Westinghouse

Westinghouse Electric Company
Nuclear Power Plants
P.O. Box 355
Pittsburgh, Pennsylvania 15230-0355
USA

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

Direct tel: 412-374-6306
Direct fax: 412-374-5005
e-mail: sterdia@westinghouse.com

Your ref: Project Number 740
Our ref: DCP/NRC2082

January 29, 2008

52-6
52-14
52-15

Subject: AP1000 COL Responses to Requests for Additional Information (TR 03)

In support of Combined License application pre-application activities, Westinghouse is submitting responses to NRC requests for additional information (RAIs) on AP1000 Standard Combined License Technical Report 03, APP-GW-S2R-010, "Extension of Nuclear Island 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.

Revision responses are provided for TR03-010,-015,-017,-020, and -022 as sent in an email from Mike Miernicki to Sam Adams dated January 7, 2008. Revision 1 responses were provided for RAI-TR03-015 and -022 under DCP/NRC1987 dated August 31, 2007. These responses complete all requests received to date for Technical Report 03. A Revision 1 response was provided for RAI-TR03-010 under DCP/NRC1954 dated July 5, 2007. Revision 1 responses were provided for RAI-TR03-017 and -020 under DCP/NRC1942 dated June 15, 2007. A Revision 0 response was provided for RAI-TR03-020 under DCP/NRC1857 dated March 29, 2007. A Revision 0 response was provided for RAI-TR03-022 under DCP/NRC1822 dated January 29, 2007. Revision 0 responses were provided for RAI-TR03-010,-015, and -017 under DCP/NRC1814 dated January 18, 2007.

Pursuant to 10 CFR 50.30(b), the responses to the 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 these responses 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.

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D063
D085
NRO

Very truly yours,

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

/Attachment

1. "Oath of Affirmation," dated January 29, 2008

/Enclosure

1. 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
	P. Ray	-	TVA	1E	1A
	P. Hastings	-	Duke Power	1E	1A
	R. Kitchen	-	Progress Energy	1E	1A
	A. Monroe	-	SCANA	1E	1A
	J. Wilkinson	-	Florida Power & Light	1E	1A
	C. Pierce	-	Southern Company	1E	1A
	E. Schmiech	-	Westinghouse	1E	1A
	G. Zinke	-	NuStart/Entergy	1E	1A
	R. Grumbir	-	NuStart	1E	1A
	J. Ewald	-	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

W. E. Cummins, being duly sworn, states that he is Vice President, Regulatory Affairs & Standardization, 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.

W. E. Cummins
Vice President
Regulatory Affairs & Standardization

Subscribed and sworn to
before me this 29th day
of January 2008.

COMMONWEALTH OF PENNSYLVANIA
Notarial Seal
Patricia S. Aston, Notary Public
Murrysville Boro, Westmoreland County
My Commission Expires July 11, 2011
Member, Pennsylvania Association of Notaries

Notary Public

ENCLOSURE 1

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-TR03-010
Revision: 2

Question:

The staff's review of Tables 4.4.1-1A and 4.4.1-1B found that Westinghouse used three soil/rock degradation models in its parametric studies for selecting site conditions: Seed and Idriss 1970 soil/rock degradation curves, Idriss 1990 soil degradation curves, and EPRI 1993 soil degradation curves. For example, Westinghouse used Seed and Idriss 1970 model for two horizontal motions and EPRI 1993 soil degradation model for two rocking motions when the parametric studies were performed for the AP1000 site selection. Westinghouse is requested to provide reasons and bases for using different soil degradation models for its parametric studies.

Westinghouse Response:

Soil structure interaction analyses on rock sites for both AP600 and AP1000 use the rock degradation curve recommended by Seed and Idriss in Reference 1. This was applied in SSI analyses for the hard rock, firm rock and soft rock sites.

Soil structure interaction analyses on soil sites for the AP1000 used the latest soil degradation curve recommended by EPRI in Reference 2. This was applied in SSI analyses for the upper bound soft to medium, soft to medium and soft soil sites. Two sets of degradation curves were used in the AP600 studies. The early analyses used the degradation recommended by Seed and Idriss in Reference 1. Later AP600 analyses performed to address NRC questions used the later soil degradation curve recommended by Idriss in Reference 3.

Westinghouse used one degradation model for soil and one for rock for the AP1000 parametric studies consistent with the latest models recommended for soil and rock sites. The soil profiles used in the generic analyses are added in DCD subsection 3.7.1.4 as shown below.

In the meeting of April 16 – 20, 2007, NRC Staff requested additional clarification of how to confirm that a specific site is enveloped by the generic seismic design basis. This clarification is provided in the revisions to DCD subsection 2.5.2 shown below.

Reference:

1. Seed, H.B. and I.M. Idriss, "Soil Moduli and Damping Factors for Dynamic Response Analysis," Report No. EERC 70-14, Earthquake Engineering Center, University of California, Berkeley, CA., 1970.
2. EPRI TR-102293, "Guidelines for Determining Design Basis Ground Motions, 1993.
3. Idriss, I.M., "Response of Soft Soil Sites during Earthquakes," H. Bolton Seed Memorial Symposium Proceedings, May 1990.

Design Control Document (DCD) Revision:

Changes to DCD Section 2.5

Revision 0 and 1 of this RAI response identified changes in DCD Section 2.5. ~~Revisions to DCD Section 2.5 were included in APP-GW-GLR-044, Rev 0, "Nuclear Island Basemat and Foundation", October 2006. These revisions to subsection 2.5.2 were further revised in the responses to RAI-TR03-018 and RAI-TR03-019. The following revision shows all changes from DCD Revision 15.~~ These changes have been incorporated in DCD Rev 16, as modified by TR134, Rev 0. Other Technical reports and RAI responses were also incorporated in this DCD section. Based on this Revision 2 of this RAI response, the next issue of this section of the DCD will read as follows. Changes from DCD Rev 16, as modified by TR134, Rev 0, are shown by redline.

2.5.2 Vibratory Ground Motion

The AP1000 is designed for a safe shutdown earthquake (SSE) defined by a peak ground acceleration (PGA) of 0.30g and the design response spectra specified in subsection 3.7.1.1, and Figures 3.7.1-1 and 3.7.1-2. The AP1000 certified seismic design response spectra (CSDRS) were developed using the Regulatory Guide 1.60 response spectra as the base and modified to include additional high frequency amplification at a control point at 25 Hz. The peak ground accelerations in the two horizontal and the vertical directions are equal.

The AP1000 is also evaluated for a safe shutdown earthquake (SSE) defined by a peak ground acceleration (PGA) of 0.30g and the design response spectra specified in Appendix 3I, and Figures 3I.1-1 and 3I.1-2. These design response spectra are applicable to certain east coast rock sites.

2.5.2.1 Combined License Seismic and Tectonic Characteristics Information

Combined License applicants referencing the AP1000 certified design will address the following site-specific information related to the vibratory ground motion aspects of the site and region:

- Seismicity
- Geologic and tectonic characteristics of site and region
- Correlation of earthquake activity with seismic sources
- Probabilistic seismic hazard analysis and controlling earthquakes
- Seismic wave transmission characteristics of the site
- SSE ground motion

The site-specific ground motion response spectra (GMRS) are determined in the free-field on the ground surface. For sites with soil layers that will be completely excavated to expose competent material, the GMRS is specified on an outcrop or a hypothetical outcrop that will exist after excavation. Motions at this hypothetical outcrop are developed as a free-surface motion, not as an in-column motion. Competent material may be defined as in-situ material having a shear wave velocity equal to or greater than 1000 fps. The Combined License applicant must demonstrate that the proposed site meets the following requirements:

1. The free field peak ground acceleration at the finished grade level is less than or equal to a 0.30g SSE.

2. The site-specific ground motion design response spectra (GMRS) at the finished grade level in the free-field are less than or equal to those the AP1000 certified seismic design spectra (CSDRS) given in Figures 3.7.1-1 and 3.7.1-2.

3. In lieu of (1) and (2) above, for a site where the nuclear island is founded on competent rock with shear wave velocity greater than 8,000 feet per second, the site-specific ground motion may be defined at the foundation level as the foundation input response spectraum (FIRS) and shown to be less than or equal to the CSDRS given in Figures 3.7.1-1 and 3.7.1-2.

4. In lieu of (1) and (2) above, for a site where the nuclear island is founded on competent rock with shear wave velocity greater than 8000 feet per second and there are thin layers of soft material overlying the rock, the site-specific peak ground acceleration and spectra may be developed at the top of the competent rock and shown at the foundation level to be less than or equal to those given in Figures 3I.1-1 and 3I.1-2.

5. Foundation material layers are approximately horizontal (dip less than 20 degrees), and the median estimate of the low strain shear wave velocity of the soil below the foundation of the nuclear island is greater than or equal to 1000 feet per second.

6. For sites where the nuclear island is founded on soil, the median estimate of the strain-compatible soil shear modulus and hysteretic damping is compared to the values used in the AP1000 generic analyses shown in Table 3.7.1-4 and Figure 3.7.1-17. Properties of soil layers within a depth of 120 feet below finished grade are compared to those in the generic soil site analyses (soft soil, soft-to-medium soil, and upper bound soft-to-medium soil).

7. In lieu of (1) to (6) above, a site-specific evaluation can be performed as described in subsection 2.5.2.3.

Where features of the site are not within the parameters specified for the AP1000, site-specific soil structure interaction analyses may be performed using the 2D SASSI models described in Appendix 3G for variations in site conditions that can be represented in these models. Results should be compared to the results of the 2D SASSI analyses described in Appendix 3G. Such analyses may be used to demonstrate that local features, such as soil degradation properties or backfill, are bounded by the design cases. If the results are not clearly enveloped then a 3D SASSI analysis may be required.

2.5.2.2 Site-Specific Seismic Structures

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

2.5.2.3 Sites with Geoscience Parameters Outside the Certified Design

If the site-specific spectra at foundation level exceed the response spectra in Figures 3.7.1-1 and 3.7.1-2 at any frequency, or if soil conditions are outside the range evaluated for AP1000 design certification, a site-specific evaluation can be performed. This evaluation will consist of a site-specific dynamic analysis and generation of in-structure response spectra to be compared with the floor response spectra of the certified design at 5-percent damping. The site design response spectra at the foundation level in the free-field given in Figures 3.7.1-1 and 3.7.1-2 were used to develop the floor response spectra. They were applied at foundation level for the hard rock site and at finished grade level for the soil sites. The site is acceptable for construction of the AP1000 if the floor response spectra from the site-specific evaluation do not exceed the AP1000 spectra for each of the locations identified below:

• Containment internal structures at elevation of reactor vessel support	Figure 3G.4-5X to 3G.4-5Z
• Containment operating floor	Figure 3G.4-6X to 3G.4-6Z
• Auxiliary building NE corner at elevation ~~135' 6"~~ 116' 6"	Figure 3G.4-7X to 3G.4-7Z
• Shield building at fuel building roof	Figure 3G.4-8X to 3G.4-8Z
• Shield building roof	Figure 3G.4-9X to 3G.4-9Z
• Steel containment vessel at polar crane support	Figure 3G.4-10X to 3G.4-10Z

Site-specific soil structure interaction analyses are performed using the 3D SASSI models described in Appendix 3G. The site-specific soil structure interaction analyses use the site-specific soil conditions (including variation in soil properties in accordance with Standard Review Plan 3.7.2). The three components of the site-specific ground motion time history must satisfy the regulatory requirements for statistical independence and enveloping of the site design spectra at 5% damping. Floor response spectra determined from the site-specific analyses should be compared against the design basis of the AP1000 described above. These evaluations and comparisons will be provided and reviewed as part of the Combined License application.

If the site-specific spectra at foundation level at a rock site exceed the response spectra in Figures 3I.1-1 and 3I.1-2 at any frequency, a site-specific evaluation can be performed similar to that described in Appendix 3I.

Changes to DCD Section 3.7

Revision 0 and 1 of this RAI response identified changes in DCD Section 3.7. These changes have been incorporated in DCD Rev 16, as modified by TR134, Rev 0. The revised section with the TR134, Rev. 0 modifications are shown below.

3.7.1.4 Supporting Media for Seismic Category I Structures

The supporting media will be described consistent with the information items in subsection 2.5.4. Seismic analyses for both rock and soil sites are described in subsection 3.7.2 and Appendix 3G.

The AP1000 nuclear island consists of three seismic Category I structures founded on a common basemat. The three structures that make up the nuclear island are the coupled auxiliary and shield buildings, the steel containment vessel, and the containment internal structures. [*The nuclear island is shown in Figure 3.7.1-14.*]* The foundation embedment depth, foundation size, and total height of the seismic Category I structures are presented in Table 3.7.1-2.

For the design of seismic Category I structures, a set of six design soil profiles (that include hard rock) of various shear wave velocities is established from parametric studies as described in Appendix 3G. These six profiles are sufficient to envelope sites where the shear wave velocity of the supporting medium at the foundation level exceeds 1000 feet per second (see subsection 2.5.2). The design soil profiles include a hard rock site, a firm rock site, a soft rock site, an upper bound soft-to-medium soil site, a soft-to-medium soil site, and a soft soil site. The shear wave velocity profiles and related governing parameters of the six sites considered are as follows:

- For the hard rock site, an upper bound case for rock sites using a shear wave velocity of 8000 feet per second.
- For the firm rock site, a shear wave velocity of 3500 feet per second to a depth of 120 feet and base rock at the depth of 120 feet.
- For the soft rock site, a shear wave velocity of 2400 feet per second at the ground surface, increasing linearly to 3200 feet per second at a depth of 240 feet, and base rock at the depth of 120 feet.
- For the upper bound soft-to-medium soil site, a shear wave velocity of 1414 feet per second at ground surface, increasing parabolically to 3394 feet per second at 240 feet,

base rock at the depth of 120 feet, and ground water at grade level. The initial soil shear modulus profile is twice that of the soft-to-medium soil site.

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

- For the soft soil site, a shear wave velocity of 1000 feet per second at ground surface, increasing linearly to 1200 feet per second at 240 feet, base rock at the depth of 120 feet, and ground water is assumed at grade level

The strain-dependent shear modulus curves for the foundation materials, together with the corresponding damping curves are taken from References 37 and 38 and are shown in Figures 3.7.1-15 and 3.7.1-16 for rock material and soil material respectively. The different curves for soil in Figure 3.7.1-16 apply to the range of depth within a soil column below grade. The strain-dependent soil material damping is limited to 15 percent of critical damping. The strain-dependent properties used in the SSI analyses for the safe shutdown earthquake are shown in Table 3.7.1-4 and Figure 3.7.1-17 for the firm rock, soft rock, upper bound soft-to-medium soil, soft-to-medium soil, and soft soil properties.

37. H.B. Seed, and I.M. Idriss, "Soil Moduli and Damping Factors for Dynamic Response Analysis," Report No. EERC-70-14, Earthquake Engineering Research Center, University of California, Berkeley, 1970.

38. EPRI TR-102293, "Guidelines for Determining Design Basis Ground Motions, 1993.

PRA Revision:
None

Technical Report (TR) Revision:
None

Table 3.7.1.4 (Sheet 1 of 4)

STRAIN COMPATIBLE SOIL PROPERTIES

Depth to Bottom of Layer (ft)	Thickness of Layer (ft)	Layer Number	Total Unit Weight (kcf)	Initial G (ksf)	Initial Vs (fps)	Final G (ksf)	Final Vs (fps)	Damping
Firm Rock								
0.0								
5.0	5.0	1	0.15	57422	3500	57030	3499	0.015
10.0	5.0	2	0.15	57422	3500	56579	3485	0.016
15.0	5.0	3	0.15	56963	3486	55961	3466	0.014
20.0	5.0	4	0.15	56963	3486	55731	3459	0.015
25.0	5.0	5	0.15	56442	3470	54894	3433	0.016
30.0	5.0	6	0.15	56442	3470	55260	3444	0.014
33.5	3.5	7	0.15	55922	3454	54564	3422	0.015
39.5	6.0	8	0.15	55922	3454	54395	3417	0.015
45.0	5.5	9	0.15	55406	3438	53708	3395	0.016
60.0	15.0	10	0.15	55406	3438	53462	3388	0.017
70.0	10.0	11	0.15	54763	3418	52285	3350	0.018
80.0	10.0	12	0.15	54763	3418	51561	3327	0.020
90.0	10.0	13	0.15	53647	3383	49794	3269	0.021
100.0	10.0	14	0.15	53647	3383	49236	3251	0.022
Bedrock			0.15	300000	8000	298137	8000	0.000

Table 3.7.1.4 (Sheet 2 of 4)

STRAIN COMPATIBLE SOIL PROPERTIES

Depth to Bottom of Layer (ft)	Thickness of Layer (ft)	Layer Number	Total Unit Weight (kcf)	Initial G (ksf)	Initial Vs (fps)	Final G (ksf)	Final Vs (fps)	Damping
Soft Rock								
0								
10	10.0	1	0.15	27214	2417	27050	2402	0.007
20.0	10.0	2	0.15	27962	2450	27533	2424	0.009
30.0	10.0	3	0.15	28720	2483	28162	2451	0.009
40.0	10.0	4	0.15	29512	2517	28865	2481	0.010
60.0	20.0	5	0.15	30696	2567	29940	2527	0.010
80.0	20.0	6	0.15	32295	2633	31422	2589	0.011
120.0	40.0	7	0.15	34795	2733	33772	2684	0.011
160.0	40.0	8	0.15	38290	2867	37094	2813	0.011
200.0	40.0	9	0.15	41925	3000	40584	2942	0.011
240.0	40.0	10	0.15	45725	3133	44259	3073	0.011
Base	-	11	0.15	47702	3200	-	-	0.011

Table 3.7.1.4 (Sheet 3 of 4)

STRAIN COMPATIBLE SOIL PROPERTIES

Depth to Bottom of Layer (ft)	Thickness of Layer (ft)	Layer Number	Total Unit Weight (kcf)	Initial G (ksf)	Initial Vs (fps)	Final G (ksf)	Final Vs (fps)	Damping
Upper Bound Soft-to-Medium Soil								
0								
5	5.0	1	0.11	6440	1373	6272	1355	0.018
10.0	5.0	2	0.11	6440	1373	5894	1313	0.027
15.0	5.0	3	0.11	8626	1589	7741	1505	0.030
20.0	5.0	4	0.11	8626	1589	7310	1463	0.037
25.0	5.0	5	0.11	11415	1828	10323	1738	0.026
30.0	5.0	6	0.11	11415	1828	10071	1717	0.029
33.5	3.5	7	0.11	13231	1968	11683	1849	0.029
39.5	6.0	8	0.11	13231	1968	11478	1833	0.031
45.0	5.5	9	0.11	15659	2141	14303	2046	0.023
52.5	7.5	10	0.11	16012	2165	14444	2056	0.025
60.0	7.5	11	0.11	16012	2165	14228	2041	0.026
66.0	6.0	12	0.11	18850	2349	16841	2220	0.026
73.0	7.0	13	0.11	18850	2349	16665	2209	0.027
80.0	7.0	14	0.11	18850	2349	16495	2197	0.028
90.0	10.0	15	0.11	22179	2548	19544	2392	0.027
100.0	10.0	16	0.11	22179	2548	19326	2379	0.028
120.0	10.0	17	0.11	22179	2548	19024	2360	0.030
130.0	10.0	18	0.11	22179	2548	18698	2340	0.032
Base			0.15	298137	8000	298137	8000	0.000

Table 3.7.1.4 (Sheet 4 of 4)

STRAIN COMPATIBLE SOIL PROPERTIES

Depth to Bottom of Layer (ft)	Thickness of Layer (ft)	Layer Number	Total Unit Weight (kcf)	Initial G (ksf)	Initial Vs (fps)	Final G (ksf)	Final Vs (fps)	Damping
Soft-to-Medium Soil								
0								
10	10.0	1	0.11	3617	1029	3074	946	0.032
20.0	10.0	2	0.11	4044	1088	2989	933	0.056
30.0	10.0	3	0.11	4486	1146	2859	912	0.077
40.0	10.0	4	0.11	4952	1204	2843	909	0.089
60.0	20.0	5	0.11	5702	1292	2977	931	0.100
80.0	20.0	6	0.11	6772	1408	3453	1002	0.102
120.0	40.0	7	0.11	8560	1583	4764	1177	0.093
160.0	40.0	8	0.12	12304	1817	7343	1399	0.085
200.0	40.0	9	0.12	15661	2050	9277	1573	0.086
240.0	40.0	10	0.12	19424	2283	11490	1750	0.086
Base	-	11	0.12	21466	2400	-	-	0.093

Soft Soil								
0								
10	10.0	1	0.11	3444	1004	2925	922	0.033
20	10.0	2	0.11	3506	1013	2472	848	0.063
30	10.0	3	0.11	3561	1021	2044	771	0.089
40	10.0	4	0.11	3617	1029	1750	713	0.108
60	20.0	5	0.11	3709	1042	1484	657	0.128
80	20.0	6	0.11	3824	1058	1530	667	0.130
120	40.0	7	0.11	4007	1083	1603	683	0.136
160	40.0	8	0.11	4262	1117	1705	704	0.150
200	40.0	9	0.11	4518	1150	1807	725	0.150
240	40.0	10	0.11	4781	1183	1912	746	0.150
Base		11	0.11	6708	1200			0.150

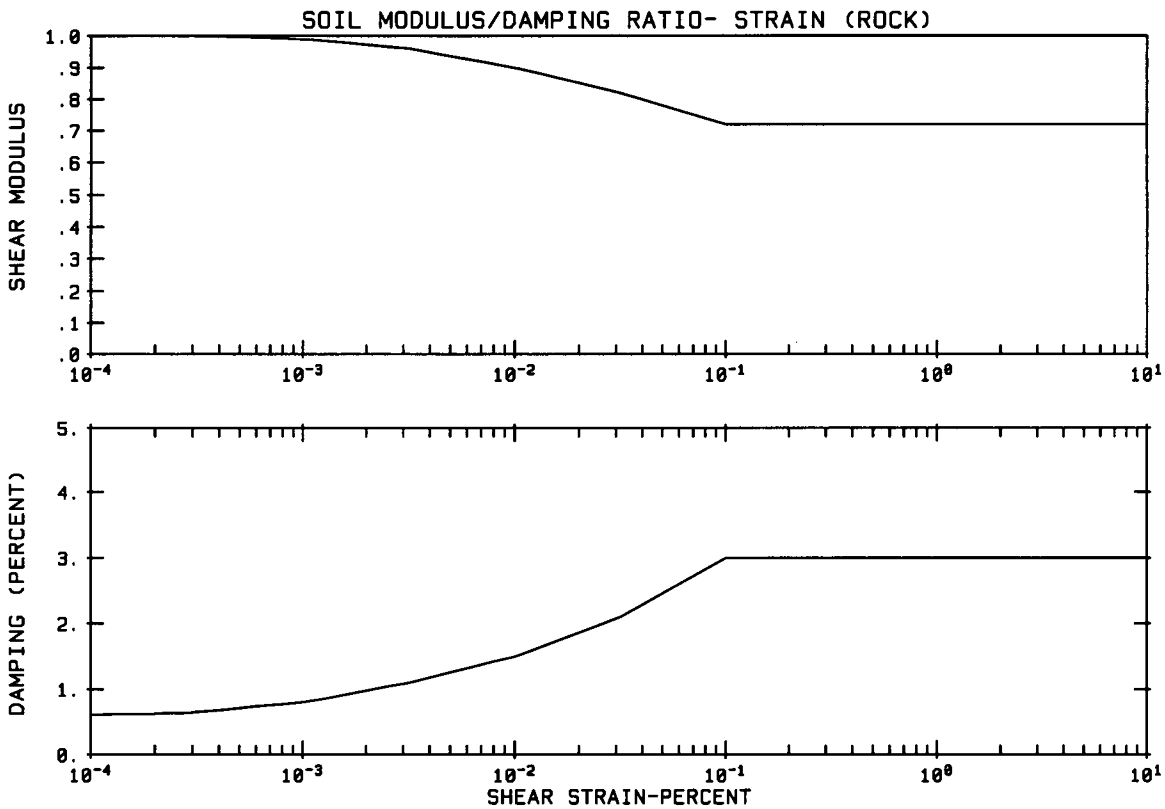
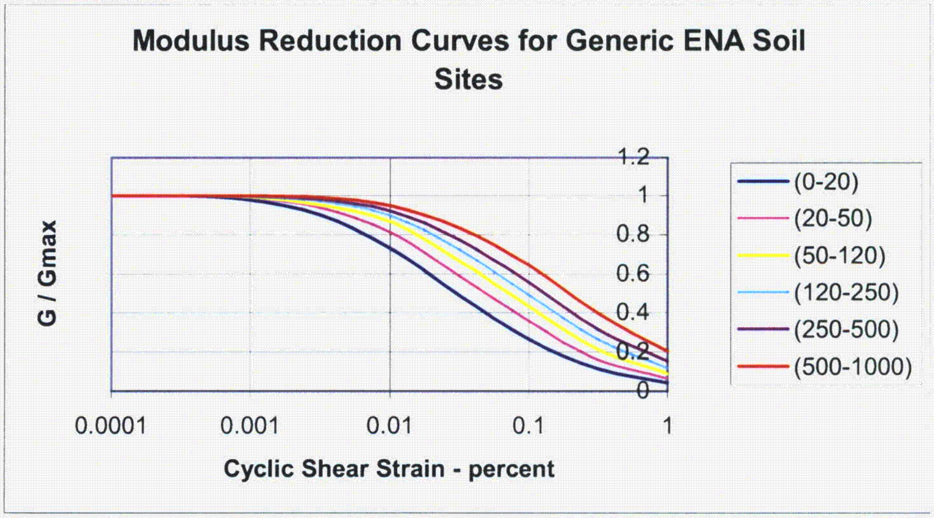


Figure 3.7.1-15
Strain Dependent Properties of Rock Material



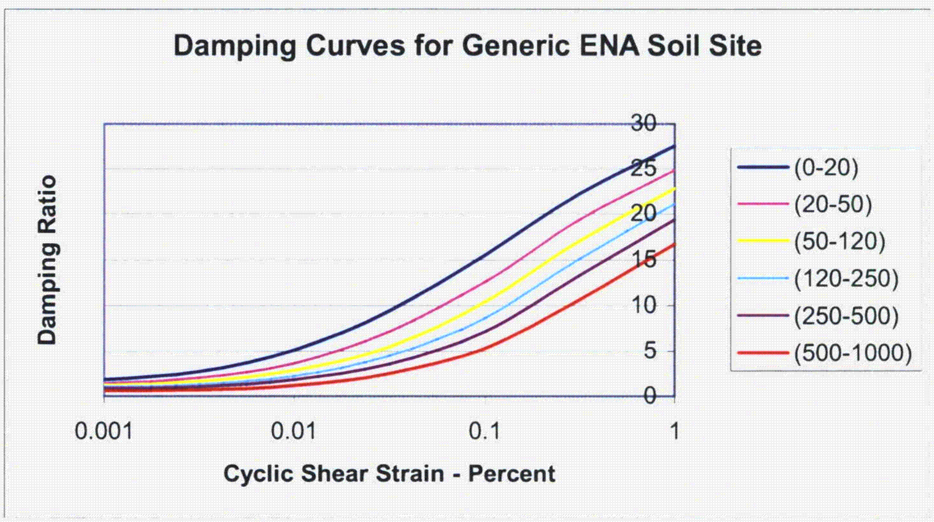
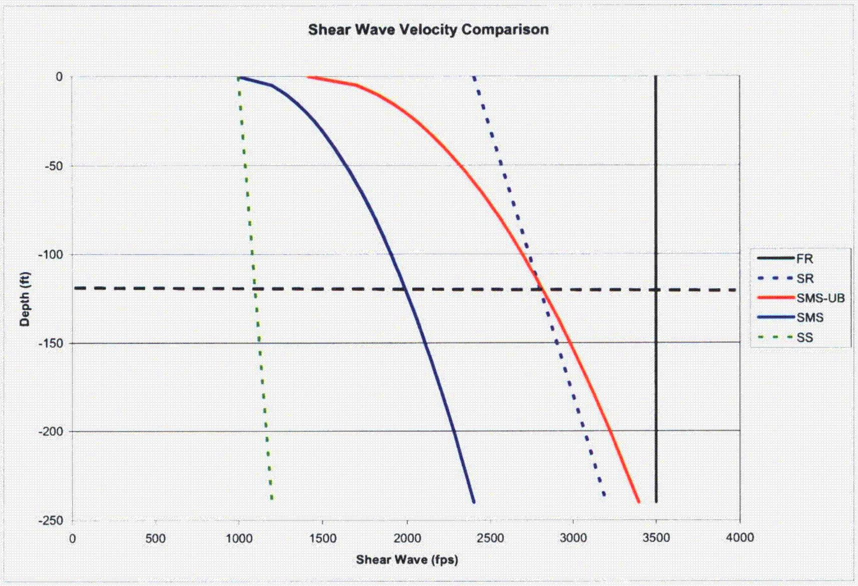
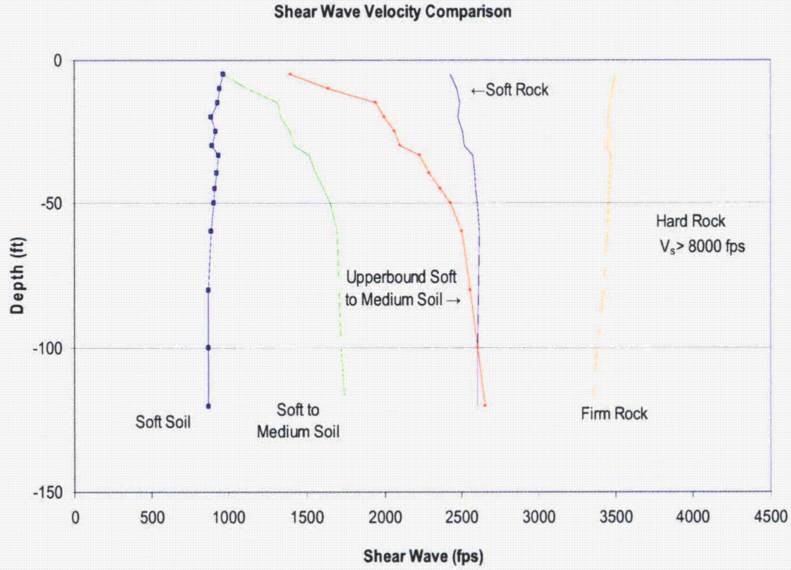


Figure 3.7.1-16
Strain Dependent Properties of Soil Material



(a) Initial Properties



(b) Strain-iterated shear wave velocity profiles

Note: Fixed base analyses were performed for hard rock sites. These analyses are applicable for shear wave velocity greater than 8000 feet per second.

Figure 3.7.1-17
Generic Soil Profiles

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR03-015
Revision: 2

Question:

In Page 48 of 154, Westinghouse illustrated that some effects (water table, soil layering, soil degradation model, etc.) are not significant to the seismic response of the nuclear island (NI) structures. Because these results are applied for the AP1000 design, the staff requests Westinghouse provide technical basis for making these conclusions. In addition, Westinghouse needs to demonstrate the combination of these effects is also insignificant to the seismic response of the NI structures.

Westinghouse Response:

Section 4.4.1.1 is amplified as shown below to provide additional technical basis for the selection of the soil parameters used in the AP1000 3D SASSI design cases. The soil cases selected for the AP1000 utilize the same parameters on depth to bedrock, depth to water table and variation of shear wave velocity with depth as those used in the AP600 design analyses. The selection of these parameters for the AP1000 is based on the results and conclusions from the AP600 soil studies summarized in Table 4.4.1-1A. These AP600 soil studies considered variations of the parameters and combinations thereof in establishing the design soil profiles. The conclusions of the AP600 soil studies are applicable to the AP1000 due to the identical footprint to the AP600 and the similarity in overall mass. The height of the shield building is increased by about 20'. The total weight of the nuclear island increases by about 10%.

Parametric analyses of the AP1000 were performed for six soil cases as described in Section 4.4.1.2. These analyses used the same assumptions for depth to bedrock, depth to water table and variation of shear wave velocity with depth as were used in the AP600 and AP1000 3D SASSI design analyses. These analyses confirm that the response of the AP1000 is similar to that of the AP600 for these soil cases with the AP1000 fundamental response occurring at lower frequencies due to the increased height and mass of the nuclear island. Based on the similar response in these analyses, it is concluded that the governing parameters obtained for the AP600 soil studies are also applicable to the AP1000.

Westinghouse has addressed soil degradation in RAI-TR03-10. Tables of strain-iterated shear wave velocity used in the generic analyses are shown. Figure RAI-TR03-15-1 shows the bounds of these strain-iterated shear wave velocity profiles. The combination of effects of the different soil parameters is reflected in these bounds. Figure RAI-TR03-15-2 shows how a COL applicant could demonstrate that the site is enveloped by generic seismic design basis. The applicant would define its site geotechnical parameters as defined in DCD Section 2.5 and would justify why the site is within the bounds of the AP1000 generic analyses that have been considered in this technical report. These parameters would include the soil profiles used in the PSHA (probabilistic seismic hazard analysis) analyses, which could then be compared to Figure

RAI-TR03-15-1. Subsequent discussions between the COL applicant and the NRC may uncover a parameter for which more justification is required to show that the impact of this parameter on the response is small. This justification could be done with the AP1000 2D model. An example of how a 2D parametric study would be used is shown in Figure RAI-TR03-15-3 and RAI-TR03-15-4. If the parametric 2D SASSI studies show that the effect could be significant (e.g., 90% of the design spectrum, see Figure RAI-TR03-15-4) when compared to the 2D design spectra, a 3D SASSI study would then be performed. If the 3D SASSI analyses show some exceedances at the critical locations, the applicant would then proceed to show that sufficient margin exists in the design to accommodate these exceedances.

The effect of water table on the seismic response of the nuclear island structures is shown in figures RAI-TR03-15-5 through RAI-TR03-15-7. Case 1 (SM) shows the results for the soft-to-medium generic case profile which assumes water table at grade. Case 2 (SM-NW) results are for the same soil condition except the water table is below the bottom of the soil profile at 120' below grade. As can be seen there is negligible difference between the two cases for the horizontal response. The vertical response due to the design profile with the water table at grade (Case 1) is more conservative than that for the dry soil profile (Case 2). This result is similar to the results in the AP600 study which are summarized in section 4.4.1.1 which states:

> "These studies showed that the change of water table elevations had insignificant effect on the horizontal results. Comparison of the vertical responses showed that the water table at the grade level controlled the responses in the frequency range of 2 to 8 hertz."

Thus, the generic analyses are conservative for sites with a lower water table.

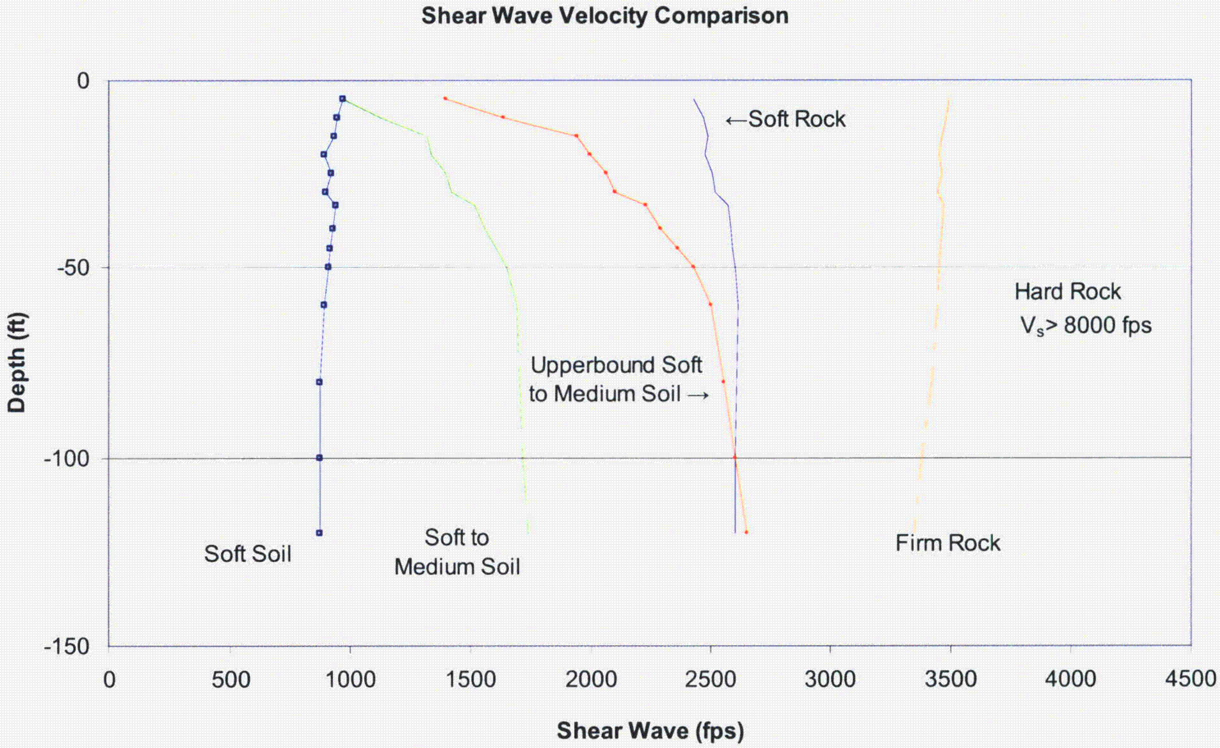


Figure RAI-TR03-15-1-Strain-iterated shear wave velocity profiles

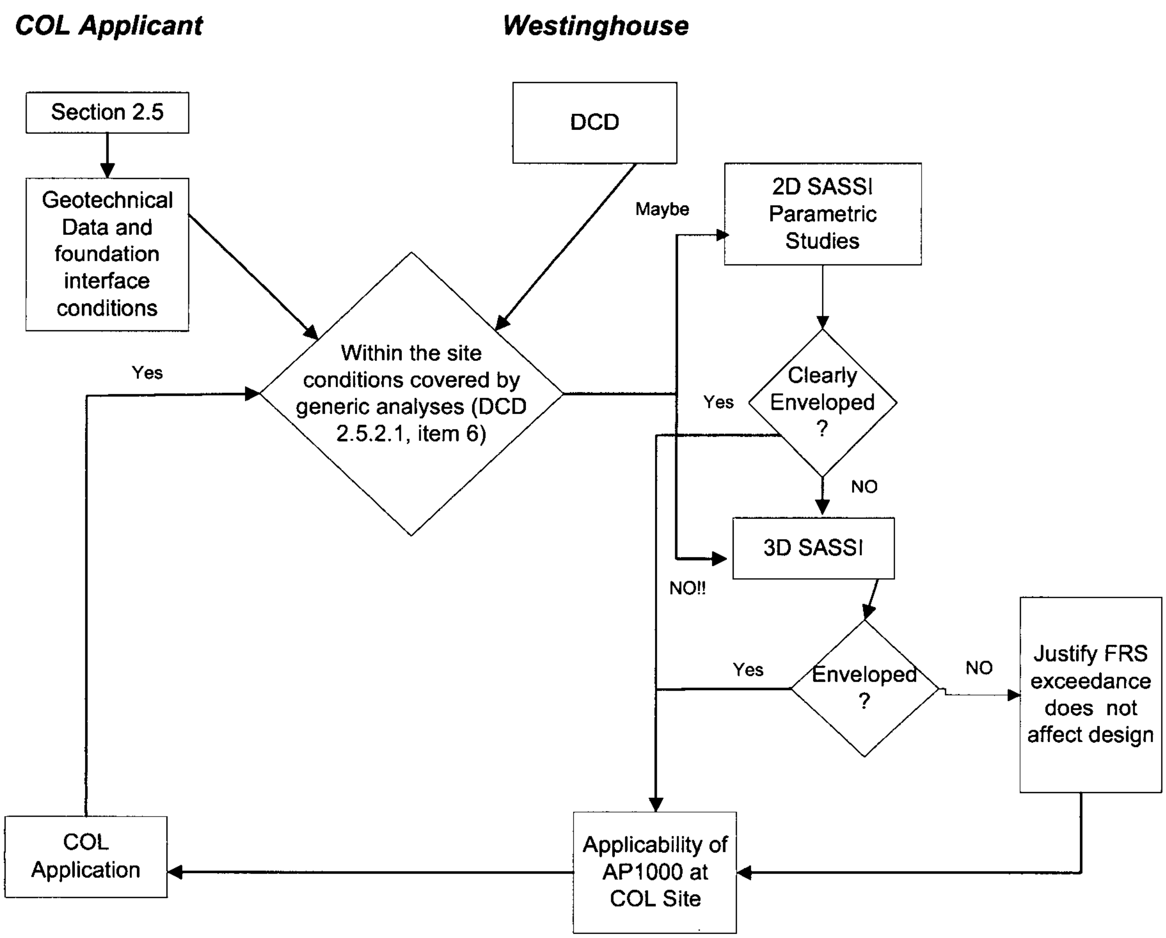


Figure RAI-TR03-15-2-COL Application process for generic design

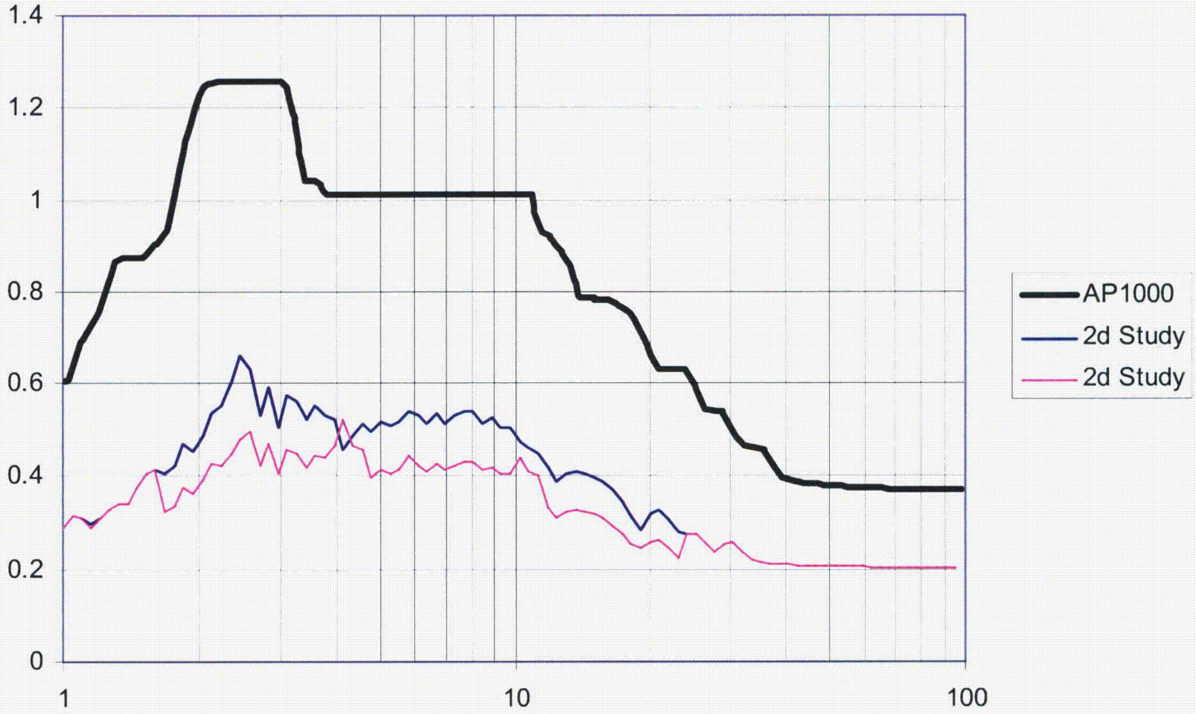


Figure RAI-TR03-15-3- 2D parametric studies demonstrate site is clearly enveloped by 2D design spectra

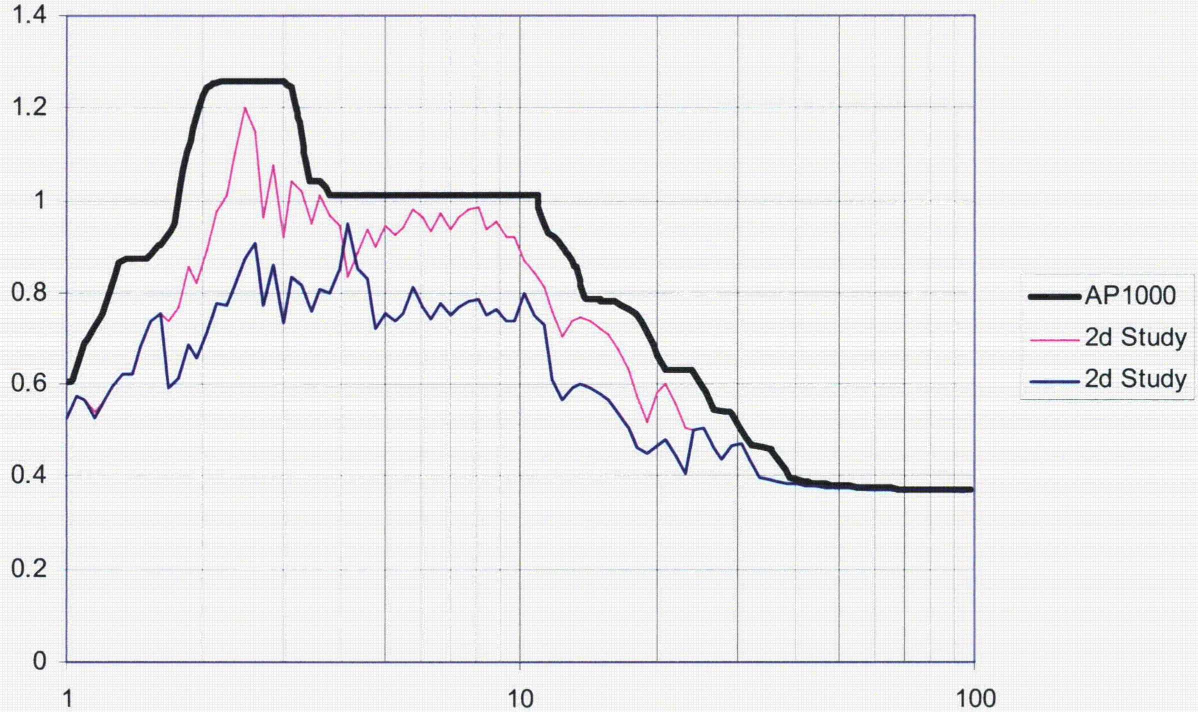


Figure RAI-TR03-15-4- 2D parametric study demonstrate that further studies may be required

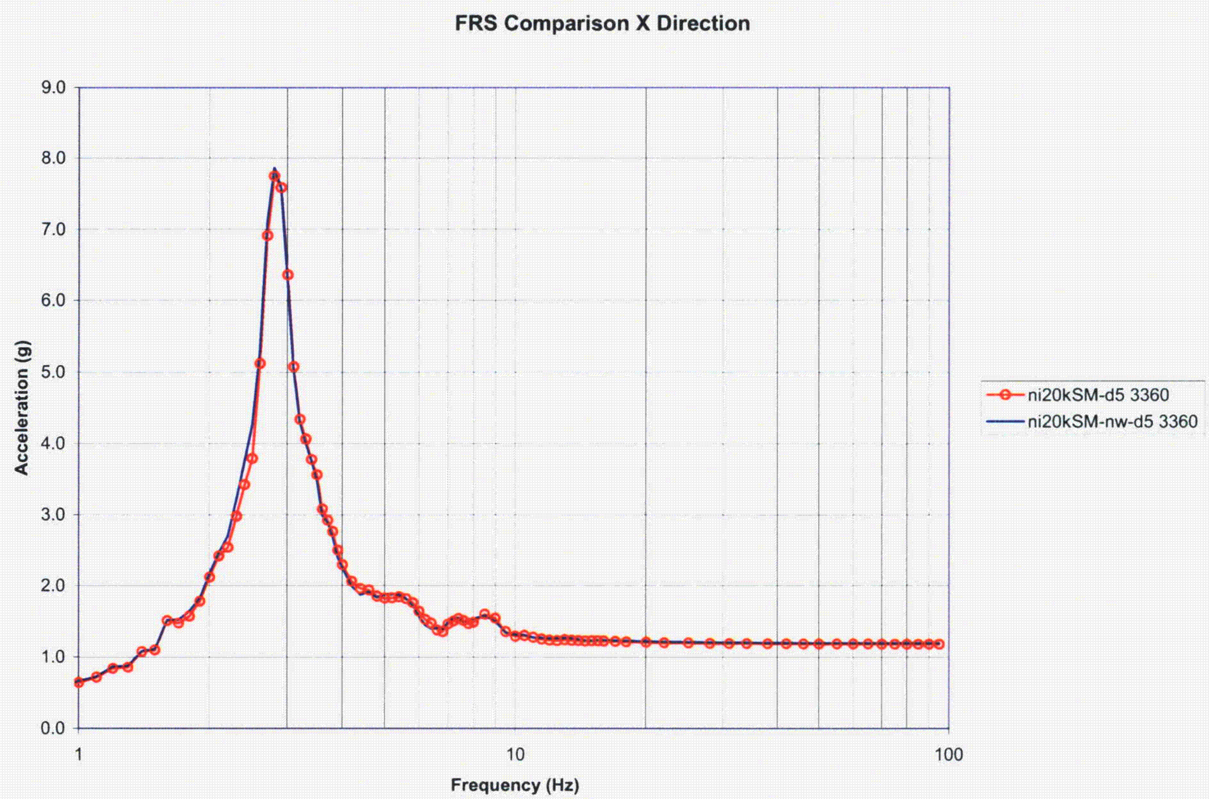


Figure RAI-TR03-15-5- Effect of water table variation in horizontal direction (X)

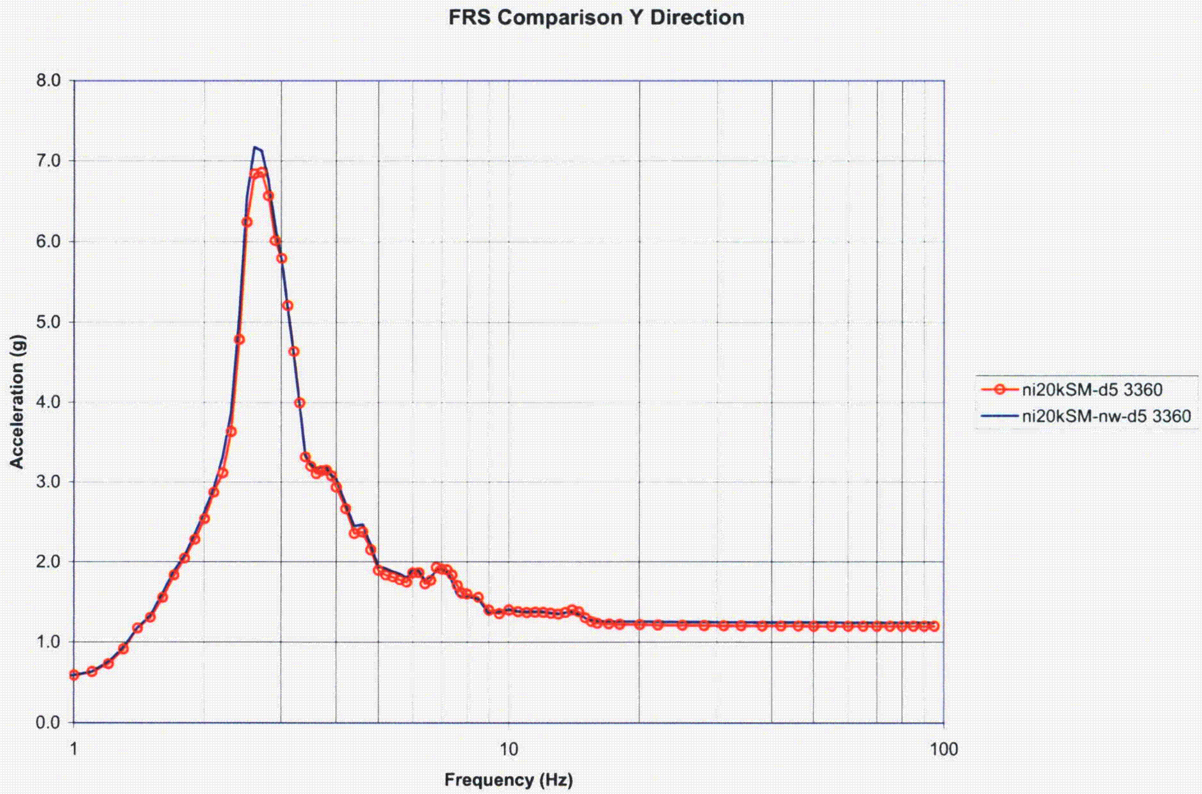


Figure RAI-TR03-15-6-Effect of water table variation in horizontal direction (Y)

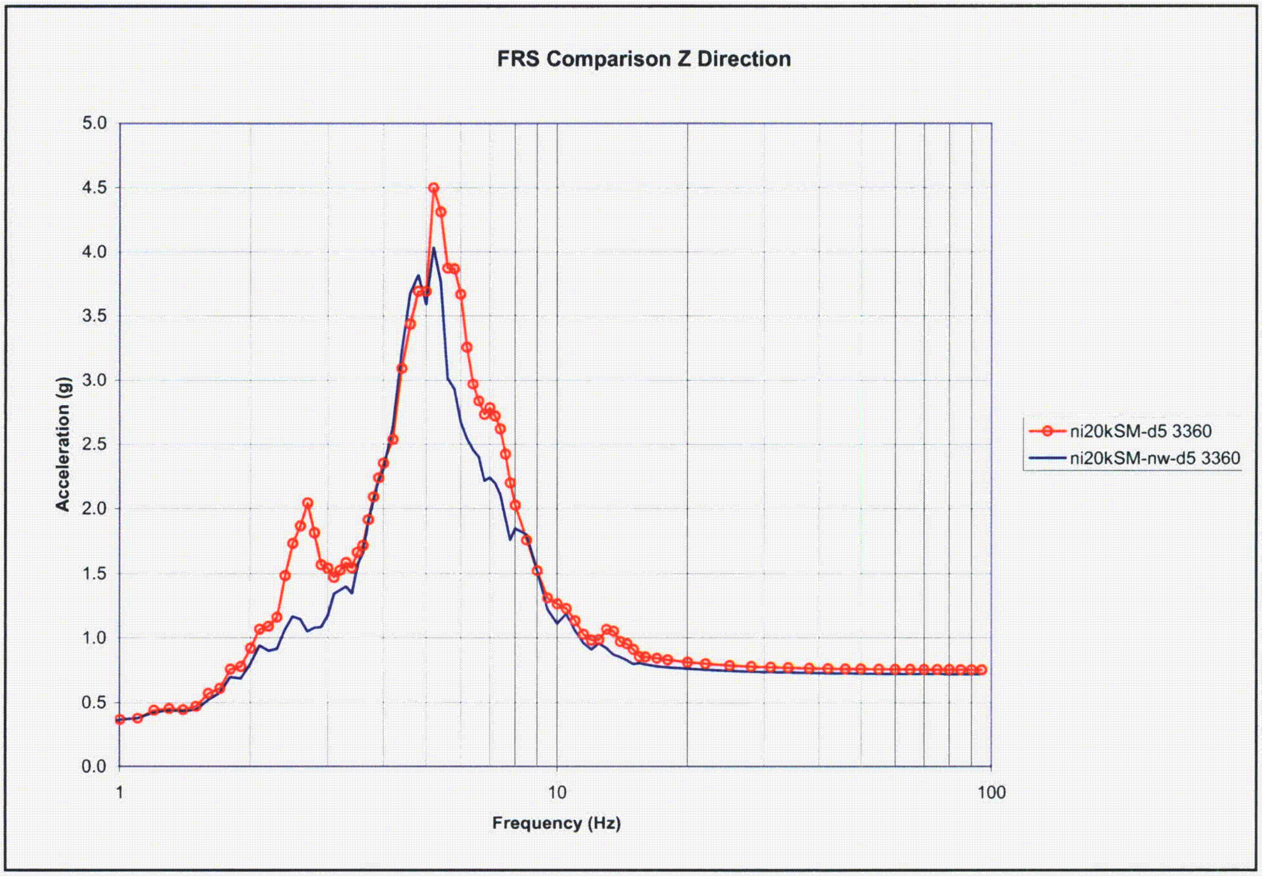


Figure RAI-TR03-15-7- Effect of water table variation in horizontal direction (Z)

Design Control Document (DCD) Revision:
None

PRA Revision:
None

Technical Report (TR) Revision:

The Technical Report will be revised to include the RAI responses in an appendix. Thus the proposed DCD revisions will also become a part of the technical report. Sections 4.4.1 and 4.4.1.1 have been revised as shown below in Revision 1 of the Technical Report.

4.4 Soil Cases and SSI Analyses

4.4.1 2D SASSI Analyses and Parameter Studies

This section describes the parametric analyses performed using 2D models in SASSI to select the design soil cases for the AP1000. The AP1000 footprint, or interface to the soil medium, is identical to the AP600. The AP1000 containment and shield building are 20' 6" taller than AP600. Results and conclusions from the AP600 soil studies are summarized since the behavior of the AP1000 is expected to be similar and results from AP600 provide guidance in the selection of the generic cases for the AP1000. Five soil and rock cases are selected as follows: hard rock; firm rock; soft rock; upper bound soft to medium soil, soft to medium soil, and soft soil. These are the same as the cases analyzed for the AP600 except that the soft soil case is added and the soft rock case (v_s =2500 feet per second) for the AP600 has been replaced by firm rock (v_s = 3500 feet per second) since the 2D SASSI parametric analyses show that the firm rock case is more significant than on AP600 due to the additional height of the shield building.

4.4.1.1 AP600 Soil Studies

The AP600 studies are summarized below. They are described in Appendices 2A and 2B of the AP600 DCD (Reference 7).

A survey of 22 commercial nuclear power plants in the United States was conducted to identify the subsurface soil profiles and the range of soil properties at these plants as part of the AP600 design certification. The survey included nuclear power plants sites both east and west of the Rocky Mountains. Based on this survey five generic soil profiles (soft soil, soft to medium soil, soft rock and step profile in Figure 4.4.1-1 plus hard rock) were established ranging from soft soil to hard rock. Using these soil profiles, 2D soil-structure interaction analyses were performed to determine site geotechnical variables which induced the highest nuclear seismic response during an earthquake.

The series of parametric studies performed using 2D SASSI models for AP600 certification is shown in Table 4.4.1-1A. Note that for AP1000, 2D SASSI parametric studies were performed and they are shown in Table 4.4.1-1B. These SASSI models consisted of 2D lumped mass stick models coupled with a 2D model of the foundation. The conclusions made based on these parametric studies for the AP600 configuration are given below.

Soil properties were specified to a depth of 240 feet below grade. Analyses were performed for various depths to base rock. In each case, the soil properties above the base rock were those of the soil and the base rock was assumed to have shear wave velocity of 8000 feet per second. The analyses performed for a depth to base rock of 240 feet are described in Table 4.4.1-1A as a deep soil site and results would also be representative of deeper soil sites. Soil sites were found to control the AP600 nuclear island response at frequencies below about 4 hertz for horizontal response and 8 hertz for vertical response while the hard rock site controls the response at higher frequencies. The studies of depth to base rock showed that the response was not very sensitive to the depth. The depth-to-base rock of 120 ft generally gave the higher response for each of the soil profiles and was therefore specified for the 3D SASSI design cases. The shallower depth models gave a higher building response at high frequencies, but these responses were lower than those for hard rock. The deeper models had greater radiation damping reducing the overall response. The dominant AP1000 building mode shapes are similar to the AP600 and the frequencies are lower. Since the response of the AP600 was relatively insensitive to depth and the dominant modes of the AP600 and AP1000 are similar, using a depth-to-base rock of 120 ft is also appropriate for the AP1000.

The soil properties associated with the lower and upper bound sandy soils (soft-to-medium soil profile) bound the range of properties associated with clays with plasticity indices from 10 to 70 as shown in Figure 2B-13 of the AP600 DCD. SSI analyses were performed for clay profiles and concluded that the responses for clay profiles were bounded by those for the design soil profiles.

The effect of depth to water table was studied for the soft-to-medium soil case with the depth to base rock of 120 feet. Cases were analyzed for water table at grade, for water table at the foundation level (40 foot depth) and for a dry site. For cases where the water table was below grade, the Poisson's ratio for soil above the water table was also varied from 0.25 to 0.35. These studies showed that the change of water table elevations had insignificant effect on the horizontal results. Comparison of the vertical responses showed that the water table at the grade level controlled the responses in the frequency range of 2 to 8 hertz. The increase in response was mainly due to an increase in foundation effective motion, which results from an increase in the P-wave velocity in conjunction with the SSI frequency for this case. Thus, the water table was specified at grade for the 3D SASSI design cases. Since the mass of the AP1000 is similar to that of the AP600 the vertical SSI frequency and response are similar. Thus, the specification of the water table at grade is also appropriate for the AP1000 soil sites.

The change in degradation curves between the 1970 Idriss and Seed and 1990 Seed degradation curves was not significant. The AP1000 uses the EPRI 93 degradation curves. These degradation curves have been used in AP1000 2D SASSI parametric analyses and do not significantly affect the SSI response, and thus should not result in a change in the selection of the generic soil profiles.

Analyses were also performed for a layered soil profile with step-wise change in shear wave velocity. The step-wise layered soil profile had a layered profile with shear wave velocity of 1000 feet per second to a 40-foot depth, 1800 feet per second between 40-foot and 80-foot depth, and 4300 feet per second for depth greater than 80 feet. The response for this profile is enveloped by the soft rock, soft-to-medium, and rigid base response. In addition the cases previously described in the depth to base rock studies showed that the sharp contrast in shear wave velocity (layering) was enveloped by the design cases with depth to base rock at 120 feet. Based on this study and the studies of depth to base rock, the step-wise layered soil profile was not included as a design case for AP600 nor need it be included for AP1000.

Analyses including adjacent buildings showed that the effect of the adjacent buildings on the nuclear island response was small. Based on this, the 3D SASSI analysis of the nuclear island can be performed without adjacent buildings. The nuclear island does affect the response of the adjacent buildings and the results of the 2D SASSI analyses are used for design of the adjacent buildings for both the AP600 and AP1000.

SASSI analyses for hard rock sites were compared to fixed base results. A fixed base analysis is adequate for sites in excess of 8000 fps.

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR03-017
Revision: 2

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 t~~The maximum acceptable change in velocity profile within a depth equal to the width of the basemat is evaluated by the comparison against the AP1000 generic soil profiles as required by item 6 of DCD subsection 2.5.2.1 (see RAI-TR03-010, Rev 2). It is noted that if there is a property inversion (i.e. stiff soil above soft soil) at a specific site, then a site specific analysis will be performed for this case. Six~~Four~~ design soil profiles are analyzed. Four of these ~~These~~ are the same profiles as were ~~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 AP600 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: 2

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. The "All Nodes" connectivity is used in the updated seismic models described in TR03, Rev 1.

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 Rev 16 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~~ holder 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 or as-built changes in component mass, center of gravity, and support configuration based on 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, does not exceed the design basis floor response spectra by more than 10 percent. The Combined License holder will complete this reconciliation prior to fuel load.

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.

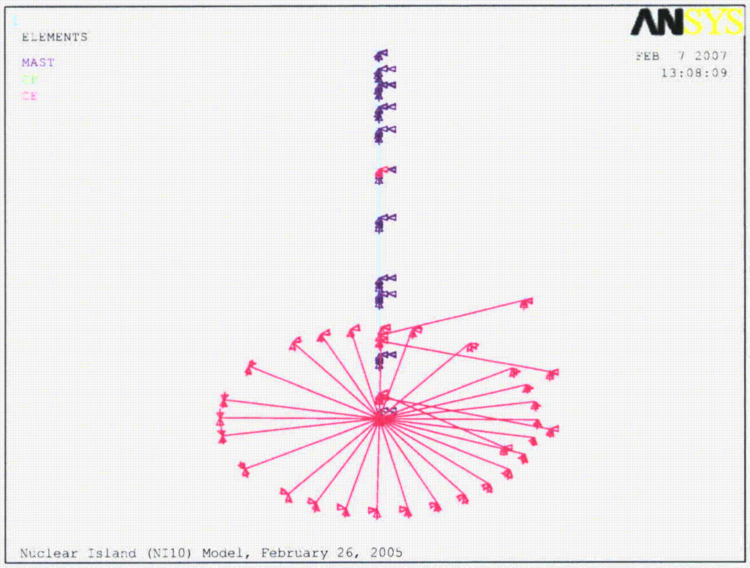


Figure RAI-TR03-020-1 - Rev 4 SCV

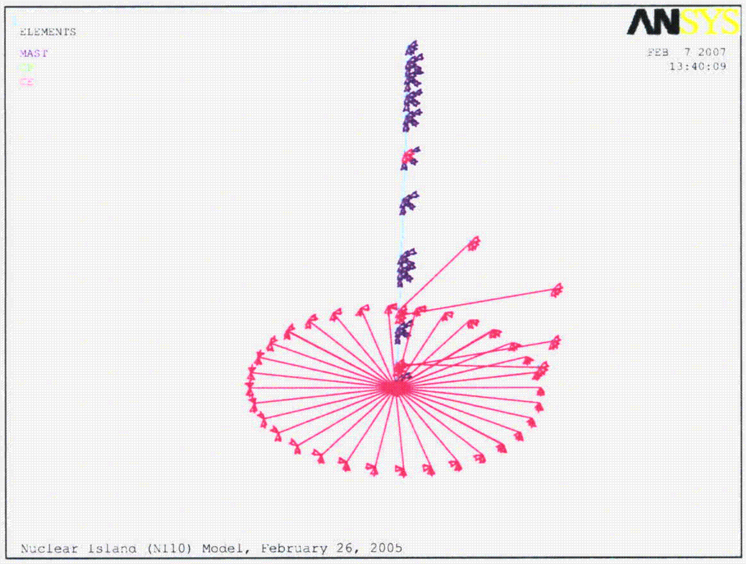


Figure RAI-TR-03-020-2 - All Nodes

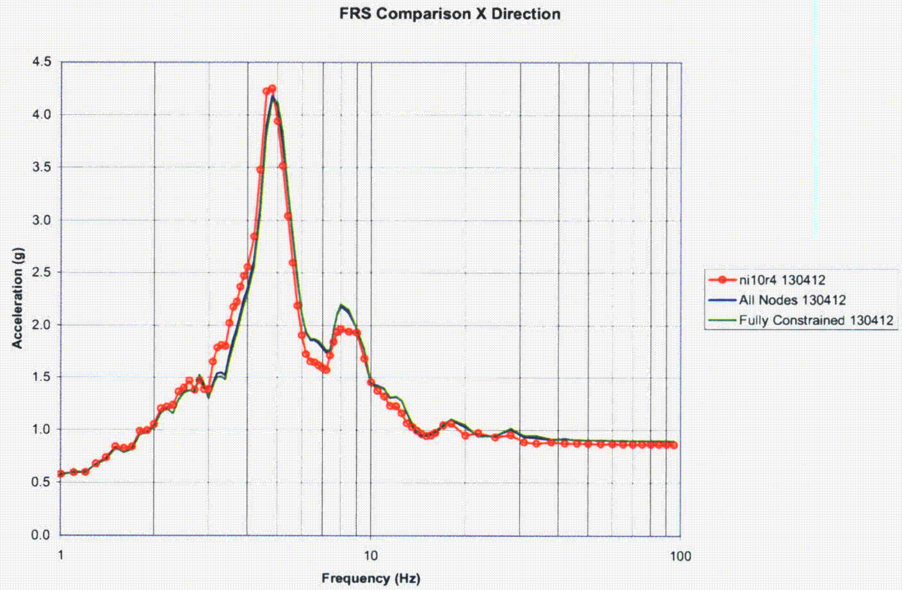


Figure RAI-TR03-020-3 – Effect of Connection - X FRS at Elevation 224'

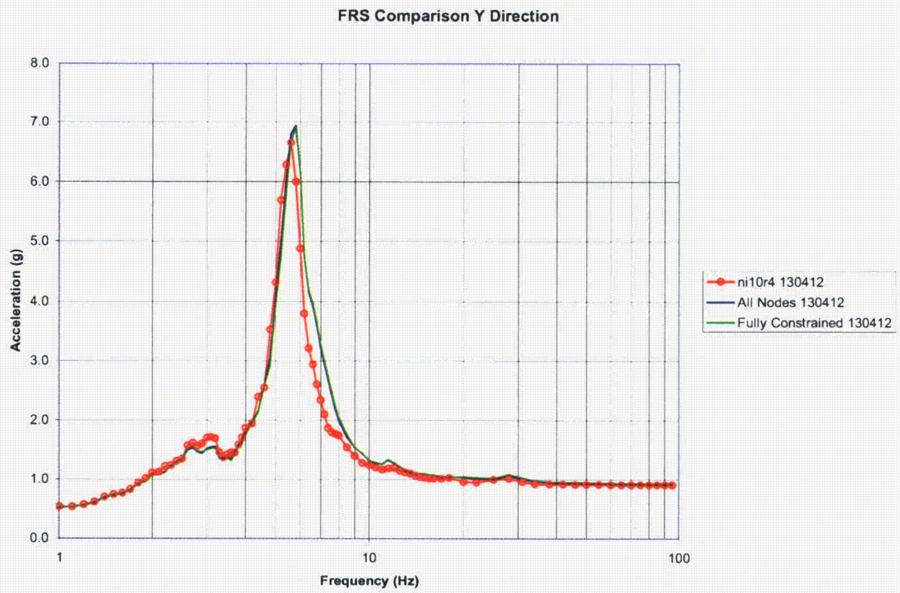


Figure RAI-TR03-020-4 – Effect of Connection - Y FRS at Elevation 224'

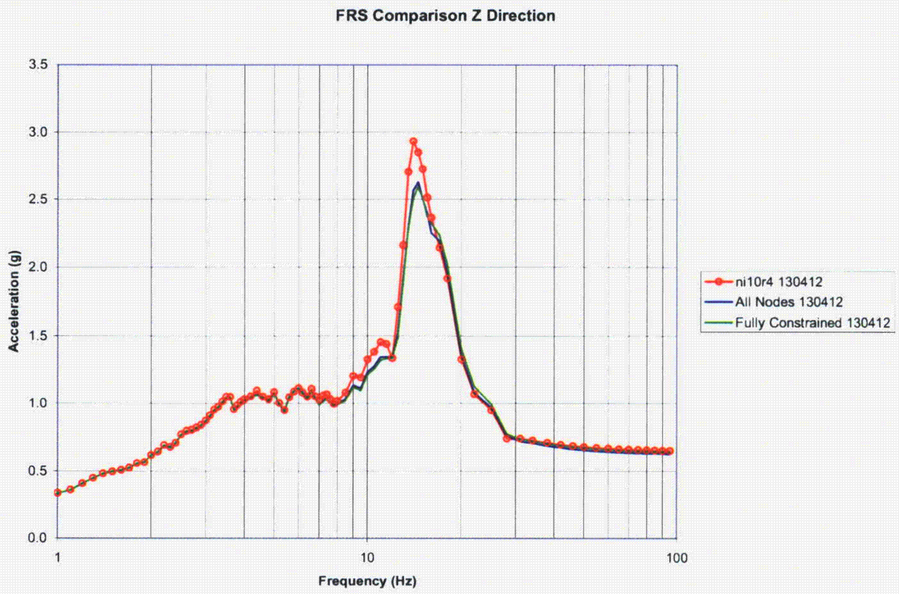


Figure RAI-TR03-020-5 – Effect of Connection - Z FRS at Elevation 224'

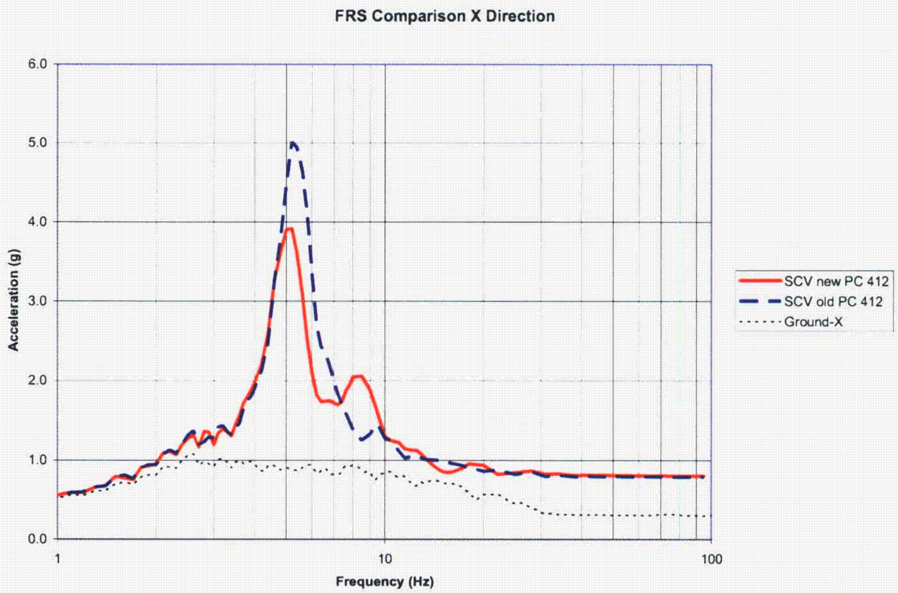


Figure RAI-TR03-020-6 – Effect of Polar Crane - X FRS at Elevation 224'

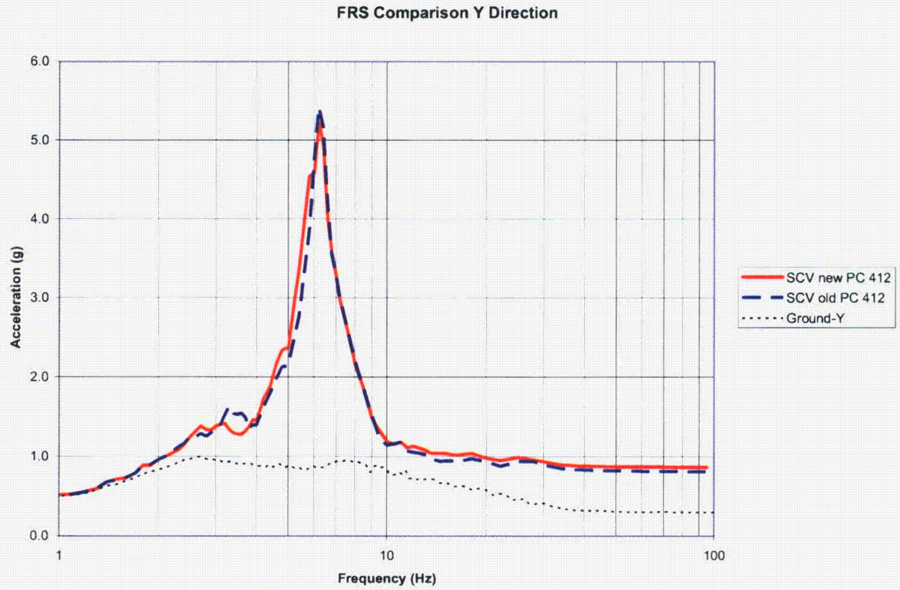


Figure RAI-TR03-020-7 – Effect of Polar Crane - Y FRS at Elevation 224'

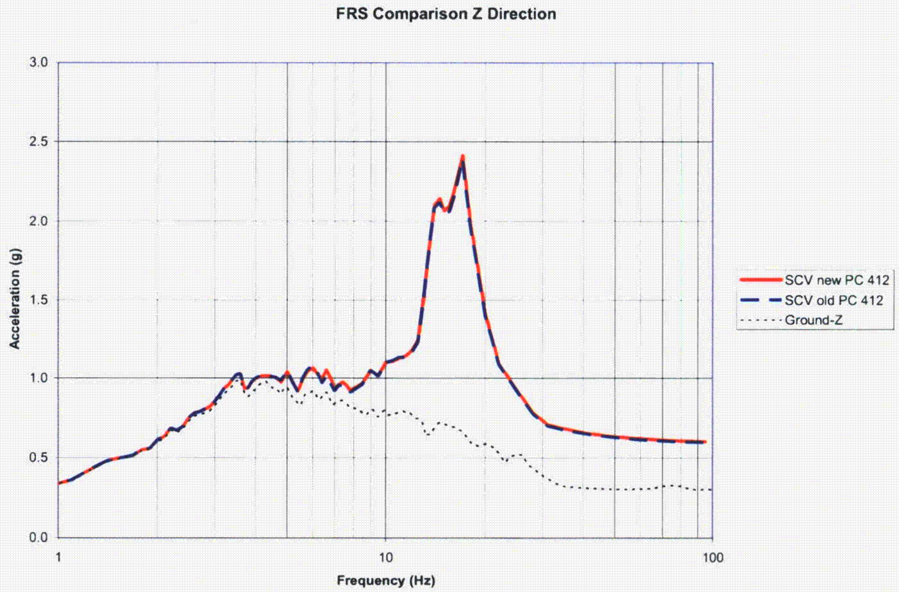


Figure RAI-TR03-020-8 – Effect of Polar Crane - Z FRS at Elevation 224'

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 in the Technical Report, Revision 1, as shown below.

In Revision 1 of the Technical Report the polar crane models and the containment vessel shell model in Table 4.2.4-1 were added 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; response spectrum 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"

Replace Figure 4.2.1-3 with the "All Nodes" model. The revised figure is shown below.

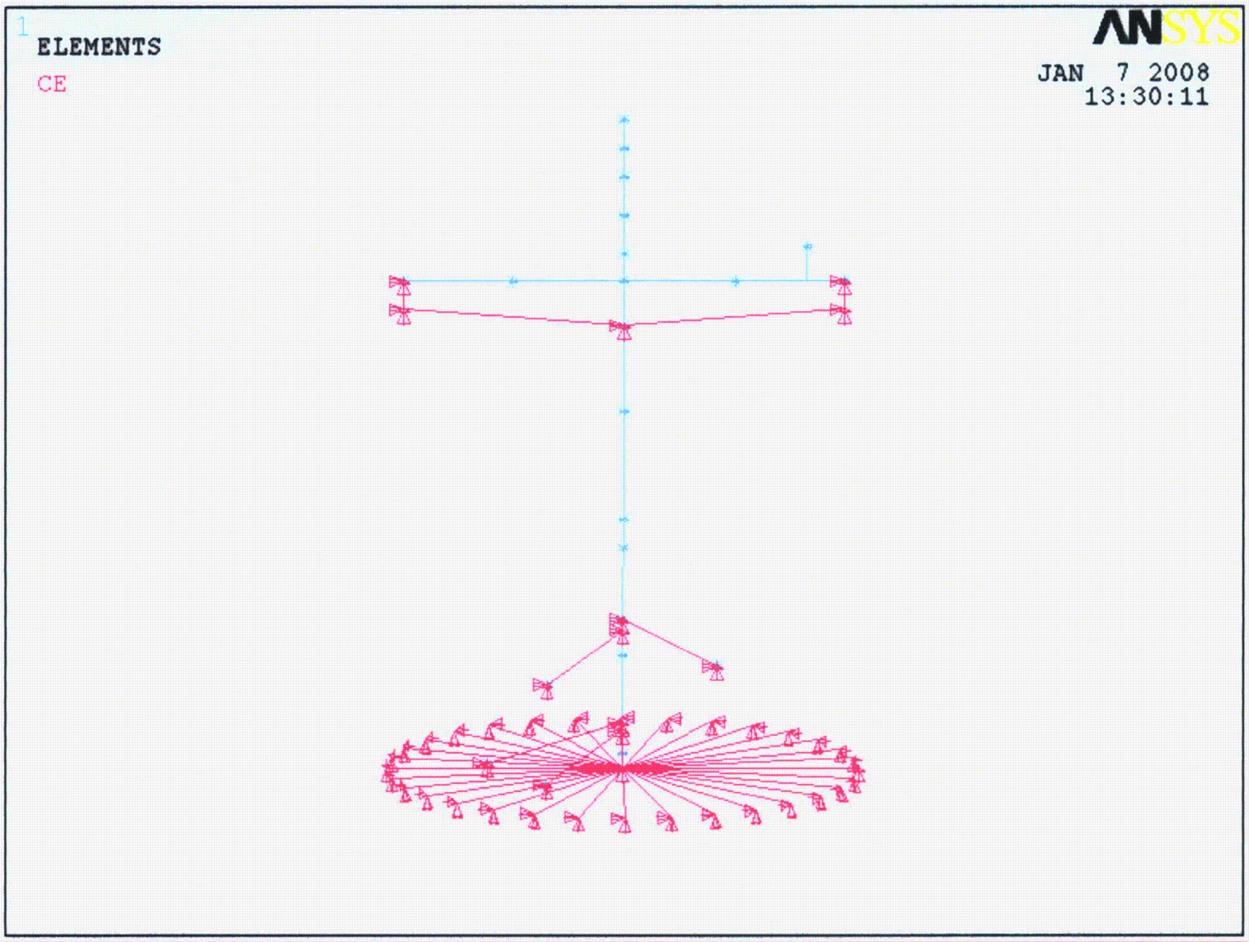


Figure 4.2.1-3 – Polar Crane and Steel Containment Vessel Nodes

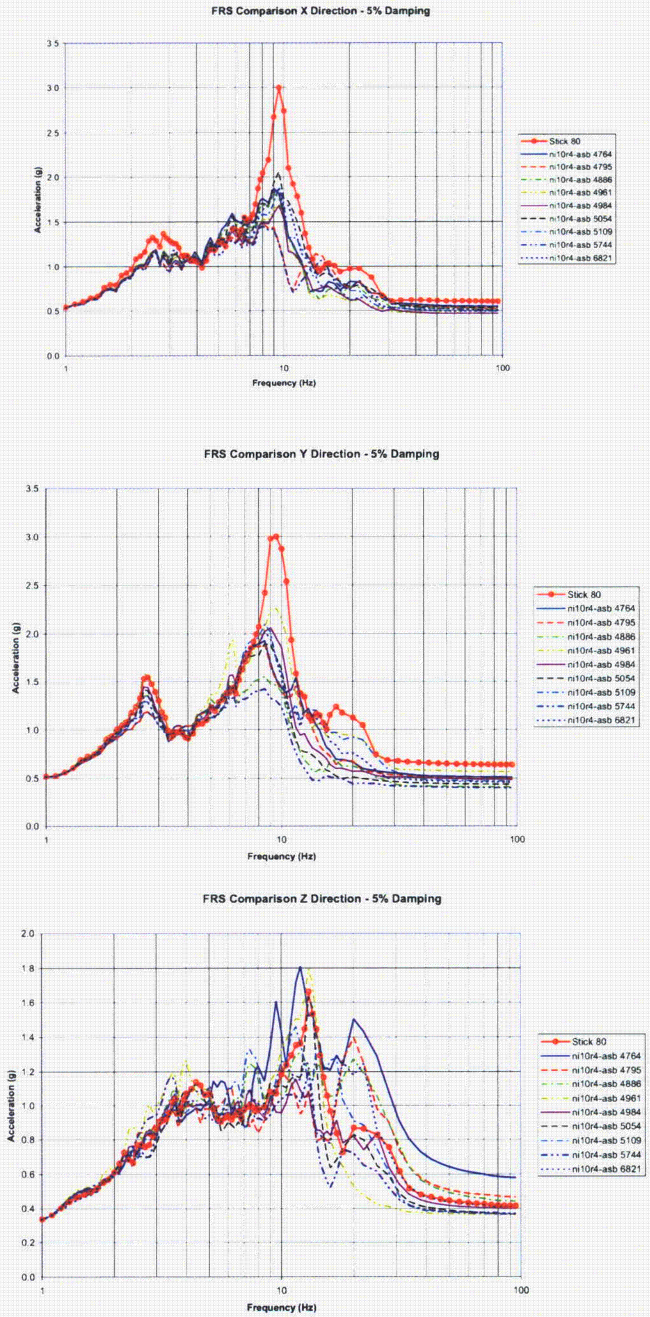


Figure 6.1- 1 - Auxiliary Building at Elevation 135 feet

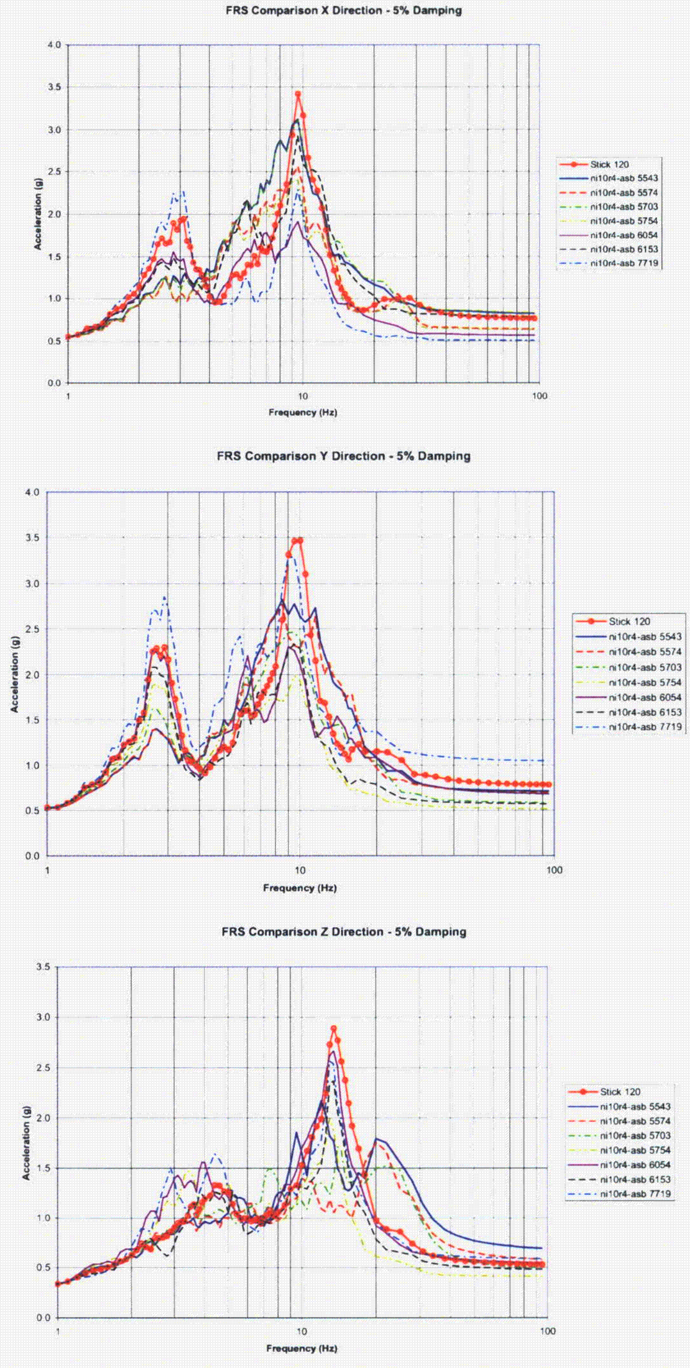


Figure 6.1- 2 - Auxiliary and Shield Building at Elevation 180 feet

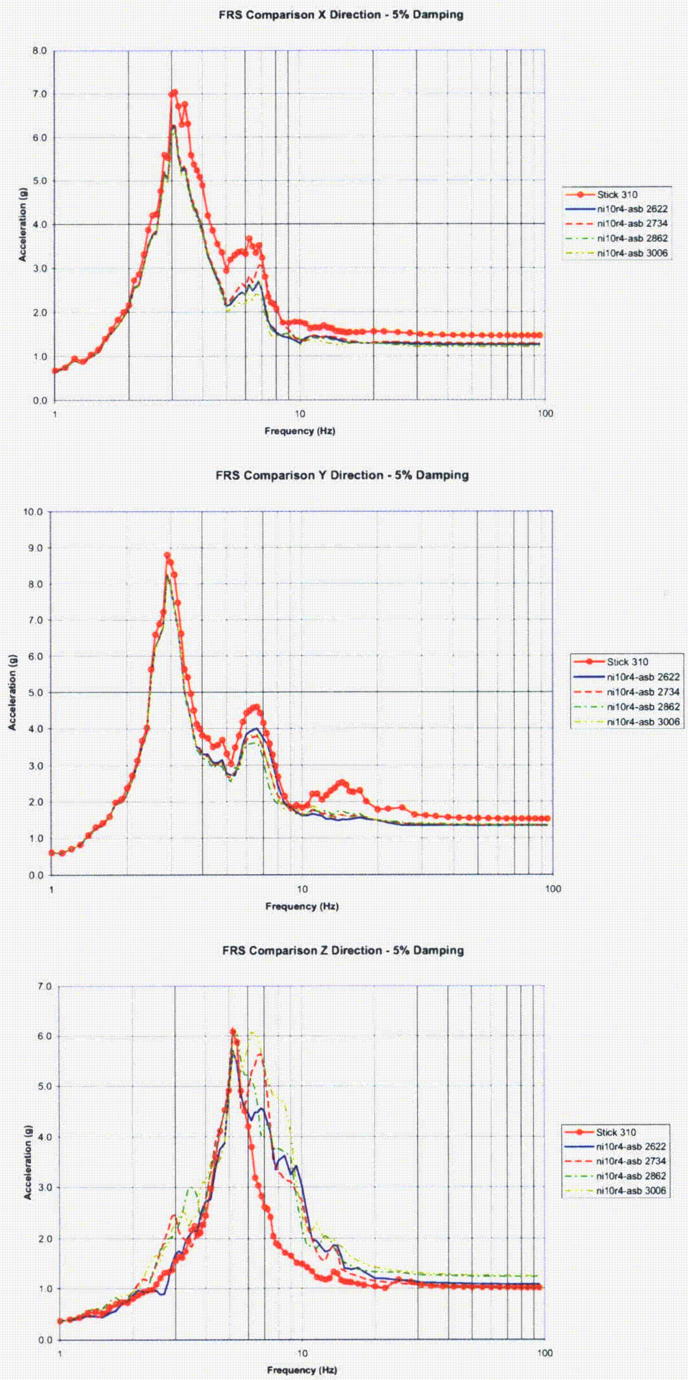
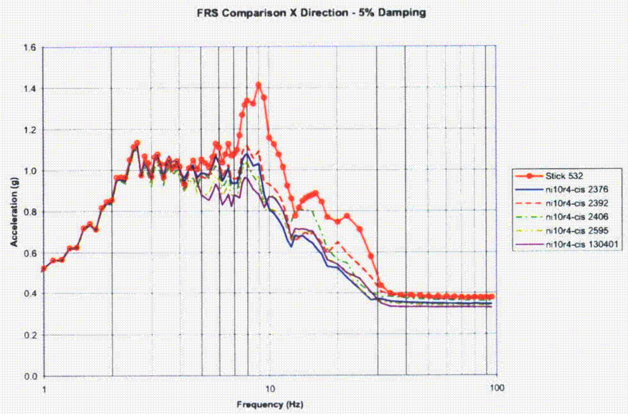
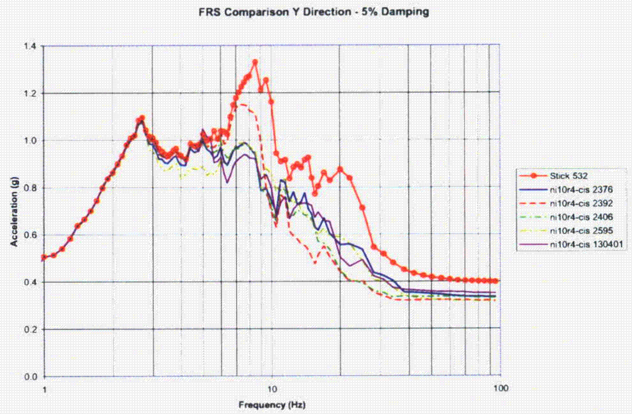


Figure 6.1- 3 - Shield Building at Elevation 333 feet





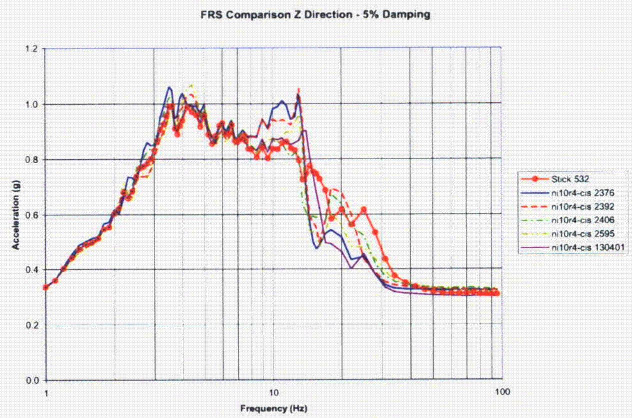


Figure 6.1-4 - CIS at Elevation 99 feet

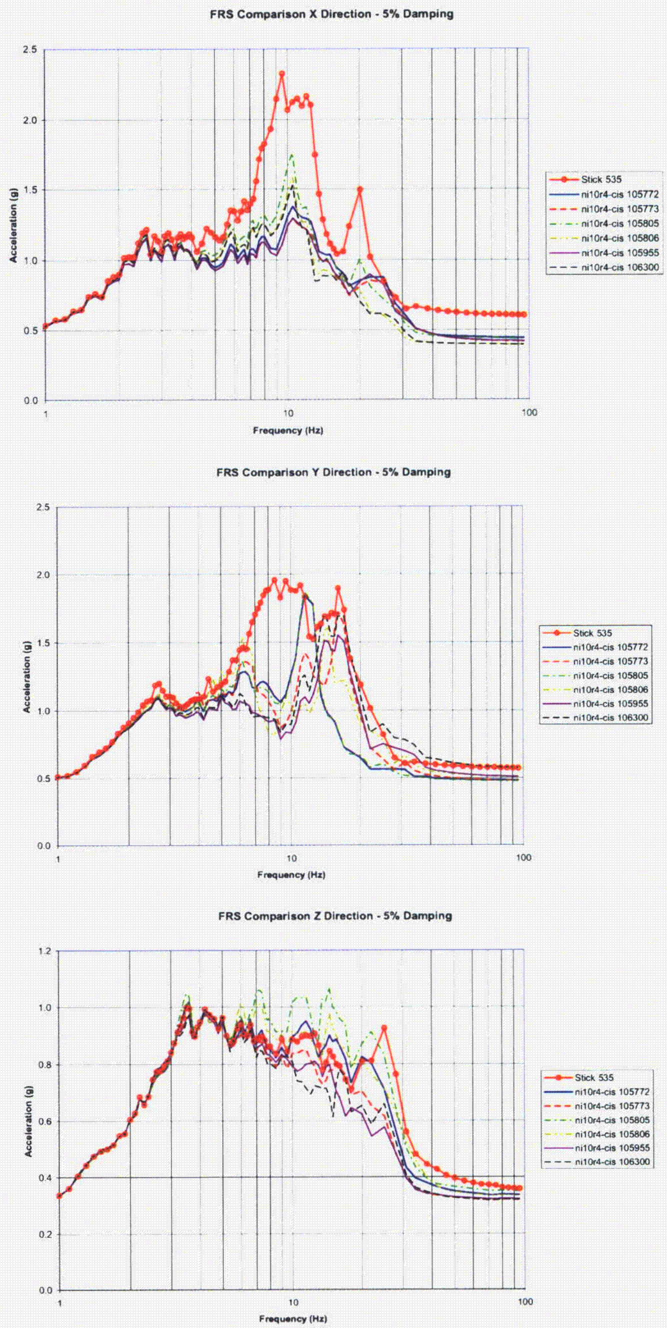


Figure 6.1- 5 - CIS at Elevation 135 feet

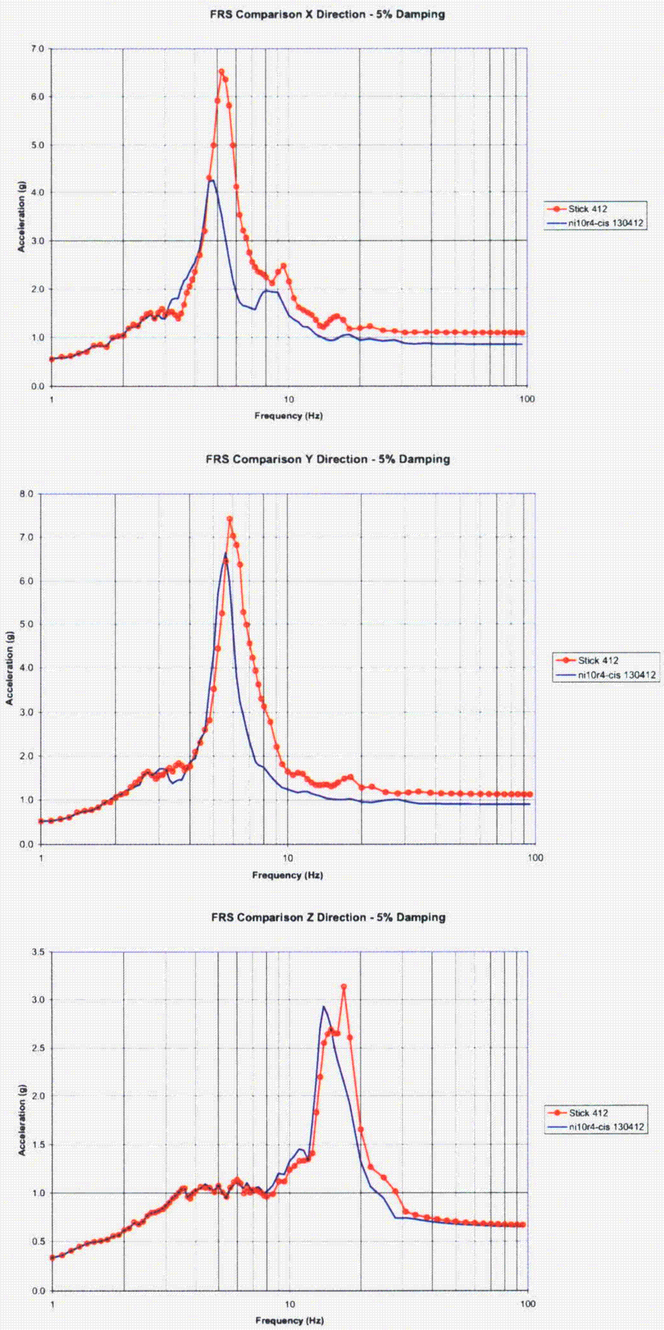


Figure 6.1-6 - SCV at Polar Crane Elevation 224 feet

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR03-022
Revision: 2

Question:

Section 6.3 states "The maximum seismic deflections that were obtained from the time history analyses and SASSI analyses given in Tables 6.3-1 to 6.3-3 for the auxiliary and shield building, containment internal structure, and steel containment vessel." For the staff to properly evaluate this information, the following additional information is needed:

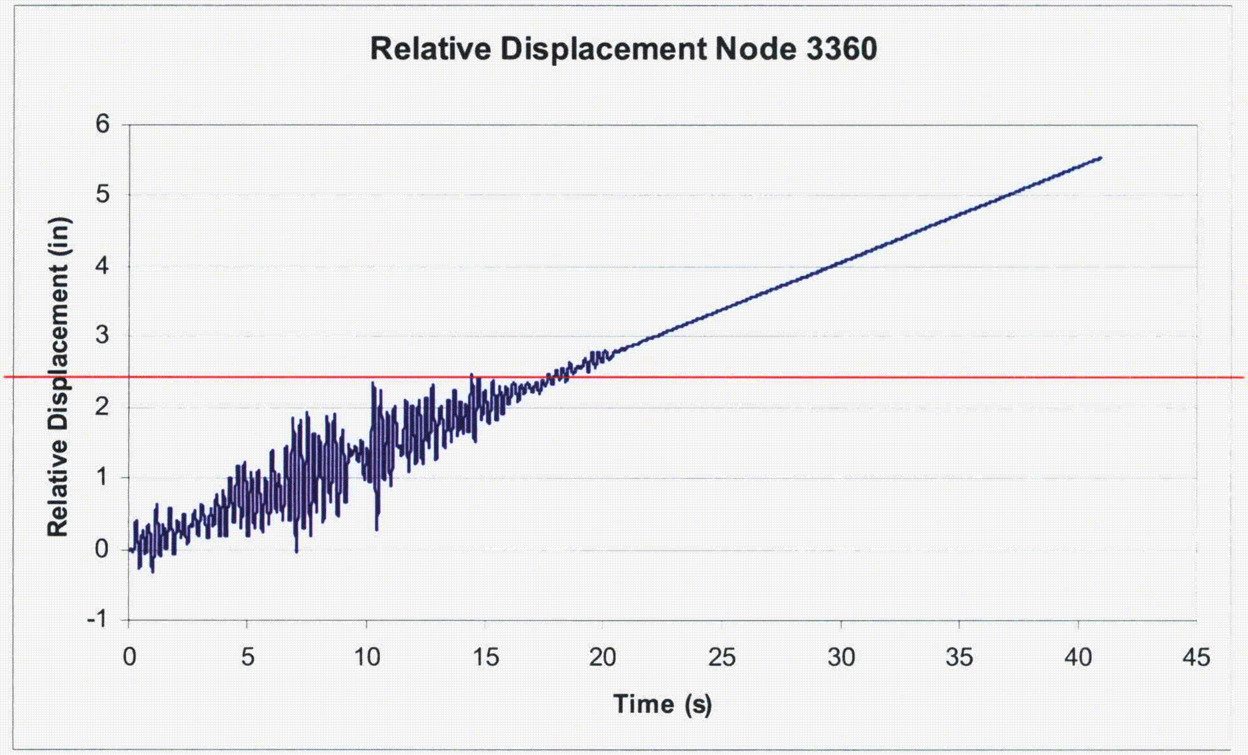
a. Are the deflections in the tables a consistent set, based on the worst-case time history result, or are they an envelope of maximum deflections from all the time history results?

b. How do these tabulated deflections compare to the corresponding deflections obtained from the equivalent static acceleration analyses? Please provide a tabulated comparison, and an explanation of any significant differences.

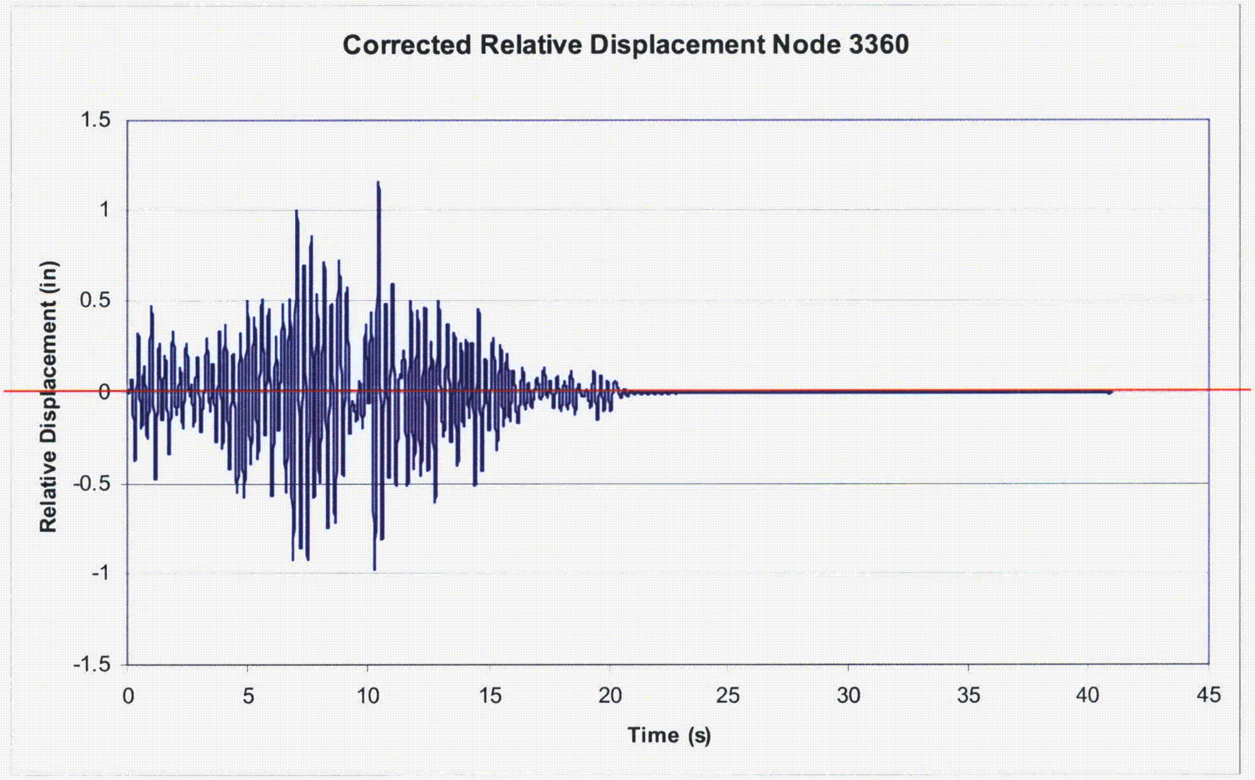
Westinghouse Response:

a. During the October 8-12, 2007 audit, the NRC requested that Westinghouse consider adjusting the deflections obtained from SSI analyses for drift in the frequency domain, and not use a baseline correction that subtracts the slope of the relative displacement multiplied by the time from the relative displacement at each time step. Westinghouse has adopted the recommended approach by calculating displacements internally within the SASSI program based on an analytical complex frequency domain approach that uses inverse fast-fourier transforms (FFT) to compute relative displacement histories instead of double numerical integration in the time domain for computing absolute displacement time histories from absolute acceleration time histories. The analytical approach is more accurate than a typical baseline correction (time integration) algorithm.

~~Deflections have been developed using the model with the robust shield building design. These displacements for the soil and hard rock cases have been obtained relative to the translation of a reference node at the bottom of the foundation and near the center of the basemat. Coordinates of this reference node are x= 993.00 ft, y= 986.00 ft and z= 60.50 ft. The deflections have been revised to remove drift. The absolute displacement time histories are calculated from the nodal time histories accelerations. When the relative displacements are plotted there is a constant slope as shown in Figure RAI-TR03-022-1. To correct this drift, the slope of the relative displacement multiplied by the time is subtracted from the relative displacement at each time step. Presented in Figure RAI-TR03-022-2 is the drift corrected relative displacement.~~

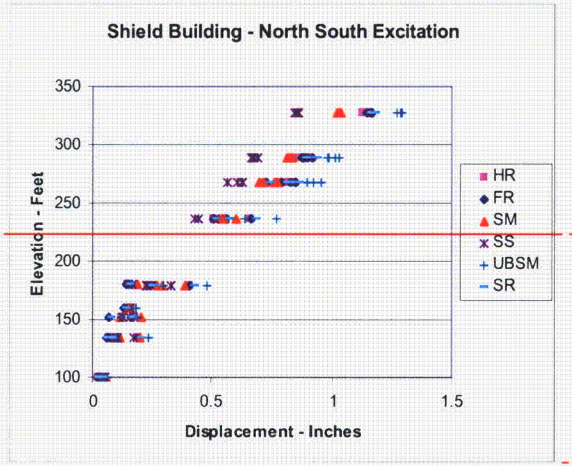


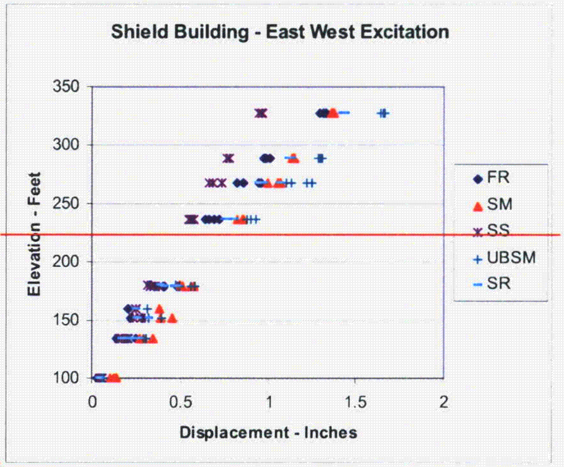
~~Figure RAI-TR03-022-1-Relative Displacement of Node 3360, top of Shield Building~~

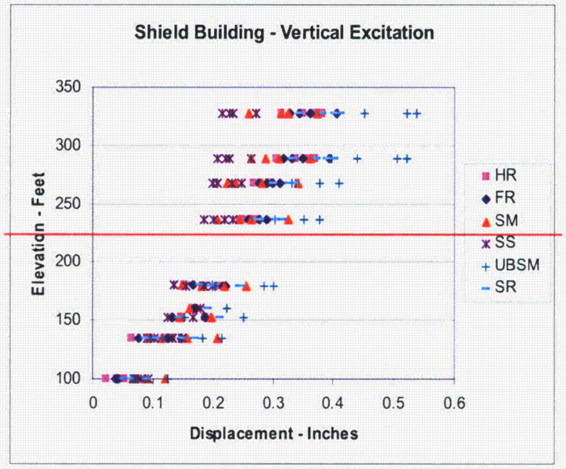


~~Figure RAI-TR03-022-2 - Corrected Relative Displacement of Node 3360, top of Shield Building~~

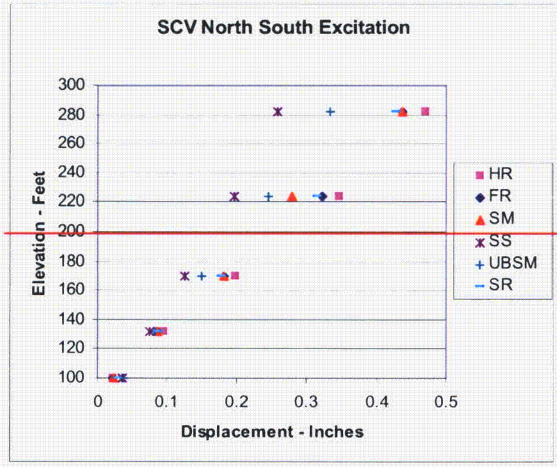
~~Figures RAI-TR03-022-3 and RAI-TR03-022-4 show the maximum deflection plots for the shield building and steel containment vessel for each of the soil cases (firm rock, FR; soft to medium, SM; soft soil, SS; Upper bound soft to medium, UBSM; and soft rock, SR) and hard rock site (HR). Figures RAI-TR03-022-5 and RAI-TR03-022-6 show deflections for the NW corner of the pressurizer compartment and the SE corner of the East steam generator compartment.~~

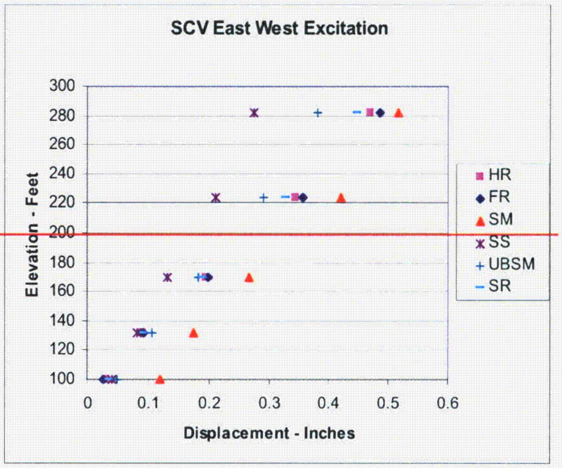


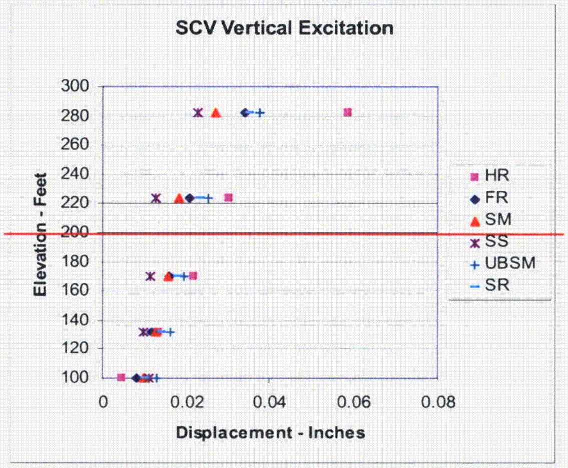




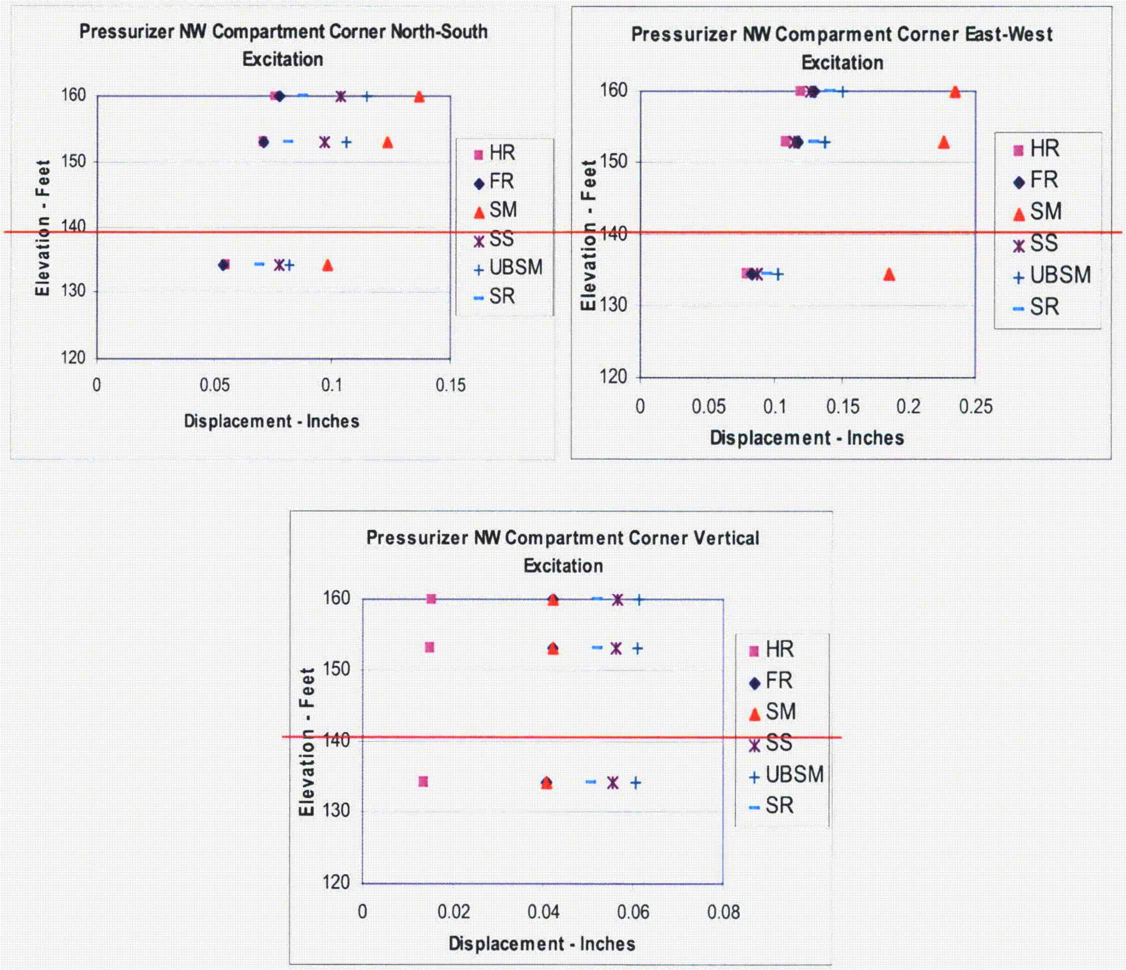
~~Figure RAI-TR03-022-3 – Deflection Plots of Shield Building for all Soil Cases~~



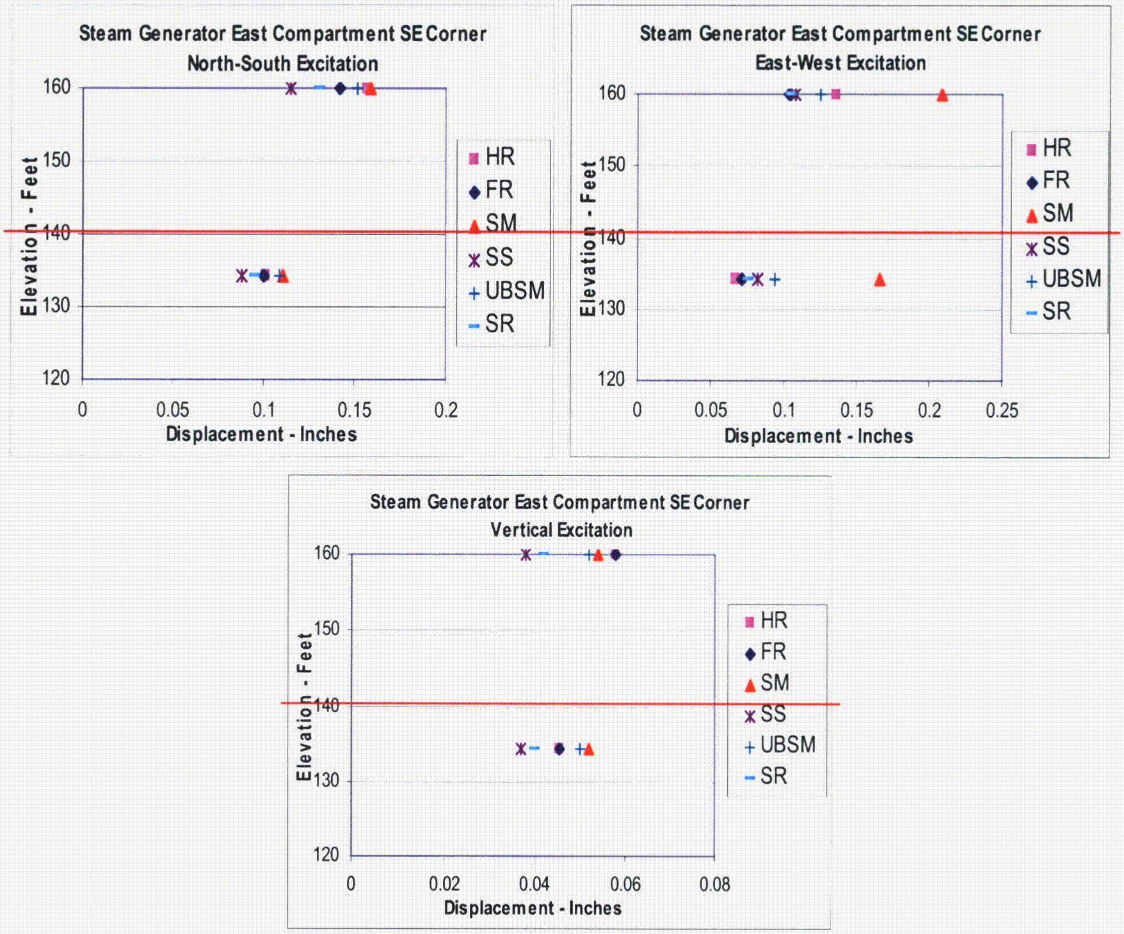




~~Figure RAI-TR03-022-4 – Deflection Plots of SCV for all Soil Cases~~



~~Figure RAI-TR03-022-5 – Deflection Plots of Pressurizer Compartment NW Corner~~



~~**Figure RAI-TR03-022-6 – Deflection Plots of East SG Compartment SE Corner**~~

b. Westinghouse has switched to a seismic response spectrum analysis and is not using equivalent static analyses. The responses for this request for additional information are no longer applicable.

Design Control Document (DCD) Revision:
None

PRA Revision:
None

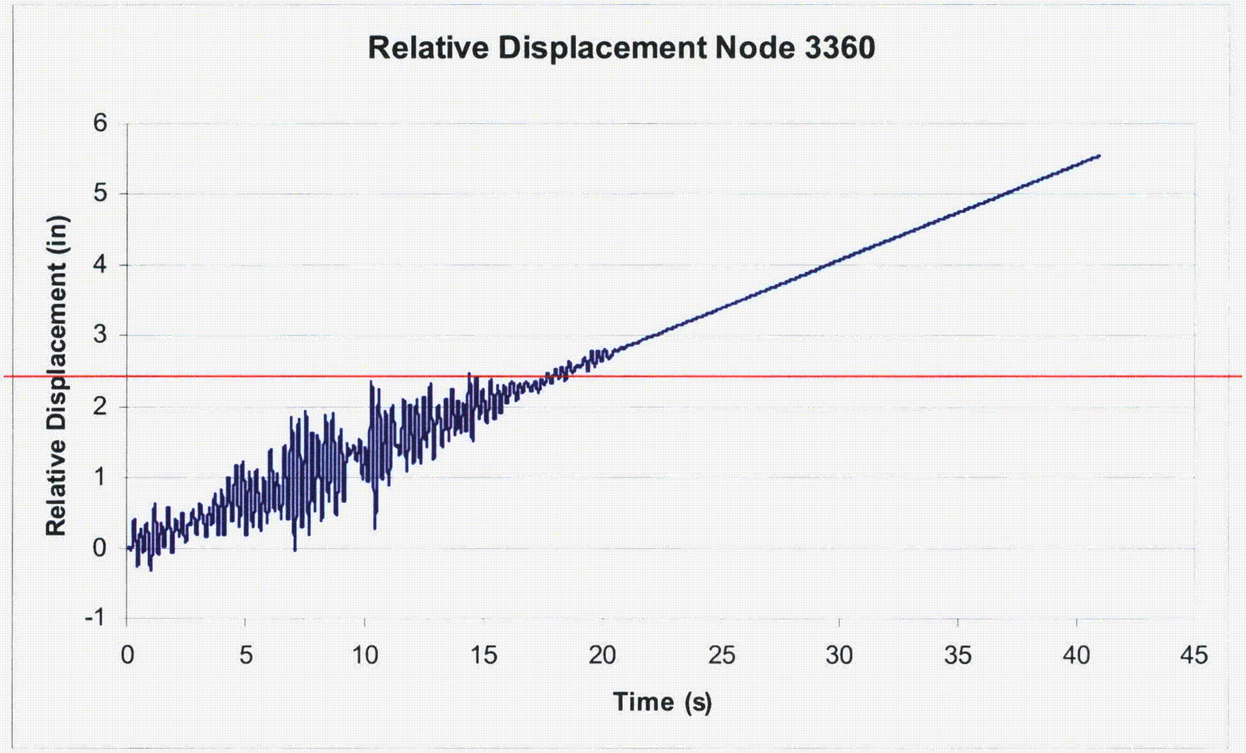
Technical Report (TR) Revision:

Section 6.3 will be replaced by the response given in the Westinghouse Response (part a).

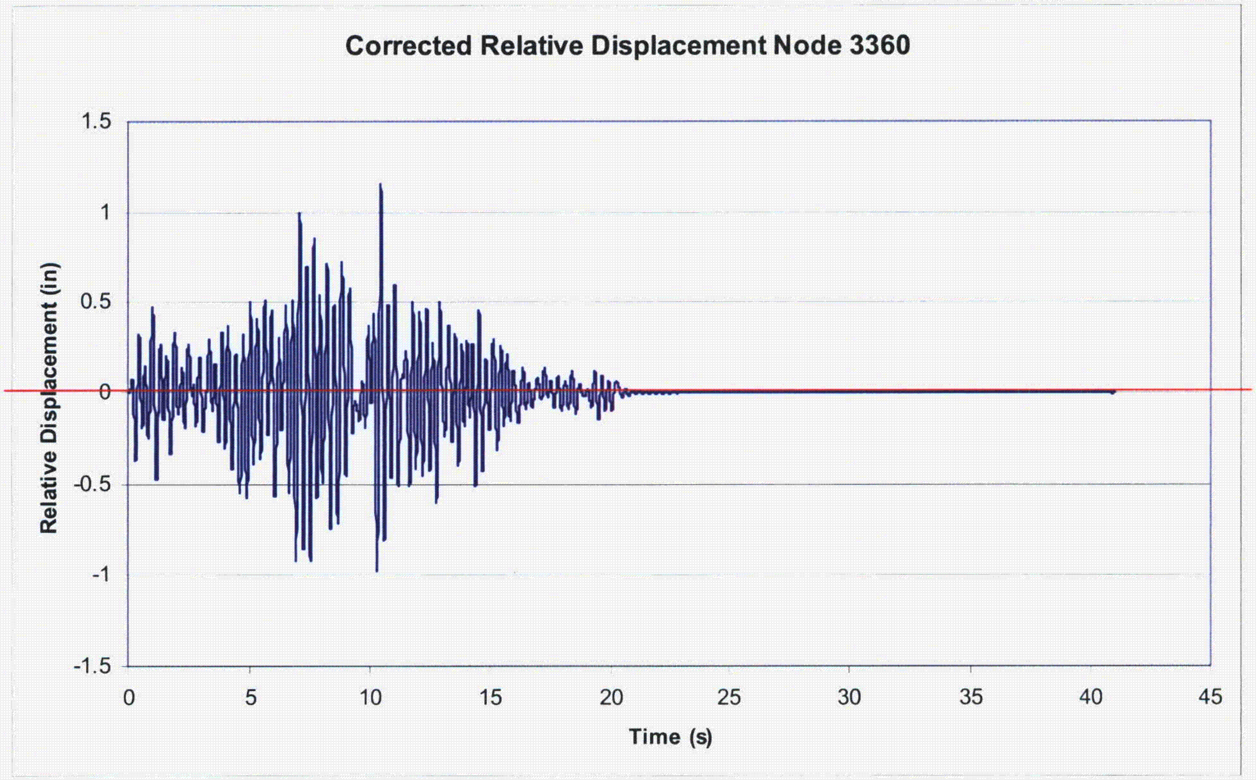
~~Deflections have been developed using the model with the robust shield building design. These displacements for the soil and hard rock cases have been obtained relative to the translation of a reference node at the bottom of the foundation and near the center of the basemat. Coordinates of this reference node are x= 993.00 ft, y= 986.00 ft and z= 60.50ft. The deflections have been revised to remove drift. The absolute displacement time histories are calculated from the nodal time histories accelerations. When the relative displacements are plotted there is a constant slope as shown in Figure 6.3-1. To correct this drift, the slope of the relative displacement multiplied by the time is subtracted from the relative displacement of each time step. Presented in Figure 6.3-2 is the drift corrected relative displacement.~~

6.3 Seismic Displacement Calculation

Westinghouse has adopted the approach that calculates displacements internally within the SASSI program based on an analytical complex frequency domain approach that uses inverse fast-fourier transforms (FFT) to compute relative displacement histories instead of double numerical integration in the time domain that computes absolute displacement time histories from absolute acceleration time histories. The analytical approach is more accurate than a typical baseline correction (time integration) algorithm.

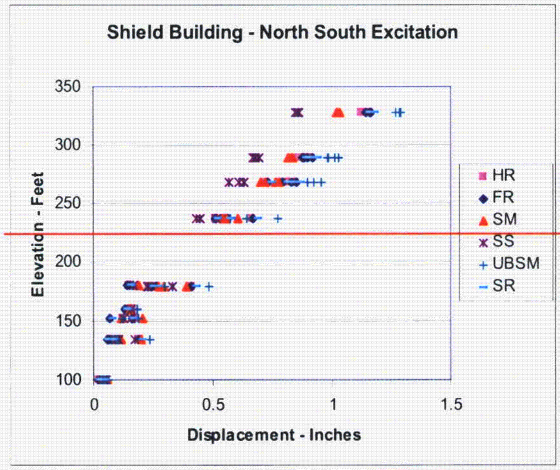


~~Figure 6.3-1 - Relative Displacement of Node 3360, top of Shield Building~~

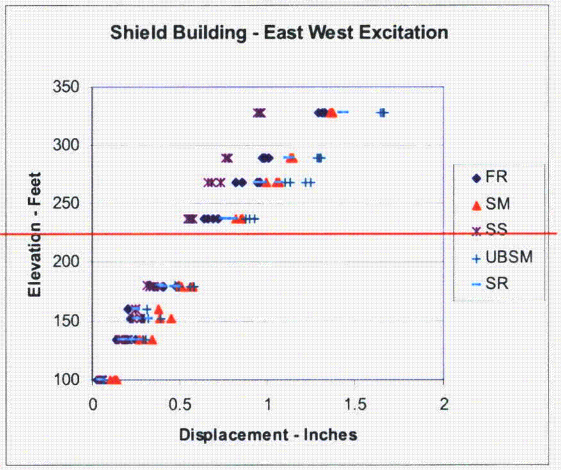


~~Figure 6.3-2 - Corrected Relative Displacement of Node 3360, top of Shield Building~~

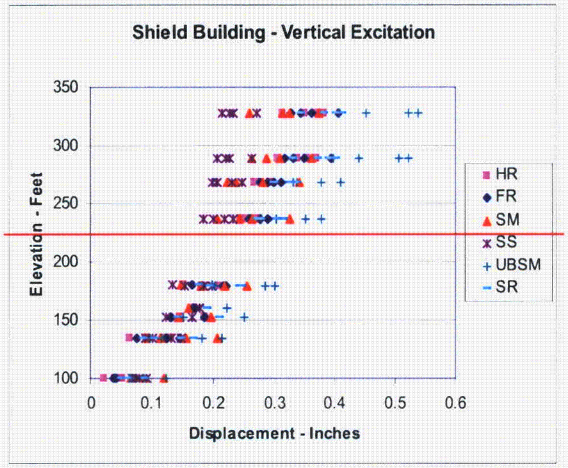
~~Figures 6.3-3 and 6.3-4 show the maximum deflection plots for the shield building and steel containment vessel for each of the soil cases (firm rock, FR; soft to medium, SM; soft soil, SS; Upper bound soft to medium, UBSM; and soft rock, SR) and hard rock site (HR). Figures 6.3-5 and 6.3-6 show deflections for the NW corner of the pressurizer compartment and the SE corner of the East steam generator compartment.~~



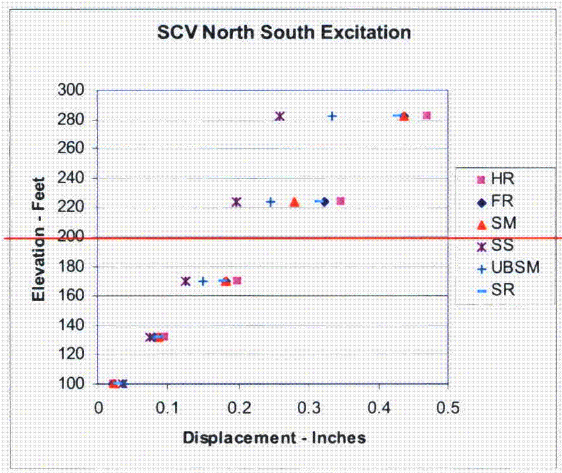
Shield Building - North South Excitation
350
300
250
200
150
100
0
0.5
1
1.5
Elevation - Feet
Displacement - Inches
HR
FR
SM
SS
UBSM
SR

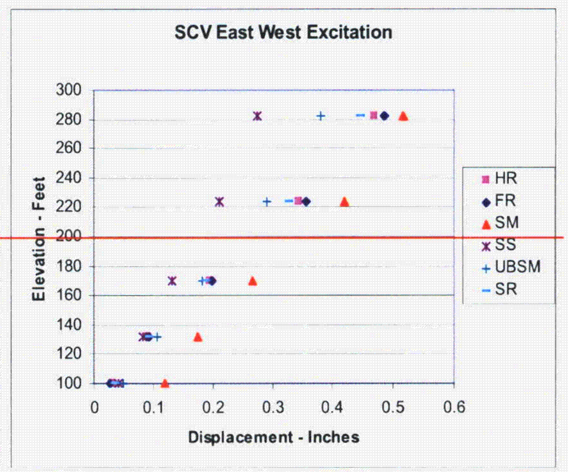


Shield Building - East West Excitation
350
300
250
200
150
100
0
0.5
1
1.5
2
Elevation - Feet
Displacement - Inches
FR
SM
SS
UBSM
SR



~~Figure 6.3-3 Deflection Plots of Shield Building for all Soil Cases~~





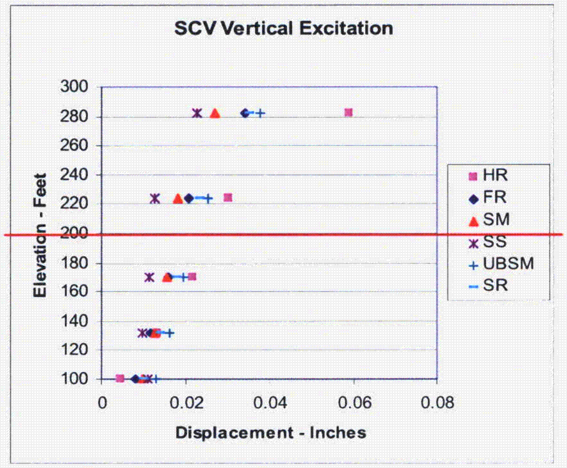
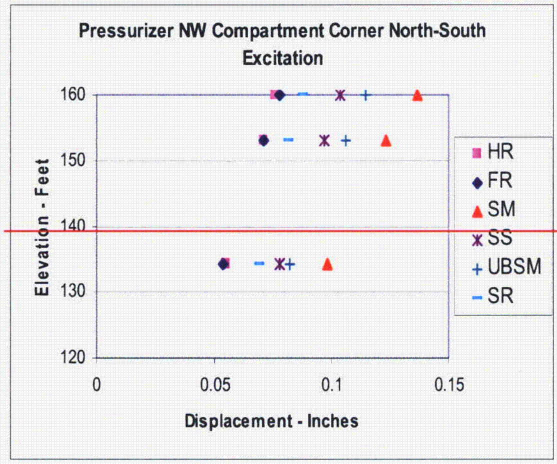
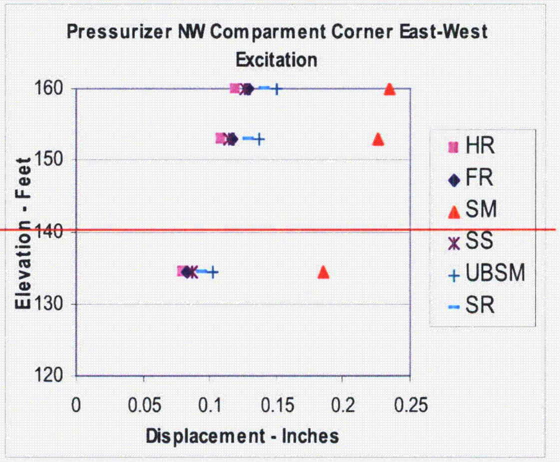
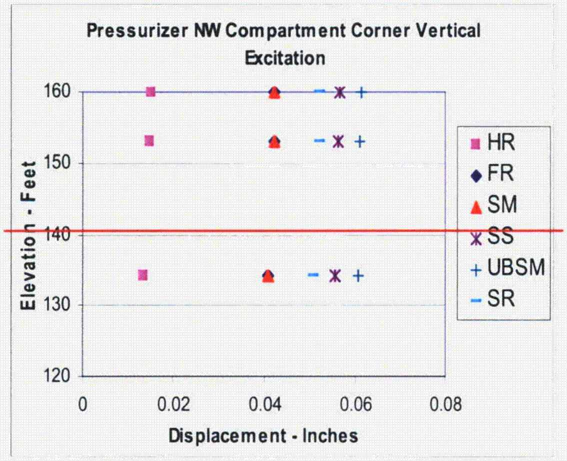


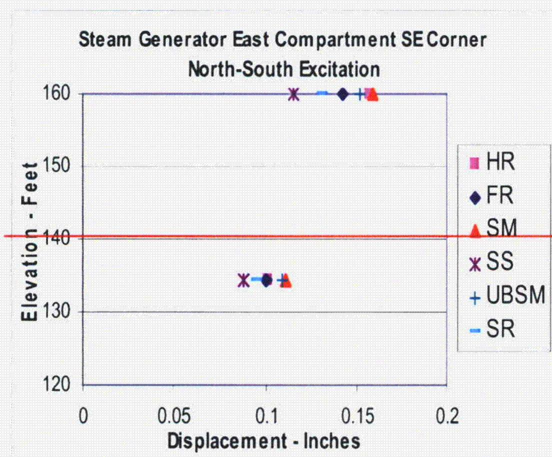
Figure 6.3-4 Deflection Plots of SCV for all Soil Cases

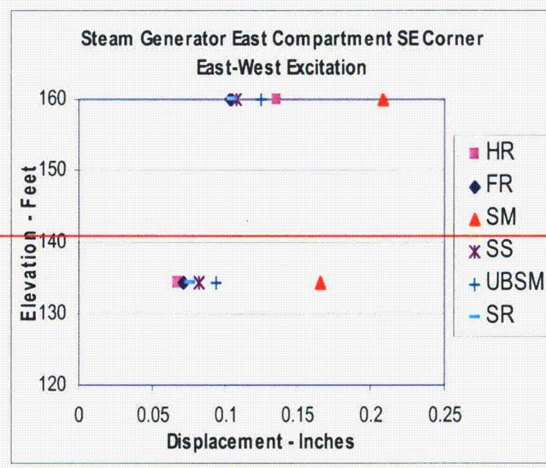


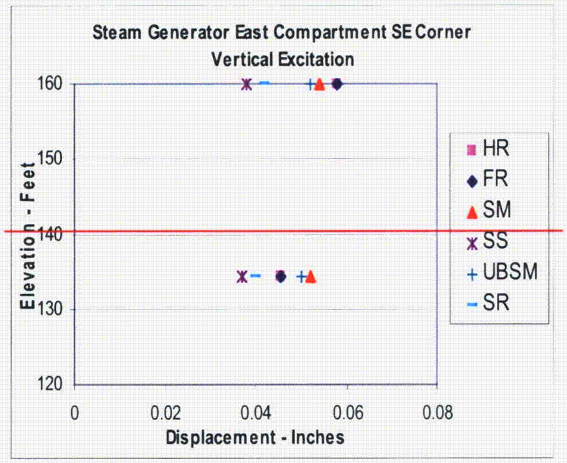




~~Figure 6.3-5 Deflection Plots of Pressurizer Compartment NW Corner~~







~~Figure 6.3-6 Deflection Plots of East SG Compartment SE Corner~~