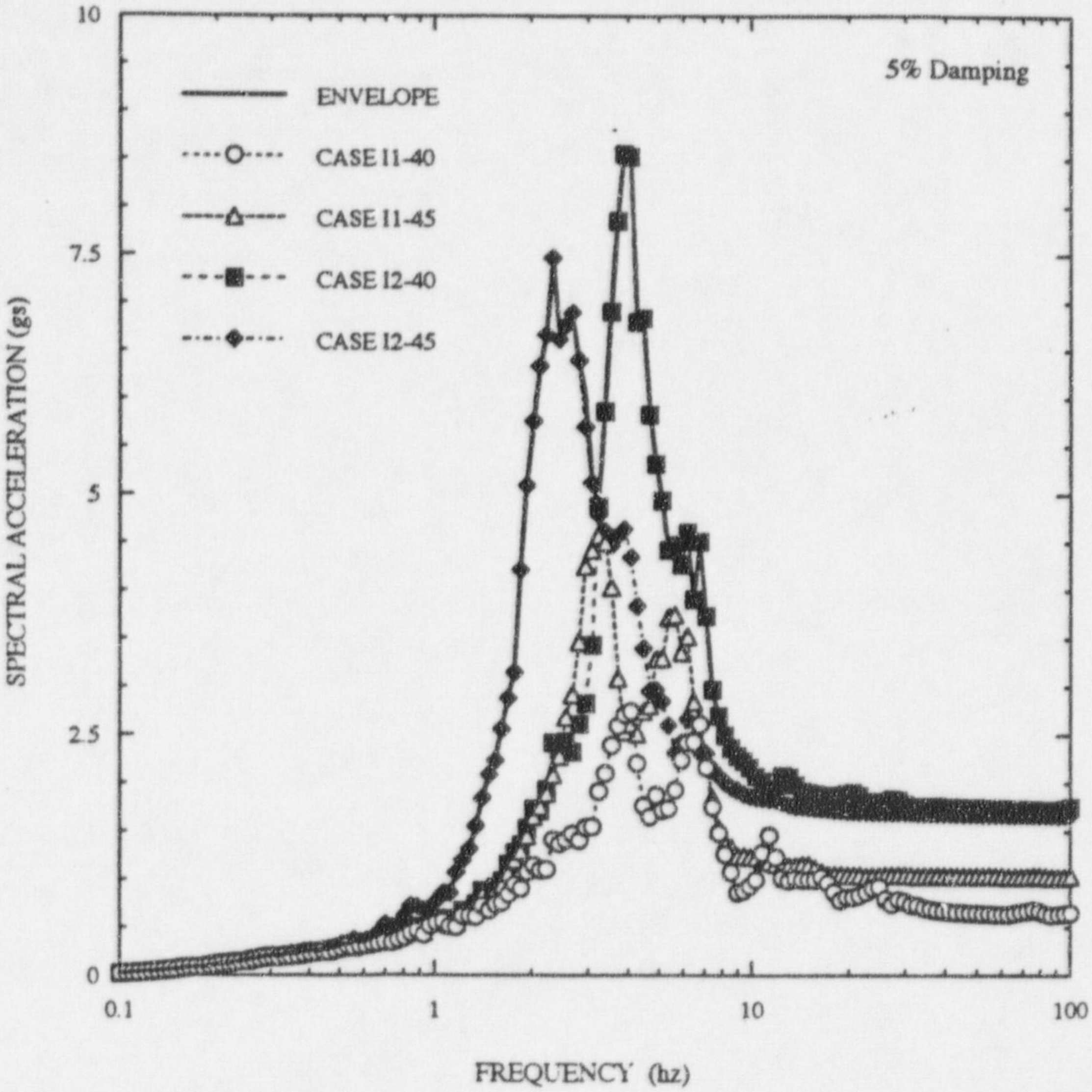

FIGURE A7 RESPONSE SPECTRA AT NODE 3016, SHIELD BUILDING ROOF, AT SHALLOW SOIL SITES

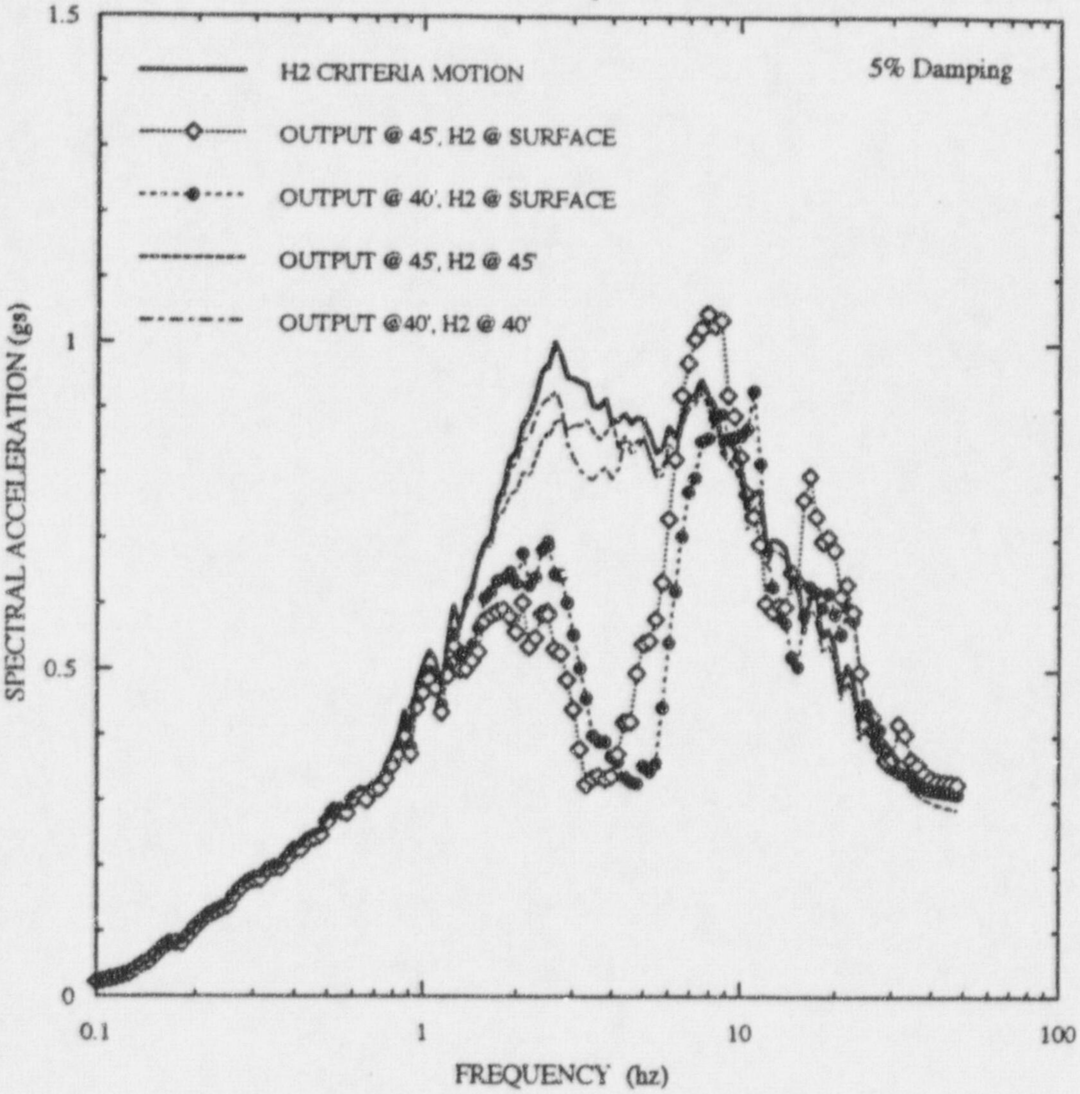

FIGURE A8 FREE-FIELD MOTIONS FOR SOFT THIN SOIL LAYER ATOP BEDROCK

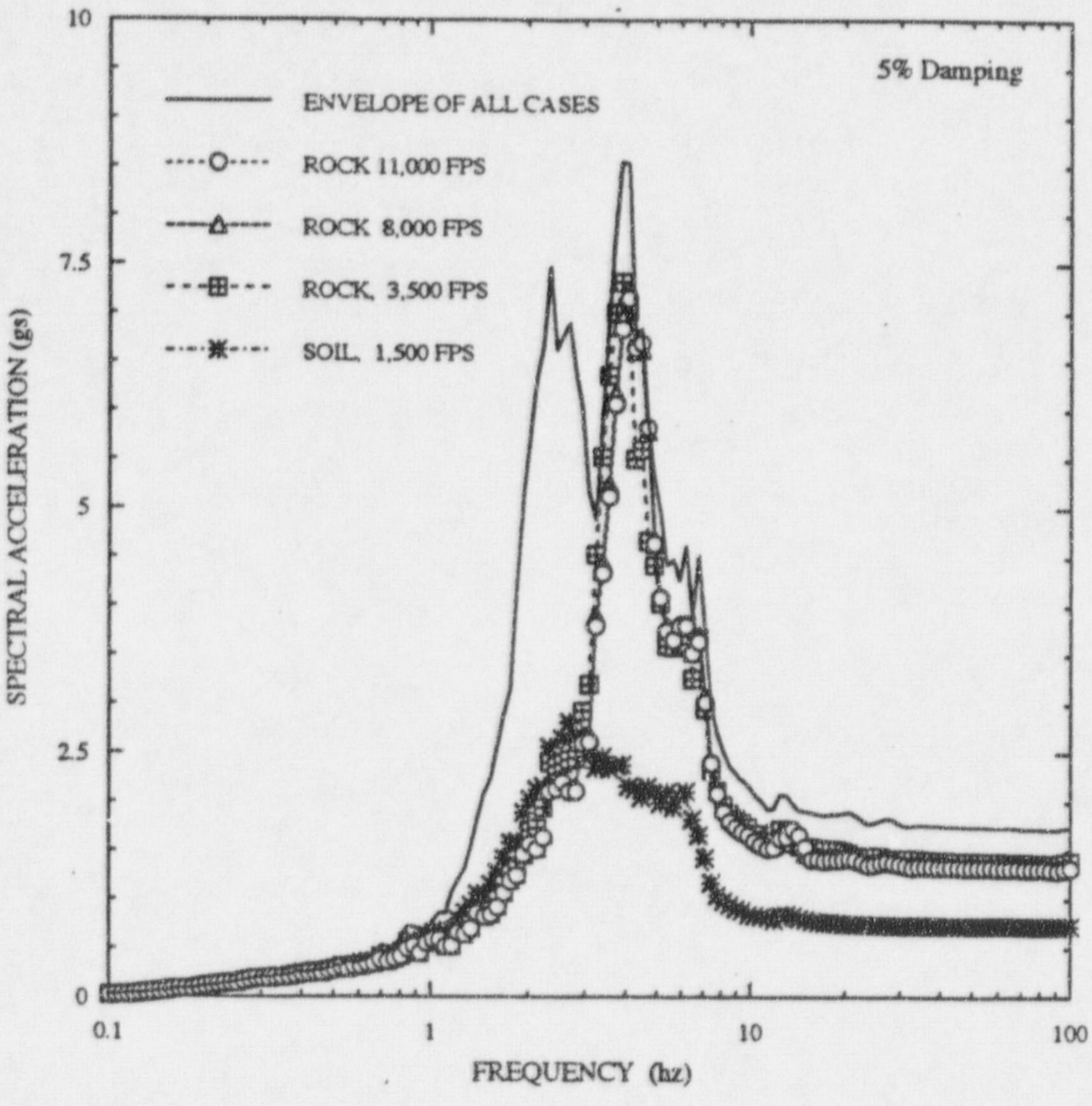

FIGURE A9 RESPONSE SPECTRA AT NODE 3016, SHIELD BUILDING ROOF, AT UNIFORM SITES

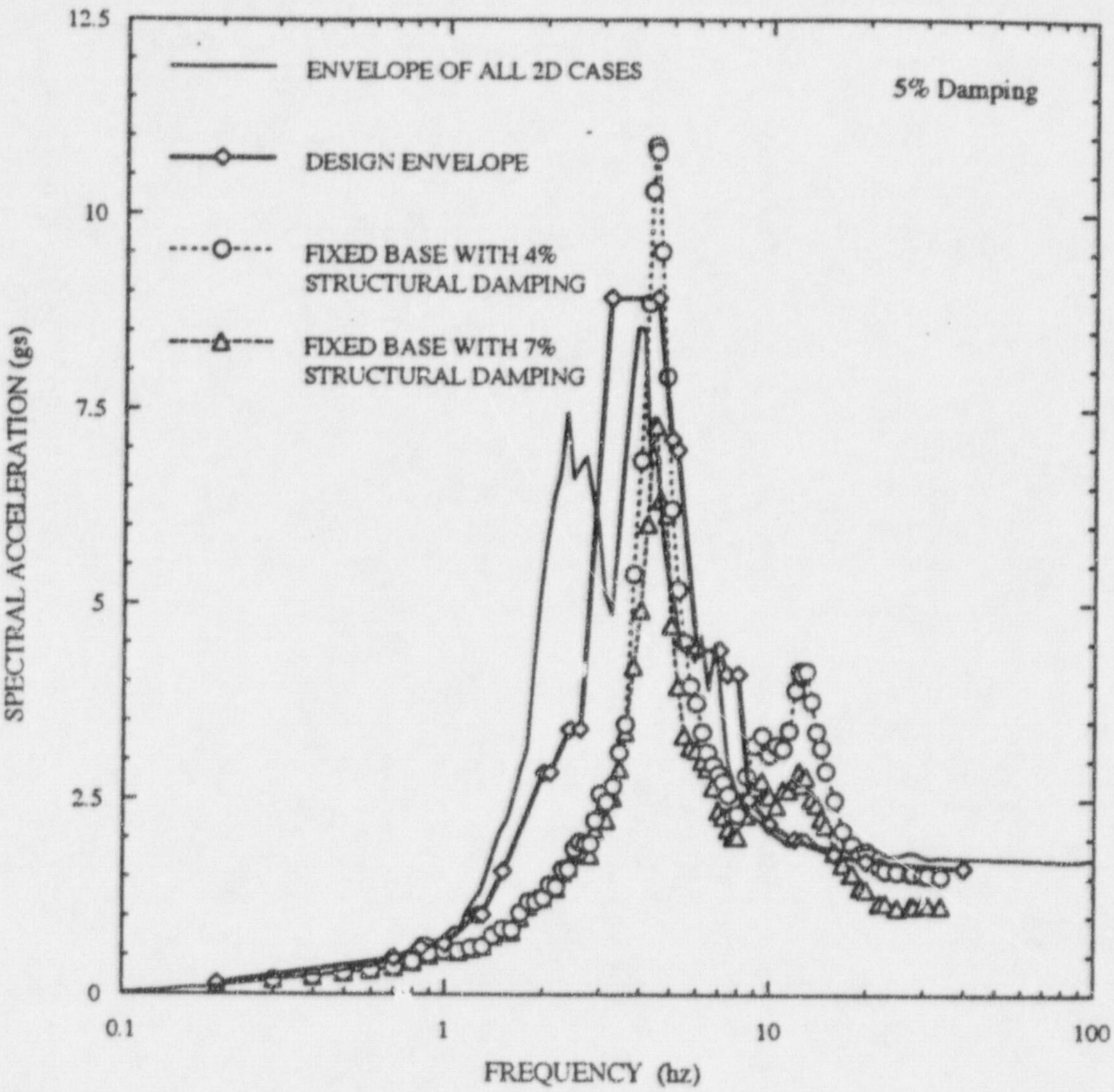

FIGURE A10 RESPONSE SPECTRA AT NODE 3016, SHIELD BUILDING ROOF, COMPARISON WITH DESIGN SPECTRUM

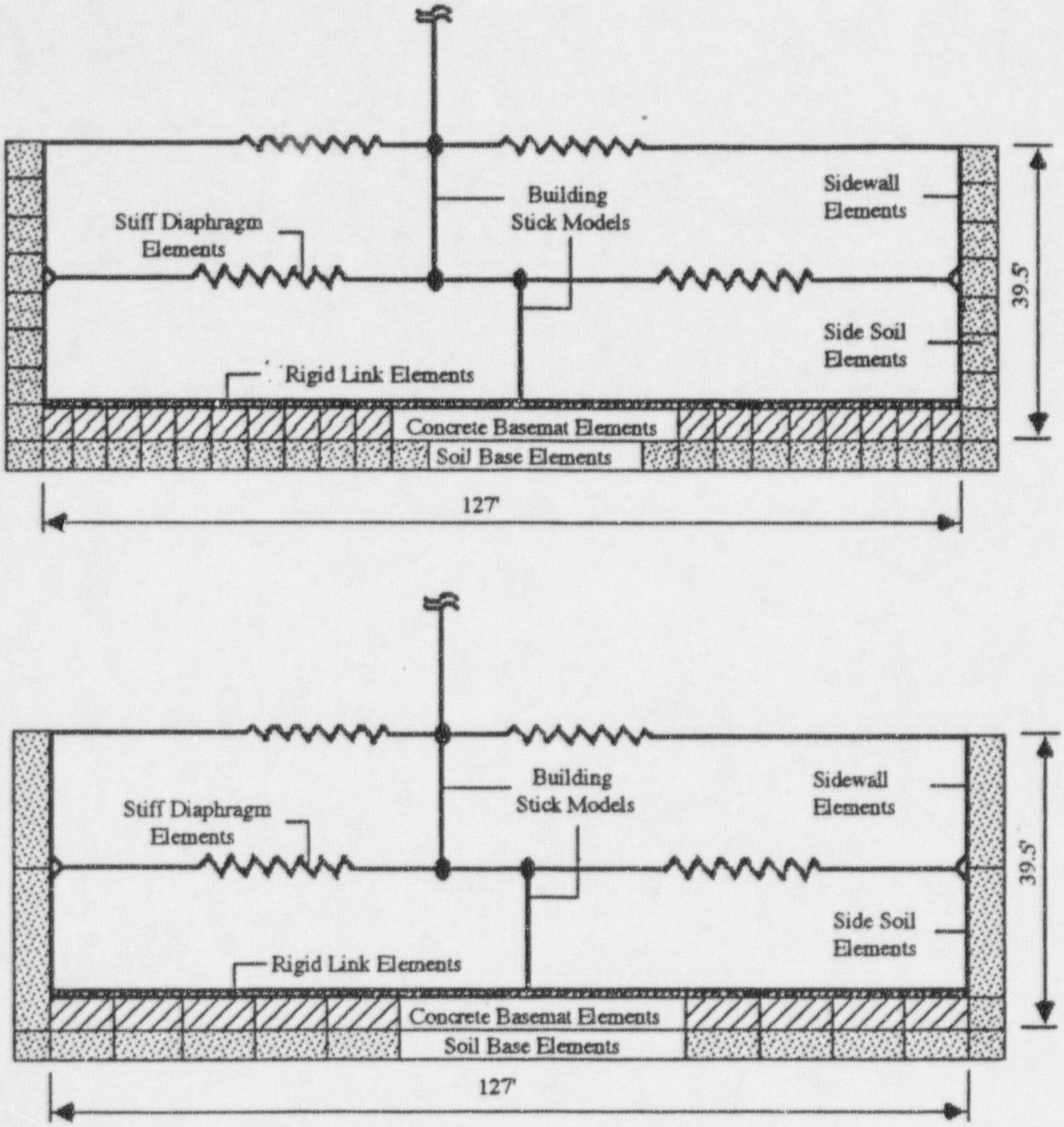

FIGURE A11 VARIATIONS IN SASSI FINITE ELEMENT DISCRETIZATION

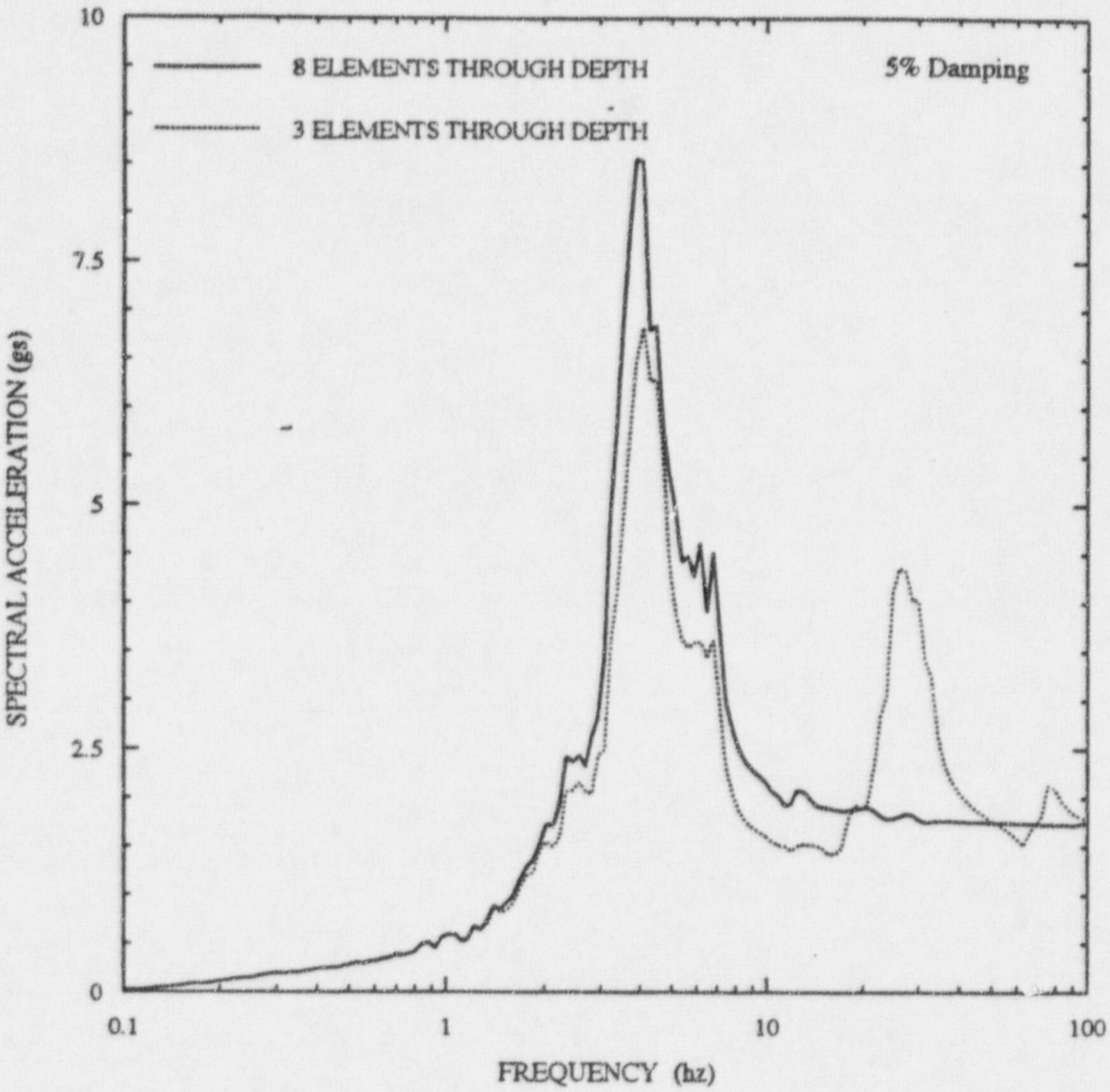

FIGURE A12 RESPONSE SPECTRA AT NODE 3016, SHIELD BUILDING ROOF, FOR DIFFERENT ELEMENT SIZES THROUGH DEPTH OF NUCLEAR ISLAND

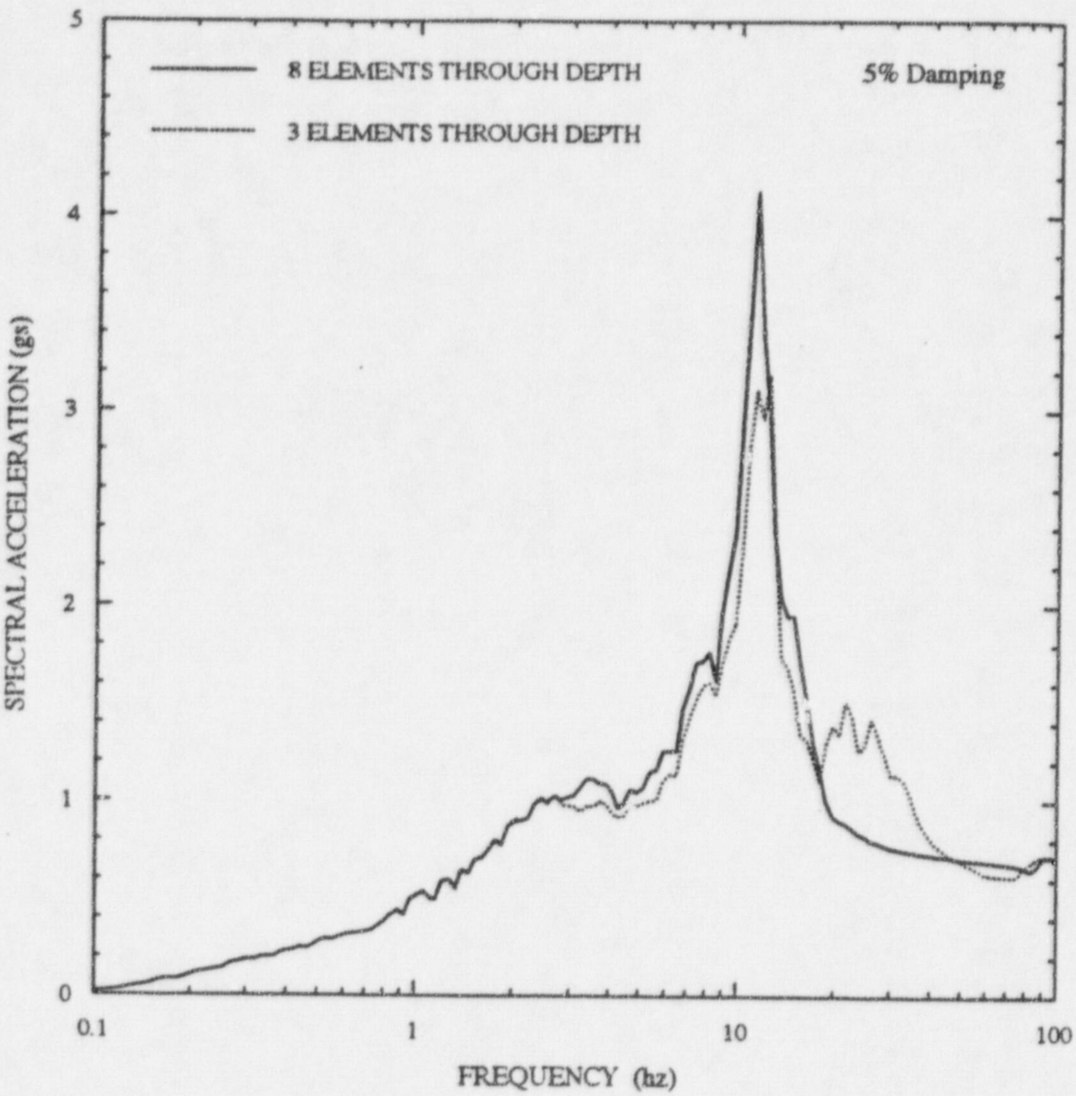

FIGURE A13 RESPONSE SPECTRA AT NODE 3204, OPERATING DECK, FOR DIFFERENT ELEMENT SIZES THROUGH DEPTH OF NUCLEAR ISLAND

Appendix B

Evaluation of Soil Pressure on Below Ground Walls

B1. INTRODUCTION

As part of the seismic assessment of the proposed AP600 advanced reactor system, investigations were undertaken by the Applicant to estimate the dynamic wall pressure that may develop along sidewalls of the nuclear island (NI) from seismic motions of the plant and surrounding soil/rock. Initial presentations to the Staff Review Team had proposed the use of the Mononobe-Okabe (MO) method to estimate both magnitude and distribution of wall pressures. The Staff Review Team disagreed with this approach since the method is formally appropriate only for retaining wall structures, could not be shown to be a conservative estimate of wall pressures and could not easily incorporate the effects of structures adjacent to the NI on dynamic wall pressures. In addition, the current Staff position clearly outlines the problems inherent with the MO method and indicates that the procedure to estimate soil pressures must be able to incorporate the potential effects of relative dynamic displacement of the NI with respect to side soils as well as the soils below the basemat, including potential rocking and rotation of the NI.

Following these discussions, it was generally agreed that dynamic pressures can best be handled within the framework of the SASSI formulation since these were to be performed anyway to evaluate seismic response of internal structures and systems within the NI. The major drawback to this formulation was the anticipated adequacy of the SASSI computations for the soil pressure computations. As discussed in Appendix A, it was determined from the many 2D SASSI models performed by both the Applicant and the Staff Review team that relatively fine finite element meshes are required to obtain reasonable accuracy in computation of dynamic wall pressures. For example, it was found from the 2-D SASSI computations that about 8 elements were required along the walls of the NI to achieve a reasonably accurate stress computation. Using such a discretization requirement in the 3-D SASSI models would lead to extremely large 3-D computational models that had to be used for the design computations. It was therefore agreed that an alternate approach would make use of the 2-D SASSI stress computations which could then be modified in a simplified manner to incorporate corrections for potential torsional motions of the NI (anticipated to be small) as well as potential corner effects. In the 2-D SASSI models, the effects of the light structures adjacent to the NI could be included to estimate any additional pressures that may be induced by differential seismic motions of the NI and the adjacent structures. In this Appendix, we summarize the results of the 2-D SASSI computations conducted by the Staff Review Team. These were done to determine the influence of the additional site conditions evaluated on the computed wall loads and to provide confirmation of the Applicant's estimates of wall pressures to ensure that seismic designs of the NI are appropriate.

In addition, an area of concern in developing these wall loading conditions has to do with the effect of ground water on induced wall pressures. The plant design condition assumes that the ground water table is at or near the operating deck level, or nominal elevation 100. The only impact of this assumption on the calculation of dynamic wall pressures is in the selection of the value of Poisson's ratio of the surrounding elastic soil material. In this approach estimates of horizontal shear wave velocity for the various site conditions are combined with an assumed P-wave velocity of approximately 5,000 fps for the soil which matches the wave velocity through water. This leads to a value of Poisson's ratio which is relatively high (approaching a value of 0.5). However, as Poisson's ratio approaches 0.5, the SASSI computation (as well as most other codes) becomes unstable since it cannot treat this incompressibility condition. attempts were made to address the issue of the inclusion of pore water effects on wall loads and if in fact the elastic SASSI type solutions provide reasonable estimates. Values of Poisson's ratio typically utilized in the SASSI runs varied from 0.45 to 0.48. The question of the adequacy of this assumption should be evaluated. Since no complete computational solution similar to SASSI is currently available to treat the water/soil system as coupled but separate systems, the results of other computations were used to try to evaluate the adequacy of the SASSI approach. These results are also described in this Appendix.

B2. SITE CONDITIONS EVALUATED

A number of SASSI calculations were performed by the Staff Review Team to determine the sensitivity of the computed responses to the various parameters of the assumed site conditions and these have been described in Appendix A. The site conditions considered in the Staff's evaluations are listed in Tables A1 and A2 and consider variations in both soil and rock site conditions. The soil site conditions considered are presented in Table A1 while the rock site assumptions are listed in Table A2. The soft thin soil cases (Case I1 and I2) fall into the class of soft soil site as defined in the SRP and requires special consideration. For the three rock cases (Cases B, C and H) considered, the NI was considered fully embedded within the rock halfspace, which extends to the ground surface. The purpose of these cases was to determine the influence of rock shear wave velocity on site response since the Applicant used only a single rock velocity of 8,000 fps in all its calculations. It should also be noted that a comparison of results from Cases B, C, H and G also allows for an evaluation of the influence of rock modulus on induced dynamic wall pressures.

B3. TYPICAL RESULTS OF 2-D SASSI ANALYSES

From the output generated from these various site conditions, the pressures generated along the walls were generated for several of the site cases. A special purpose postprocessor was written which converts the stresses generated by SASSI in the soil elements adjacent to the NI to obtain peak pressures applied to the wall with depth as well as the total forces generated along the wall with time. From this computation, the maximum force was determined as well as the pressure distribution along the wall at the time of the peak total horizontal force. Figure B1 presents a plot of the peak computed tensile dynamic pressures (in ksf) generated along the wall for the various rock cases while Figure B2 presents the same data for the maximum compressive dynamic pressures. These pressures do not occur at the same instant of time, but are the maximum at the particular levels shown. For these runs, eight wall elements were used to compute pressures which were assumed to be uniformly distributed over the width of the element. In addition, the maximum induced pressures generated from all the runs (both soil and rock) is also plotted in the figures. It can be noted that the maximum pressures developed along the wall are developed from the hard rock case, Case B, which has a shear wave velocity of 11,000 fps. As the shear wave velocity decreases, the pressures on the wall similarly decrease.Maximum dynamic pressures are always induced at the top of the wall indicating the participation of the walls in resisting rocking of the NI. The values of pressure for the hard rock case are similar in magnitude to the values generated by the Applicant.

It should be noted that the static at-rest pressures are also plotted on these figures and it is clear that on the tensile side, the dynamic pressures exceed the static over a major part of the wall, indicating the tendency for separation of the NI from the surrounding rock. If this is in fact true, the tensile forces developed by SASSI cannot be sustained, and either the dynamic loads are transferred to the compressive side, increasing their value or are transferred through other parts of the NI wall/basemat system to the surrounding rock. This issue has not been previously discussed and should be evaluated by the Applicant.

The results from the soft soil cases (Cases I1 and I2) are shown in Figure B3. Again, the peak pressures occur at the top of the wall. It is clear that the induced wall pressures are generally lower than those developed by the stiff rock case, although for the cases with the criteria motion input at bedrock, the induced pressures are significantly higher than the results from the other soil cases. In all cases, however, the dynamic pressures again exceed the at-rest condition, indicating that some load transfer must occur.

It should be noted that the results from the other 2D runs using a coarser finite element mesh through the wall lead to significantly different wall pressure distributions. This is obviously the case as the variation in wall pressures is generally so sharp at the top of the wall. Using a larger element would then smear this distribution, leading to potentially incorrect wall design loads. This lends further support for using the process of modifying the 2-D SASSI results rather than using the results from the coarser meshes of the 3D models.

B4. INFLUENCE OF PORE WATER ON SEISMIC RESPONSE

The effect of pore water on seismic response of the AP600 or any other plant plant is of interest from two different perspectives. The considerations for AP600 were of interest since one of the design conditions was that the ground water table was at or near the ground surface, with the plant exterior walls then being entirely below the ground water table. The implications of this assumption on seismic response can be summarized as follows.

The first issue has to do with the effect of saturation on the dynamic seismic response of structures. For most ordinary buildings, soil-structure interaction (SSI) has not been an important issue in evaluating dynamic response and structural response can be adequately determined by fixed base analyses. However, for those structures where SSI is important (such as the NI structure), previous studies have been performed for NRC (Ref. B1 and B2) to determine the effect of pore water on SSI interaction coefficients. A special finite element computer code (POROSLAM) was developed which performs a frequency domain linear finite element analysis of a two dimensional plane strain elastic medium, similar to the SASSI formulation. The Code treats both the solid and fluid fractions separately and accounts for their interaction through Biot's two-phase theory. Coupling between the soil and water takes place by accounting for both permeability as well as volume compressibility effects. The analysis is similar to considering both intergranular and pore water in ordinary static soil mechanics effects to arrive at the total stresses developed in the ground or against a wall.

The effects of soil saturation on dynamic response can be shown by considering the response of a rigid footing founded at the surface of an elastic two-phased half space (Figure B4). The forces developed on the footing for unit harmonic footing motions are then the impedance functions that are generated from similar one-phase solutions, such as with SASSI. Figures B5 and B6 indicate the comparison of the rocking and vertical interaction coefficients as functions of frequency for the fully dry and fully saturated soil cases. As you may note, differences can be significant. Fortunately, the differences that exist for the horizontal impedance coefficients are not as great as for the vertical and rocking cases. These differences in impedance functions can have a significant impact on seismic response and critical frequencies of some facilities, particularly when SSI is an important component in the seismic response.

The typical way that water is included in the one-phase analyses, such as a SASSI type seismic response analysis, assumes that the soil/water act as one material. The approach is to select the shear modulus of the material to match S-wave properties of the soil and the Poisson's ratio to match the P-wave velocity of the water. This often results in selecting values of Poisson's ratio between 0.45 and 0.48. The problem then becomes one of numerical stability as the value of Poisson's ratio approaches 0.5. Therefore, most people select Poisson's ratio no greater than 0.48. Some comparisons (Ref. B2) of the correct fully saturated two-phase analysis with the one-phase approximation using the fully incompressible assumption with Poisson's ratio equal to 0.5 indicates significant differences in impedance functions, particularly at higher frequencies. The importance of these differences obviously depends on the particular geometries and frequencies of interest or any one project. They have not been evaluated on any of the advanced reactor programs.

The second issue of importance involves the computation of lateral soil pressures computed on vertical walls developed by dynamic shaking of the facility. Again, the typical approach is to use a code like SASSI (one-phase approximation) with a Poisson's ratio of about 0.48 to match P-wave velocity through the pore water as described above. We have attempted to estimate the potential magnitude of such effects by using a simple 1D constrained rod analysis. The model is shown in Figure B7, in which the left end of the constrained rod represents the vertical wall that moves horizontally into the saturated soil. Obviously, this model assumes that the wall pressures are only influence by compressional waves moving through the solid and water fractions of the soil. A comparison of results is shown in the bottom of Figure B7. In the analysis, a steady state wall movement of 1" maximum magnitude is moved into the saturated soil at a given frequency. The stresses in both the saturated and dry cases developed at the wall have been computed and are shown in Figure B7. A comparison of stresses is made with the dry solution using different values of Poisson's ratio. For this case, to match the total stresses developed on the wall, the Poisson's ratio used in the dry analysis must reach a value of between 0.49 and 0.50, a value higher than typically used in SASSI type analyses. For lower values of Poisson's ratio, the computed stresses in the dry case are lower than the saturated results. The impact at regions closer to the free-surface where dissipation of pore pressure is easier and where total p[ressures are higher are expected to be less severe.

B5. CONCLUSIONS

The tentative conclusion reached support the approaches that are being performed by the Applicant to generate wall pressures on the vertical walls of the NI. Since the discretization requirements are so severe for the 3-D SASSI models, the approach of using the 2-D SASSI results with modifications based upon rational engineering approaches is felt to be more appropriate than atrtempting to extend the capability of the 3-D SASSI model. Secondly, the pressures computed using the one-phase soil-water representation is felt to lead to conservative estimates of wall loadings, particularly at or near the ground surface where computed pressures are largest.

B6. REFERENCES

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B2. C. J. Costantino, "Soil-Structure Interaction: Influence of Ground Water", NUREG/CR-4588, Volume 3, Brookhaven National Laboratory, April, 1986

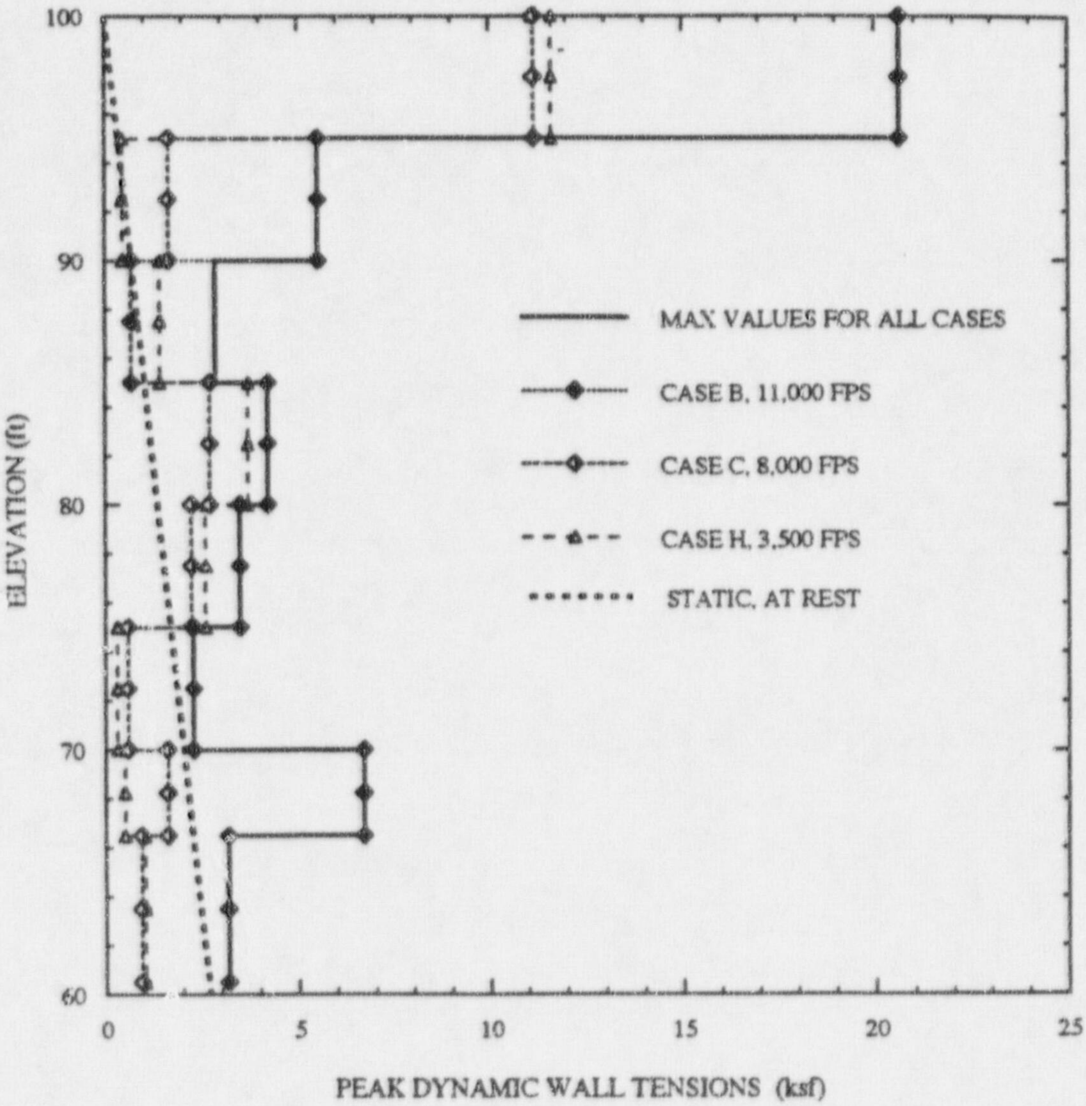


FIGURE B1 COMPUTED DYNAMIC WALL TENSIONS FROM 2D SASSI PROBLEM SET FOR UNIFORM ROCK SITES

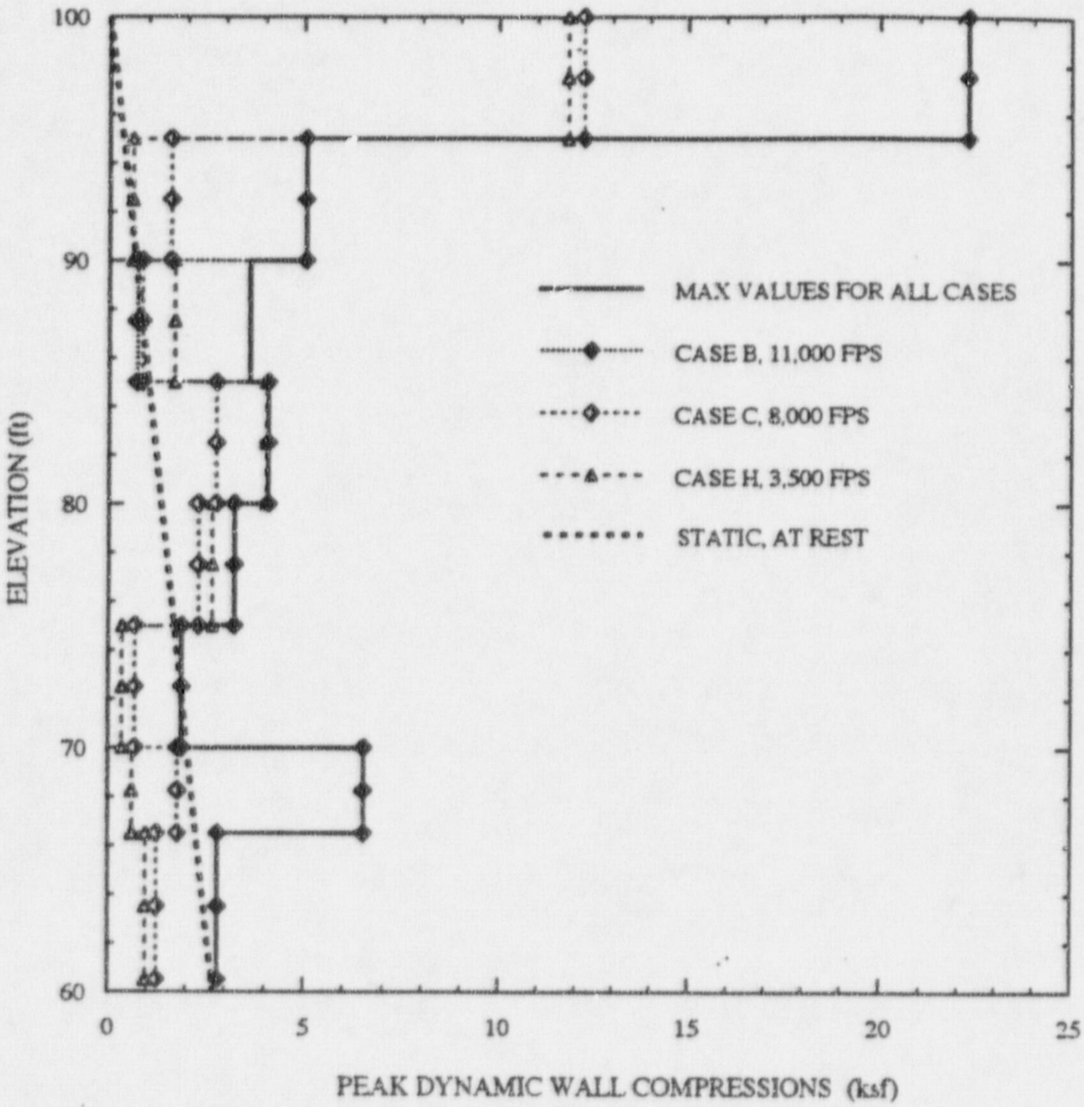


FIGURE B2 COMPUTED DYNAMIC WALL COMPRESSIONS FROM 2D SASSI PROBLEM SET FOR UNIFORM ROCK SITES

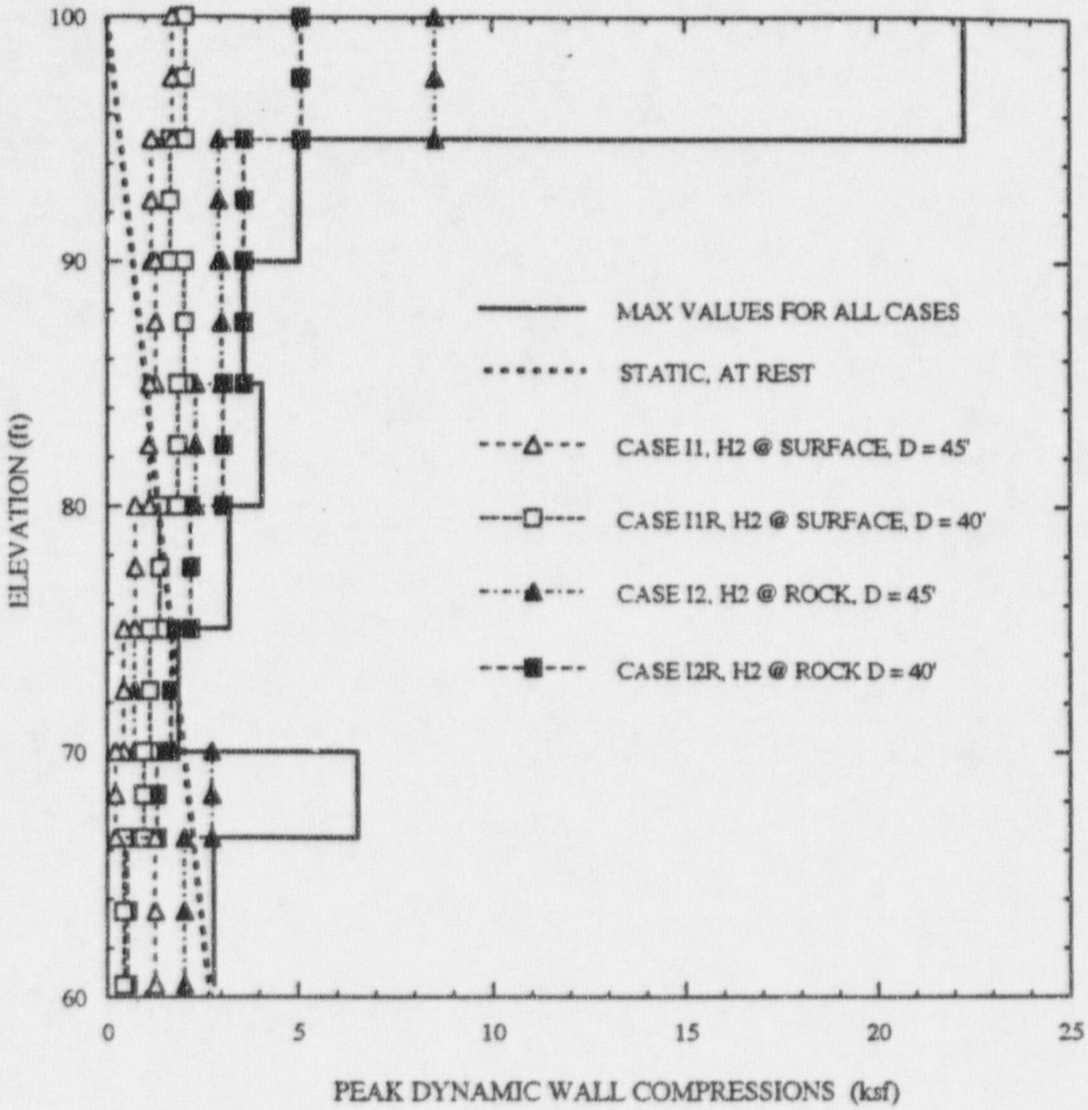


FIGURE B3 COMPUTED DYNAMIC WALL COMPRESSIONS FROM 2D SASSI PROBLEM SET FOR SOFT SOIL CASES

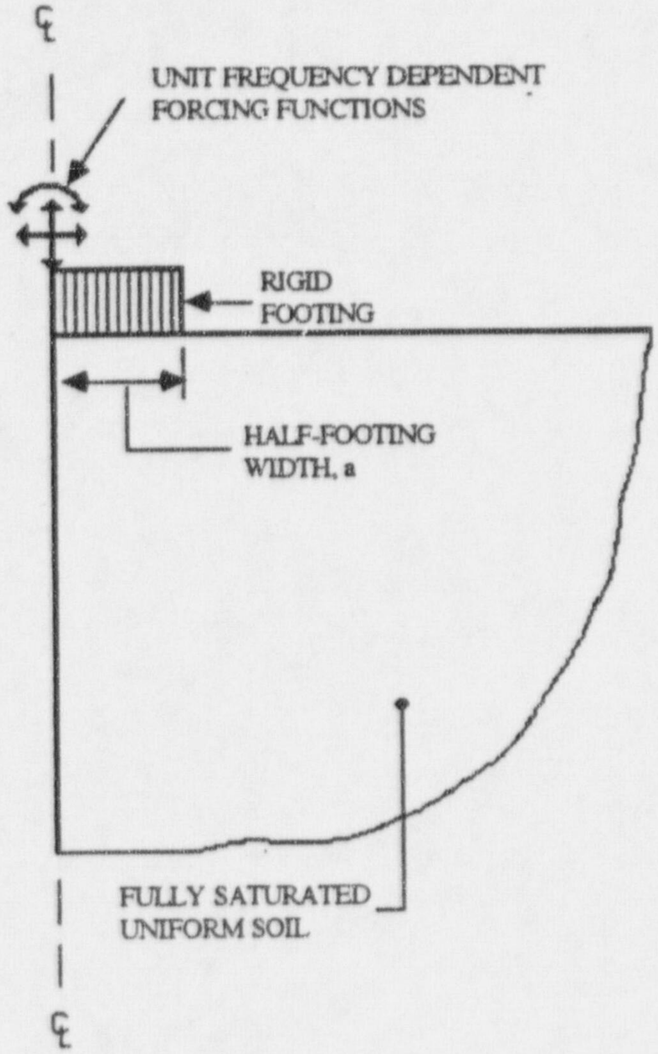


FIGURE B4 RIGID FOOTING ATOP AN ELASTIC, FULLY SATURATED SOIL IN A TWO DIMENSIONAL PLANE STRAIN CONFIGURATION

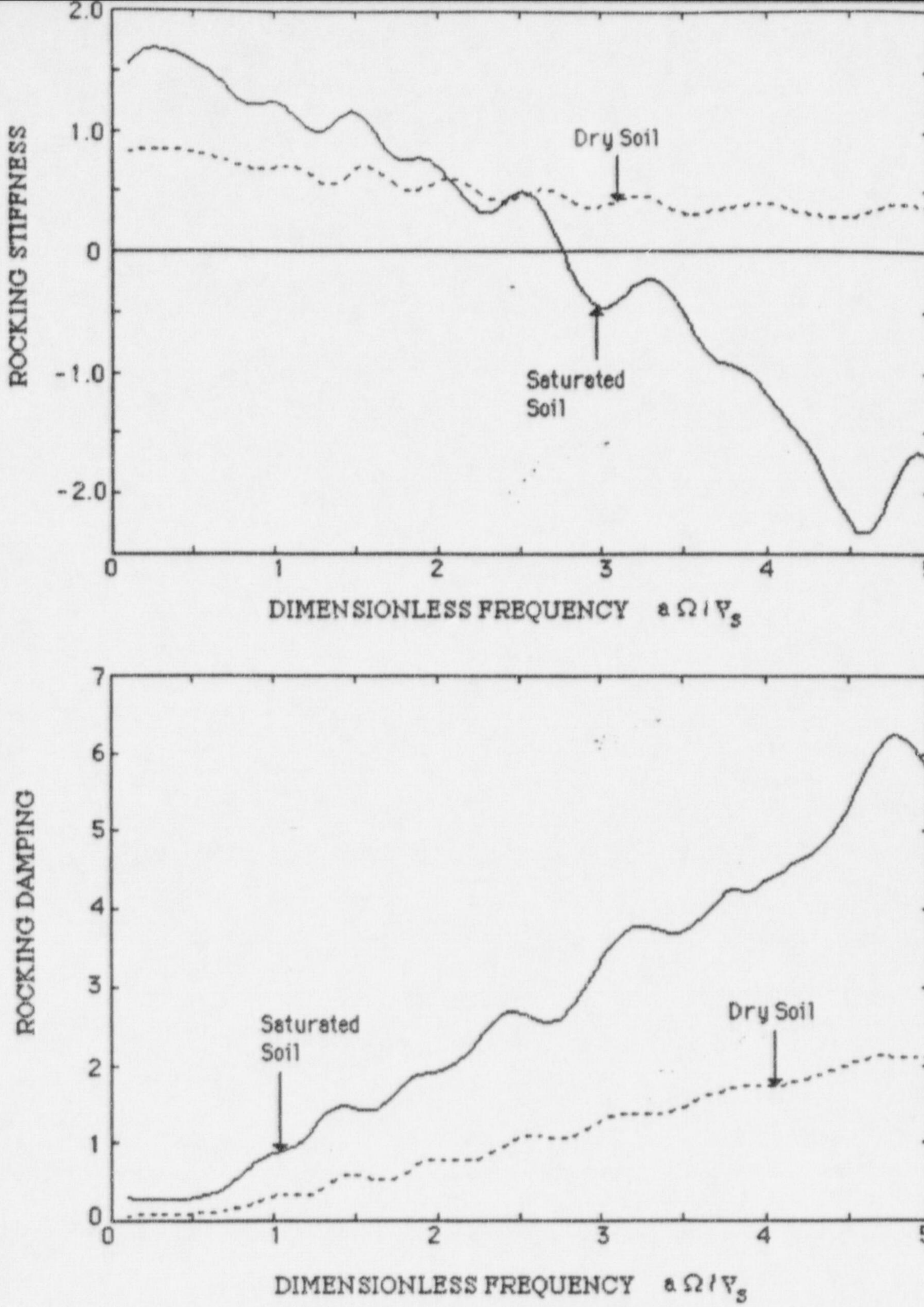


FIGURE B5 COMPARISON OF ROCKING IMPEDANCES FOR DRY AND FULLY SATURATED CASES

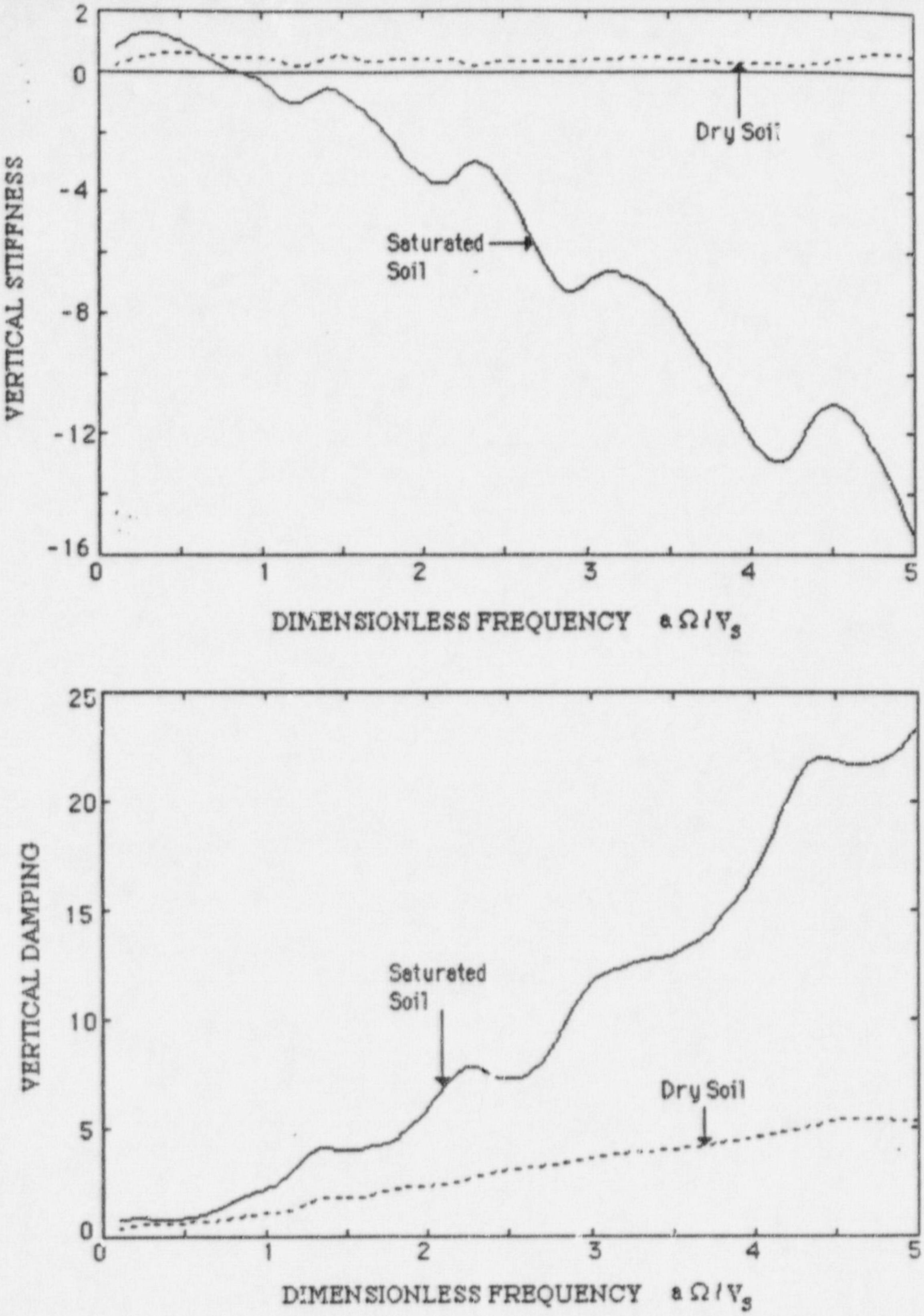


FIGURE B6 COMPARISON OF VERTICAL IMPEDANCES FOR DRY AND FULLY SATURATED CASES

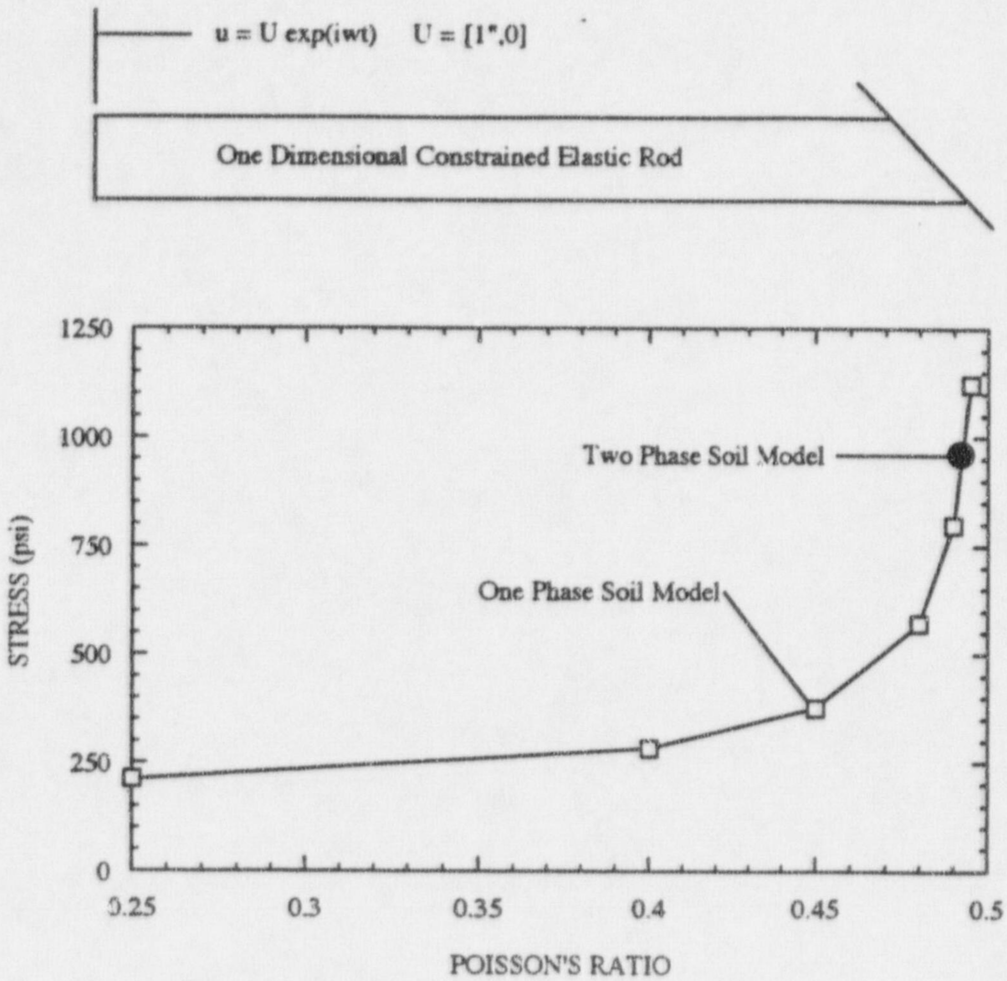


FIGURE B7   COMPARISON OF DRY AND SATURATED SOLUTIONS FOR ONE DIMENSIONAL CONSTRAINED SOIL

Appendix C

Parametric Studies using CARES Computer Code

C1. INTRODUCTION

As part of the seismic assessment of the proposed AP600 advanced reactor system, several efforts were undertaken by the Staff Review Team to investigate additional problems associated with soil-structure interaction (SSI) aspects of the seismic review. One of these areas of investigation was concerned with the critical depth of soil overburden to bedrock since it was found from the 2D SASSI computational set that this parameter was a significant player in the seismic response of the nuclear island (NI). As described in Appendix A, the Applicant performed 2D SASSI computations which had a limited variation in the rock depth. However, it was found from the SASSI analyses performed by the Staff Review Team that when the rock interface approached the bottom of the NI basemat, critical response could be calculated which may exceed the design response spectra, particularly at low frequencies. To evaluate this impact using further SASSI computations would require significant additional time and expense.

A second issue that arose during the review of the Applicant's computations had to do with the amount of out-of-plane motion that could be developed at critical locations on the NI, and the impact of variations in assumed direction of seismic input motions on these responses. The primary variational study performed by the Applicant made use of 2-D SASSI models which obviously cannot be used to evaluate this issues. Only the 3-D SASSI model can be used to evaluate this issue and since so few D problems were to be evaluated, the issue could not be properly evaluated.

For both these issues, the Staff Review Team performed several simplified analyses, making use of the CARES Computer Code (Ref. C1) to perform approximate confirmatory studies to indicate if potential problems may exist. CARES is a simplified computer code developed for the NRC to treat in a simplified fashion the SSI problem of interest for plant response analyses. The approach captures the primary effects of SSI but cannot be used for design evaluations except for simplified site and structural configurations. If the calculated responses from CARES for either issue indicated potential problems in NI responses, the more exact SASSI studies would have to be undertaken by the Applicant to further evaluate these issues. The following paragraphs attempt to summarize these additional studies.

C2. INFLUENCE OF DEPTH TO BEDROCK

CARES is designed to operate in a relatively quick and cheap manner to determine both free-field site and structural responses. In order to do this, however, it makes use of approximate interaction coefficients which are available from the open literature for simplified configurations of both the structure (circular or rectangular plan areas) and the site (uniform site conditions to bedrock). For the problem of interest herein, the plan area of the NI (see Figure A1) is not rectangular although it was felt that this deviation was not significant and the equivalent rectangle shown in Figure A1 was used in the CARES computations. However, the impedance function data incorporated into CARES and available in the literature are appropriate for the case of relatively deep depths to bedrock. For the shallow rock depths of concern for the AP600 problem of interest herein, the data on impedance functions had to be extended to make it appropriate for the shallow depths of interest.

Approximate impedance functions were developed by performing the following finite element studies. First, static FE analyses were performed with the ANSYS Computer Code as indicated in Figure C1. A number of meshes were developed varying the depth, d, the width of the mesh, L, and the FE size, keeping the parameters (L, W) of the embedded plant constant. From each computer run, the sidewall and basemat stiffness functions were determined. The impact of these parameters on the computed stiffnesses was determined to obtain a reasonable assessment of adequacy in the computation as indicated in Figures C2 and C3. From these results, the variation of stiffness with depth below the basemat was estimated as indicated in Figure C4, with the estimated parameters obtained from the computer output. The corresponding values of damping impedances was determined as shown in Figure C4, using the available results for the deeper depths to bedrock. The conversion of this data to treat the two layer (soft over stiff site) conditions of interest was made using the superposition argument shown schematically in Figure C5. These approximations to the impedance functions were then treated as frequency independent parameters and input to the CARES Code for use in the NI response analyses.

Prior to running the variations on bedrock depths, a run was made with the AP600 model for the uniform rock problem to ensure that CARES output provides a reasonable estimate of NI responses. Figures C6 through C9 show some comparisons with the more accurate 2D SASSI analysis for this problem. Some differences can be noted, particularly in Figure C8 for the Shield Building Roof node, where the SASSI spectral peak exceeds the CARES result, indicating some differences in impedances controlling rocking response of the NI. However, the results are considered reasonable for assessment of general trends. Figure C9 indicates a comparison with the fixed base BSAP results obtained by the Applicant.

Using these approximations, the CARES runs were made varying the depth to bedrock below the basemat from 1[illegible] to 100'. The available data from the 2-D SASSI computations indicated significant differences in response between these depth ranges. The results at two nodes of the AP600 stick are shown in Figures C10 and C11 which indicate the general trend in the results. Figure C10 presents spectral output at the Shield Building Roof, Node 3016, for the various depths to bedrock while Figure C11 presents similar output at the Steel Containment Vessel, Node 3110. It can be noted that as the depth to bedrock decreases, a fundamental shift in character of the response changes, with peaks of the response spectra shifting from the higher mode of the stick response to a lower mode. This shift is caused by the change in effective damping of the impedance function as the depth to rock decreases. It should be noted that this result is in keeping with the 2D SASSI results obtained for the shallow rock case. These same CARES results have been replotted in Figures C12 and C13 in the form of amplification ratios of the spectra at a particular depth to that at the deeper depth. In both cases, the results indicate that when the depth to bedrock is within 20' of the NI basemat, the lower frequency peak of the response increases significantly and may exceed the design allowables.

C3. OUT-OF-PLANE STRUCTURAL RESPONSES

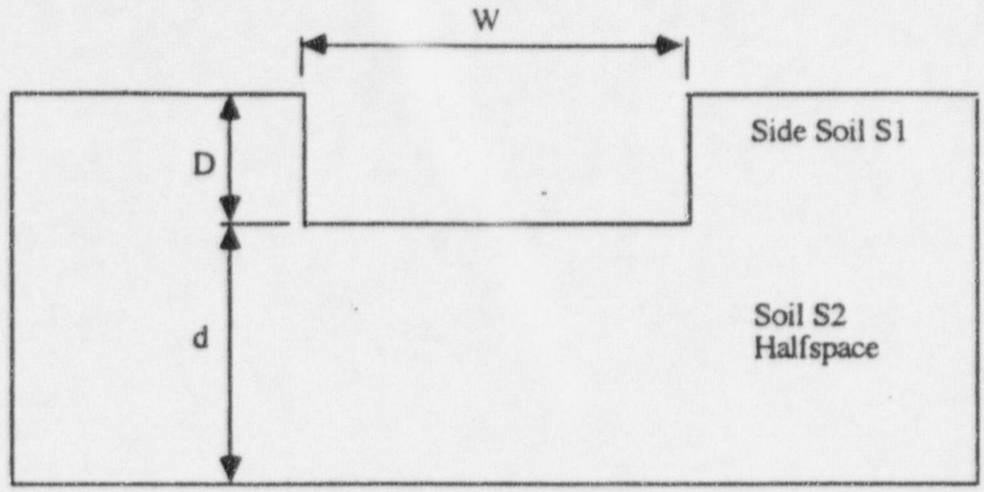
The variation in NI response due to a variation of direction of the input horizontal motions was considered by again using the simplified CARES model, but in this case using a 3-D model of the NI. The purpose of these results was to investigate the potential for significant out-of-plane response at critical locations of the NI, in which case further evaluation of direction of inputs may be of interest. Several runs were made using both the H1 (X-direction) and H2 (Y-direction) input motions and determining the nodal responses. A typical result is shown in Figure C14, in which it can be noted that only minor or negligible out-of-plane response is generated. This is a typical result for all the nodes evaluated. It should be noted that similar results were obtained from the one 3-D SASSI model performed by the Staff Review Team.

C4. CONCLUSIONS

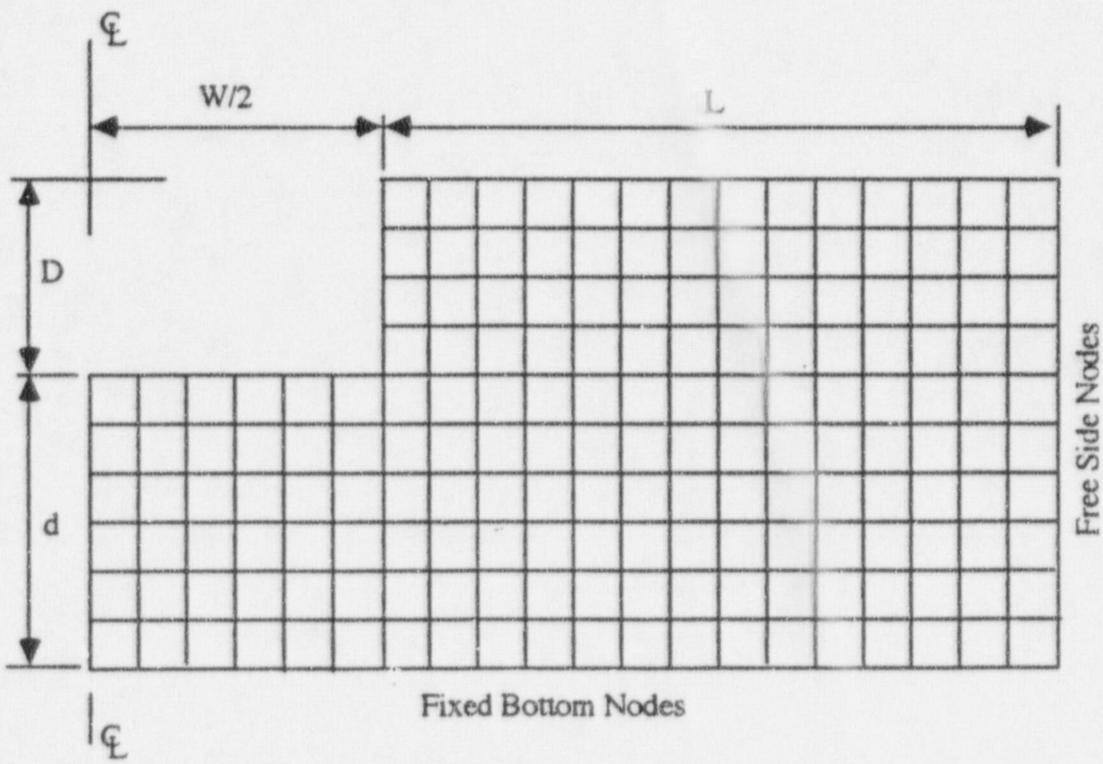
On the basis of these simplified results using the CARES Computer Code, two general conclusions were reached. First, in developing the range of site conditions considered appropriate for siting the AP600 reactor system, the depth to bedrock is a significant parameter in defining plant response and must be properly evaluated by the Applicant. The current site descriptions presented in the SSAR are primarily concerned with ranges of shear wave velocity to define acceptable site conditions. However, the depth of the bedrock below the basemat of the NI may be important, particularly for the softer site soil conditions. It is recommended that the Applicant specifically consider this parameter and evaluate its potential effect on the design. Secondly, it was determined that out-of-plane responses are generally not a significant concern when evaluating plant responses.

C5. REFERENCES

C1. Costantino, C. J., C. A. Miller, E. Heymsfield and A. Yang,"CARES: Computer Analysis for Rapid Evaluation of Structures" Version 1.2, Draft Report, Civil Engineering Dept., City College of New York, for U.S. Nuclear Regulatory Commission, September 1995

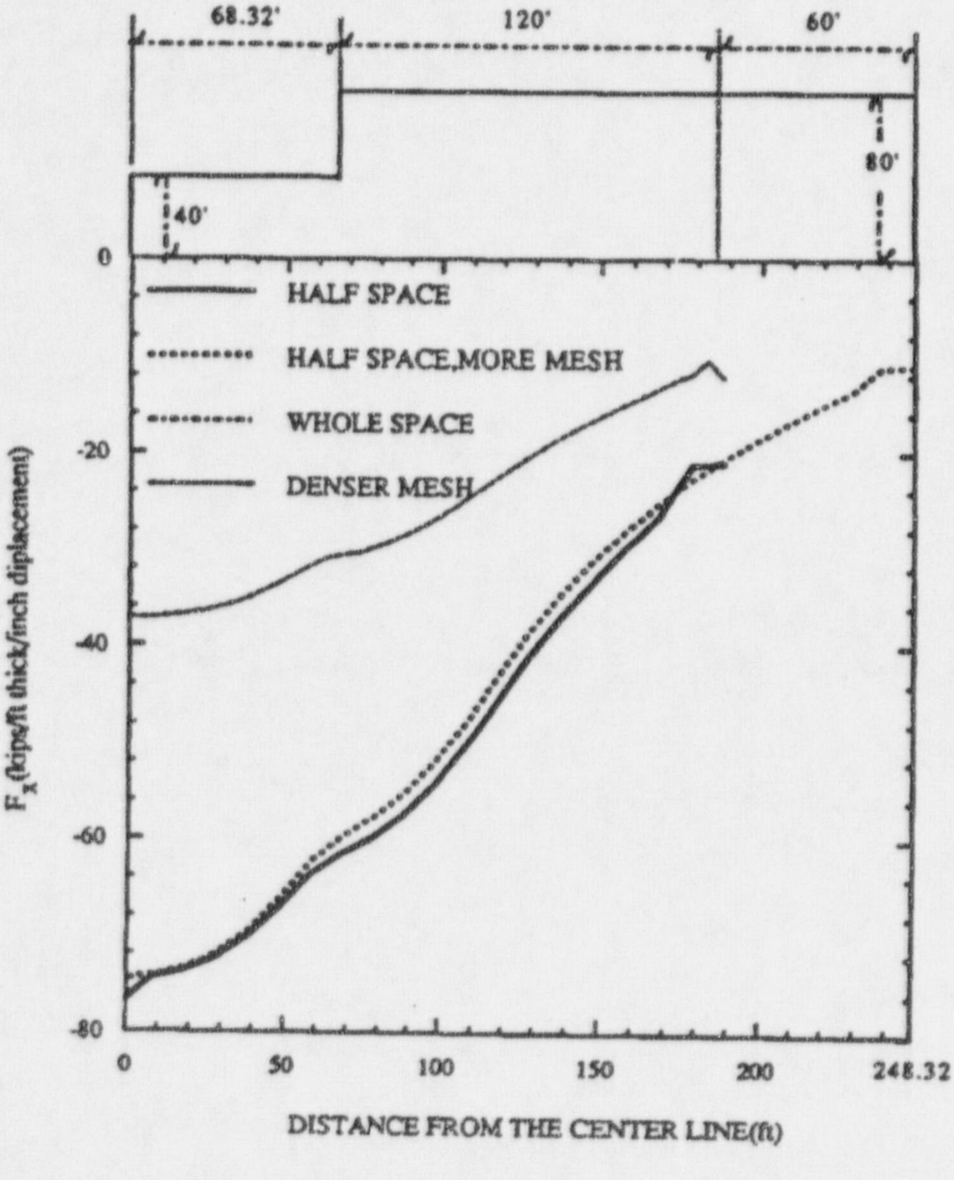


(a) General Site Configuration of Interest



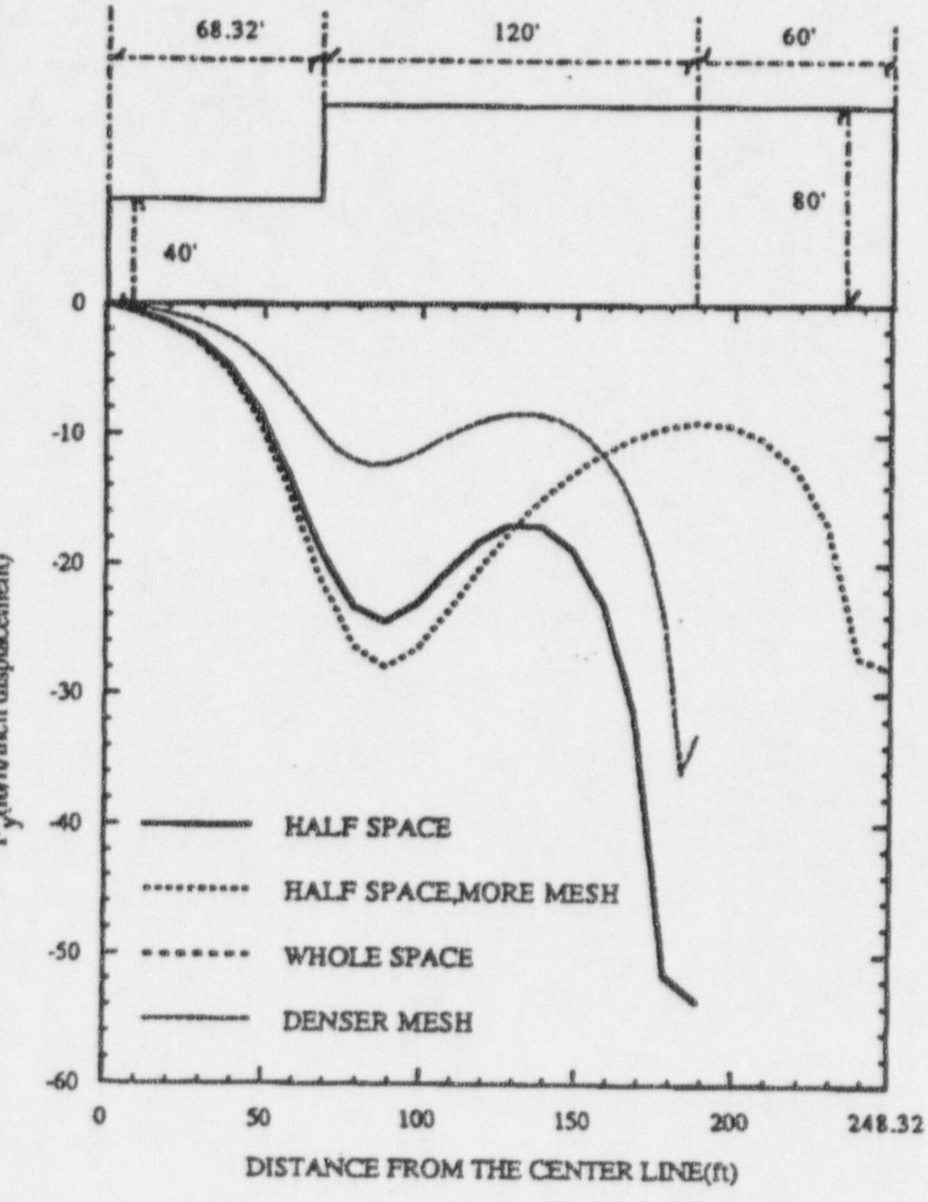
(b) ANSYS Finite Element Generic Model

FIGURE C1 CONFIGURATION FOR CARES CONFIRMATORY ANALYSES



HORIZONTAL NODAL FORCE ALONG THE BASE
("+" is the right direction)

FIGURE C2



VERTICAL NODAL FORCE ALONG THE BASE
("+" is the up direction)

FIGURE C3

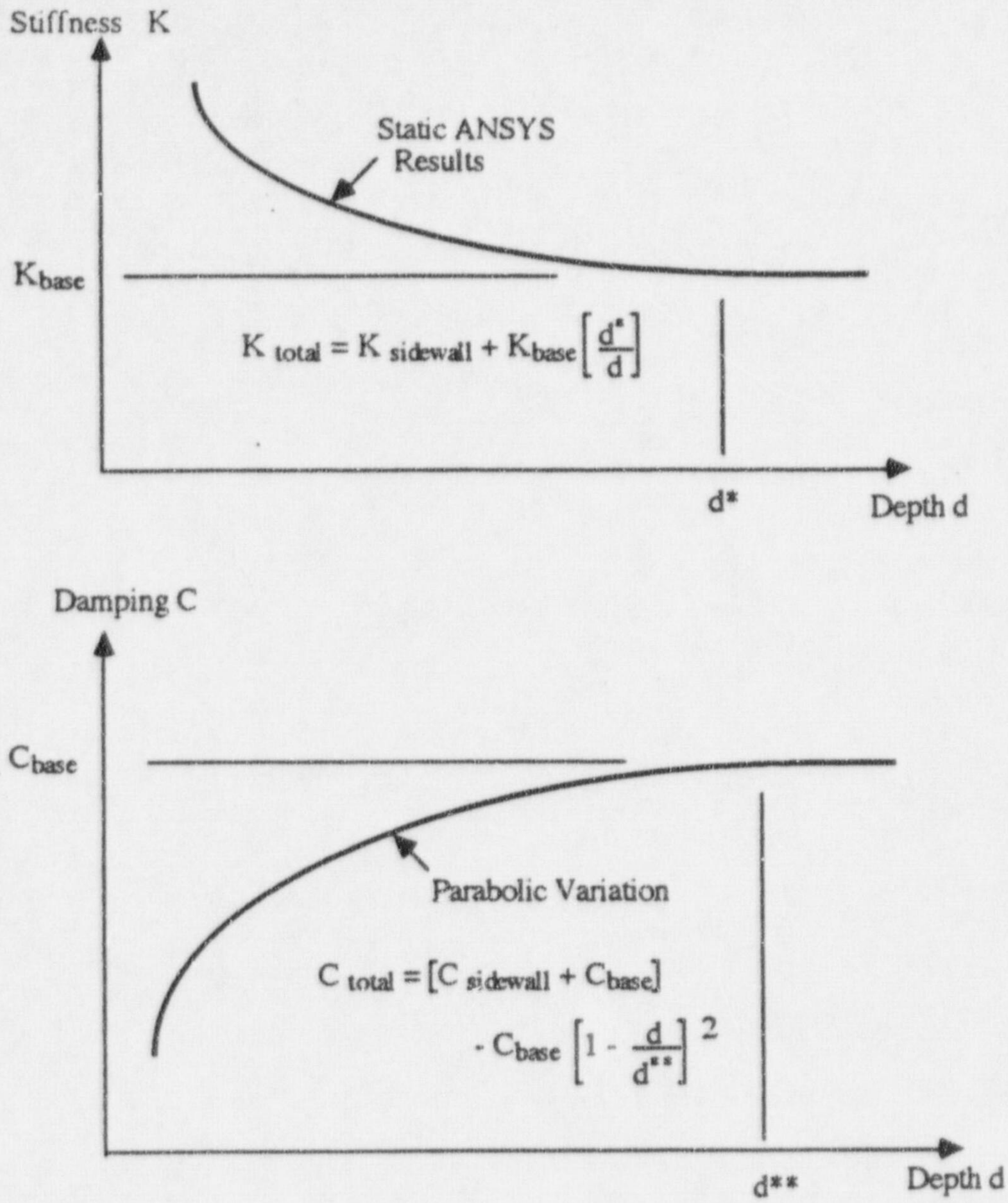


FIGURE C2 VARIATION OF IMPEDANCE FUNCTIONS WITH DEPTH
4 TO BOTTOM BOUNDARY

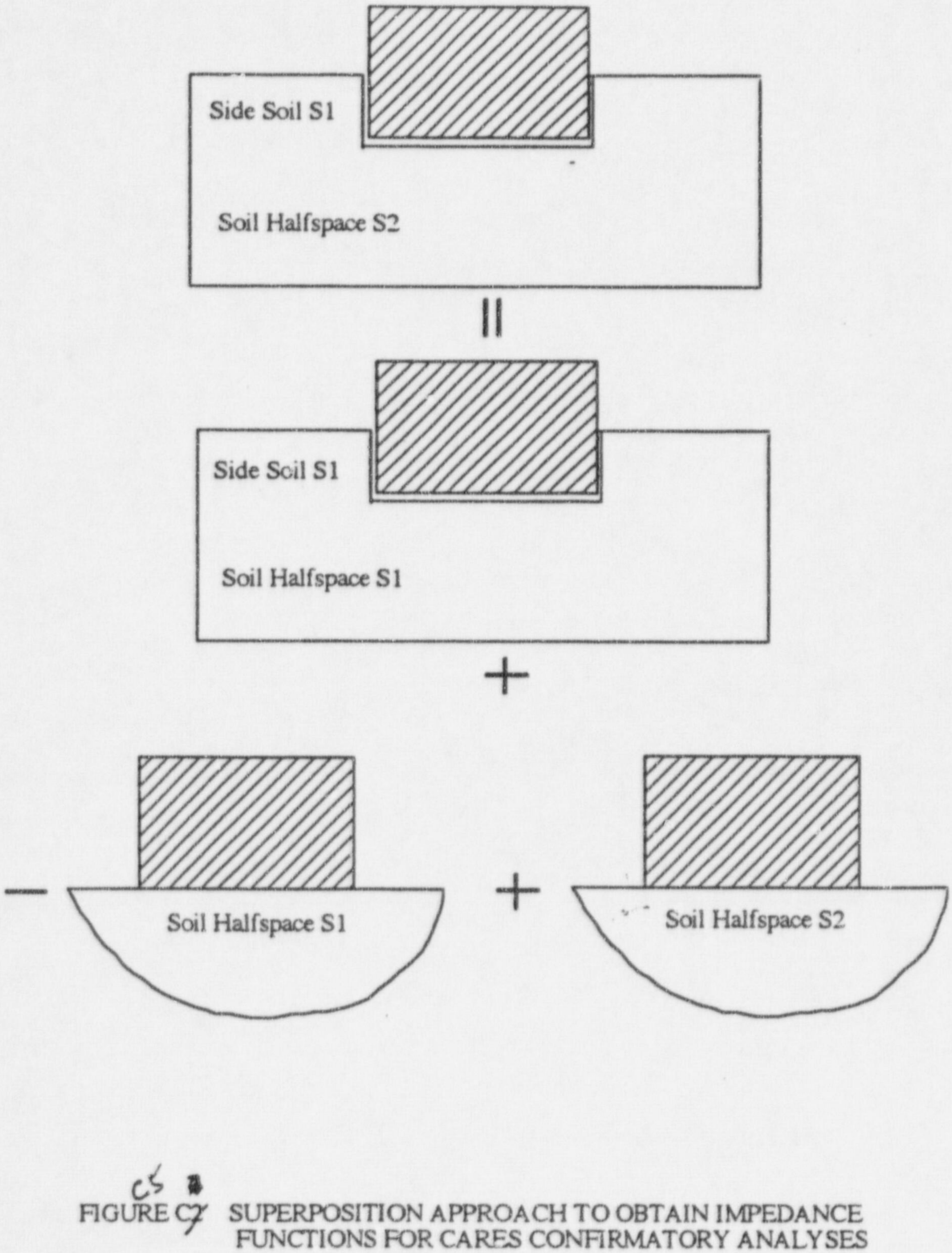
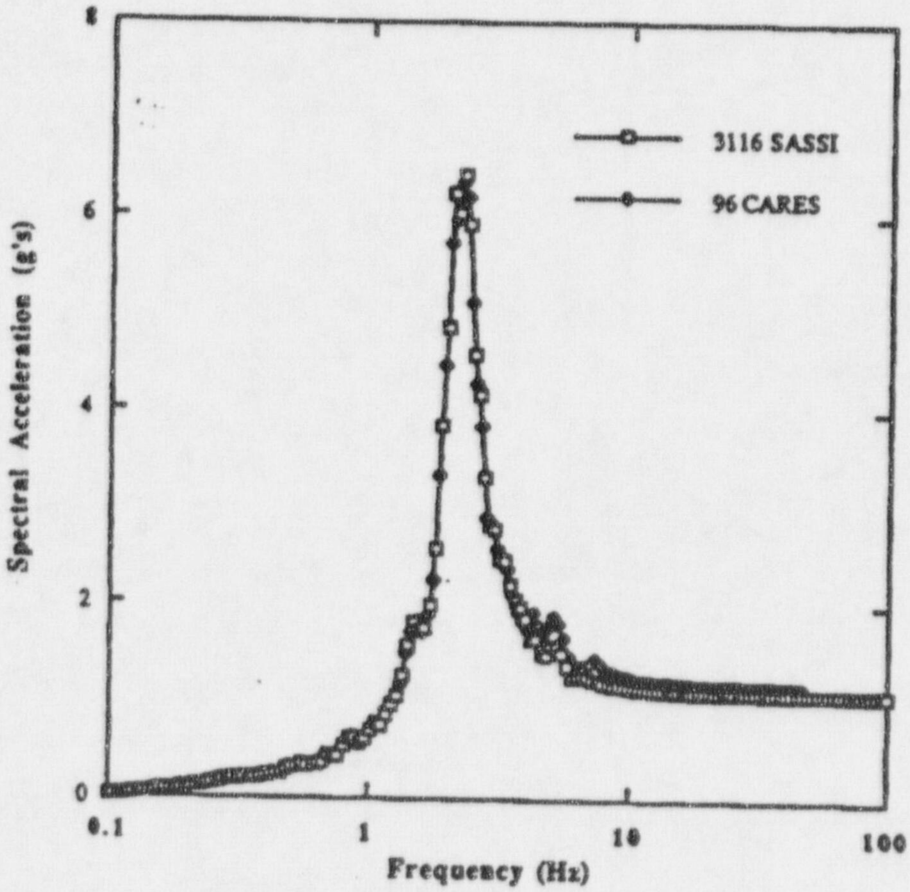
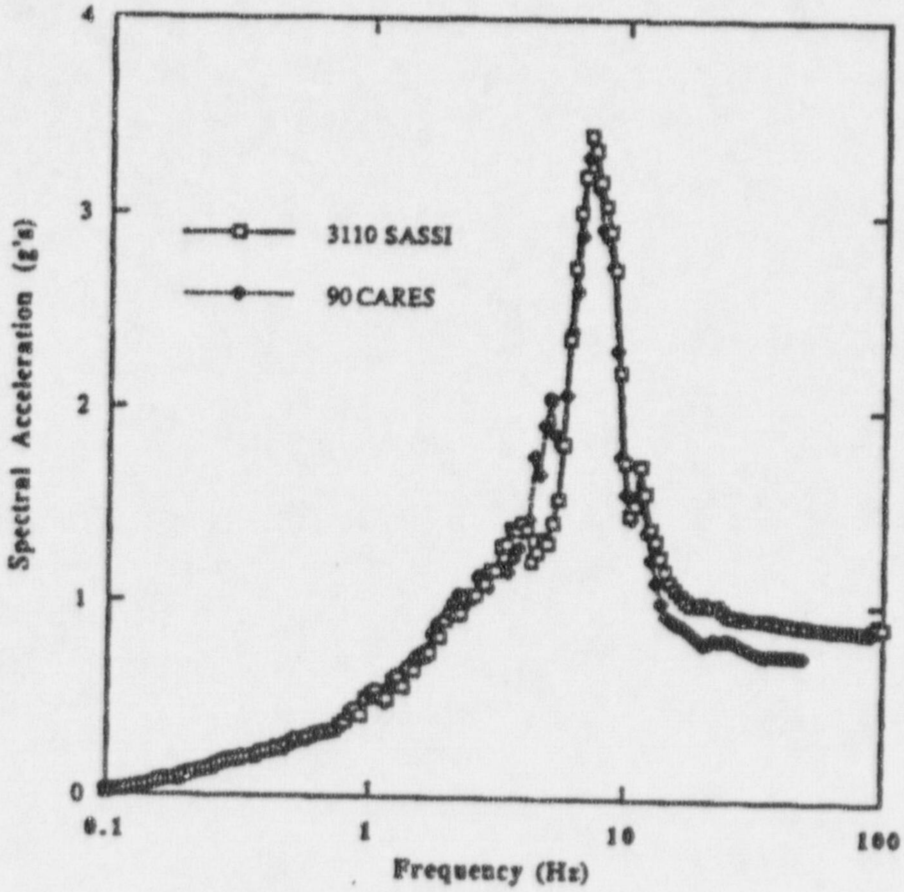


FIGURE C~~7~~ C5 SUPERPOSITION APPROACH TO OBTAIN IMPEDANCE FUNCTIONS FOR CARES CONFIRMATORY ANALYSES



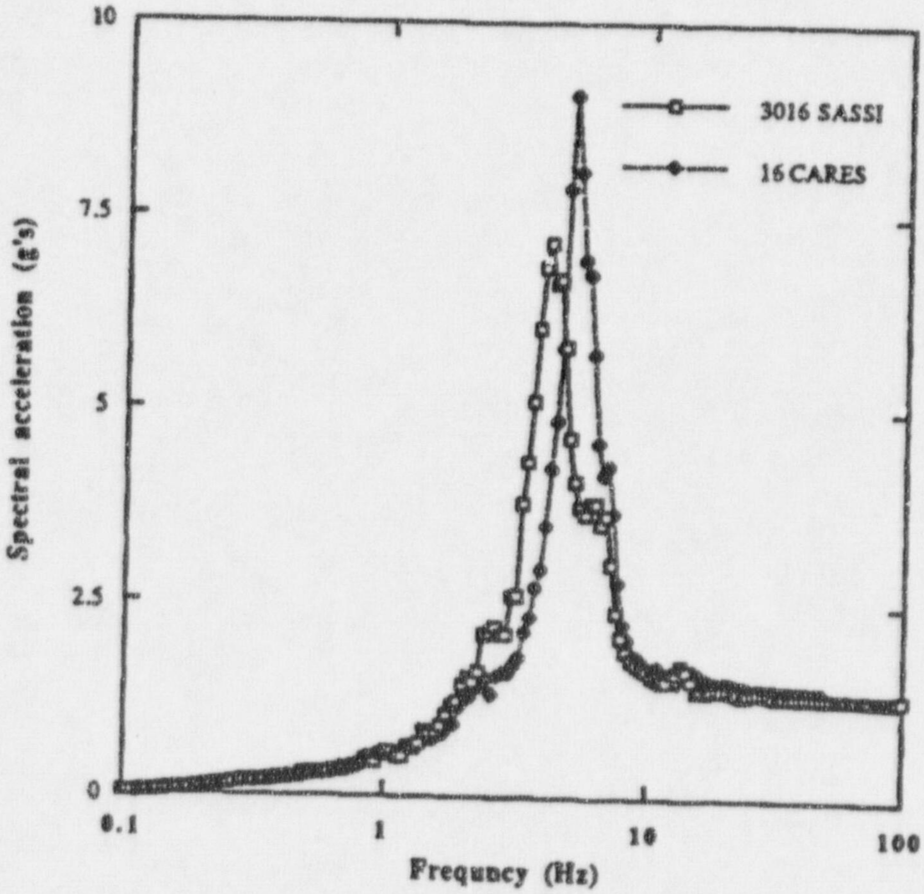
AP600 RESPONSE SPECTRA (5% DAMPING)
ROCK SITE WITH Vs=11,000 fps (CASE B)
POLAR CRANE
NODE 3116 (X=0.0, el=205.3')

FIGURE G6



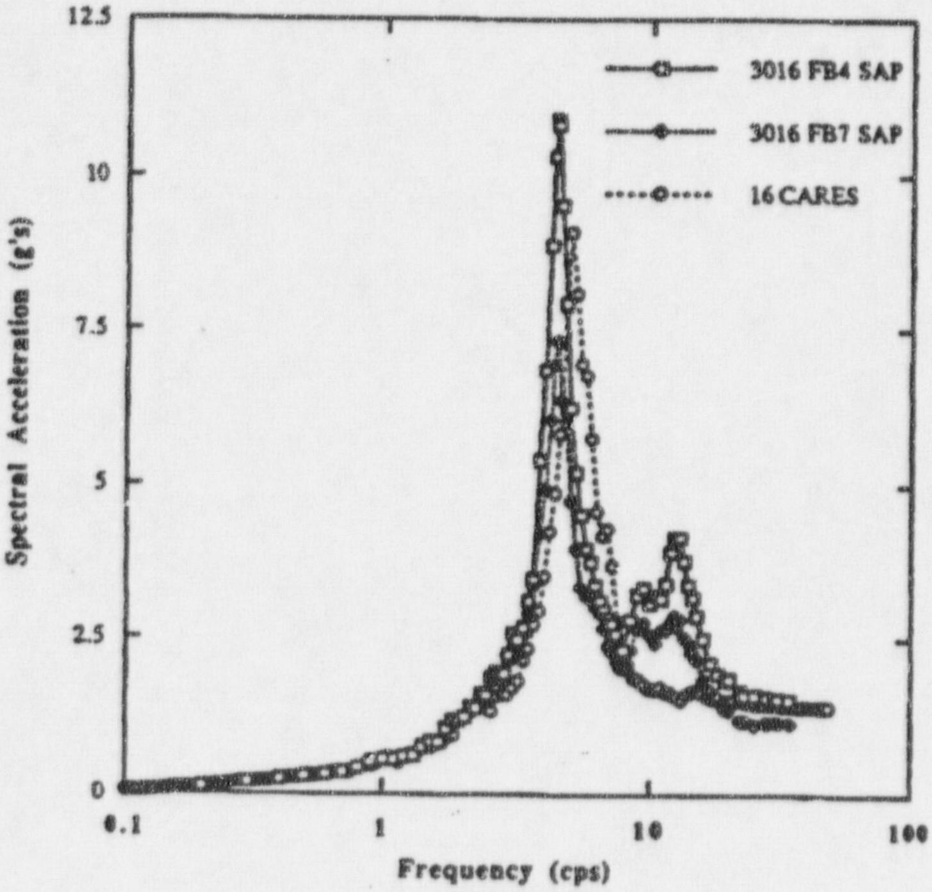
AP600 RESPONSE SPECTRA (5% DAMPING)
ROCK SITE WITH Vs=11,000 fps (CASE B)
STEEL CONTAINMENT VESSEL
NODE 3110 (X=0.0, el=205.3')

FIGURE C7



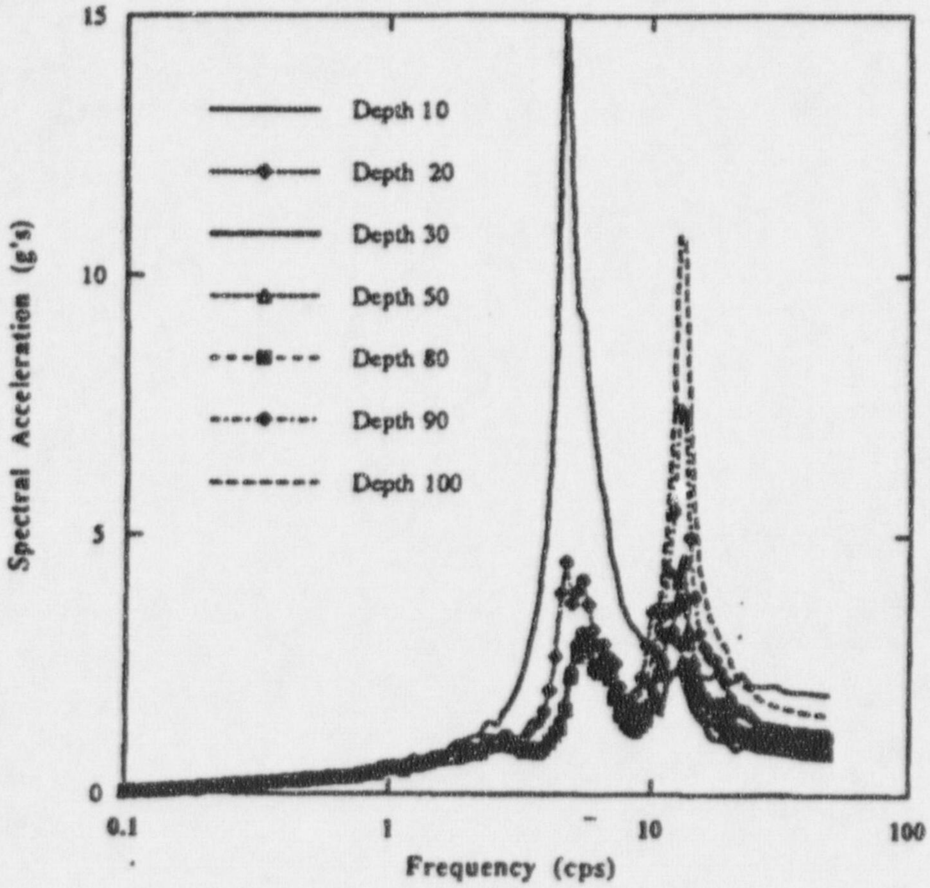
AP600 RESPONSE SPECTRA (5% DAMPING)
ROCK SITE WITH Vs=11,000 fps (CASE B)
SHIELD BUILDING ROOF
NODE 3016 (X=0.0, el=307.3')

FIGURE C8



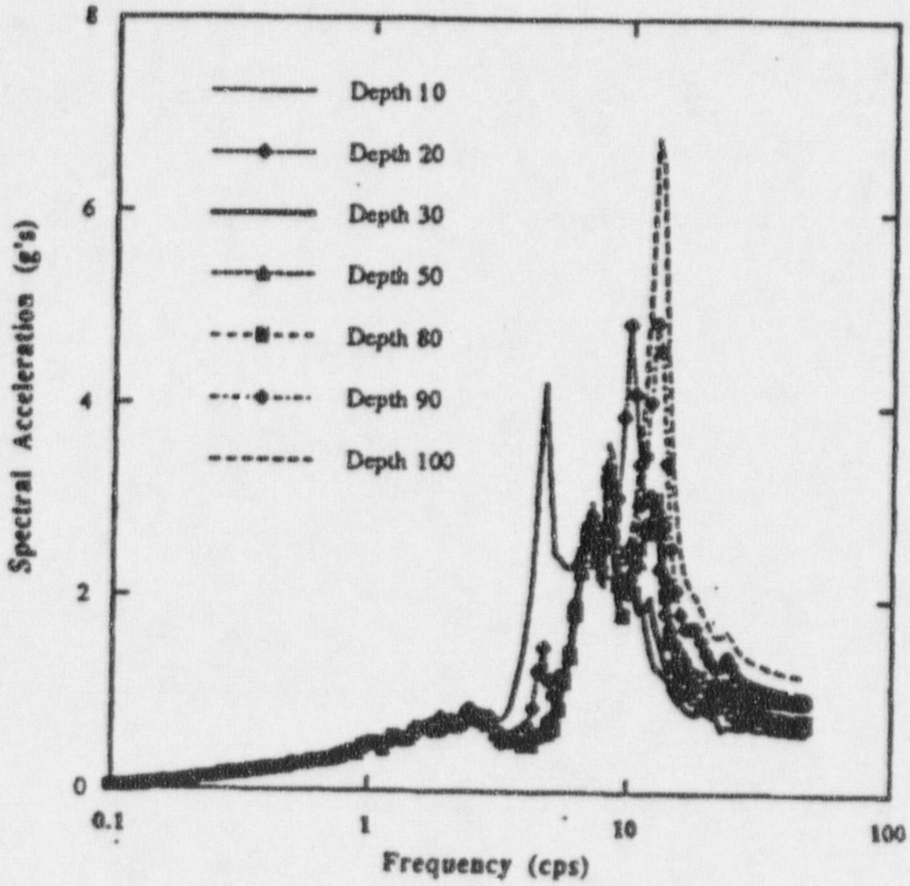
AP600 RESPONSE SPECTRA (5% DAMPING)
ROCK SITE WITH Vs=11,000 fps (CASE B)
SHIELD BUILDING ROOF
NODE 3016 (X=0.0, el=307.3')

FIGURE C9



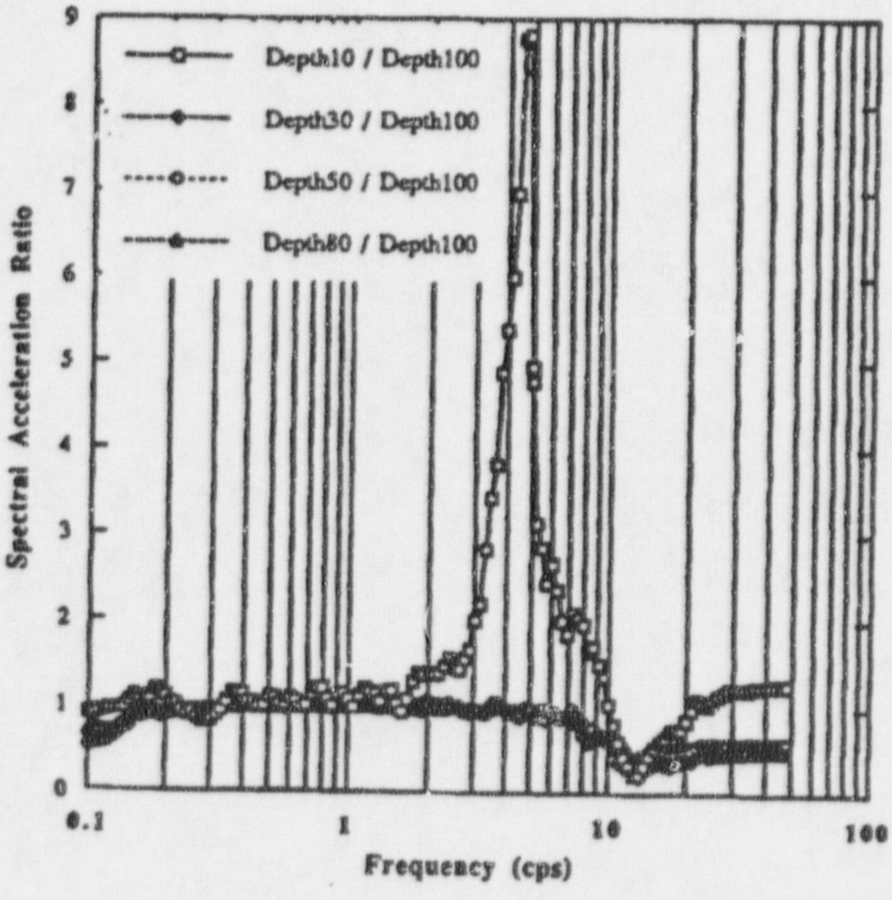
AP600 RESPONSE SPECTRA (5%DAMPING)
SHALLOW SOIL SITE (h=45') WITH
Vs=1,000fps
H2 MOTION @ SOIL SURFACE (CASE I1)
SHIELD BUILDING ROOF
NODE 3016 (X=0.0, el=307.3')

FIGURE C10



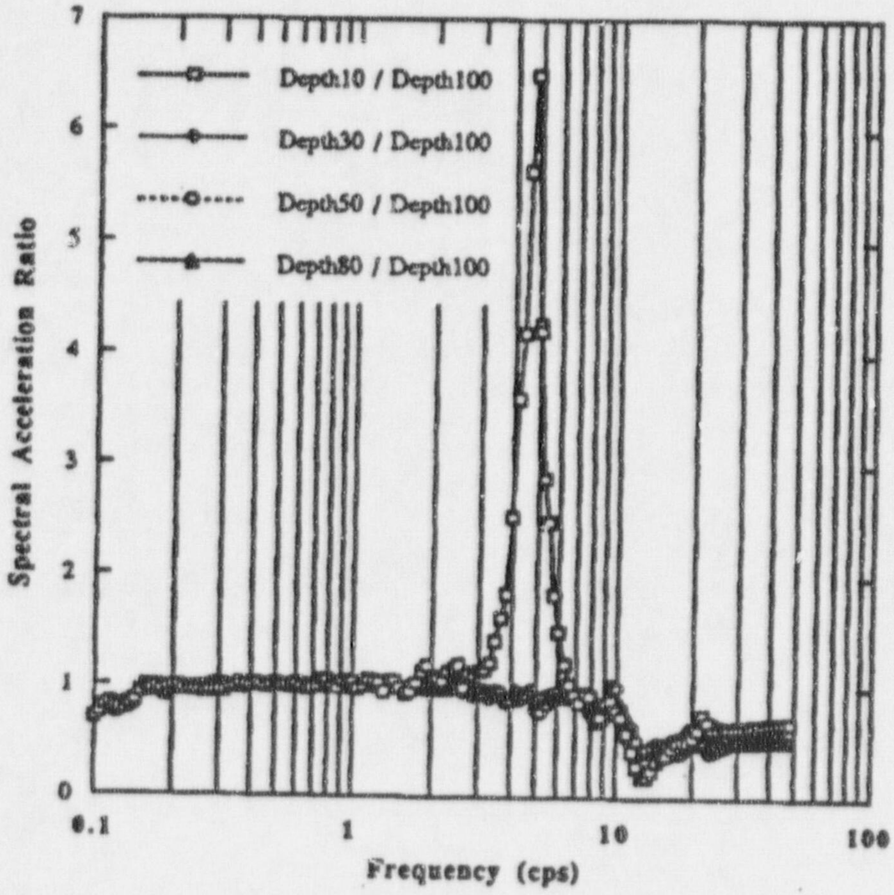
AP600 RESPONSE SPECTRA (5% DAMPING)
SHALLOW SOIL SITE (h=45') WITH
Vs=1,000fps
H2 MOTION @ SOIL SURFACE (CASE 11)
STEEL CONTAINMENT VESSEL
NODE 3110 (X=0.0, el=205.3')

FIGURE C11



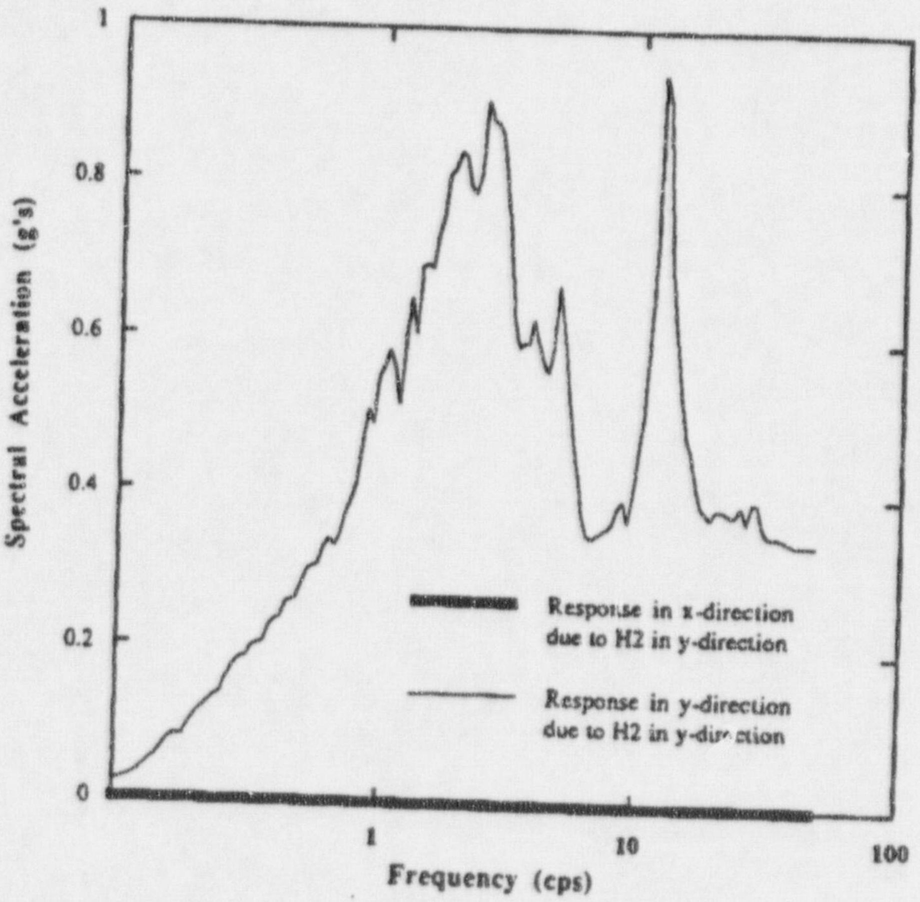
AP600 RESPONSE SPECTRAL ACCELERATION
RATIOS (5% DAMPING)
SHALLOW SOIL SITE (h=45') WITH
Vs=1,000fps
H2 MOTION @ SOIL SURFACE (CASE I1)
SHIELD BUILDING ROOF
NODE 3016 (X=0.0, el=307.3')

FIGURE C12



AP600 SPECTRAL ACCELERATION
RATIOS (5% DAMPING)
SHALLOW SOIL SITE (h=45') WITH
Vs=1,000fps
H2 MOTION @ SOIL SURFACE (CASE II)
STEEL CONTAINMENT VESSEL
NODE 3110 (X=0.0, el=205.3')

FIGURE C13



AP600 RESPONSE SPECTRA (5% DAMPING)
DEEP SOIL SITE (h=120') WITH
Vs=1000fps-2400fps @ 240' VARYING
PARABOLICALLY WITH DEPTH (CASE D)
SHIELD BUILDING ROOF
NODE 16 (3016)
(x=0.0, y=0.0, el=307.3')

FIGURE C14

Appendix D

Evaluation of Response Spectra at Basemat Level

D1. INTRODUCTION

Section 3.7.2 of the SRP (Ref. D1) indicates that when considering variations in amplitude and frequency content at depth for partially embedded structures, the free-field motion at foundation depth shall not be less than 60% than the corresponding amplitude of the design motion at the finished grade. The 60% limitation may be satisfied by considering the envelope of the three foundation level spectra in the free-field corresponding to the three soil cases considered. If the analysis does not include consideration of the rotational components of the motion at depth, no reduction is permitted.

In previous years, when simplified SSI methods of analysis were used (constant SSI spring/dashpot models with simple structural sticks), no reduction in input ground motions was allowed in SSI analyses. Concerns with abilities to predict incoming seismic wave fields, completeness of the methods of SSI analysis and uncertainties in specifying site properties all played a part in this position. As methods of SSI analysis improved, the concern with the determination of allowable levels of reductions of the free-field motions at depth have evolved since most recorded data at depth did indicate some amount of reduction as compared to the measured surface spectrum. However, uncertainties in the completeness of the methods of SSI analyses as well as variability in site conditions still remained. Current analysis methods are now considered complete enough such that the concern with rotational input is no longer an issue, leaving the definition of incoming wave fields as well as treatment of variability of soil properties in quantifying free-field motions at depth as the prime driver in maintaining the 60% requirement. Some indicate that the 60% requirement is no longer required since potential variability in ground motion at depth is fully captured by the variability in soil column properties (1/2 to 2 times best estimate properties) considered in the usual SSI analyses. The current revised version of ASCE 4-86 (Ref. D2) has apparently dropped this concern in the Commentary section of this guideline. The SSI procedure recommended in ASCE 4-86 indicates that at least 3 analyses be performed, using lower bound, best estimate and upper bound soil properties together with the best estimate structural model.

Recent experience with other evaluations of critical facilities, however, indicate that this approach may not be acceptable for those cases where SSI effects are important; that is, where critical responses are strongly influenced by the SSI effects. By uniformly changing the shear modulus of the soil column by a constant factor, it has been found that both the foundation level free-field spectrum as well as the SSI frequencies may all be proportional to the shear moduli. Therefore, if in a lower bound calculation, for example, it is found that the structural response frequency falls "in the valley" of the free-field spectrum, the response may always be in that valley as the soil column is uniformly stiffened since all parameters controlling the response are proportional to the shear modulus. This is illustrated schematically in Figures D1, D2 and D3 in which both the frequency of the free-field and the SSI mode of the rigid structure are found to be proportional to the site shear wave velocity.

Of course, if it is found that other parameters of the lower bound, best estimate and upper bound analyses control the relative locations of the "valleys" of the in-structure spectra, then the concern with this issue may be lessened. In the AP600 review, this concern has been raised since each assumed site condition has been performed for best estimate values only. The counter argument presented by the Applicant is that the free-field spectra at a depth of 40' (bottom of the basemat) from all the cases studied exceed the 60% requirement. However, whether they have met the intent of the rule is not clear until the issue of relative spread of the SSI frequencies is also addressed. It is not clear how a simple procedure can be developed that will work to overcome this concern. A procedure that may work is to first envelope the free-field motion at depth and use this broad-banded motion as the input into a single SSI analysis of the nuclear island (NI). An alternate would be to use random selection of soil properties with depth such that the shear moduli are not uniformly modified. However, this would lead to an inordinate amount of SSI SASSI computations.

D2. AMPLIFIED INPUT MOTIONS

In this section we present the implication of an interpretation of the 60% rule on computed NI responses. In this approach, the foundation level motions are maintained at or greater than the 60% surface spectral values for all variations in site properties, and this single enhanced motion is used as input to a single SSI response analysis. For the one problem evaluated, the amplified ground motion was developed to satisfy the requirement of no greater than a 60% reduction in ground motion at the foundation level as described in the Standard Review Plan (SRP). An additional 2D SASSI calculation was performed for the soft to medium stiff soil site (Case A of Appendix A) but using an enhanced H2 free-field ground motion defined at the ground surface. In this evaluation, a typical soil variation in the free-field deconvolution calculation was performed, varying the shear moduli from 1/2 to 2 times their best estimate shear wave velocities defined by the Case A configuration.

The original free-field seismic input motion used is the H2 time history generated by the Applicant and described in Appendix A. The free-field spectrum at the foundation level depth of 40 feet was then computed for each site condition assumed (best estimate, lower bound and upper bound) and the results at the 40' depth are shown in Figure D4. It can be noted that the spectra at depth fall below the 60% criteria spectrum over a frequency range of from about 1.5 hz to about 10 hz. In this problem, the foundation level spectrum in this frequency range was then amplified to match the 60% target spectrum as shown in Figure D5, a new time history generated (Figure D6) which was then convolved to the ground surface using the best estimate soil column properties to arrive at an enhanced surface H2 motion, as noted in Figure D7. This ground motion can then be used as the input ground motion to the 2D SASSI model and presumably would satisfy the intent of the SRP guidance. The resulting response spectra are shown in Figures D8 through D11, and show exceedances over the corresponding fixed base spectra which have typically controlled the design.

D3. CONCLUSIONS

It is not clear that the process followed by the Applicant of using the resulting spectra at the 40' depth from each site response analysis to show that the design satisfies the intent of the 60% rule at foundation depth. It is recommended that they consider this problem further to indicate that in fact the relationship between the "valleys" of the free-field spectra at depth do not all bear the same relationship to the SSI frequencies in each of the 2-D problems investigated.

D4. REFERENCES

D1. "Standard Review Plan", NUREG-0800, Office of Nuclear Reactor Regulation, U.S. Nuclear Regulatory Commission, Revision 2, August, 1989

D2. American Society of Civil Engineers Standard 4-86, "Seismic Analysis of Safety Related Nuclear Structures and Commentary on Standard for Seismic Analysis of Safety Related Nuclear Structures", 1986

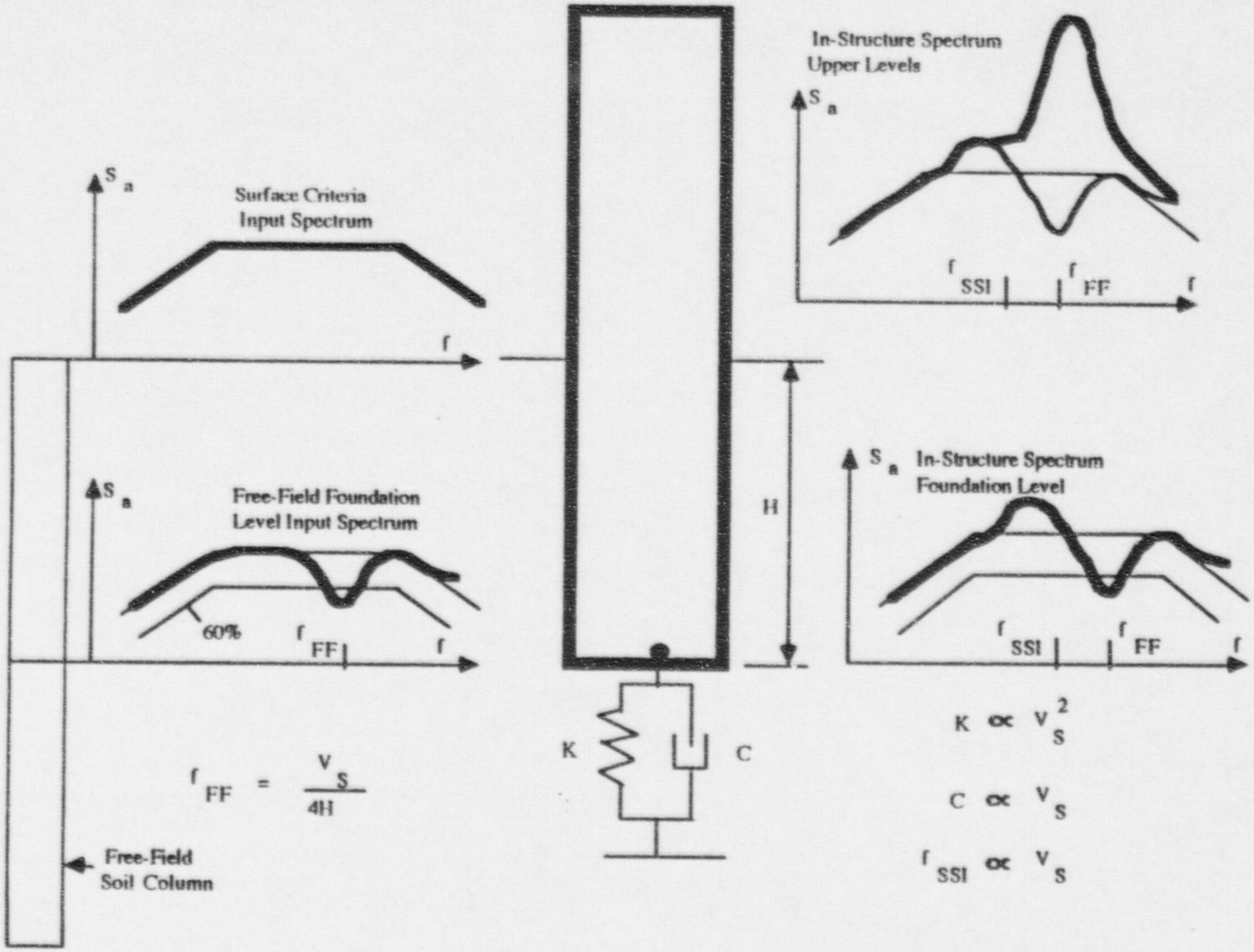
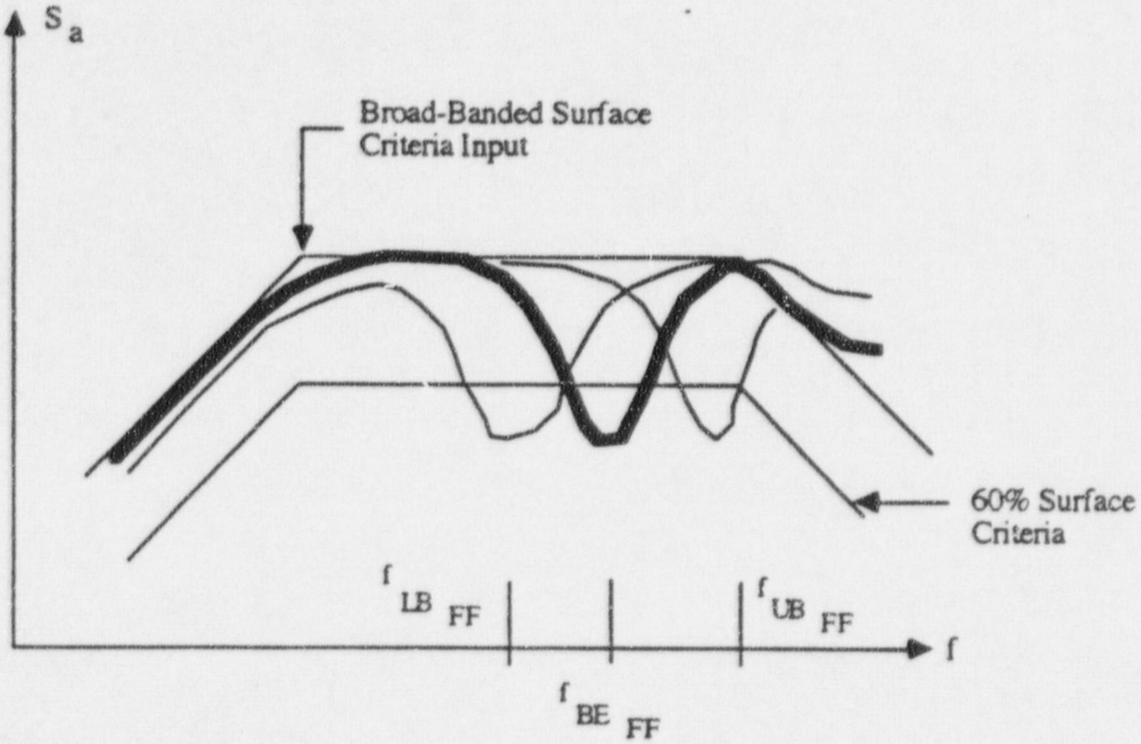


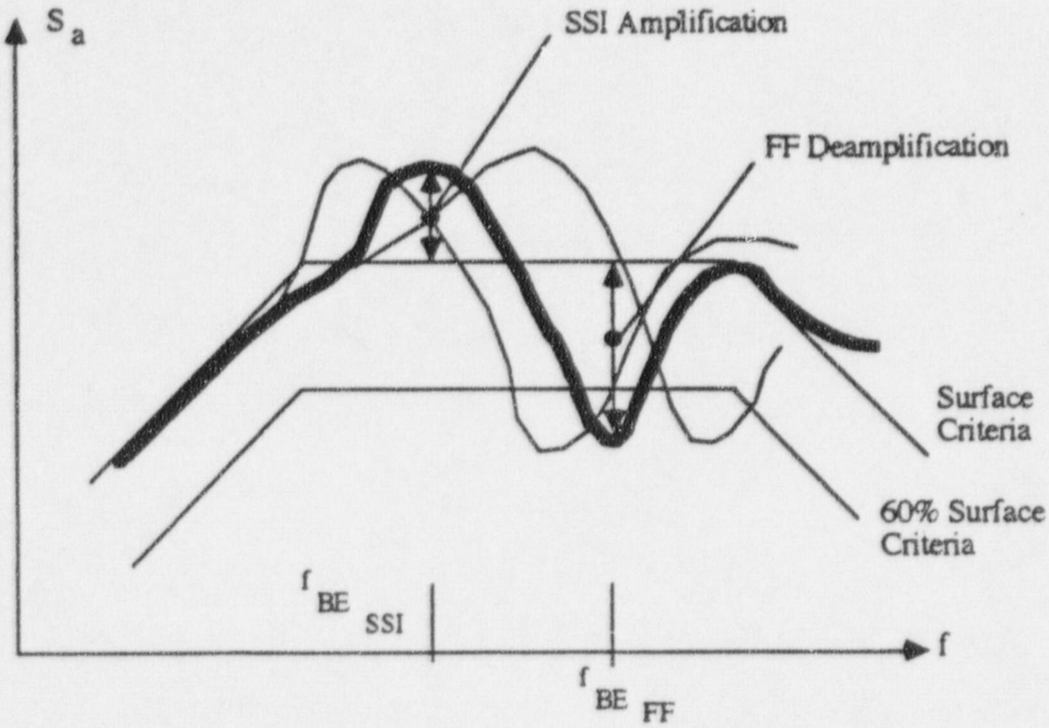
FIGURE 1 GENERAL SSI RESPONSE ANALYSES



$$\frac{f_{LB_{FF}}}{f_{BE_{FF}}} = 0.71 \qquad \frac{f_{UB_{FF}}}{f_{BE_{FF}}} = 1.41$$

Interpretation: Envelope of Input Spectra at Foundation Level Is Greater Than 60% of Surface Criteria Spectrum

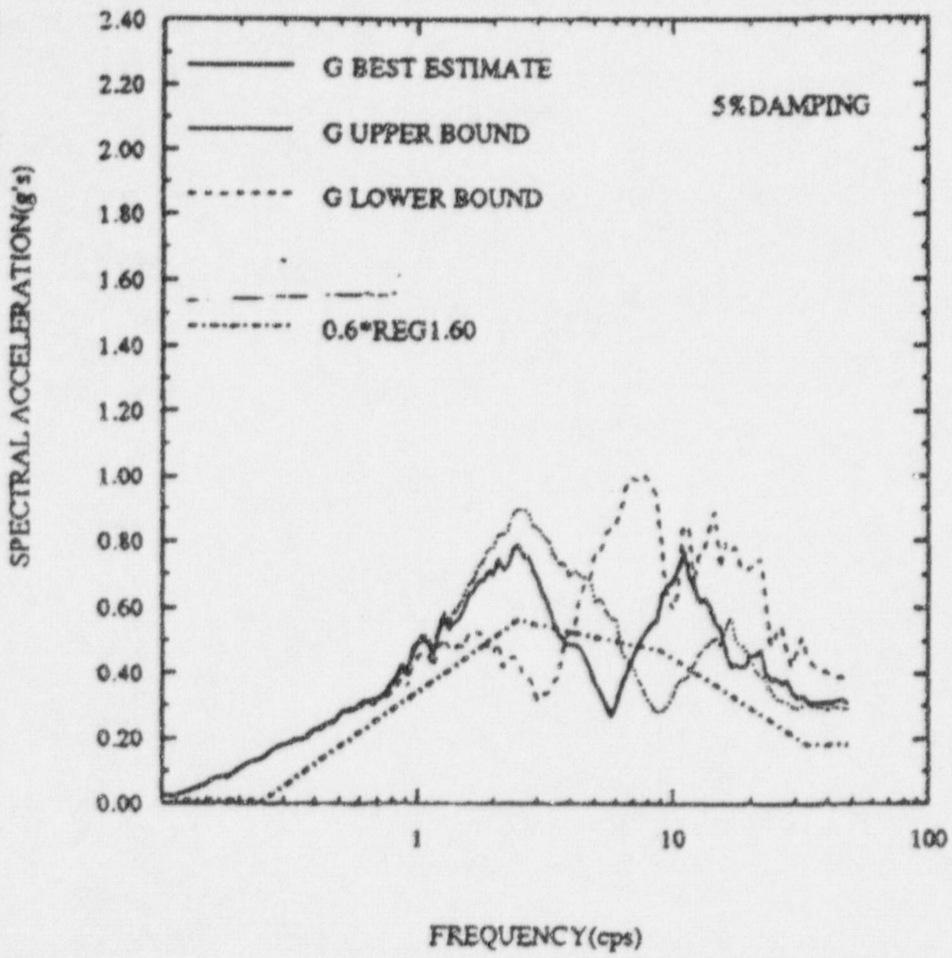
FIGURE 2 FREE-FIELD SPECTRA AT FOUNDATION LEVEL



$$\frac{f_{LB_{SSI}}}{f_{BE_{SSI}}} = 0.71 \qquad \frac{f_{UB_{SSI}}}{f_{BE_{SSI}}} = 1.41$$

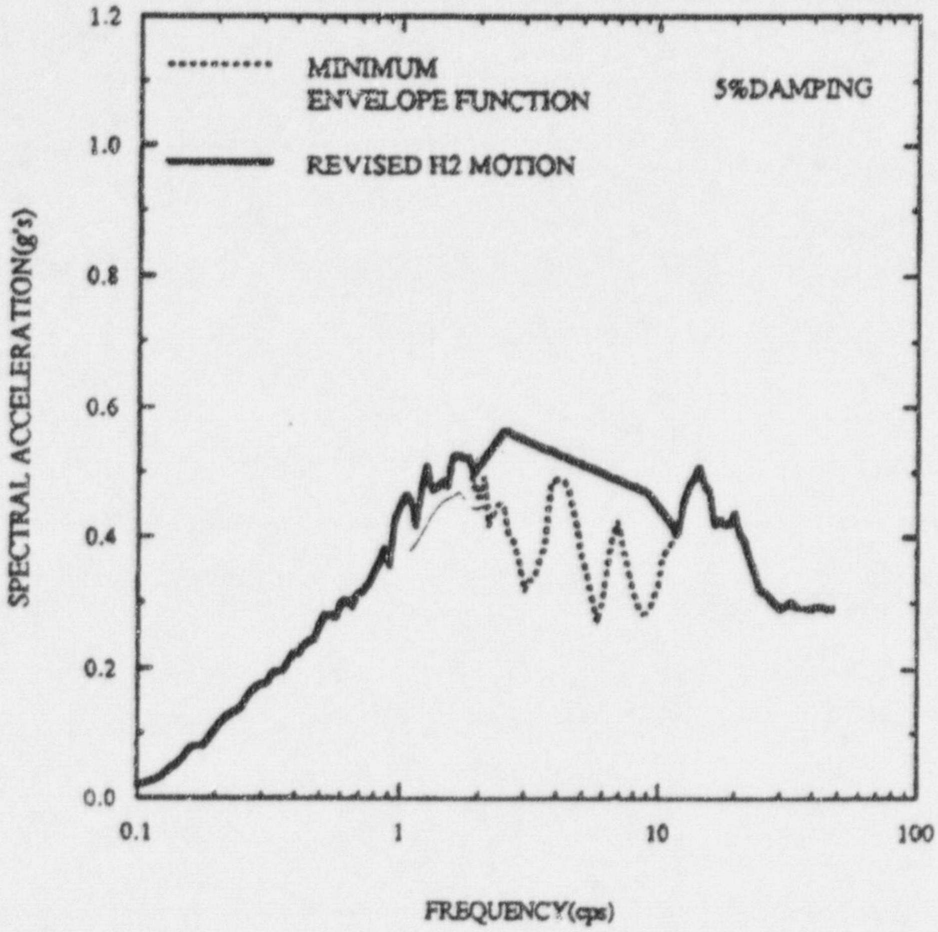
- Envelope (Widened ± 15%) of Instructure Spectra Defined as Design Spectrum
- Question of Similar Approach to Determine Design Structural Loads and Element Forces

FIGURE 3 INSTRUCTURE SPECTRA AT FOUNDATION LEVEL



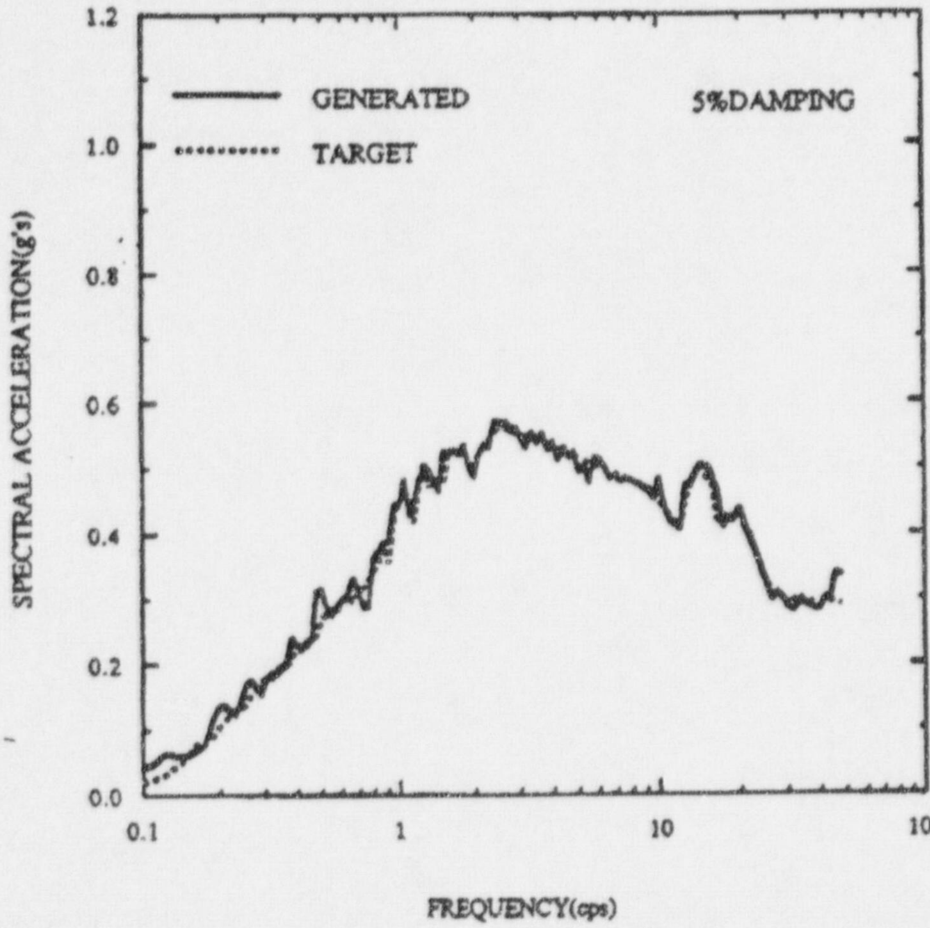
AP600:CASE A
RESPONSE SPECTRA AT 40'
DUE TO H2 MOTION AT SURFACE

⊙ Fig. D4
6



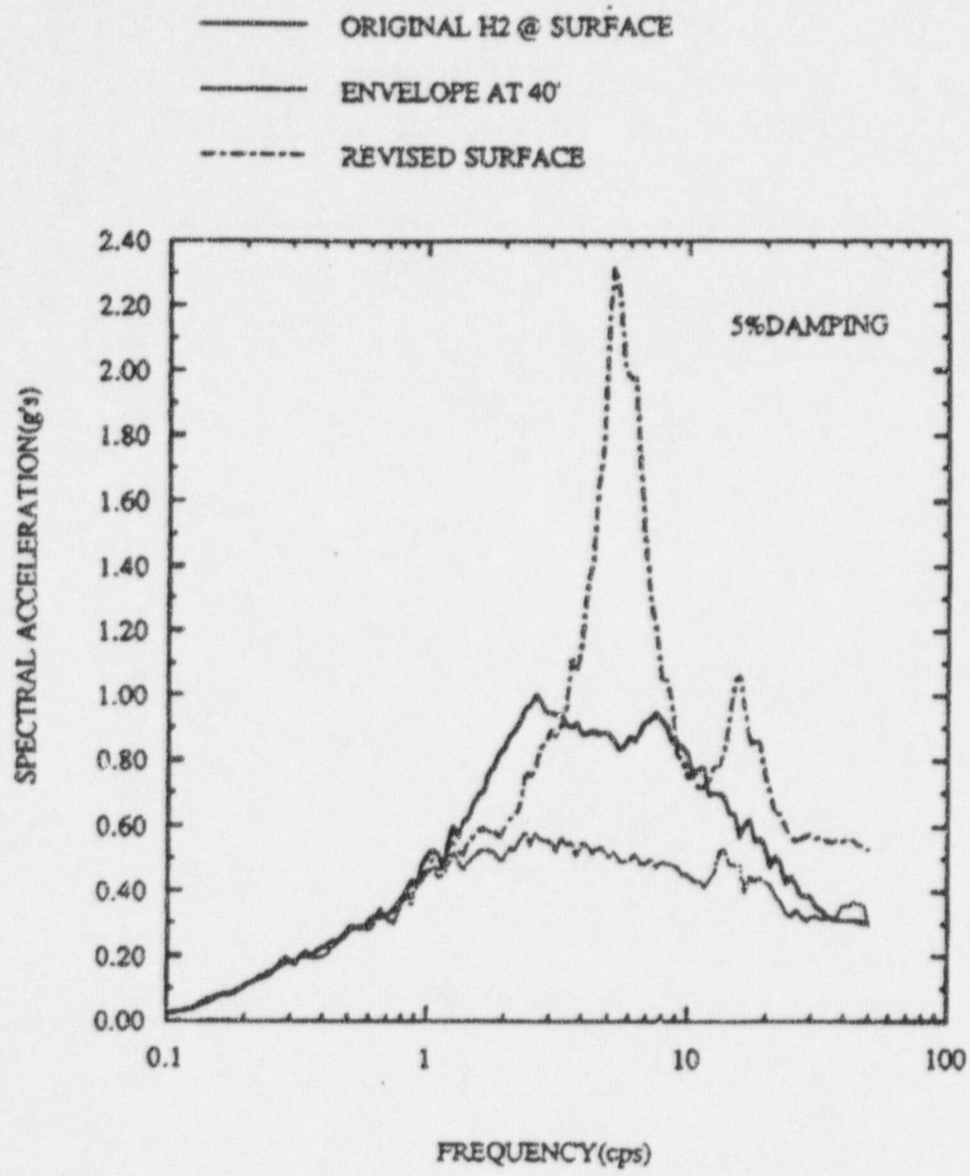
AP600:CASE A
RESPONSE SPECTRA @40'FOR
THE REVISED H2 MOTION&
MINIMUM RESPONSE SPECTRA ENVELOPEFUNCTION

Fg D5
6



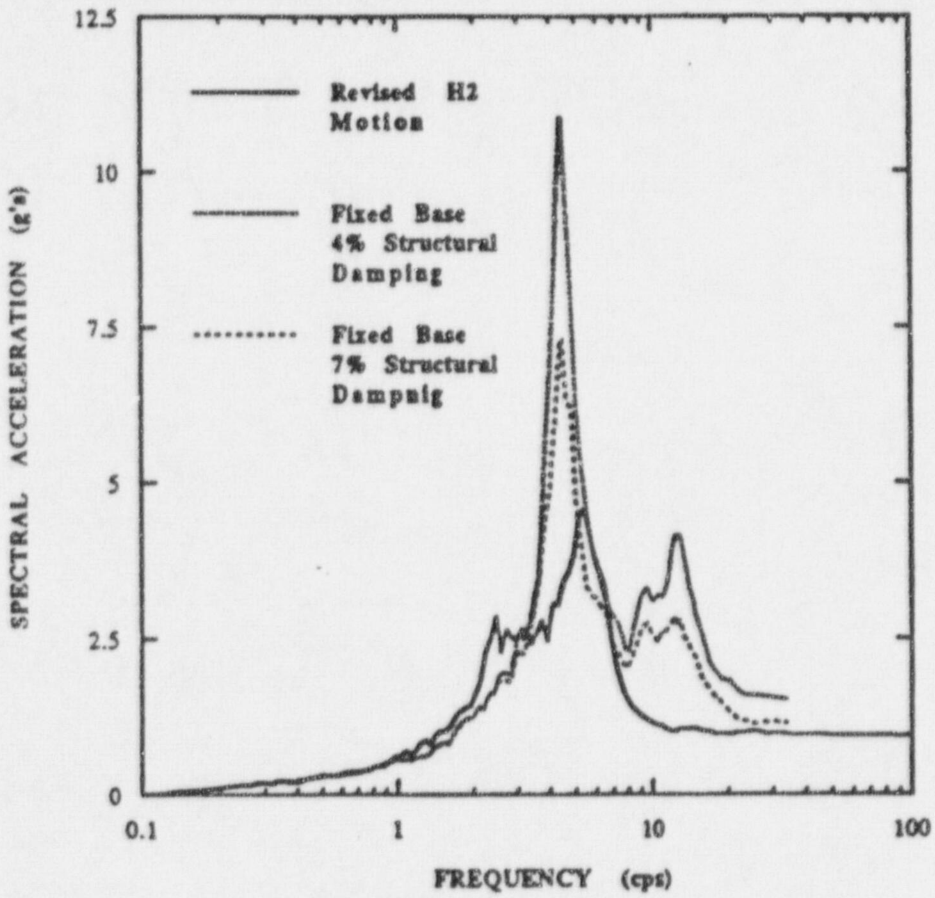
AP600:CASE A
TARGET AND GENERATED SPECTRA @40' OF
THE REVISED H2 MOTION

Fig D6



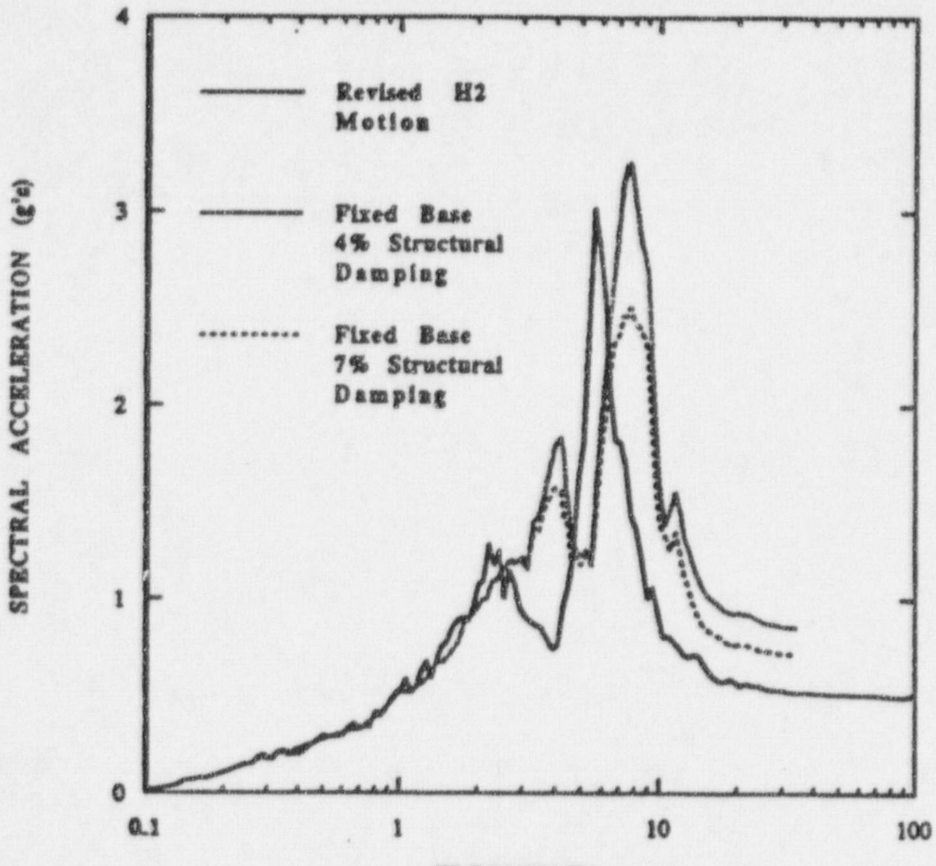
AP600: REVISED CASE A

Fig. #7



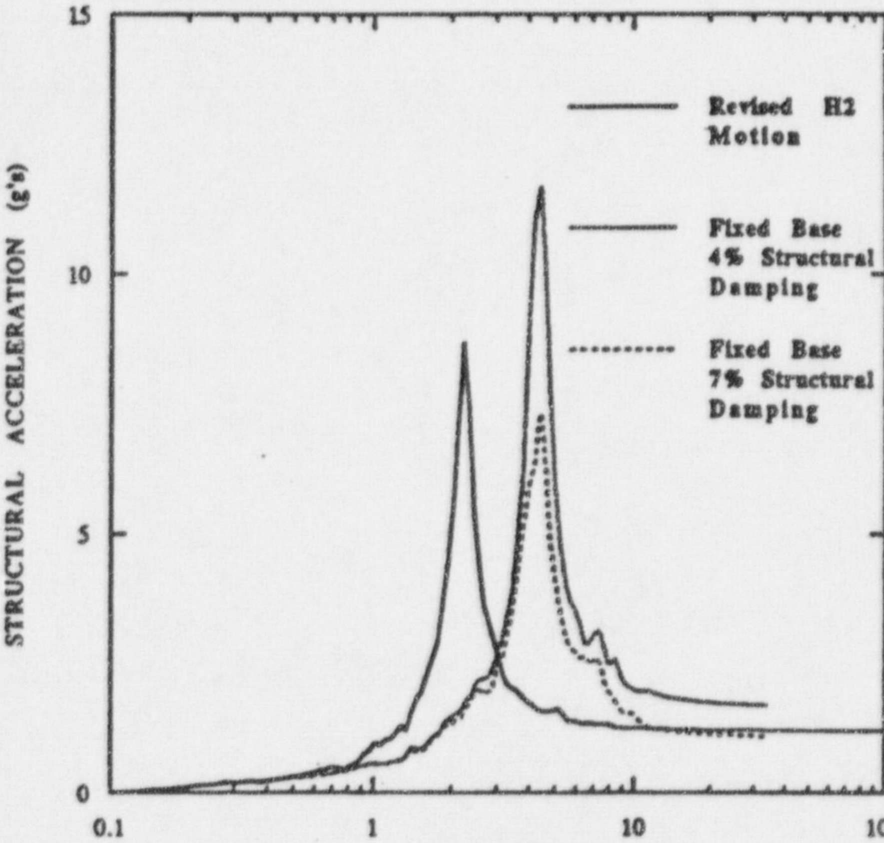
AP600 RESPONSE SPECTRA (5% DAMPING)
COMPARISON BETWEEN H2 MOTION AND
REVISED H2 MOTION FOR CASE A
SHIELD BUILDING ROOF
NODE 3016 (x=0.0, eL=307.3')

D8
Fig 8



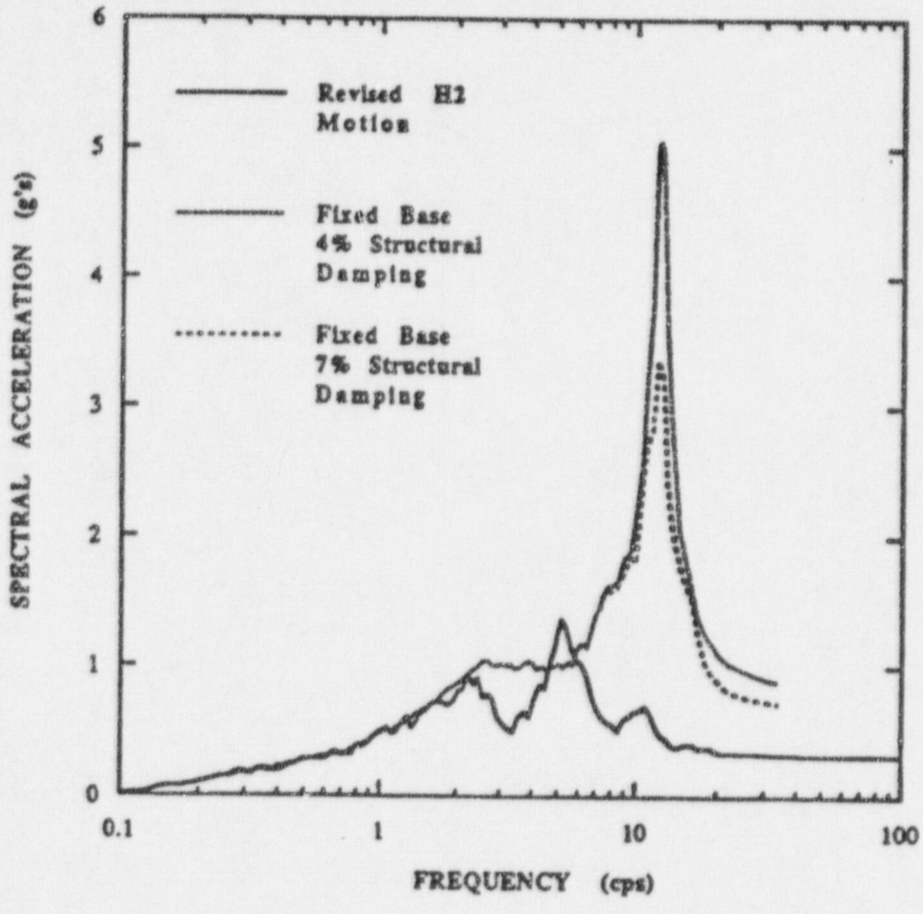
AP600 RESPONSE SPECTRA (5% DAMPING)
COMPARISON BETWEEN H2 MOTION AND
REVISED H2 MOTION FOR CASE A
STEEL CONTAINMENT VESSEL
NODE 3110 (x=0.0, el.=205.3')

Fig D4



AP600 RESPONSE SPECTRA (5% DAMPING)
COMPARISON BETWEEN H2 MOTION AND
REVISED H2 MOTION FOR CASE A
POLAR CRANE
NODE 3116 (x=0.0, el.=205.3')

Fig D10



AP600 RESPONSE SPECTRA (5% DAMPING)
COMPARISON BETWEEN H2 MOTION AND
REVISED H2 MOTION FOR CASE A
CONTAINMENT INTERNAL STUCTURE
NODE 3204 (x=14.5, el.=135.3')

D11
Fig 8c

Appendix E

3-D SASSI Analysis

E1. INTRODUCTION

Following the completion of the 2-D SASSI computations performed by the Staff Review Team, discussions were held with the Applicant concerning the appropriate set of 3-D SASSI analyses that were required to be performed to provide confidence in the adequacy of the design responses for the nuclear island (NI). On the basis of the Staff's parametric 2-D calculations, and discussions among the Staff Review Team and with the Applicant, it was decided that one additional confirmatory 3-D SASSI computation should be performed, using an assumed site condition having a uniform shear wave velocity of 3,500 fps. This site category can be considered a soft rock case and would fall between the soft to medium soil and hard rock cases proposed by the Applicant.

E2. RESULTS OF 3-D SASSI ANALYSIS

Using the new 3-D structural model provided by the Applicant, which contained some minor variations from the models previously used in the studies, together with the soft rock site condition, three separate accesses were made to the 3-D SASSI model, one for each direction of motion using the H1, H2 and V criteria motions defined at the ground surface. From these runs, response spectra were generated at several key locations mutually agreed upon with the Applicant, these being Nodes 3004 (Control Room @ elevation 117.5'), 3016 (Shield Building Roof @ elevation 307'), 3110 (Polar Crane Support @ elevation 205') and 3115 (Top of Containment Vessel @ elevation 256').

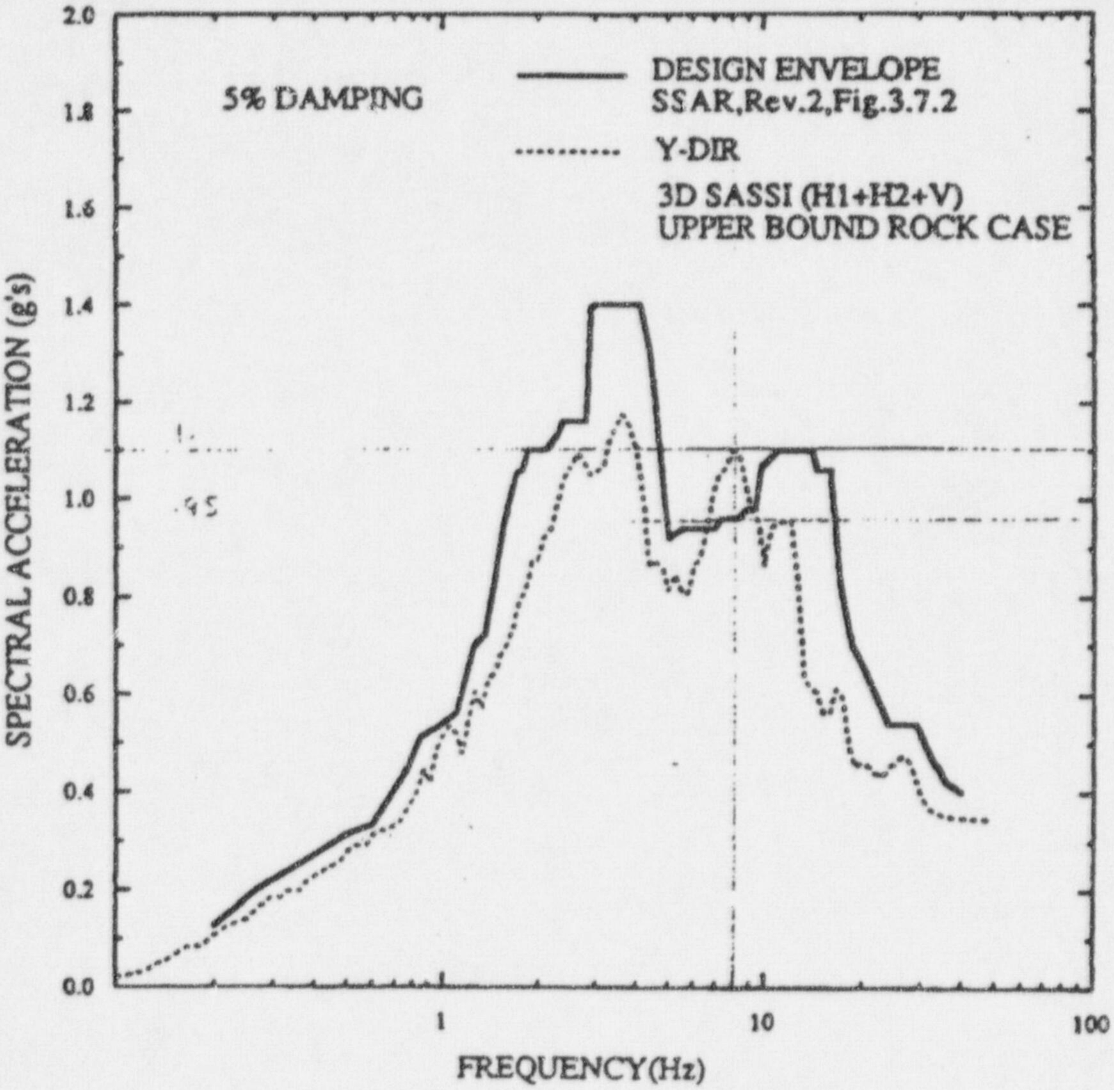
Both 2% and 5% response spectra were generated at these locations and compared with the design spectra proposed by the Applicant in the SSAR which were described as being suitably amplified and widened. Comparison of some typical results are shown in Figures E1 through E3. It can be noted that the computed spectra generated from the output for the soft rock case exceed the design spectra defined by the Applicant. Therefore, it is concluded that the design did not satisfy the requirements specified in the SSAR. Recently, the Applicant has performed additional computations and developed modified design spectra. A comparison of the new spectra for the same nodes with the output from the soft rock case are shown in Figures E4 through E6 and indicate that the new design spectra suitably envelope the computed responses.

E3. CONCLUSIONS

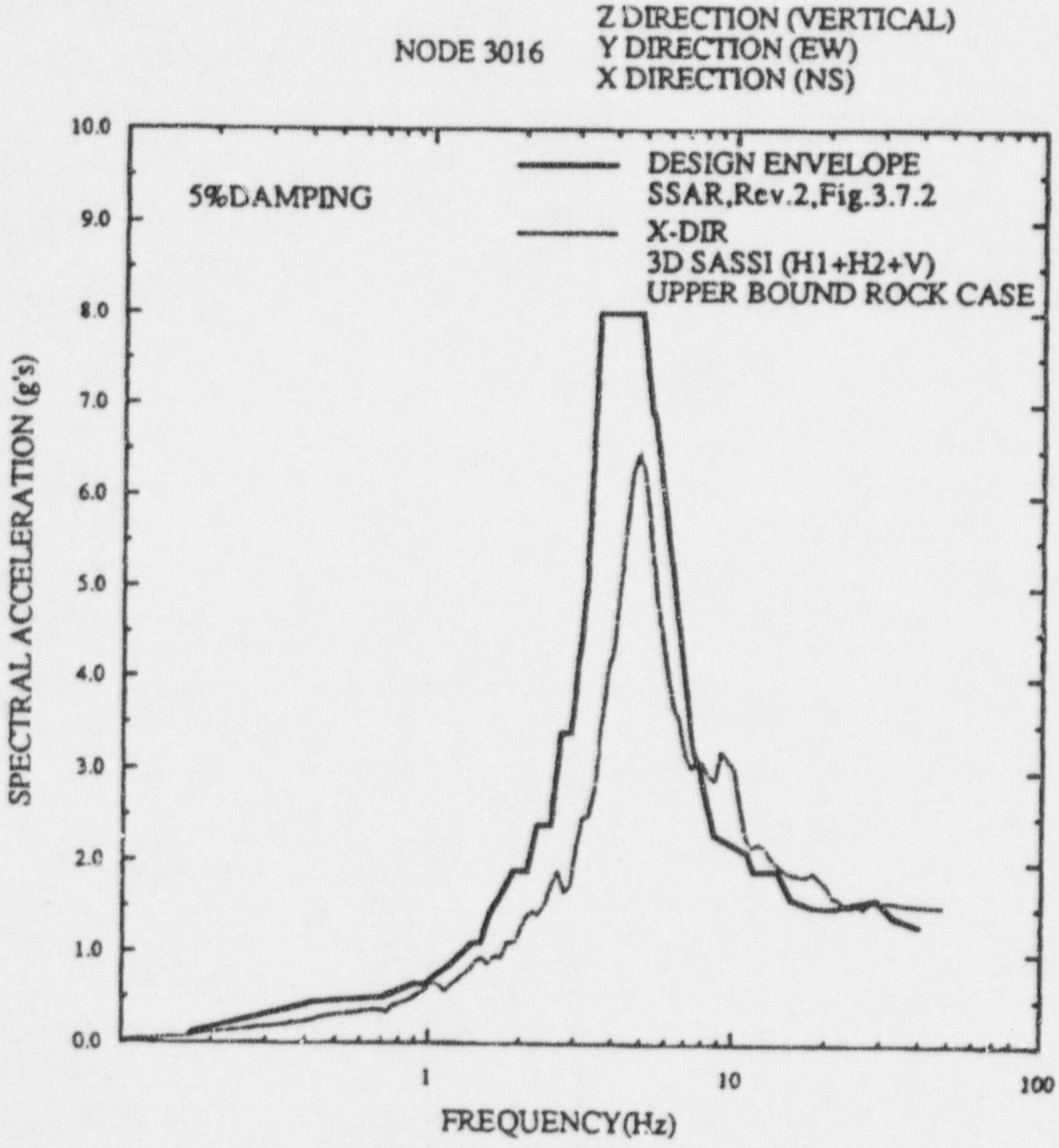
On the basis of these results, the modified design response spectra appear appropriate for design of internal equipment and components. The issue of design of the exterior walls and basemat of the nuclear island remains to be evaluated.

NODE 3004

Z DIRECTION (VERTICAL)
Y DIRECTION (EW)
X DIRECTION (NS)



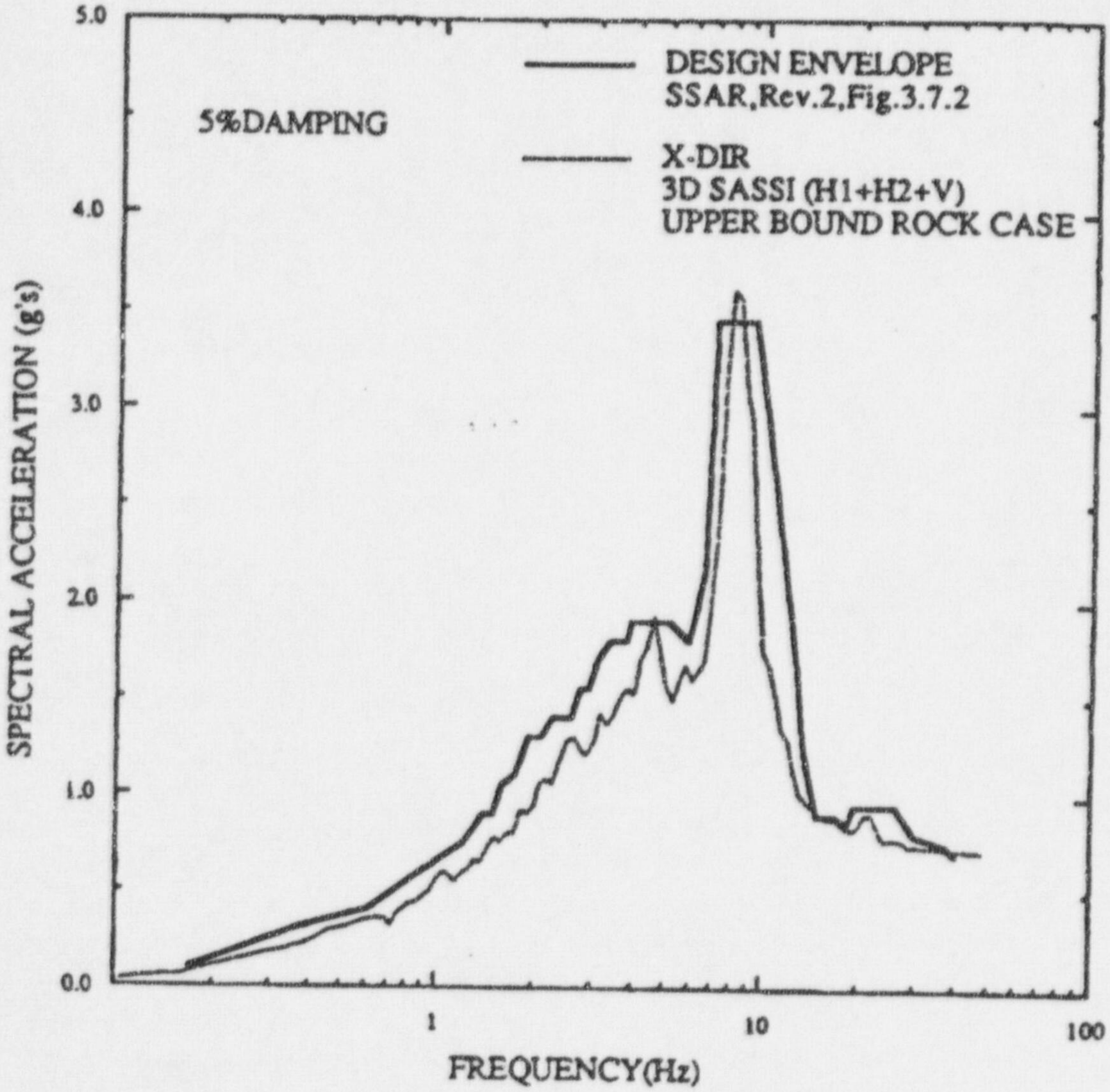
RESPONSE SPECTRA@NODE 3004
AP6003D MOTION ALGEBRAIC SUM



RESPONSE SPECTRA@NODE 3016
AP6003D MOTION ALGEBRAIC SUM

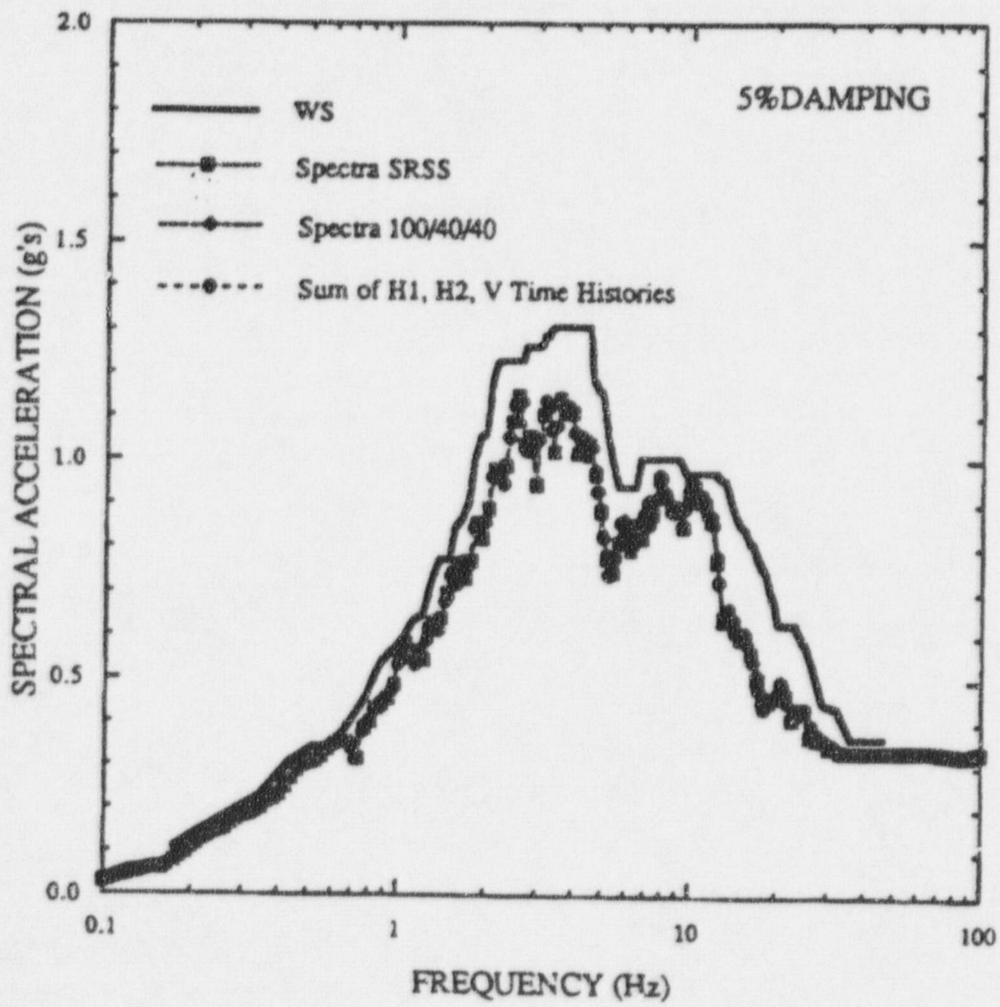
NODE 3110

Z DIRECTION (VERTICAL)
Y DIRECTION (EW)
X DIRECTION (NS)



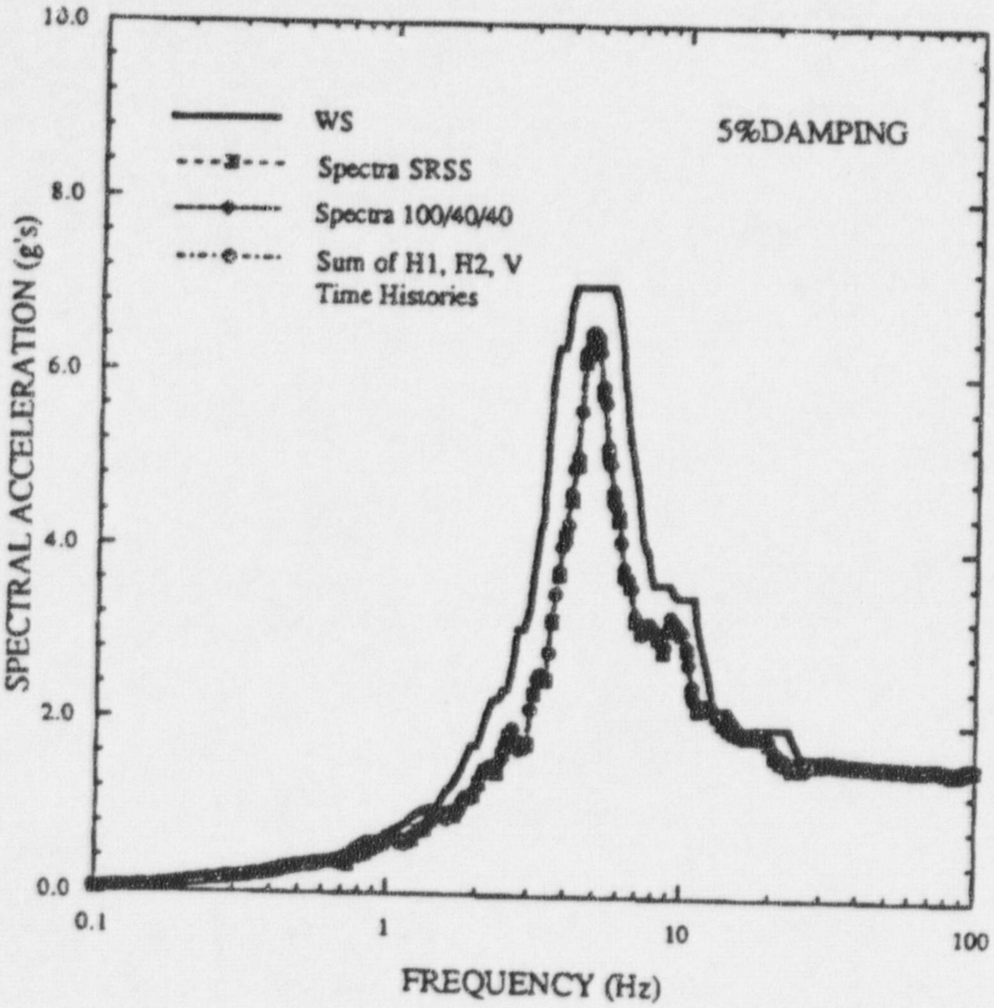
RESPONSE SPECTRA@NODE 3110
AP6003D MOTION ALGEBRAIC SUM

F 3

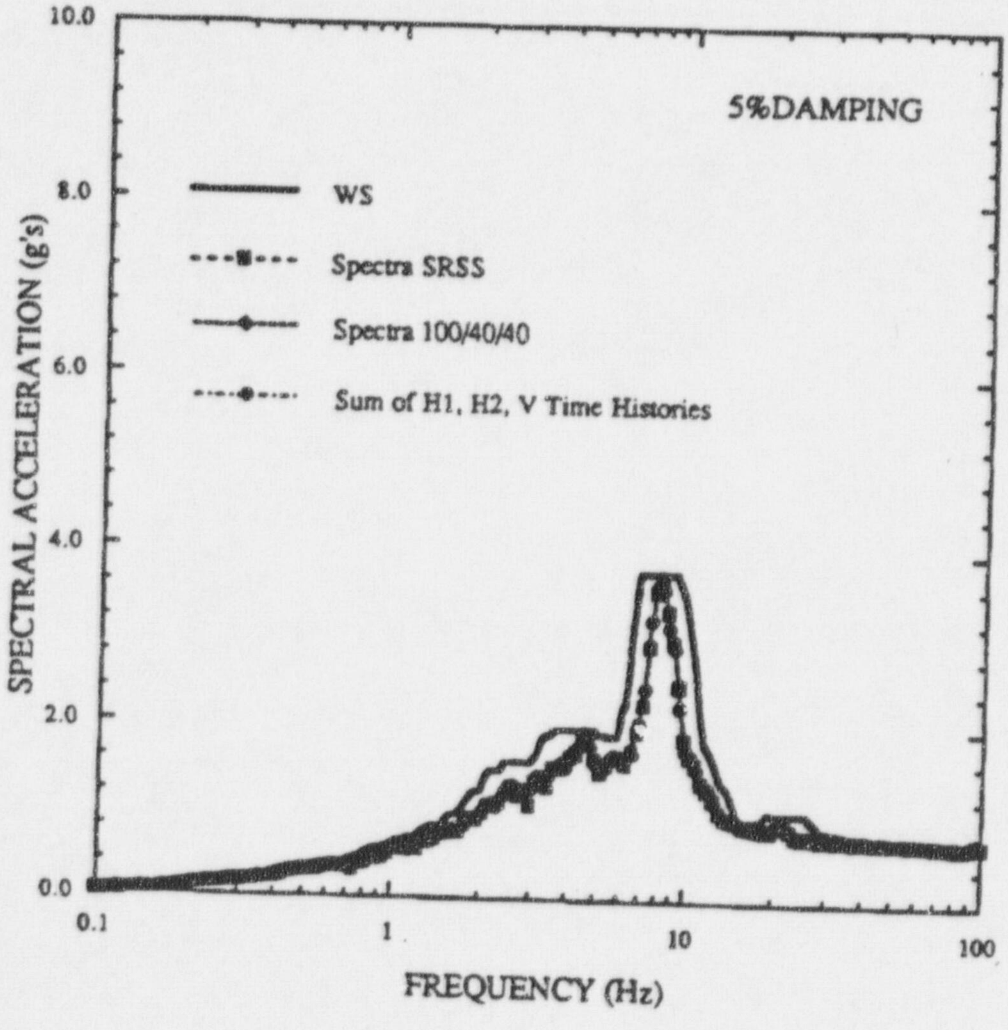


RESPONSE SPECTRA @NODE 3004, X-DIRECTION

E4



RESPONSE SPECTRA @NODE 3016, X-DIRECTION



RESPONSE SPECTRA @NODE 3110, X-DIRECTION

E6