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Your ref: Project Number 740
Our ref: DCP/NRC1942

June 15, 2007

Subject: AP1000 COL Response to Request for Additional Information (TR #03)

In support of Combined License application pre-application activities, Westinghouse is submitting revised responses to the NRC requests for additional information (RAI) on AP1000 Standard Combined License Technical Report 03, APP-GW-S2R-010, Rev. 0, Extension of NI Structures Seismic Analysis to Soil Sites. These RAI responses are submitted as part of the NuStart Bellefonte COL Project (NRC Project Number 740). The information included in the responses is generic and is expected to apply to all COL applications referencing the AP1000 Design Certification.

Revised responses are provided for requests TR03-2, TR03-5, TR03-13, TR03-16, TR03-17, TR03-20, TR03-21, TR03-26, TR03-27, TR03-28, TR03-30, and TR03-32 transmitted in NRC letter dated December 5, 2006 from Steven D. Bloom to Andrea Sterdis, Subject: Westinghouse AP1000 Combined License (COL) Pre-application Technical Report 03 – Request for Additional Information (TAC No. MD2358).

Pursuant to 10 CFR 50.30(b), the revised responses to requests for additional information on Technical Report 03 are submitted as Enclosure 1 under the attached Oath of Affirmation.

Questions or requests for additional information related to the content and preparation of this response should be directed to Westinghouse. Please send copies of such questions or requests to the prospective applicants for combined licenses referencing the AP1000 Design Certification. A representative for each applicant is included on the cc: list of this letter.

Very truly yours,

D.A. Lindgren /FOR

A. Sterdis, Manager
Licensing and Customer Interface
Regulatory Affairs and Standardization

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D079
D063
NRO

/Attachment

1. "Oath of Affirmation," dated June 15, 2007

/Enclosure

1. Revised responses to Requests for Additional Information on Technical Report No. 03

cc:	D. Jaffe	-	U.S. NRC	1E	1A
	E. McKenna	-	U.S. NRC	1E	1A
	G. Curtis	-	TVA	1E	1A
	P. Grendys	-	Westinghouse	1E	1A
	P. Hastings	-	Duke Power	1E	1A
	C. Ionescu	-	Progress Energy	1E	1A
	D. Lindgren	-	Westinghouse	1E	1A
	A. Monroe	-	SCANA	1E	1A
	M. Moran	-	Florida Power & Light	1E	1A
	C. Pierce	-	Southern Company	1E	1A
	E. Schmiech	-	Westinghouse	1E	1A
	G. Zinke	-	NuStart/Entergy	1E	1A
	B. LaPay	-	Westinghouse	1E	1A

ATTACHMENT 1

"Oath of Affirmation"

ATTACHMENT 1

UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

In the Matter of:)

NuStart Bellefonte COL Project)

NRC Project Number 740)

APPLICATION FOR REVIEW OF
"AP1000 GENERAL COMBINED LICENSE INFORMATION"
FOR COL APPLICATION PRE-APPLICATION REVIEW

B. W. Bevilacqua, being duly sworn, states that he is Vice President, New Plants Engineering, for Westinghouse Electric Company; that he is authorized on the part of said company to sign and file with the Nuclear Regulatory Commission this document; that all statements made and matters set forth therein are true and correct to the best of his knowledge, information and belief.

Bruce W. Bevilacqua

B. W. Bevilacqua
Vice President
New Plants Engineering

Subscribed and sworn to
before me this 15th day
of June 2007.

COMMONWEALTH OF PENNSYLVANIA
Notarial Seal
Debra McCarthy, Notary Public
Monroeville Boro, Allegheny County
My Commission Expires Aug. 31, 2009
Member, Pennsylvania Association of Notaries

Debra McCarthy
Notary Public

ENCLOSURE 1

Revised Responses to Requests for Additional Information on Technical Report No. 03

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR03-002
Revision: 1

Question:

The last second sentence of Item 2 in Page 3 of 154 states that the walls and basemat inside containment for this model is shown in DCD Figure 3.7.2-2. Since the height of the pressurizer cubical walls was reduced, this statement is no longer valid. Clarification is needed.

Westinghouse Response:

Item 2 on page 3 of 154 is describing the ANSYS finite element shell model of the containment internal structure that was used for analysis during the hard rock licensing phase. Therefore, this statement is valid for this section of the report. It is correct that the pressurizer cubicle walls height was reduced from elevation 169' to elevation 160' and this is reflected in the models described in Section 4 of the report.

Reference:

None

Design Control Document (DCD) Revision:
None

PRA Revision:
None

Technical Report (TR) Revision:
None

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR03-005
Revision: 1

Question:

The second sentence of the third paragraph in Page 9 of 154 states that the (concrete) modulus of elasticity is reduced to 80% of its value to reduce stiffness to simulate cracking. Westinghouse is requested to clarify whether this reduced stiffness was used in both the dynamic seismic response analyses for generation of floor response spectra, and the equivalent static acceleration analyses for design of the structural members. If different stiffness assumptions were used, provide the technical basis for this decision. Also provide the technical basis for using 80%. Discuss this in relation to current industry guidance (e.g., ASCE 43-05, ASCE 4-98). Were any sensitivity studies conducted to determine the effect of varying the concrete stiffness on (1) the floor response spectra, and (2) the design of structural members?

Westinghouse Response:

The reduction to 80% is described in DCD subsection 3.7.2.3 as shown below and was reviewed during the hard rock Design Certification. This reduction reflects the observed behavior of concrete when stresses do not result in significant cracking. This reduction is applied in both the updated dynamic ANSYS analyses on hard rock sites as well as in the SASSI analyses on soil sites. The reduction is also applied in the equivalent static acceleration analyses for design of the structural members and the nuclear island basemat.

> *The finite element models of the coupled shield and auxiliary buildings, and the containment internal structures are based on the gross concrete section with the modulus based on the specified compressive strength of concrete. When the finite element or stick models of these buildings are used in time history or response spectrum dynamic analyses, the stiffness properties are reduced by a factor of 0.8 to consider the effect of cracking as recommended in Table 6-5 of FEMA 356 (Reference 5).*

Section 3.7.2.3 (page 3-81) of the FSER accepts this approach and states:

> *The use of FEMA recommendations to modify the member stiffness of the seismic model of the NI structures is consistent with current industry practice and is reasonable and acceptable.*

Reference:

5. FEMA 356, "Prestandard and Commentary for the Seismic Rehabilitation of Buildings," Federal Emergency Management Agency, November 2000.

Design Control Document (DCD) Revision:
None

PRA Revision:
None

Technical Report (TR) Revision:

Revise second paragraph of section 4.0 as follows:

It is noted that Concrete structures are modeled with linear elastic uncracked properties. However, the modulus of elasticity is reduced to 80% of its value to reduce stiffness to reflect the observed behavior of concrete when stresses do not result in significant cracking as recommended in Table 6-5 of FEMA 356.

RAI Response Number: RAI-TR03-013
Revision: 1

Question:

The first row of the first column of Table 4.2.4-1 describes the shield and auxiliary building model as "3D finite element coarse shell model of auxiliary and shield building [NI20] (including steel containment vessel, polar crane, RCL, and pressurizer). The staff's question is that should the CIS also be included in the model?

Westinghouse Response:

This RAI is similar to RAI-TR03-008 and a response is provided in the response to RAI-TR03-008.

Revision 0 of this RAI number was used to track the proposed revisions to the DCD to address the material in the seismic report and the RAIs thereon.

During the meeting in Monroeville on April 16-19, 2007, it was agreed that the proposed DCD revision provided in Revision 0 of this RAI response would not be reviewed by the NRC Staff. Westinghouse is planning to make an additional design change that will affect the results of the seismic analysis. Westinghouse is therefore withdrawing the DCD revisions identified in Revision 0 of this response. Westinghouse will issue Revision 1 to Technical Report TR03 to include the following:

- Changes to the report identified in responses to RAIs. These changes from Revision 0 of the report will be identified with a bar in the margin.
- Changes to the report resulting from updated seismic analyses. These changes from Revision 0 of the report will be identified with a bar in the margin.
- An appendix containing the latest revision of each RAI response.

Revisions will be included in Section 3.7 of DCD Rev 16 based on the material in Revision 1 of the technical report. Revisions will made to subsections 3.7.1 and 3.7.2 and a new Appendix 3G will be added.

Reference:
None

Design Control Document (DCD) Revision:
See above

PRA Revision:
None

Technical Report (TR) Revision:
See above

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR03-016
Revision: 1

Question:

The first sentence of the fourth paragraph in Page 50 of 154 states that maximum member forces are shown in Figures 4.4.1-2 through 4.4.1-5. These figures indicate that the equivalent static analysis always results in highest member forces when compared with SASSI results based on other site conditions. The staff requests Westinghouse to identify which site condition was selected to develop the equivalent static acceleration profile used to perform the equivalent static analysis.

In addition, the staff's review of the report APP-GW-GLR-009, "Containment Vessel design Adjacent to Large Penetrations," found that the containment vessel was designed for seismic loads by applying equivalent static accelerations at each elevation based on the maximum acceleration from the fixed-base NI stick models tabulated in DCD Table 3.7.2-6. Based on the ZPAs shown in Table 4.4.1-2 and seismic loads shown in Figures 4.4.1-2 through 4.4.1-7, Westinghouse should demonstrate that the seismic loads used for the containment vessel design are the worst loading condition.

Westinghouse Response:

The equivalent static acceleration profile used in the parametric studies described in subsection 4.4.1.2 with member force results designated as EQ in Figures 4.4.1-2 to 4.4.1-5 is based on the maximum acceleration values obtained from the 2D ANSYS time history modal analyses of the same stick model on hard rock described in Section 7.1 of the report. These ANSYS analyses used the same model as the 2D SASSI analyses. The accelerations in Table 4.4.1-2, the member forces shown in Figures 4.4.1-2 to 4.4.1-5, and the floor response spectra in Appendix D are all from the 2D parametric analyses and are evaluated in the selection of the design soil cases as described in the fourth paragraph on page 50 of 154.

The equivalent static acceleration profiles specified for the design of the nuclear island structures are described in subsection 6.2 of the technical report. The accelerations given in Table 6.2-4 for the containment vessel are the envelope of the maximum accelerations obtained from the updated nuclear island analyses for the four design soil cases described in the technical report. The design analyses of the containment vessel were initially performed during the hard rock design certification using equivalent static accelerations tabulated in DCD Table 3.7.2-6 (based on fixed base stick models). The reconciliation of the design of the containment vessel for seismic input for soil sites is described in report APP-GW-GLR-005, "Containment Vessel Design Adjacent to Large Penetrations," (Reference 1). As discussed in the April 16-20 meeting, this reconciliation should be considered as part of the review of Reference 1.

Reference:

1. APP-GW-GLR-005, "Containment Vessel Design Adjacent to Large Penetrations," Rev. 0.

Design Control Document (DCD) Revision:
None

PRA Revision:
None

Technical Report (TR) Revision:

Revise third paragraph on page 50 of 154 as follows:

Maximum member forces from the 2D SASSI analyses are shown in Figures 4.4.1-2 to 4.4.1-5. These figures also show member forces for an equivalent static acceleration profile (EQ) based on the maximum acceleration values obtained from 2D ANSYS time history modal analyses of the same stick model on hard rock as described in Section 7.1 of the report. These 2D ANSYS analyses used the same model as the 2D SASSI analyses. Floor response spectra from the 2D SASSI analyses associated with nodes 41, 120, 310, 411 and 535 for the six AP1000 soil cases are shown in Appendix D, Figures D-1 to D-10.

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR03-017
Revision: 1

Question:

Wording in DCD Table 2-1 "Site Parameters" indicates that best estimate low-strain shear wave velocity shall be greater than 1,000 fps and that variability across the site shall be less than 100 fps (10%). It is presumed that this DCD commitment is based on SASSI results for a uniform half-space below the plant basemat. Westinghouse is requested to a include statement on maximum acceptable change in velocity profile within a depth equal to the width of the basemat in the definition of "Site Parameters."

Westinghouse Response:

The variability in shear wave velocity of 10% across the site was established to limit variability in the soil pressures used in design of the basemat. This was based on AP600 basemat analyses. The analyses for the AP1000 are described in the "Nuclear Island Basemat and Foundation" report (Reference 1) submitted in October 2006. The variability specified for the AP600 is retained for the AP1000. Section 5 of Reference 1 shows proposed revisions to DCD Chapter 2. Subsection 2.5.4.5.3, Site Foundation Material Evaluation Criteria, describes the evaluation of the variability in each layer. If the shear wave velocity at the foundation level varies in plan, the minimum value must satisfy the requirement that the best estimate low-strain shear wave velocity shall be greater than 1,000 fps.

There is no limit on the maximum acceptable change in velocity profile within a depth equal to the width of the basemat. Four design soil profiles are analyzed. These are similar to the four cases analyzed for the AP600. For the AP600 a number of soil profiles were included in parametric studies including soil with various depths to rock and a "stepped" profile. Responses on the nuclear island for these cases were bounded by the four design soil profiles. Further discussion is given related to the applicability of these studies to the AP1000 plant in the responses to RAI-TR03-014 and RAI-TR03-015.

Reference:

1. APP-GW-GLR-044 Revision 0, "Nuclear Island Basemat and Foundation", October, 2006.

Design Control Document (DCD) Revision:
None

PRA Revision:
None

Technical Report (TR) Revision:
None

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR03-020
Revision: 1

Question:

a. Comparison of Figure 6.1-4 to Figure 6.1-6, and comparison of the stick model results to the FE model results at the top of the SCV in Figure 6.1-6, raises a question about the connectivity of the bottom of the SCV stick to the CIS FE model, at node 130401. The staff requests Westinghouse to provide a detailed technical explanation for the following:

   a. Why is the x-direction spectral peak at node 130412 reduced by 1/3 (approx. 4.2 vs. 6.3), while the y-direction spectral peak at node 130412 is only reduced by 1/11 (approx. 6.6 vs. 7.2)? What mechanism has caused the ratio of y to x to change from 1.09 for the stick model to 1.57 for the FE model?

   b. Why does the vertical spectrum comparison in Figure 6.1-6 show (1) an increase in spectral peak for the FE model, compared to the stick model, and (2) a significant shift in the frequency of the peak?

Westinghouse Response:

The connection of the bottom of the SCV stick to the CIS finite element model at node 130401 was reviewed. The connectivity, via constraint equations, is shown in Figure RAI-TR03-020-1. As seen the connectivity (identified as "Rev 4 model") is not symmetric around the SCV model. This connectivity was changed by adding six more connections so that it is symmetric. It is identified as "All Nodes" and is shown in Figure RAI-TR03-020-2. The vertical motion for the CIS interface nodes is tied rigidly to the vertical motion and rotation about the x-axis and y-axis of Node 130401 at the base of the SCV stick model. The tangential motion is tied rigidly to the horizontal motion and rotation about the z-axis of the same node. No constraints were placed for the radial direction of the CIS.

An additional case was considered that added constraints in the radial motion of the CIS to the SCV. This additional case is titled "Full Connection". The SCV bottom connectivity is the same as the "all nodes" case shown in Figure RAI-TR03-020-2.

Time history fixed base analyses were performed for each case on the nuclear island NI10 model. Response spectra shown in Figures RAI-TR03-020-3 to RAI-TR03-020-5 were generated on the containment vessel stick at the elevation of the polar crane girder (elevation 224', node 130412) for each case and compared to the spectra obtained from the Nuclear Island Rev 4 model. As seen from these spectra, the results for the "All Nodes" and "Fully Constrained" cases are almost identical. The Rev 4 model with the unsymmetrical constraint equations has minor differences.

It can be concluded from this study that the connectivity in the Rev 4 model is adequate. The "All Nodes" connectivity is better and also permits radial deformation of the CIS at the interface to the containment vessel.

Provided below are the responses for parts a and b of the RAI.

a. The reduction in response of the containment vessel in the x-direction is due primarily to the change in interaction between the polar crane and the containment vessel. The model of the polar crane was updated to reflect additional definition of the polar crane wheel assemblies. The fundamental mode of the old model in the x-direction has a frequency of 5.387 hertz with an effective mass of 175.274 kips.sec^2/ft. The update in the polar crane model resulted in two x-direction modes in the coupled model as follows:

- frequency of 5.09 hertz with an effective mass of 151.50 kips.sec^2/ft.
- frequency of 8.11 hertz with an effective mass of 32.01 kips.sec^2/ft.

The effect of this change in frequency is shown in Figures RAI-TR03-020-6 to RAI-TR03-020-8. These results are for analyses of the SCV stick and PC fixed at the bottom of the containment vessel stick using the AP1000 ground motion. The change in the updated polar crane model discussed above is primarily in the x- direction, along the axis of the polar crane that is parked in the north-south direction, so there is little effect on the Y and Z direction response. The peak response in the X- direction reduces by ~20% from 5.0g to 3.9g.

b. Figure RAI-TR03-020-8 shows that the stick model of the steel containment vessel and polar crane has two significant frequencies in the vertical direction. The mode at 16.4 Hz has an effective mass of 166.3 kips.sec^2/ft.and the mode at 17.5 Hz has an effective mass of 13.3 kips.sec^2/ft. The response shown in Figure RAI-TR03-020-5 matches that of the stick model at the first peak. The second peak in the stick model has much lower effective mass and is attenuated in the more detailed models (NI10 or NI20). This is the effect of the finite element model of the nuclear island. The shell models of the nuclear island provide a more realistic response of the Nuclear Island in the vertical direction than the stick models.

The evaluations have shown that the seismic response is sensitive to the configuration of the polar crane. This will be reconciled using as-procured crane data in accordance with DCD subsection 3.7.5.4 which is shown below.

3.7.5.4 Reconciliation of Seismic Analyses of Nuclear Island Structures

The Combined License applicant will reconcile the seismic analyses described in subsection 3.7.2 for detail design changes at rock sites such as those due to as-procured equipment information. Deviations are acceptable based on an evaluation consistent with the methods and procedure of Section 3.7 provided the amplitude of the seismic floor response spectra including the effect due to these deviations, do not exceed the design basis floor response spectra by more than 10 percent.

Due to the sensitivity of the response to the crane properties, the floor response spectra specified for design of piping and miscellaneous items attached to the containment vessel will conservatively envelope the results in the two horizontal directions. The horizontal spectra in the X and Y directions will be enveloped and the resulting envelope specified for use in two orthogonal directions. The spectra may be applied either in the X and Y directions or in the radial and tangential directions depending on the component being evaluated.

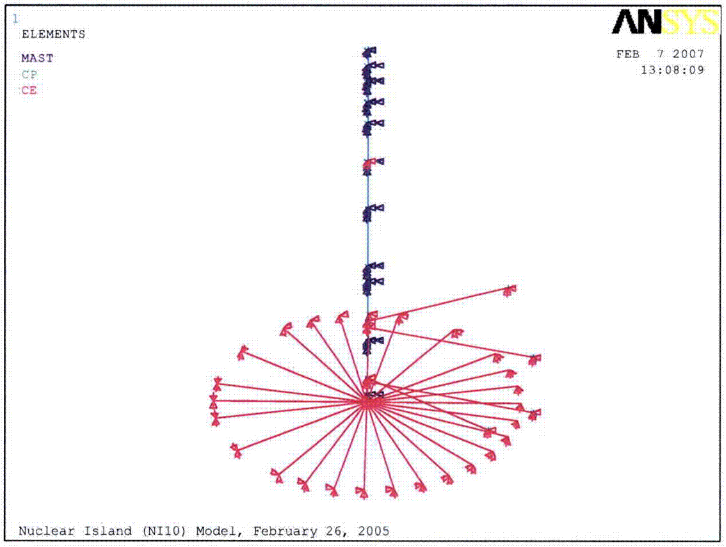


Figure RAI-TR03-020-1 - Rev 4 SCV

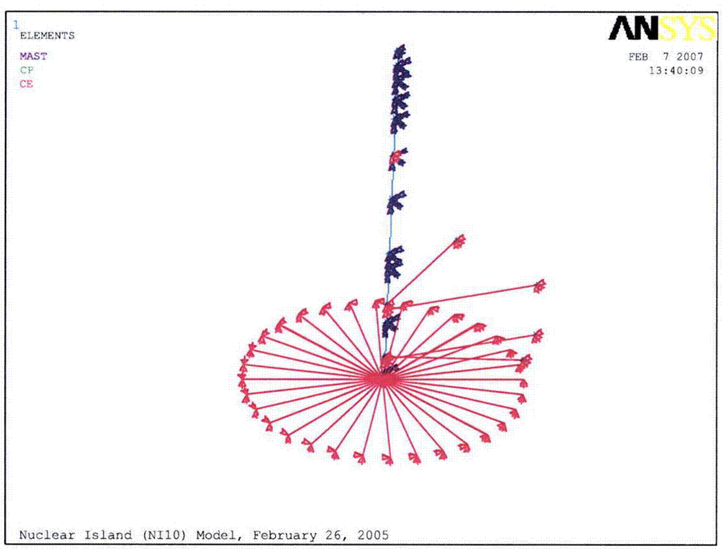


Figure RAI-TR-03-020-2 - All Nodes

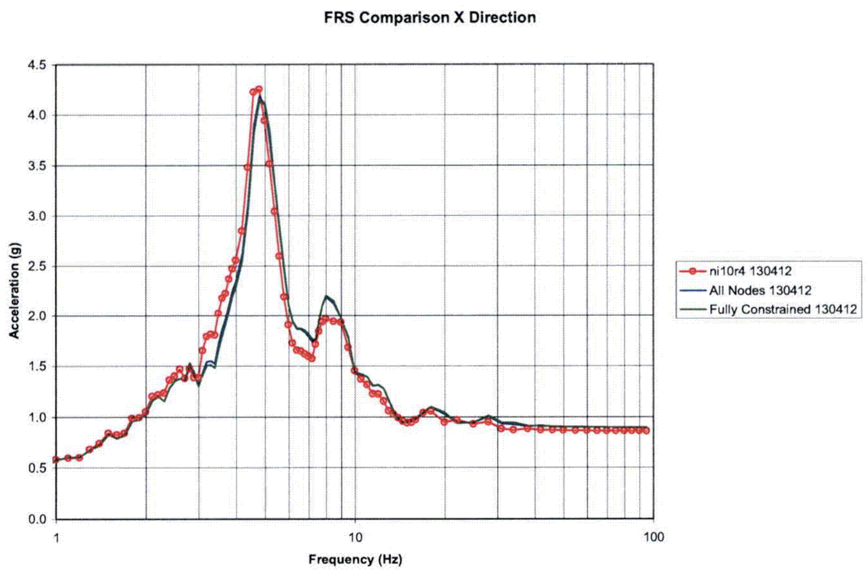


Figure RAI-TR03-020-3 – Effect of Connection - X FRS at Elevation 224'

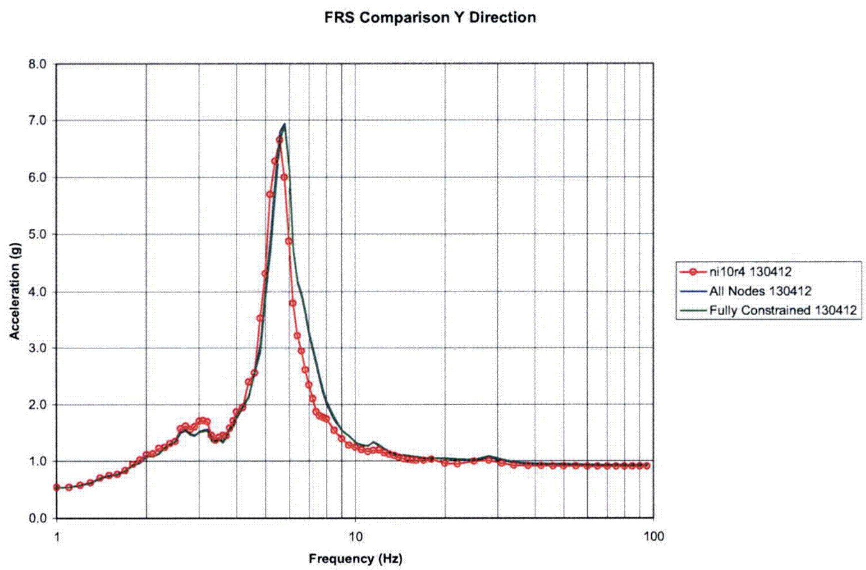


Figure RAI-TR03-020-4 – Effect of Connection - Y FRS at Elevation 224'

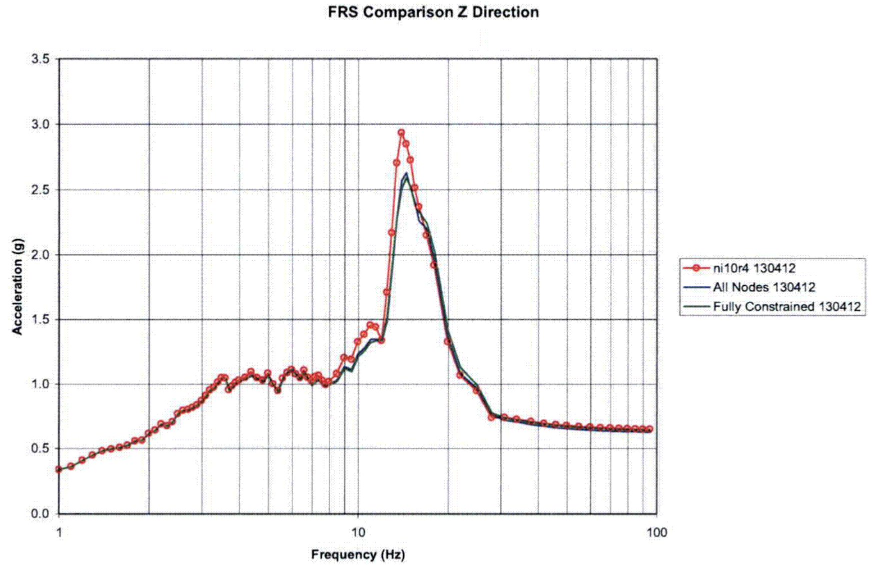


Figure RAI-TR03-020-5 – Effect of Connection - Z FRS at Elevation 224'

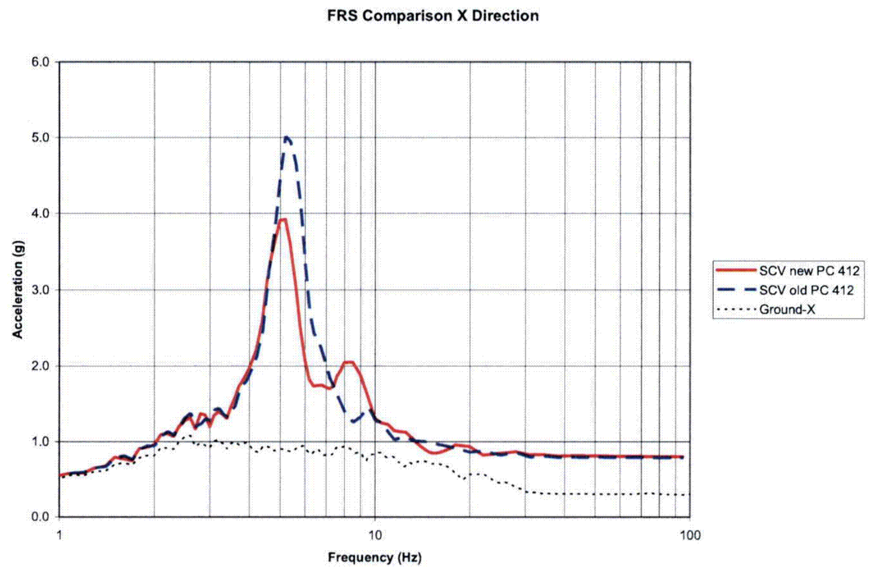


Figure RAI-TR03-020-6 – Effect of Polar Crane - X FRS at Elevation 224'

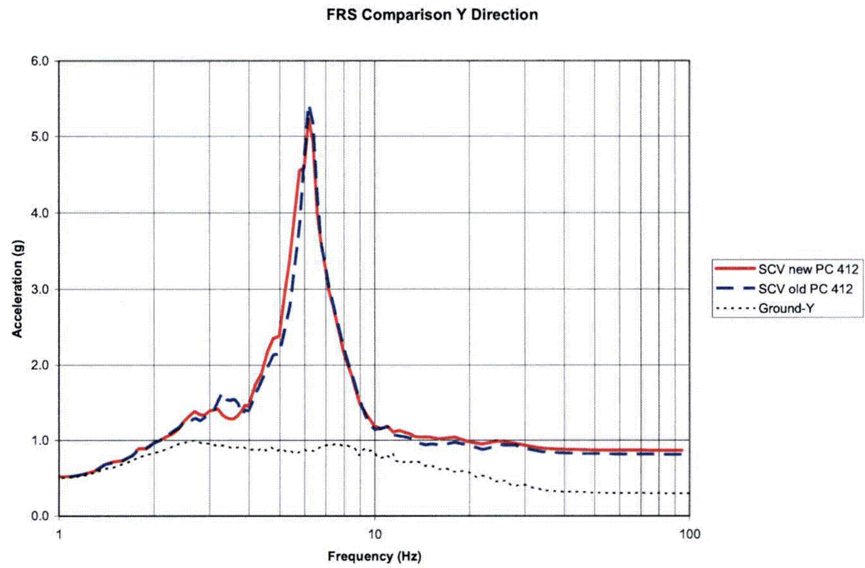


Figure RAI-TR03-020-7 – Effect of Polar Crane - Y FRS at Elevation 224'

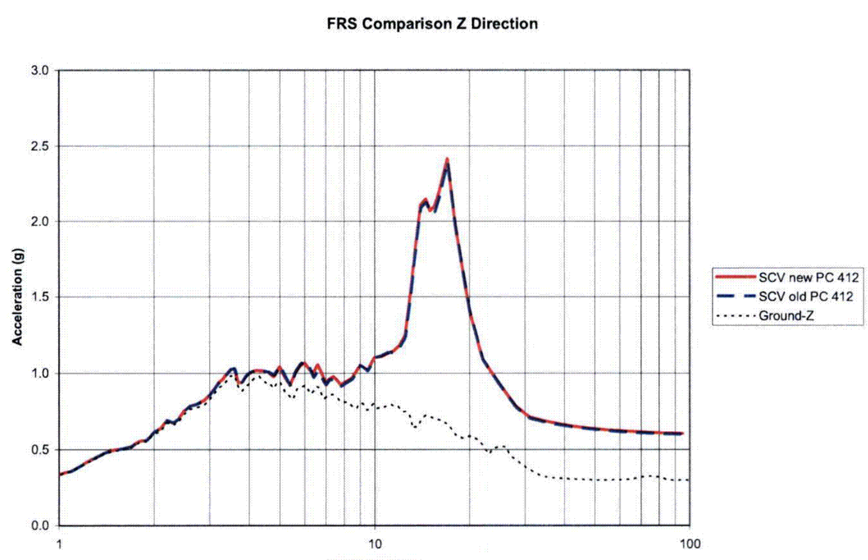


Figure RAI-TR03-020-8 – Effect of Polar Crane - Z FRS at Elevation 224'

Reference:

None

Design Control Document (DCD) Revision:
None

PRA Revision:
None

Technical Report (TR) Revision:

During the investigations of the polar crane response it was found that some of the results plotted in the comparisons for the stick model were not those from the DCD. The spectra for the stick model in Figures 6.1-1 to 6.1-6 are corrected as shown below.

Add the polar crane models and the containment vessel shell model in Table 4.2.4-1 as follows:

3D lumped mass detailed model of the polar crane	Modal analysis	ANSYS	To obtain dynamic properties. Used with 3D finite element shell model of the containment vessel
3D lumped mass simplified (single beam) model of the polar crane		ANSYS	Used in the NI10 and NI20 models
3D finite element shell model of containment vessel (1)	Mode superposition time history analysis Static analysis	ANSYS	Used with detailed polar crane model to obtain acceleration response of equipment hatch and airlocks To obtain shell stresses in vicinity of the large penetrations of the containment vessel

Note: 1) The 3D finite element shell model of the containment vessel is described in report APP-GW-GLR-005, "Containment Vessel Design Adjacent to Large Penetrations"

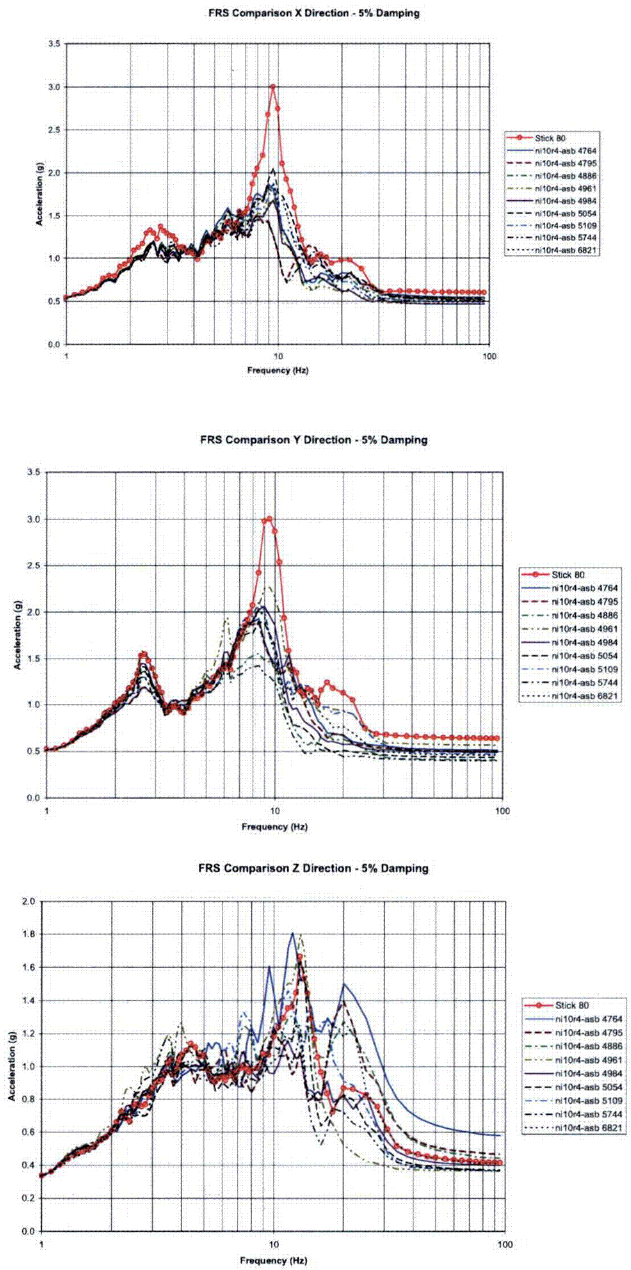


Figure 6.1- 1 - Auxiliary Building at Elevation 135 feet

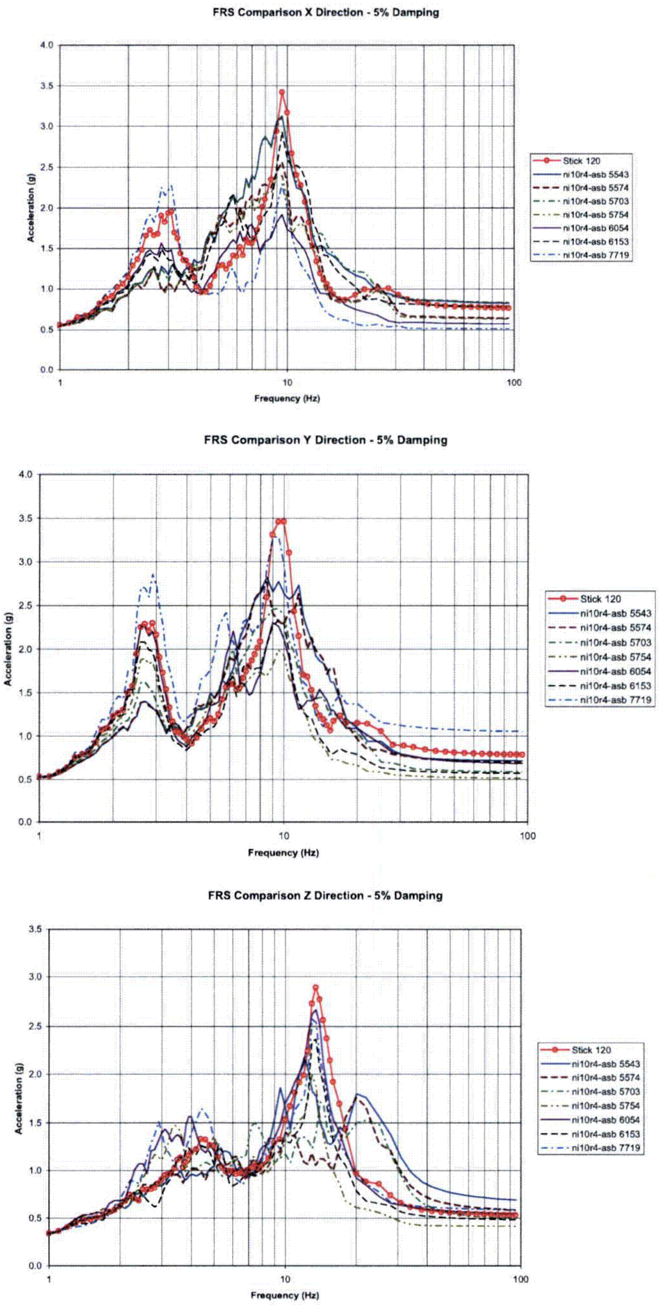


Figure 6.1- 2 - Auxiliary and Shield Building at Elevation 180 feet

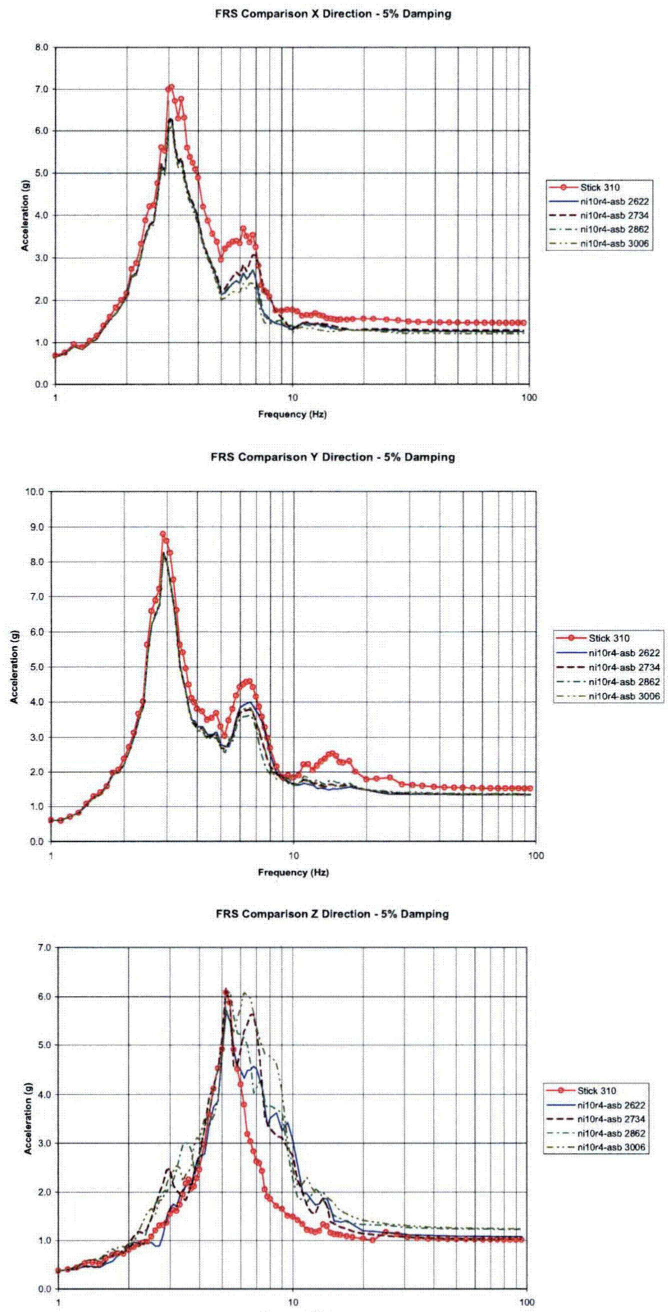


Figure 6.1- 3 - Shield Building at Elevation 333 feet

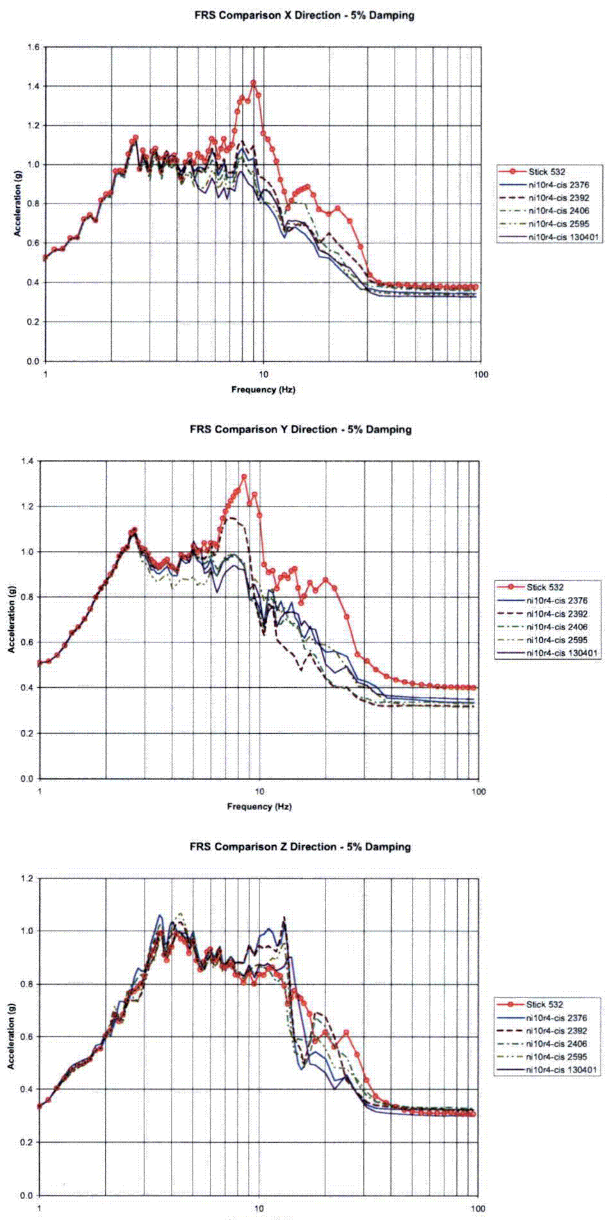


Figure 6.1-4 - CIS at Elevation 99 feet

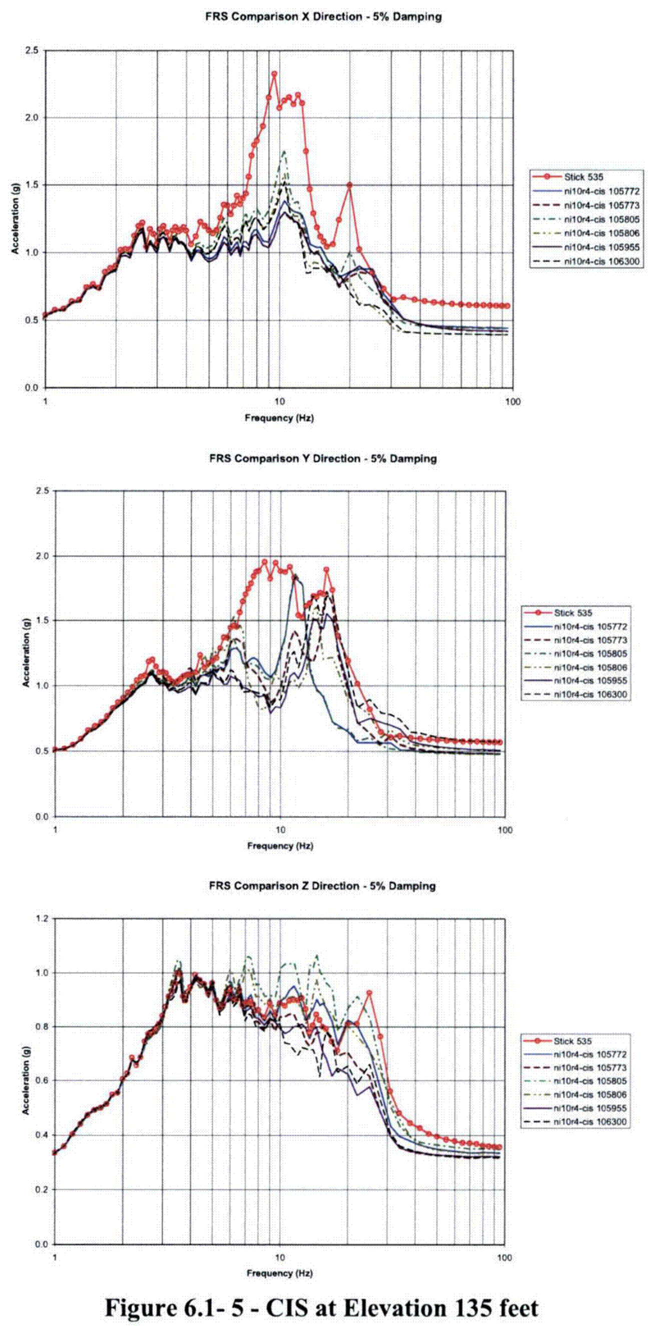


Figure 6.1- 5 - CIS at Elevation 135 feet

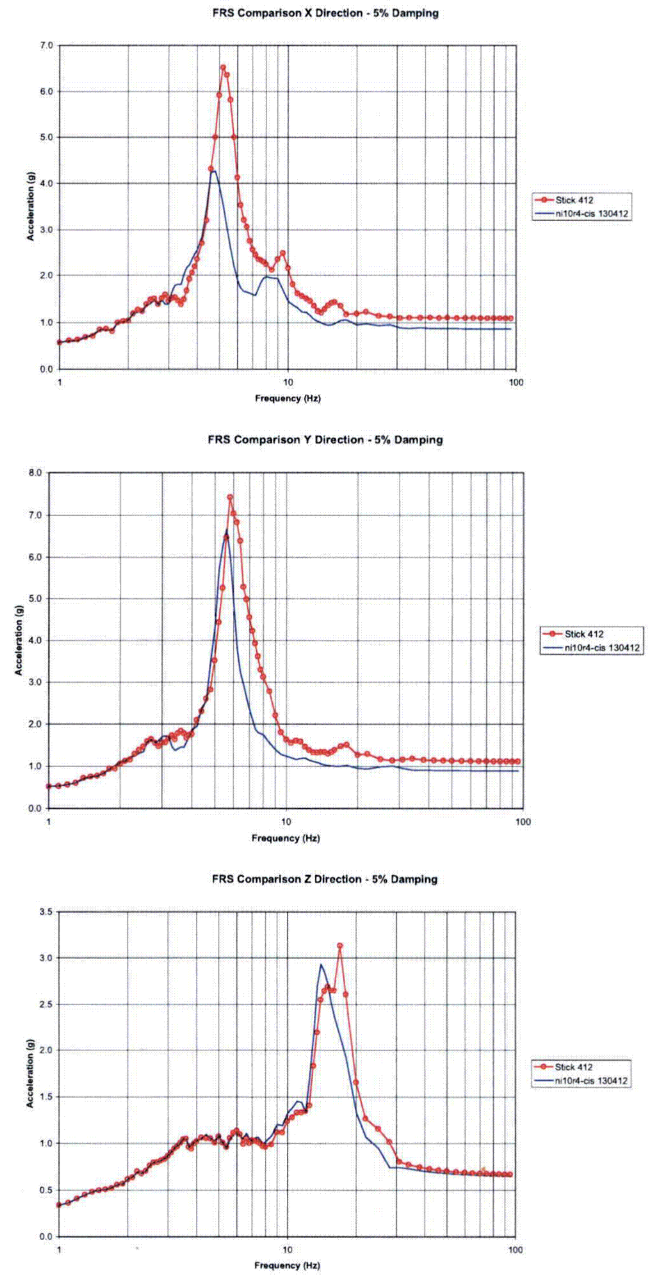


Figure 6.1-6 - SCV at Polar Crane Elevation 224 feet

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR03-021
Revision: 1

Question:

21. The staff's review of Section 6.2 identified a number of items in need of clarification or explanation. The staff requests Westinghouse to address the following:

   a. The fourth paragraph of page 91 of 154 states "In Section 6.3 a comparison of member forces obtained from seismic static and time history analyses is given." Please confirm that the reference should be to Section 6.4.

   b. The last paragraph of page 91 of 154 states "For those local flexible structures that are amplified, apply an additional acceleration to these structures equal to the difference between the average uniform amplified component accelerations and rigid body component equivalent static accelerations. These accelerations are to be considered in local design of the flexible portion of the structure but do not need to be considered in areas of the structure away from the local flexibility. They can be applied in a series of individual load vectors." It is not obvious to the staff how this methodology has been implemented, and whether the effects of increased accelerations on locally flexible structures can be ignored in areas of the structure away from the locally flexible structures. The sum total of all the flexible masses times the corresponding acceleration increments may impose non-negligible additional loads on the overall structure, in the two horizontal directions and in the vertical direction. Therefore, Westinghouse is requested to (1) describe in greater detail the implementation of this methodology, including a numerical example; and (2) provide a quantitative technical basis for the conclusion that the effects of increased accelerations on locally flexible structures can be ignored in areas of the structure away from the locally flexible structures.

   c. The top paragraph of Page 93 of 154 states "The vertical equivalent static seismic accelerations at (Shield Bldg) elevations 294.93 ft and 333.13 ft are obtained directly from the maximum time history results by taking the average of locations at opposite ends of a diameter. The vertical accelerations from the 3D finite element model at the shield building edges at these elevations are significantly influenced by the horizontal loading. If they are used for the vertical equivalent accelerations, the horizontal response would be double counted in the vertical direction." It is not obvious to the staff how this methodology has been implemented, and whether it is even appropriate. Therefore, Westinghouse is requested to submit a numerical example, based on elevation 333.13 ft of the Shield Building, to demonstrate the implementation of this methodology. In this example, please also include the vertical acceleration value that would be obtained if this methodology was NOT implemented.

d. Confirm that in Table 6.2-7, the referenced table numbers should be 6.2-3, 6.2-4, 6.2-5, and 6.2-6.

e. In Page 99, under the heading "Seismic Accelerations for Evaluation of Building Overturning," states "The dynamic response of the structure affecting overturning and basemat lift off is primarily the first mode response at about 3 hertz on hard rock. This reduces to about 2.4 hertz on soil sites as shown in the 2D ANSYS and SASSI analyses. The higher auxiliary building accelerations of Table 6.2-2 are not considered in overturning since they are from higher frequency modes greater than 2.4 hertz. Amplified response of individual walls in the Auxiliary Building and the IRWST need not be considered since they are local responses that do not effect overturning." For the overturning analysis, the staff is concerned that the methodology employed may not predict an overall moment on the basemat that envelops the maximum overturning moment for all site conditions. Westinghouse is requested to provide its technical basis for the conservatism of the methodology employed.

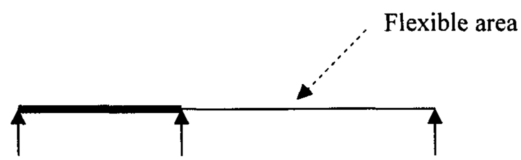
Westinghouse Response:

a. It is confirmed that the reference should be Section 6.4 and not Section 6.3.

b. The methodology being used does not neglect the effect of the locally flexible structures on the structures away from the flexible areas. The wording in the technical report is changed to avoid any confusion. The new wording is given below:

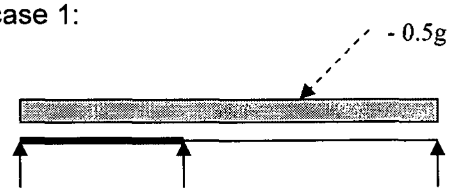
"For those local flexible structures that are amplified, apply an additional acceleration to these structures equal to the difference between the average uniform amplified component acceleration and the rigid body component equivalent static acceleration. These accelerations are to be considered in local design of the flexible portion of the structure. The effect of these additional accelerations on the seismic loadings in areas of the structure away from the local flexibility are to be considered in design."

The methodology being used allows the analyst the ease of applying the inertia loads by first applying the seismic accelerations using the accelerations of the associated structure as if it is not flexible. Then, using an additional load case, apply the incremental acceleration to the flexible portion. This procedure is shown below using a simple two span beam with three supports and one flexible area. This structure is subjected to vertical seismic excitation. The equivalent static acceleration for the beam at this elevation is equal to 0.5g vertical, and the flexible area has an average uniform vertical seismic acceleration of 0.8g.

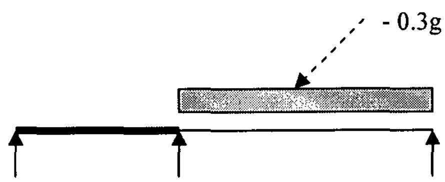


The following load cases are considered in this example with the vertical load being down. Other, cases of course would be with the vertical excitation up.

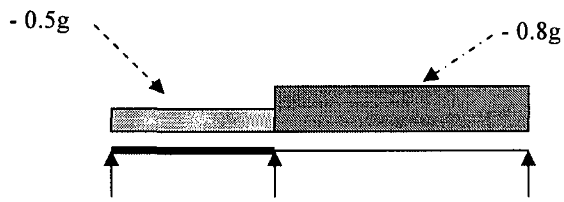
Load case 1:



Load Case 2: This load case is applied only to the flexible area. The inertia loading is the incremental portion above 0.5g that will act only on the flexible area (0.8g - 0.5g = 0.3g). It is noted that there are separate load cases for each of the flexible floors so that worst loading on the structures away from the flexible areas is obtained.



The results of the local load cases are combined absolutely with the results of the "rigid" portion in the same direction. The three directions are then combined by SRSS. The resultant member forces, that include the "rigid" portions, reflect the total seismic inertial load on the structure.



Therefore, the effects of increased accelerations on locally flexible structures are not ignored in areas of the structure away from the locally flexible structures.

The part b response will be added to Revision 2 of calculation APP-1000-S2C-070.

c. The seismic response of the shield building roof has been reviewed. It has been concluded that a seismic component associated with the rotational response of the PCCS tank should also be included in addition to the translational seismic acceleration component. The AP1000 shield building roof design is being modified as part of the evaluation of an airplane crash. The rotational response of the PCCS tank will be addressed in the redesign of the shield building roof. The response to this part of the RAI will be addressed in RAI-TR03-036.

d. It is confirmed that in Table 6.2-7, the referenced table numbers should be 6.2-3, 6.2-4, 6.2-5, and 6.2-6. This will be corrected in the report. It is noted that the values given in Table 6.2-6 have been revised. The new values will be updated as noted in the section addressing Technical Report Revisions.

e. The conservatism of the overall moment on the basemat is addressed in Section 2.6.1.2 of the Nuclear Island Basemat and Foundation report (Reference 1). This part of the RAI should be considered during the review of this report.

Reference:

1. APP-GW-GLR-044, Rev 0, “Nuclear Island Basemat and Foundation”, October, 2006

Design Control Document (DCD) Revision:
None

PRA Revision:
None

Technical Report (TR) Revision:

The last paragraph of page 91 of 154 will be modified to:

"For those local flexible structures that are amplified, apply an additional acceleration to these structures equal to the difference between the average uniform amplified component acceleration and the rigid body component equivalent static acceleration. These accelerations are to be considered in local design of the flexible portion of the structure. The effects of these additional accelerations on the seismic loadings in areas of the structure away from the local flexibility are to be considered in design."

In Section 6.2, the discussion related to the shield building will be revised to reflect the proposed change to the shield building roof design.

Revise Table 6.2-6 to the following:

Table 6.2-6 – CIS Equivalent Static Seismic Accelerations
Units: g [(1)]

Elevation [(2)]	East Side			West Side		
	X	Y	Z	X	Y	Z
66.5	0.33	0.36	0.36	0.33	0.36	0.36
82.5	0.33	0.36	0.36	0.33	0.36	0.36
99	0.35	0.36	0.36	0.35	0.36	0.36
103	0.36	0.36	0.36	0.36	0.36	0.36
107.17	0.37	0.36	0.36	0.37	0.36	0.36
134.25	0.58	0.56	0.39	0.59	0.56	0.39
153	0.71	0.59	0.39	0.74	0.66	0.40
164.95				0.85	0.83	0.41

Notes to Table 6-.2-6:

(1) X = North-South; Y = East-West; Z = Vertical
(2) Linear interpolation between elevations is acceptable.

Revise Table 6.2-7, the referenced table numbers should be 6.2-3, 6.2-4, 6.2-5, and 6.2-6.

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR03-026
Revision: 1

Question:

The description provided in Section 7.2 indicates that spring/dashpot values were selected based on parameters for a uniform half-space. However, for a soil site with hard rock located at a depth of 120 ft below the basemat, the resulting SSI radiation damping value would be expected to be significantly lower than that for a uniform half-space solution. Westinghouse should evaluate what is the impact of this difference on the computed seismic response?

Westinghouse Response:

The vertical springs were not selected based on a uniform half space. As stated in the second paragraph of Section 7.2 the springs were calculated for elastic layers of finite depth by means of the Steinbrenner approximation. The soil properties were those used in the SASSI analyses described in Section 4.4.1.2 of the report with hard rock located at a depth of 120 feet below grade.

The horizontal springs were calculated from the vertical springs assuming the ratio of horizontal to vertical springs was equal to that for a uniform half space.

For a soil site with hard rock located at a depth of 120 ft below the basemat, the resulting SSI radiation damping value would be lower than that for a uniform half-space solution. Soil spring stiffness was calculated using the Steinbrenner approximation, which does not provide a damping value. Preliminary time history analyses were performed with the identified soil spring stiffness with zero soil spring damping. Comparison of these preliminary time history analysis results to those from the 2D SASSI analyses confirmed the soil spring stiffness. Member forces/moments in these preliminary analyses were higher than the 2D SASSI results due to the neglect of soil damping. Since the SASSI analyses account for the soil damping including the effect of embedment and the hard rock at elevation 120', damping in the soil springs in the ANSYS analyses was selected by iterative modal analyses to match the overturning member forces in the SASSI analyses. The resulting damping values are shown in Table 7-1 of the report. The 30% value for damping for soft soil was the value obtained to match the 2D SASSI results. This value was not used in any subsequent analyses since the overall response on a soft soil site is significantly lower than on the soft to medium soil case selected for the non-linear liftoff analyses.

The soft to medium soil case analyzed for the AP1000 assumes bedrock at a depth of 120 feet. This depth was established based on the parametric studies described in section 4.4.1.1 of the technical report and in the response to TR03-RAI-015.

Design Control Document (DCD) Revision:
None

PRA Revision:
None

Technical Report (TR) Revision:

Revision 1

Revise section 7.2 as shown below.

7.2 Soil sites

The effect of liftoff during the safe shutdown earthquake of 0.3g and the review level earthquake of 0.5g was evaluated using the same approach described in section 7.1 for the hard rock site. The analyses used the East-West lumped-mass stick model of the nuclear island structures supported on a rigid basemat with nonlinear springs. The actual footprint of the basemat was used in these analyses of the East-West model (see Figure 7.2-3).

Table 7-1 summarizes the properties of soil springs and dampers used in this calculation. The stiffness of the soil springs in the vertical direction in the ANSYS models were calculated for elastic layers of finite depth by means of the Steinbrenner approximation. This same approach was used for calculation of the soil springs in the AP600 nuclear island basemat analyses. The depth to bedrock was 120 feet. The stiffness of soil springs in the horizontal direction was calculated from that in the vertical direction assuming that the ratio of horizontal and vertical stiffness for the layered site has the same relationship as for a semi-infinite medium.

Damping was modeled in the ANSYS analyses using Rayleigh damping to match modal damping at 3 and 25 hertz. The value of modal damping shown in Table 7.1 was selected to match member forces from the corresponding 2D SASSI analyses described in section 4.4.1. The soil damping is 5% for the soft to medium soil.

FRS comparisons of the ASB stick were performed to check the adequacy of the calculated soil spring properties. The peaks match reasonably for all cases. However, the 2D ANSYS results are significantly higher in the high frequency range compared with the 2D SASSI results. The calculated soil spring stiffness and damping are considered adequate because the results of the 2D ANSYS analyses match the peaks of FRS and member forces/moments reasonably to the 2D SASSI analyses.

Linear analyses of the ANSYS models showed that the soft-to-medium soil case gave the maximum base shear force and overturning moment. Hence, a non-linear lift off analysis was performed for the soft-to-medium soil case. Linear and non-linear (liftoff) analyses were performed for the SSE input of 0.3g and the RLE (review level earthquake) input of 0.5g. The linear analysis uses linear soil springs, and the non-linear (liftoff) analysis uses non-linear soil springs that are inactive when a basemat node is higher than its initial location without loads.

Basemat Displacements

Figure 7.2-1 shows the time history of uplift displacements at the basemat edges. Maximum uplift at the east edge occurs at the time around 5 seconds for both linear and non-linear (liftoff) analyses. Maximum lift off is 0.31 inches. This is higher compared with the hard rock case result of 0.07 inches described in section 7.1. The increase ratio is about equal to the inverse of the soil spring stiffness (1000 versus 6267 kcf).

Floor Response Spectra

Figure 7.2-1 compares the SSE FRS between linear and non-linear (liftoff) analyses. The lift off effect on FRS is similar with those for the hard rock case; it is visible but insignificant. Figure 7.2-2 compares RLE FRS between linear and non-linear (liftoff) analyses. The liftoff effect on FRS is similar with those for the hard rock case; it is insignificant in the horizontal direction and visible in the vertical direction at high frequency range.

Table 7-1 - ANSYS Soil Spring Property

	Assumption of Soil Conditions				
	Soil Material Property		ANSYS Soil Spring Property		
	Density pcf	Poisson's Ratio	Stiffness kcf		Damping %
			Vertical	East-West	
~~*Soft Rock*~~	~~150~~	~~0.25~~	~~3200~~	~~2782~~	~~2~~
Soft-to-medium Soil	110	0.35	1000	814	5
~~*Soft Soil*~~	~~110~~	~~0.40~~	~~300~~	~~234~~	~~30~~

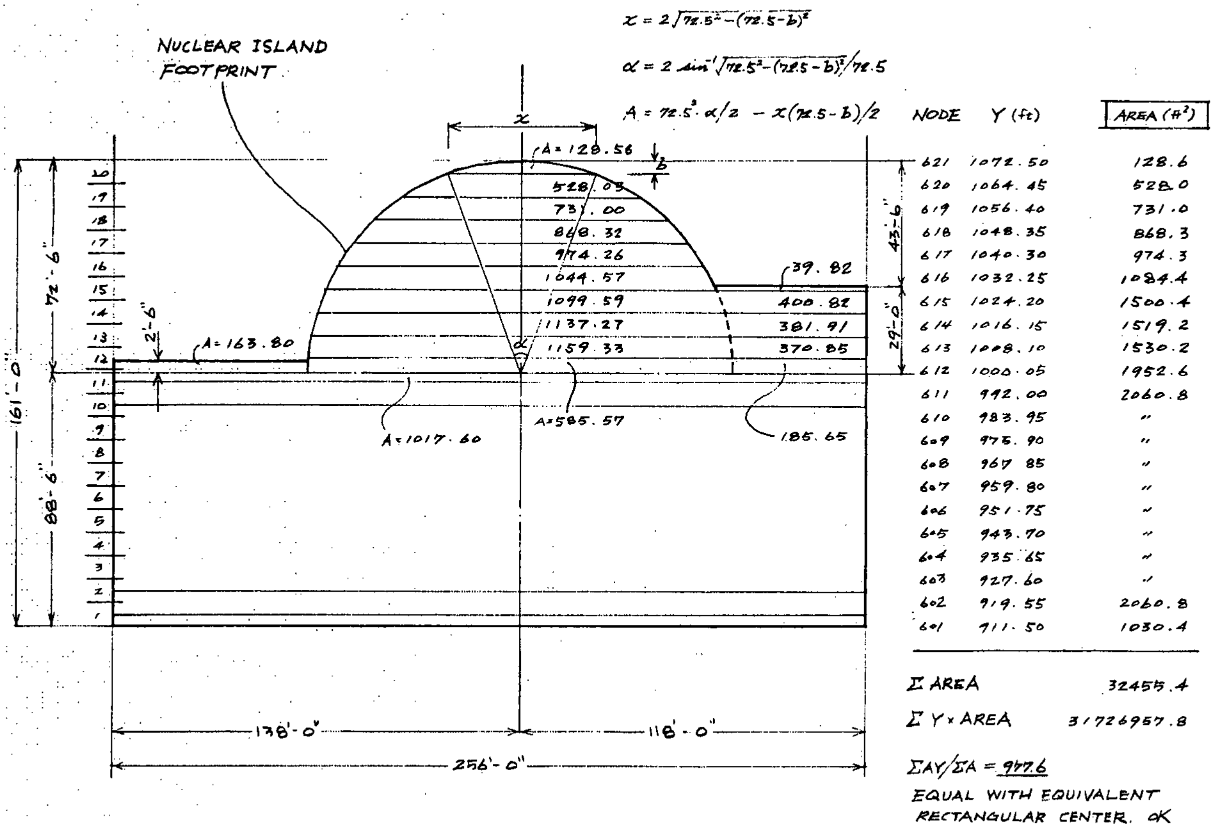


Figure 7.2-3 Modeling of Actual Footprint in East-West Model

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR03-027
Revision: 1

Question:

Section 7.1 indicates that direct integration was used to obtain computed results. Section 7.2 indicates that modal analysis was used to obtain solutions requiring the computation of equivalent modal damping accounting for both element and SSI damping. Westinghouse should describe how was the modal analysis method used to account for lift-off? Do the resulting modal damping values satisfy the limitations recommended in ASCE 4-98?

Westinghouse Response:

Non-linear lift-off analyses were performed in ANSYS using direct integration. Linear (no lift-off) time history modal analyses were performed to compare the ANSYS model on soil springs to the SASSI model on layered soil. These ANSYS analyses were also used to select a soil damping to match the ANSYS overturning member forces to the SASSI results. These damping values are shown in Table 7-1 of the report.

The basis for selection of the damping values is described in the response to RAI-TR03-026.

Design Control Document (DCD) Revision:
None

PRA Revision:
None

Technical Report (TR) Revision:
None

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR03-028
Revision: 1

Question:

Westinghouse is requested to describe in Section 7 of this report that were the three directions of motion (H1, H2 and V) used to generate liftoff responses in all cases analyzed?

Westinghouse Response:

The H2 component (east west direction) and the vertical components of the time histories were used to generate liftoff response in the 2D analyses of the East-West lumped mass stick model. They were applied simultaneously.

Reference:
None

Design Control Document (DCD) Revision:
None

PRA Revision:
None

Technical Report (TR) Revision:
The first paragraph in Section 7.2 is modified as follows:

The effect of liftoff during the safe shutdown earthquake of 0.3g and the review level earthquake of 0.5g was evaluated using the same approach described in section 7.1 for the hard rock site. The analyses used the East-West lumped-mass stick model of the nuclear island structures supported on a rigid basemat with nonlinear springs. The H2 and vertical components of the time histories were used to generate liftoff response in the 2D analyses. They were applied simultaneously. The actual footprint of the basemat was used in these analyses.

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR03-030
Revision: 1

Question:

Table 7.1 indicates that a damping of 30% was selected for the soft soil site. Westinghouse is requested to explain what is the basis for this selection? How does the viscous damping values shown in this table compare with the hysteretic material damping values typically found for iterated soils based on site responses?

Westinghouse Response:

Table 7.1 has been revised to remove the soft soil case as described in the response to TR03-RAI-026, Rev 1.

Design Control Document (DCD) Revision:
None

PRA Revision:
None

Technical Report (TR) Revision:
None

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR03-032
| Revision: 1

Question:

The staff's review of the text and figures in Appendix C of AP1000 Document No. APP-GW-S2R-010, Revision 0, June 2006, "Extension of Nuclear Island Seismic Analyses to Soil Sites," identified the need for a number of clarifications and explanations of the results presented. The staff requests Westinghouse to address the following:

a. In paragraphs 4 and 5, an explanation is provided why the SASSI NI20 model produces higher results in the high frequency region than the ANSYS NI20 model, for a hard rock site condition. The explanation would appear to apply on a generic basis. However, comparison of Figures C-1 through C-6 to Figures C-7 through C-12, respectively, indicates that this effect is not generically demonstrated. Only the first three of the six locations demonstrate this behavior. Please (a) provide a detailed explanation why this effect occurs only at three locations, and not at all six locations; (b) describe how it was determined that the explanation provided in paragraph 4 and 5 is accurate; and (c) confirm that all other potential sources for the differences (e.g., modeling error) have been investigated and eliminated as the source of the difference.

b. Paragraph 2 states:

> "Both finite element models give comparable results below 10 hertz. However, the results from the coarse model are not as good at high frequencies (above about 15 hertz). Therefore the hard rock FRS were generated from the fine NI10 model, and the coarse NI20 model was used for the soil site analyses where frequencies of interest are below 10 hertz."

Paragraph 6 states:

> "In a few cases it is found that the soil cases analyzed in SASSI using the NI20 model give higher results than the hard rock case using the NI10 model for frequencies above 10 Hz (see for example Figure 4.4.3-9). Although these cases are believed to be due to conservatism in the SASSI results at high frequency, the SASSI results are used in developing the broadened envelope design response spectra."

Apparently, the hard rock results obtained from the NI10 ANSYS model do not always envelop the soil site results obtained from the SASSI NI20 model at frequencies above 10 hertz, as one might easily conclude from paragraph 2. From paragraph 6, it appears that there is considerable uncertainty about the validity of the SASSI results above 10 hertz. This is in contrast to the "matter-of-fact" statements made in paragraphs 4 and 5. Please clarify the Westinghouse position, including the technical basis, on the validity of SASSI

NI20 model results above 10 hertz for all site conditions, including a hard rock site. Is the NI20 grid sufficiently refined to accurately predict response above 10 hertz? Have any SASSI soil site analyses been performed using a refined grid comparable to the NI10 model, to study the effect of element size on the solution results?

c. Explain what studies were performed to establish that the NI10 model refinement is sufficient to accurately account for high frequency response effects at all critical locations. It is not obvious from the results shown in Figure C-1 that convergence with element size has been achieved.

Westinghouse Response:

a) The NI20 model uses solid elements for the mass concrete below grade inside the shield building. Other parts of the model use shell elements. The difference in ANSYS and SASSI results is most noticeable at the three lowest elevations where the response is most affected by the solid elements below grade.

The explanation provided in Paragraphs 4 and 5 were based on detailed checking of the models and on a series of studies. The explanation was confirmed by a study comparing the SASSI and ANSYS responses using a reduced model with only the solid elements in the NI20 model.

b) Paragraph 2 does not imply that NI10 ANSYS model envelopes the soil site results obtained from the SASSI NI20 model at frequencies above 10 hertz. It is discussing the comparison of the NI10 and NI20 models on hard rock. The paragraph states explicitly that the results of the NI20 model on hard rock are not as good at high frequencies.

The RAI is correct when it says that the hard rock results obtained from the NI10 ANSYS model do not always envelop the soil site results obtained from the SASSI NI20 model at frequencies above 10 hertz. This can be seen by review of the floor response spectra in Figures 4.4.3-1 to 4.4.3-18. The higher SASSI responses are generally responses in the vertical direction. An extreme example is seen in Figures 4.4.3-9 where the firm rock exhibits a higher response at about 25 hertz. As seen in Figure C-3 on hard rock the NI20 model has a similar higher response so this higher response is due to the coarser modeling of NI20; however, the higher SASSI results were conservatively enveloped in developing the broadened envelope design response spectra.

The comparisons of the NI10 and NI20 results in Figures C-1 to C-6 show the NI20 model is acceptable for responses above 10 hertz. However, as stated in paragraph 2, the NI10 model gives more accurate results and is used in the fixed base analyses for hard rock. The comparisons of NI10 to NI20 were performed in ANSYS. Analyses have not been performed in SASSI with more refined models than the NI20 model.

The FRS for the NI10, NI20 (ANSYS & SASSI) given in Appendix C are compared on the same plots in Figures RAI-TR03-032-1 to RAI-TR03-032-6. The node numbers are the same as shown in Table C1 of the technical report. The pertinent information from Table C1 is reproduced in Table RAI-TR03-032-1. The NI10 ANSYS FRS are used as the design basis for hard rock.

c) The NI10 model is described in DCD subsection 3.7.2 (Item 5) and is the basis for the vertical floor response spectra for hard rock. The model was reviewed and accepted as part of the hard rock design certification. During development of the model detail studies with greater element refinement were performed for the floor above the control room and the adjacent bays to confirm the adequacy of the model.

Table RAI-TR03-032-1– Key Nodes at Location

Location	NI10 Node	NI20 Coarse Model Nodes	NI20 Sassi	Figure ANSYS & SASSI FRS Comparaison	General Area	Elevation (feet)
CIS at Reactor Vessel Support Elevation	130401	465	1397	RAI-TR03-032-1	RPV Center	100.00
CIS at Operating Deck	105772	981	1913	RAI-TR03-032-2	SG West compartment, NE	134.25
ASB NE Corner at Control Room Ceiling	5109	1115	2047	RAI-TR03-032-3	NE Corner	134.88
ASB Corner of Fuel Building Roof at Shield Building	5754	1433	2365	RAI-TR03-032-4	NW Corner of Fuel Bldg	179.19
ASB Shield Building Roof Area	2862	2022	2954	RAI-TR03-032-5	South side of Shield Bldg	333.12
SCV Near Polar Crane	130412	1546	2478	RAI-TR03-032-6	SCV Stick Model	224.00

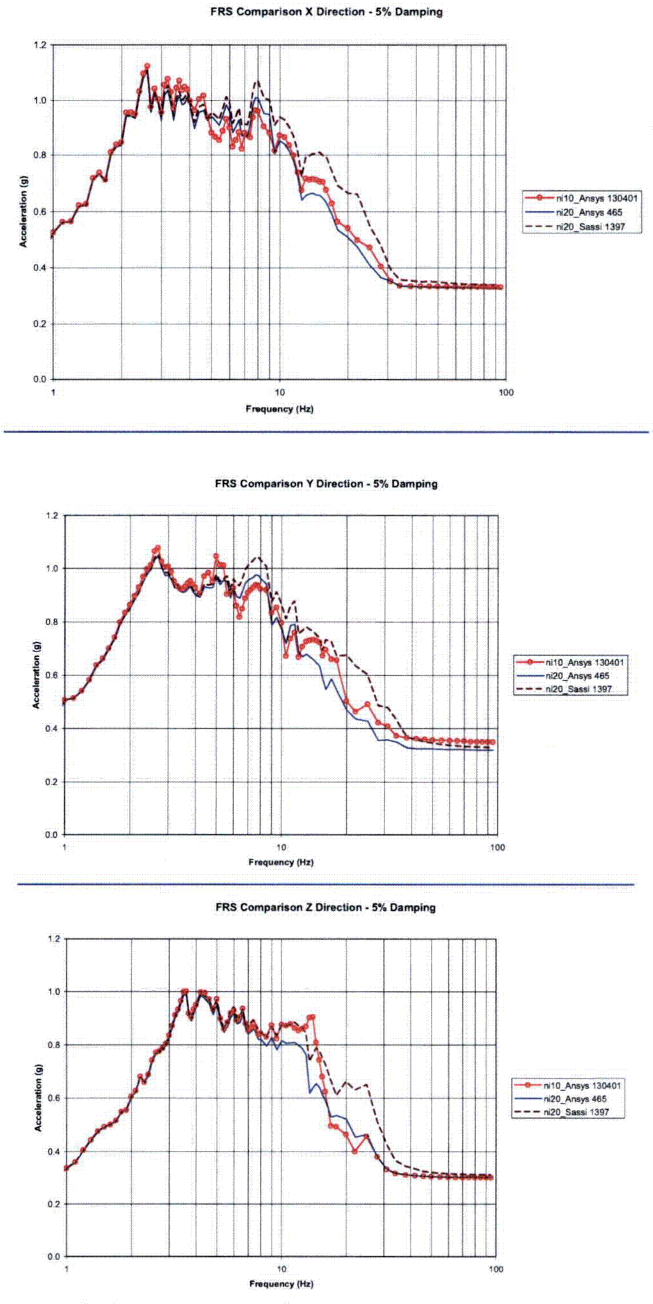
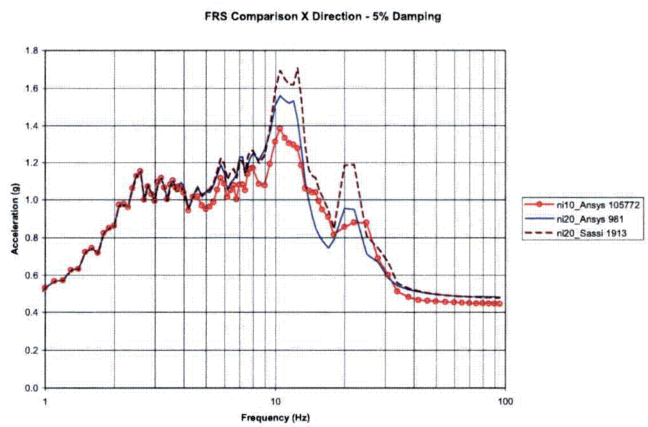
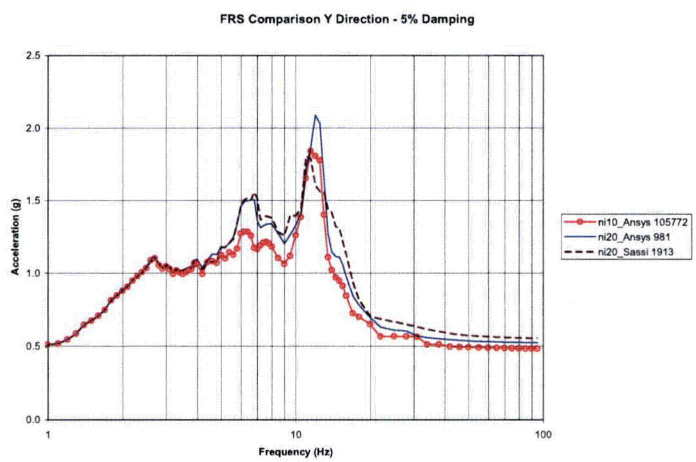


Figure RAI-TR03-032-1 - CIS at Reactor Vessel Support Elevation


FRS Comparison X Direction - 5% Damping
Acceleration (g)
Frequency (Hz)
ni10_Ansys 105772
ni20_Ansys 981
ni20_Sassi 1913


FRS Comparison Y Direction - 5% Damping
Acceleration (g)
Frequency (Hz)
ni10_Ansys 105772
ni20_Ansys 981
ni20_Sassi 1913

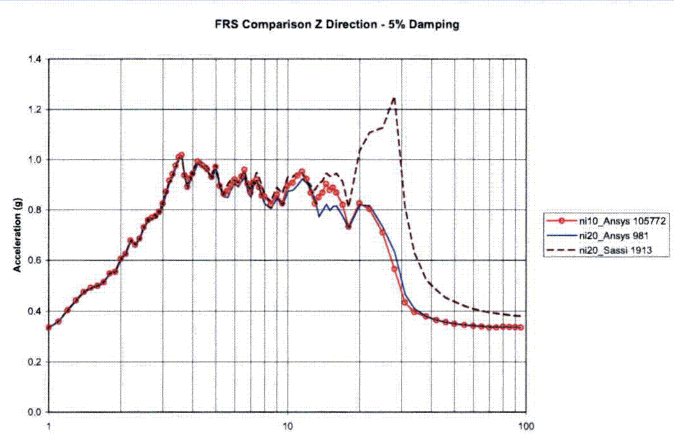

FRS Comparison Z Direction - 5% Damping
Acceleration (g)
Frequency (Hz)
ni10_Ansys 105772
ni20_Ansys 981
ni20_Sassi 1913

Figure RAI-TR03-032-2 - CIS at Operating Deck

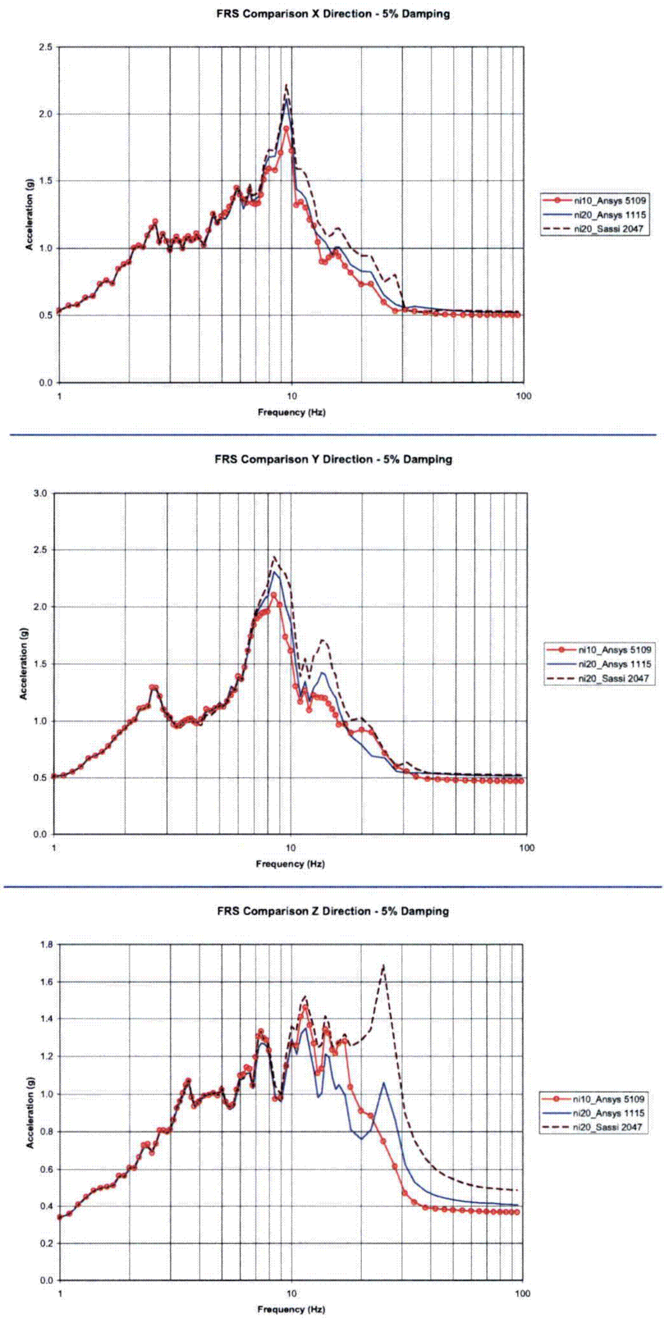


Figure RAI-TR03-032-3 - ASB NE Corner at Control Room Ceiling

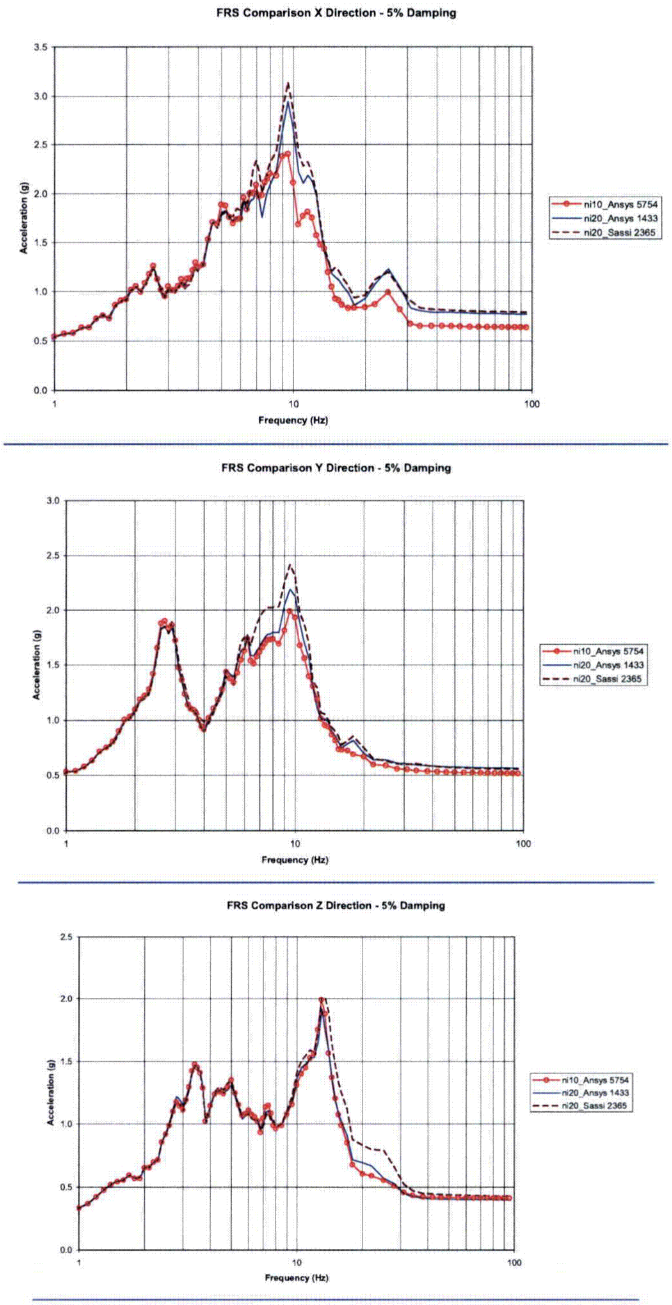


Figure RAI-TR03-032-4 - ASB Corner of Fuel Building Roof at Shield Building

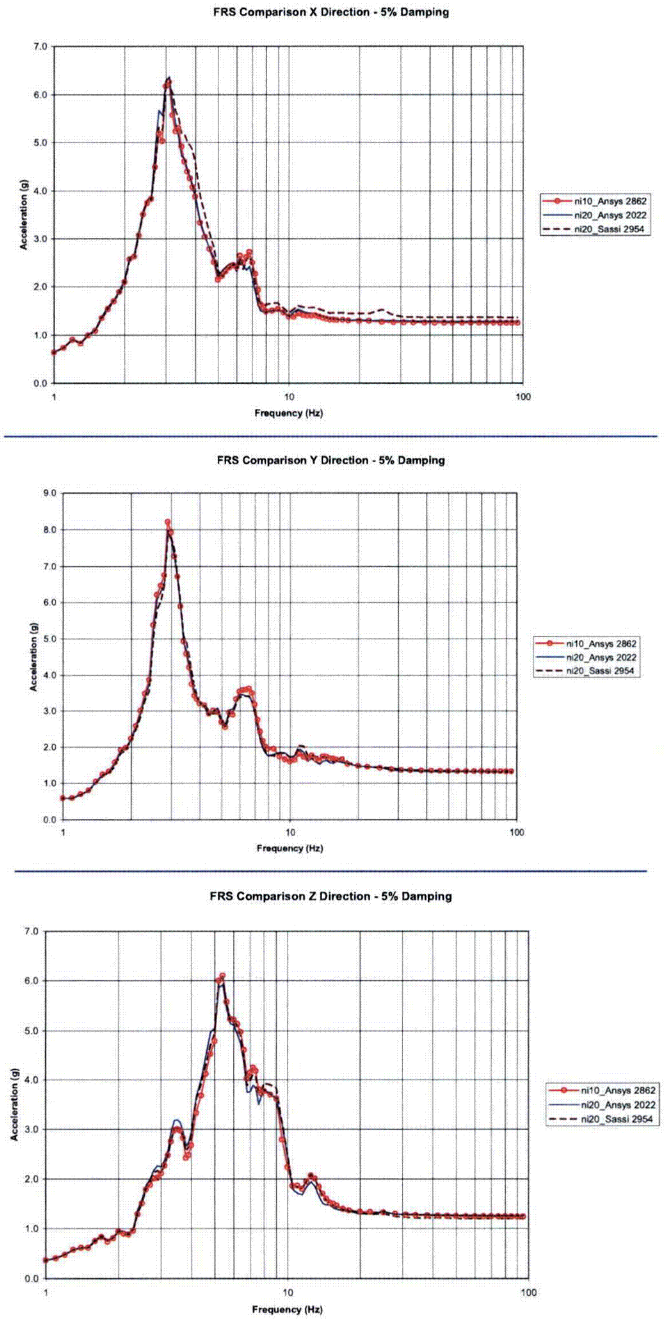


Figure RAI-TR03-032-5 - ASB Shield Building Roof Area

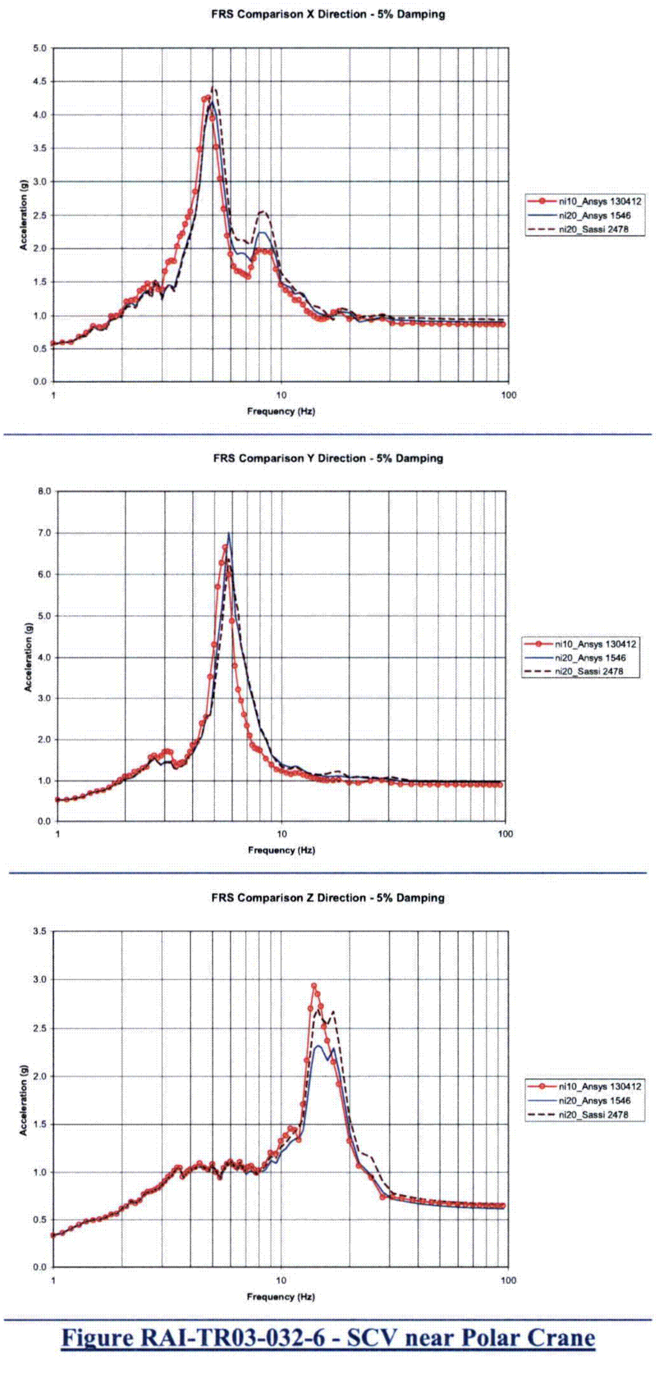


Figure RAI-TR03-032-6 - SCV near Polar Crane

Design Control Document (DCD) Revision:
None

PRA Revision:
None

Technical Report (TR) Revision:
Revise Appendix C as shown below:

Appendix C - Comparison of NI10 and NI20 Responses

In this appendix the fine (NI10) and coarse (NI20) model seismic responses are compared. Seismic response spectra were developed for both models using a fixed base (hard rock) case. Also in this section the NI20 ANSYS is compared to the SASSI analysis results.

Figures C-1 to C-6 compare response spectra for ANSYS analyses of the NI10 and NI20 models at the interface seismic response key nodes (see Section 4.4.3). These locations are given in Table C-1. Also shown in this table are the figures where the comparison spectra are given. Both finite element models give comparable results below 10 hertz. However, the results from the coarse model are not as good at high frequencies (above about 15 hertz). Therefore the hard rock FRS were generated from the fine NI10 model, and the coarse NI20 model was used for the soil site analyses where frequencies of interest are below 10 hertz.

A Time History Analysis for the Nuclear Island SASSI Surface Structure Model and the Embedded Structure Model is carried out with the seismic input in three orthogonal directions. The acceleration response spectra for 5% damping are generated at the interface locations identified in Table C-1. Figures C-7 to C-12 compare the Nuclear Island SASSI Surface Structure Model and the Embedded Structure Model results with the Nuclear Island ANSYS Coarse Model (NI20) results for hard rock conditions.

As seen from the comparison (see Figures C-7 to C-12), for the horizontal response, the SASSI and ANSYS results for NI20 are very similar to about 15 Hz horizontal and about 10 Hz vertical. At the higher frequencies SASSI calculates higher accelerations. The NI20 model uses solid elements for the mass concrete below grade inside the shield building. Other parts of the model use shell elements. The difference in ANSYS and SASSI results is most noticeable at the three lowest elevations where the response is most affected by the solid elements below grade. This behavior was investigated in a study comparing the SASSI and ANSYS responses using a reduced model with only the solid elements in the NI20 model. One reason for this conservatism in the SASSI results is the different formulation in the solid elements. Another difference is due to the different way the two computer programs calculate the dynamic response. ANSYS performs the dynamic response in the time domain. SASSI converts the time history input (time domain) to the frequency domain, solves the response in the frequency domain, and then converts the output back to the time domain.

SASSI also needs to specify key frequencies to perform its transfer function calculations. For such a large model, resting on a very stiff soil (hard rock), SASSI gives conservative results at high frequencies. The significant responses for soil cases occur at less than 10 Hz. Therefore, the SASSI Model is adequate for the AP1000 Soil-Structure Interaction analyses to be performed.

In a few cases it is found that the soil cases analyzed in SASSI using the NI20 model give higher results than the hard rock case using the NI10 model for frequencies above 10 Hz (see for example Figure 4.4.3-9). The reason for this is two-fold: mesh size and SASSI approximation. The NI20 SASSI model is a much coarser model than the NI10, at higher frequencies it cannot capture the local behavior as well as the NI10 and this causes some of the response to be higher. SASSI uses a limited number of transfer functions to obtain the dynamic response. This limited number (up to 100 frequencies) is an adequate approach when the medium that you are considering is soil, where only a few significant modes need to be captured to obtain the building response. At higher frequencies, in a shell models, many modes (or transfer frequencies) are required to obtain the building response. Although these cases are due to conservatism in the SASSI results at high frequency, the SASSI results are used in developing the broadened envelope design response spectra.