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Your ref: Docket No. 52-006
Our ref: DCP_NRC_002987

July 30, 2010

Subject: AP1000 Response to Request for Additional Information (SRP TR85 and 3.8.3)

Westinghouse is submitting a response to the NRC request for additional information (RAI) on SRP Section SRP TR85 and 3.8.3. This RAI response is submitted in support of the AP1000 Design Certification Amendment Application (Docket No. 52-006). The information included in this response is generic and is expected to apply to all COL applications referencing the AP1000 Design Certification and the AP1000 Design Certification Amendment Application.

Enclosure 1 provides the response for the following RAI(s):

RAI-SRP-3.7.1-SEB-1-19
RAI-SRP-3.8.3-SEB-1-03
RAI-TR03-001
RAI-TR03-005

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. C. Lundgren / FOR

Robert Sisk, Manager
Licensing and Customer Interface
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/Enclosure

1. Response to Request for Additional Information on Section SRP TR85 and 3.8.3

D063
NRO

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ENCLOSURE 1

Response to Request for Additional Information on SRP TR85 and 3.8.3

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-SRP3.7.1-SEB1-19
Revision: 0

Question:

With DCD, Revision 17, the applicant changed the design of the shield building (SB) from reinforced concrete (RC) construction (7% SSE damping in DCD Table 3.7.1-1) to concrete-filled steel module (SC) construction (5% SSE damping in DCD Table 3.7.1-1). The staff performed a review of TR-03, Revision 4, to verify the critical structural damping values used in shield building dynamic analysis. The applicant did not provide information in this regard, so staff issued RAI-SRP3.7.1-SEB1-19 (a) (draft sent 4/29/2010), requesting the applicant to define the damping value(s) used for the SC module walls and to describe how this value is assigned in the ANSYS and SASSI models.

The staff also found that the applicant reduced the SB concrete modulus to 80% of nominal value, to account for concrete cracking. The 80% value is recommended by FEMA when there is little load-induced cracking. Therefore, the staff is assuming that the applicant has confirmed the seismic response level of the shield building is sufficiently low such that only minor concrete cracking occurs due to seismic loading. Damping has been recognized for many years as being a function of the structural response level. At low response levels, lower effective viscous damping has been observed; at high response levels, higher effective viscous damping has been observed The staff is concerned that SSE damping may not be appropriate to use in the shield building dynamic analyses that are relied on for generation of in-structure response spectra (ISRS). In RAI-SRP3.7.1-SEB1-19 (b), the staff requested the applicant to document in TR-03 the technical basis for the damping value assigned, in light of the observed dependence of damping on response level.

Westinghouse Response:

The models presented in TR03 follow the damping criteria stated in DCD Table 3.7.1-1 using 5% SSE damping for all steel composite (SC) structures, including the Shield Building wall and modules, and 7% SSE damping for the remaining reinforced concrete (RC) structures throughout the Nuclear Island (NI). The linear structural damping values were defined in SASSI and ANSYS as a parameter of material property defined for each element. This form of structural damping is used for seismic time history analyses. The use of these damping values is justified if the element stresses in the model approach their cracking limit. A typical SB element is provided in Figure RAI-SRP3.7.1-SEB1-19-02, where cracking is present at a tensile stress of around 43 ksf.

The question of whether 80% Em is an appropriate value can be assessed using Figure RAI-SRP3.7.1-SEB1-19-01. Figure RAI-SRP3.7.1-SEB1-19-02 shows the idealized in-plane behavior of an SC wall. The figure shows a region that would be considered minimal cracking, and where the secant stiffness would be very close to 0.8 Em value. The figure shows the

stress strain curve for a typical element in the SC cylindrical wall. It shows that only minor cracking is occurring for that element. These results are consistent with those shown in the Shield Building report, Appendix B, where isometric views of the nuclear island show that although cracking occurs throughout the nuclear island the level of cracking is minimal.

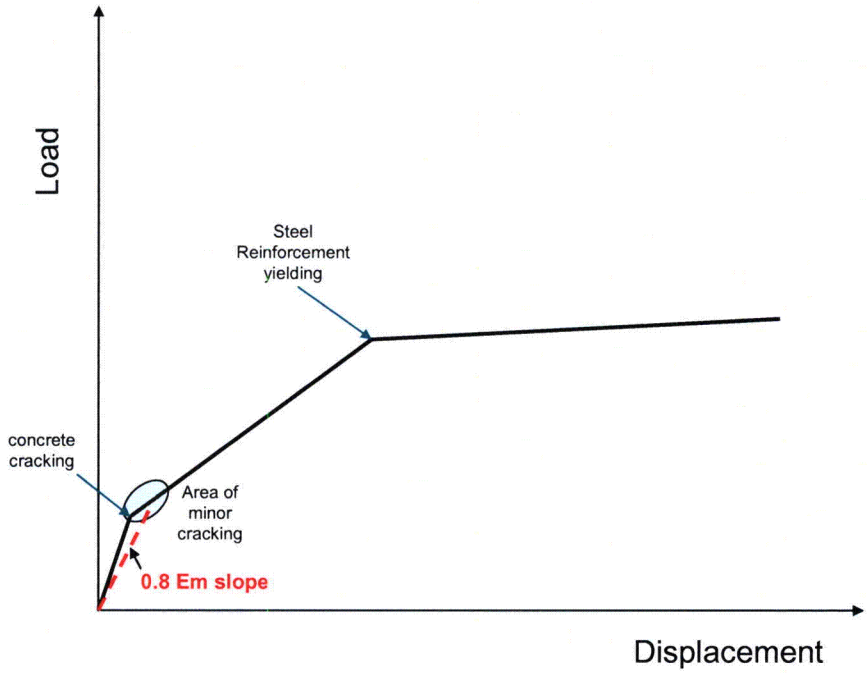

Figure RAI-SRP3.7.1-SEB1-19-01: In Plane Shear load deformation behavior

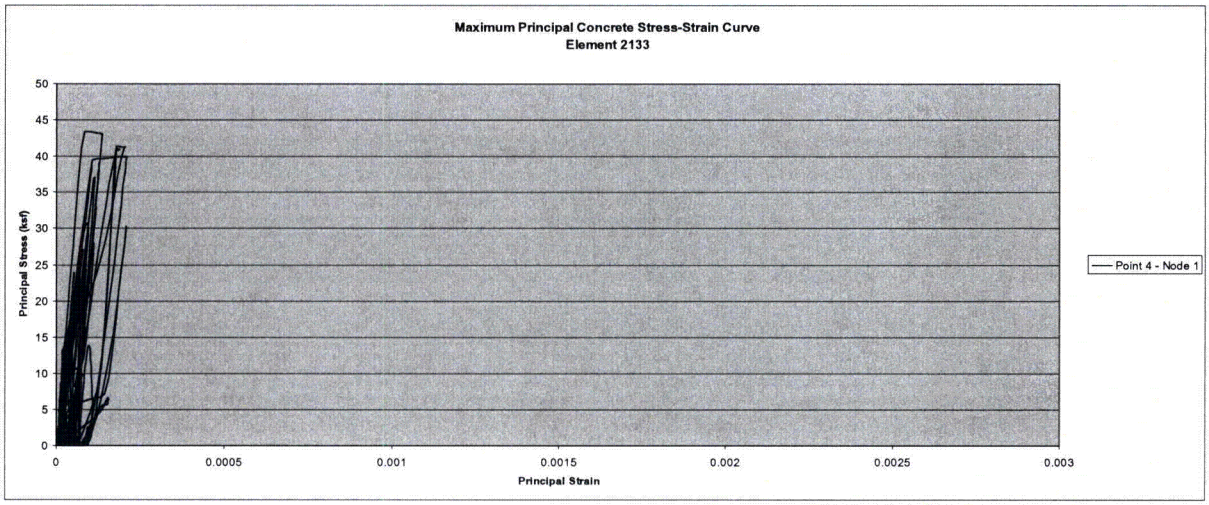

Figure RAI-SRP3.7.1-SEB1-19-02: Stress strain curve of an element showing minimal cracking

Westinghouse has confirmed only minor cracking in the nuclear island structure during an SSE event by performing an ABAQUS time history non-linear analysis. The ABAQUS model uses material properties that have been benchmarked against the Purdue University SC tests presented in the Shield Building report. The purpose of this analysis was to compare the response spectra and stress distribution of a linear model of the nuclear island using 80% stiffness to a non-linear model using the benchmarked materials for the SC and RC elements.

The Shield Building walls SC sections were modeled as layered shell elements with steel as the exterior plies and concrete material properties in the middle as a simplified representation of a SC structure (Figure RAI-SRP3.7.1-SEB1-19-03). The concrete material properties are capable of reaching the cracking limit in the three principal directions and redistributing stresses once the maximum tensile stress in the element has been reached. ABAQUS also has the ability to model the opening and closing of cracks in the concrete element during a cyclic loading case.

To verify that the non-linear model was producing cracked concrete results, maximum stresses in four locations were calculated. The four locations are presented in Figures RAI-SRP3.7.1-SEB1-19-04 and RAI-SRP3.7.1-SEB1-19-05, with the maximum principal stresses provided in Figure RAI-SRP3.7.1-SEB1-19-06 through RAI-SRP3.7.1-SEB1-19-09. Based off of the benchmarked information provided in the Shield Building Report, the concrete in the SC elements initially crack at a tensile stress around 300 psi or 43.2 ksf, shown as a red line in the figures. Each figure provides the tensile stress in concrete at the four nodes used to define the element and at three of the integration nodes located in the concrete. The concrete in the element is still capable of carrying a tensile stress in the concrete material after approaching the initial cracking limit. This is due to the element's ability carry principal stress in three directions. Additionally, it can been seen in the time-based stress-strain curve provided in Figure RAI-SRP3.7.1-SEB1-19-02 that the element is mostly acting in the linear material property zone, meaning the concrete is cracking, but the cracks are closing and still capable of carrying tension until a complete failure.

Figure RAI-SRP3.7.1-SEB1-19-10 shows the critical node selected for the Shield Building roof response spectra comparison. The roof elevation is the area most affected by the wall material properties. The response spectra comparison between linear 80% stiffness and the non-linear cracked concrete model is provided in Figures RAI-SRP3.7.1-SEB1-19-11 through RAI-SRP3.7.1-SEB1-19-13. The non-linear ABAQUS model is enveloped by the conservative linear 80% stiffness model which supports cracking due to SSE is minimal.

This RAI response presents a non-linear ABAQUS model of the Nuclear Island placed under SSE dynamic loading. The model exhibits cracked concrete in typical SC sections shown in Figures RAI-SRP3.7.1-SEB1-19-04 and RAI-SRP3.7.1-SEB1-19-05. These elements are reaching the cracked limit but not completely yielding and the cracks are closing. The response spectra of the non-linear model is compared to, and enveloped by, the linear 80% stiffness showing the linear model is a conservative approach for time history analyses. The presence of the cracked concrete elements in the non-linear model allows the use of the defined SSE

damping values presented in the beginning response. The fact that the cracking is limited, does not extend significantly past the cracking limit, justifies the use of the 80% Em values in linear modeling.

In summary, the use of 80% Em to model cracked elements and 5% damping for SC structures and 7% damping for RC structures is justified based on:

- The use of these damping values is justified since the element stresses in the model approach their cracking limit. A typical Shield Building SC element is provided in Figure RAI-SRP3.7.1-SEB1-19-02, which shows that cracking is present at a tensile stress of around 43 ksf.

- The use of 80% Em is an appropriate value is justified since the stresses due to an SSE loading only result in minimal cracking in the concrete. Figure RAI-SRP3.7.1-SEB1-19-02 shows that although cracking occurs throughout the nuclear island the level of cracking is minimal and does not significantly exceed the tensile limit.

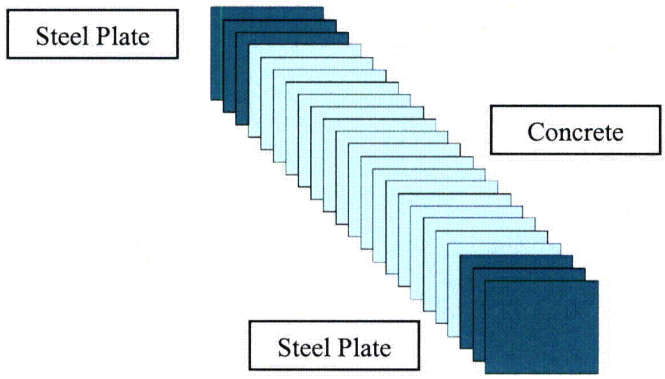

Figure RAI-SRP3.7.1-SEB1-19-03: ABAQUS Benchmarked Layered Shell Element

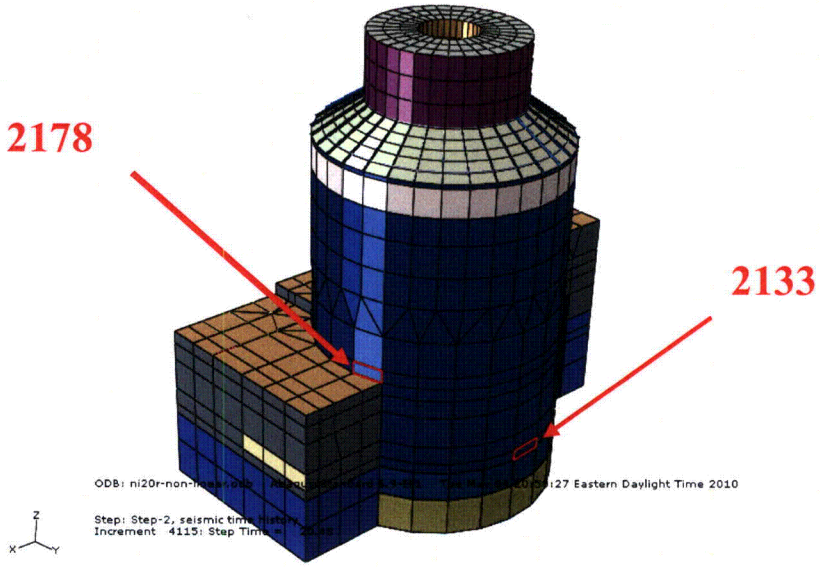

Figure RAI-SRP3.7.1-SEB1-19-04: Critical Shield Building Elements Part 1

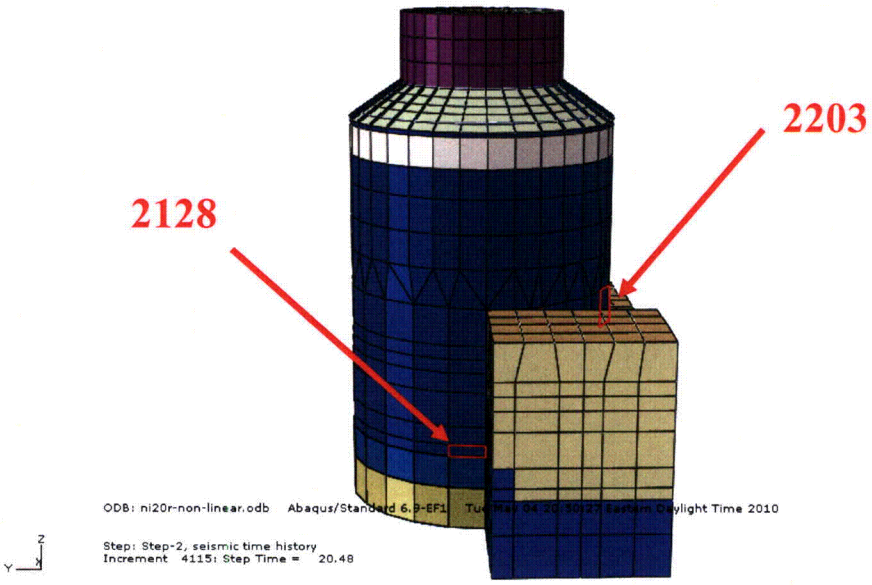

Figure RAI-SRP3.7.1-SEB1-19-05: Critical Shield Building Elements Part 2

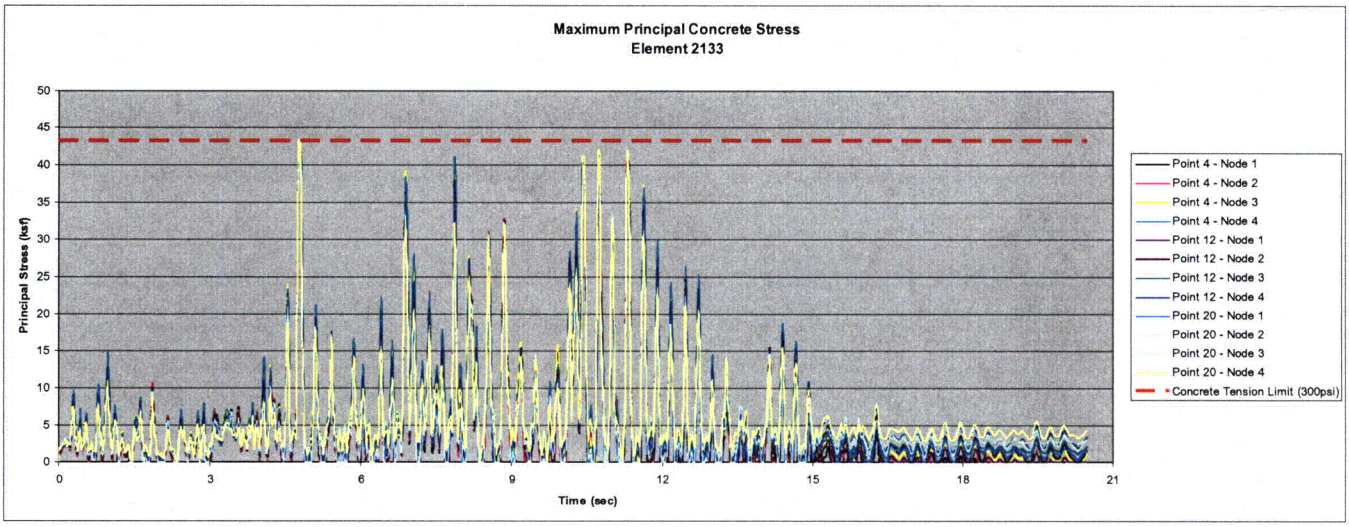

Figure RAI-SRP3.7.1-SEB1-19-06: Maximum Principal Stresses – Element 2133 – West Side

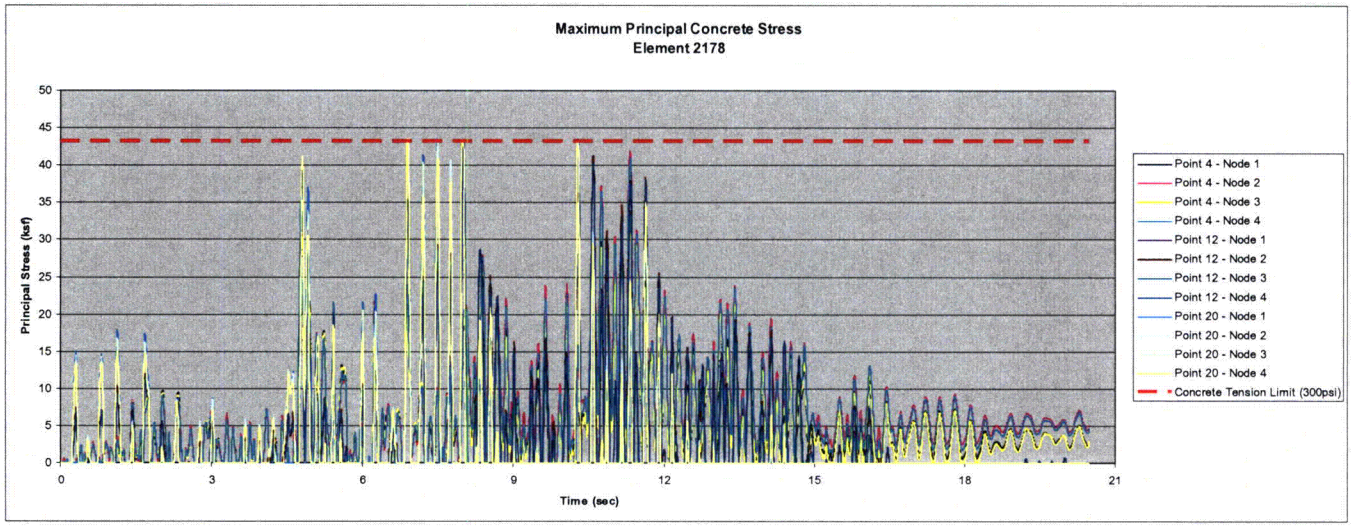

Figure RAI-SRP3.7.1-SEB1-19-07: Maximum Principal Stresses – Element 2178 – North Side

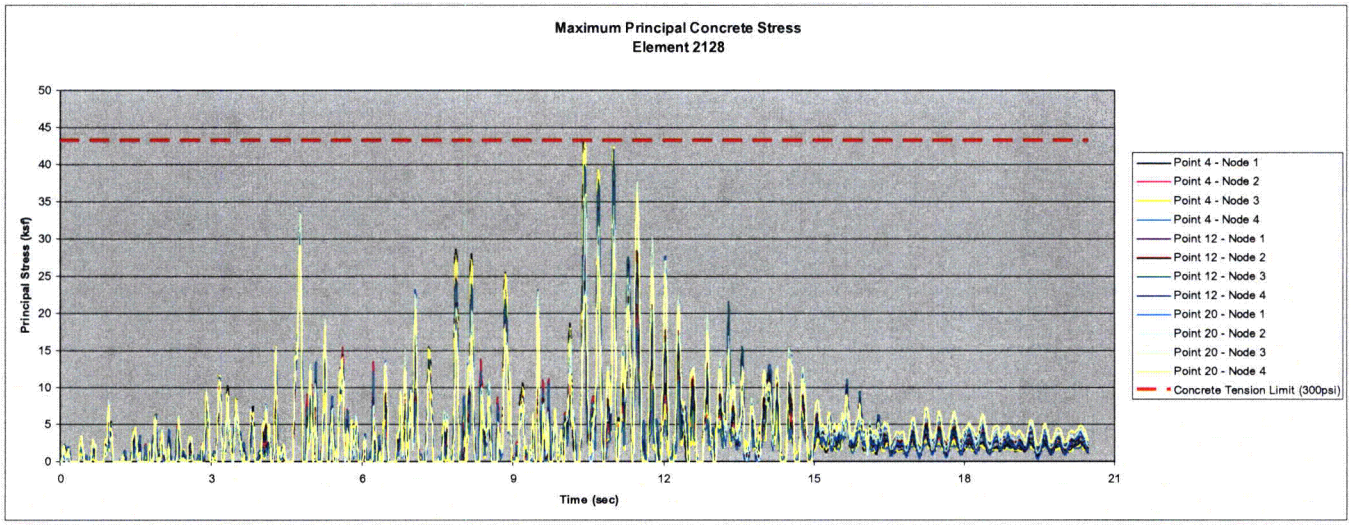

Figure RAI-SRP3.7.1-SEB1-19-08: Maximum Principal Stresses – Element 2128 – South Side

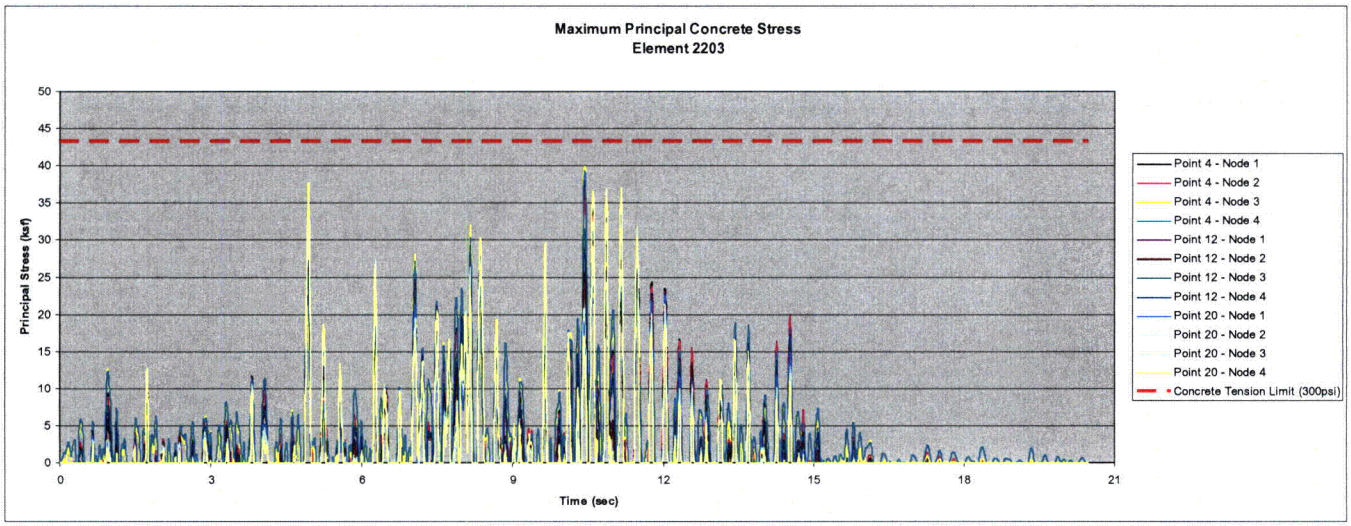

Figure RAI-SRP3.7.1-SEB1-19-09: Maximum Principal Stresses – Element 2203 – East Side

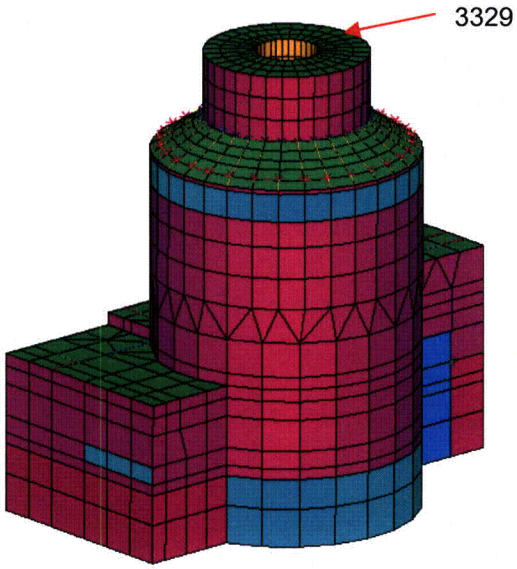

Figure RAI-SRP3.7.1-SEB1-19-10: ASB Roof Elevation 327.41'

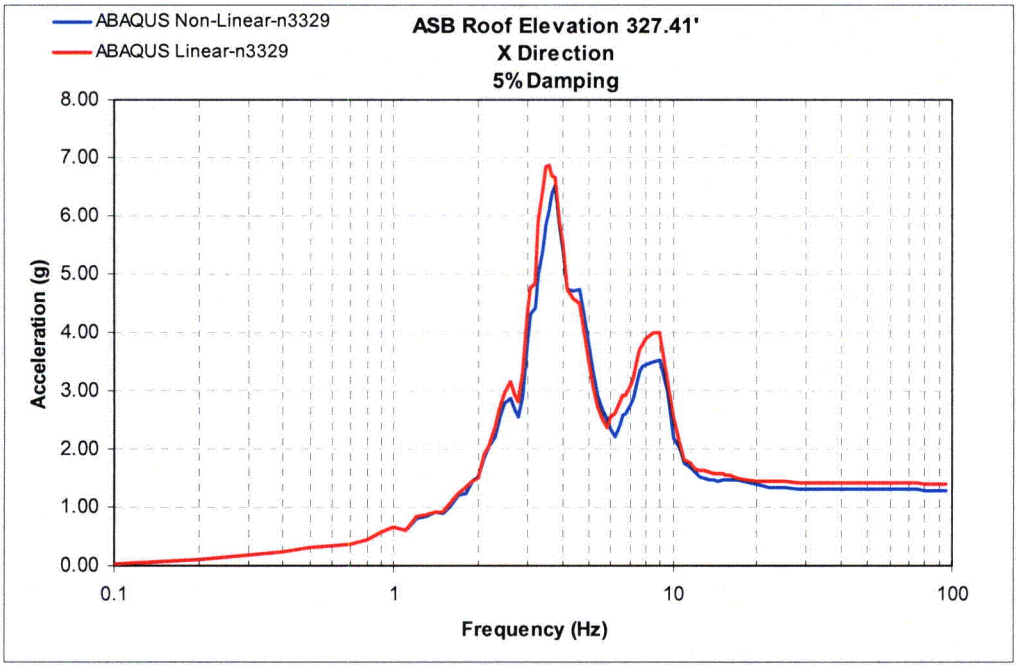

Figure RAI-SRP3.7.1-SEB1-19-11: X-Direction - ASB Roof Elevation 327.41'

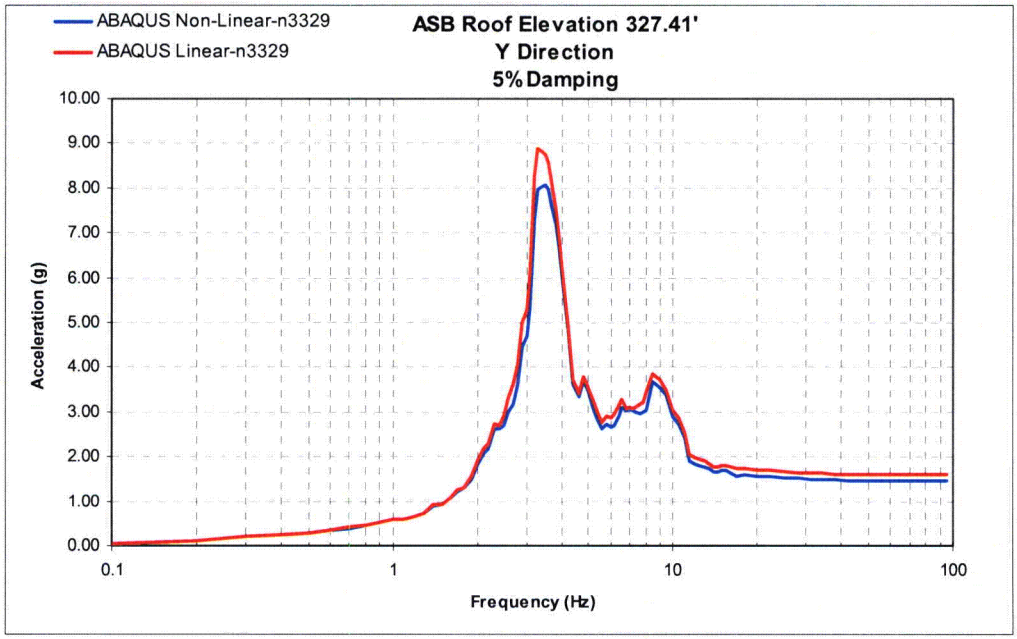

Figure RAI-SRP3.7.1-SEB1-19-12: Y-Direction - ASB Roof Elevation 327.41'

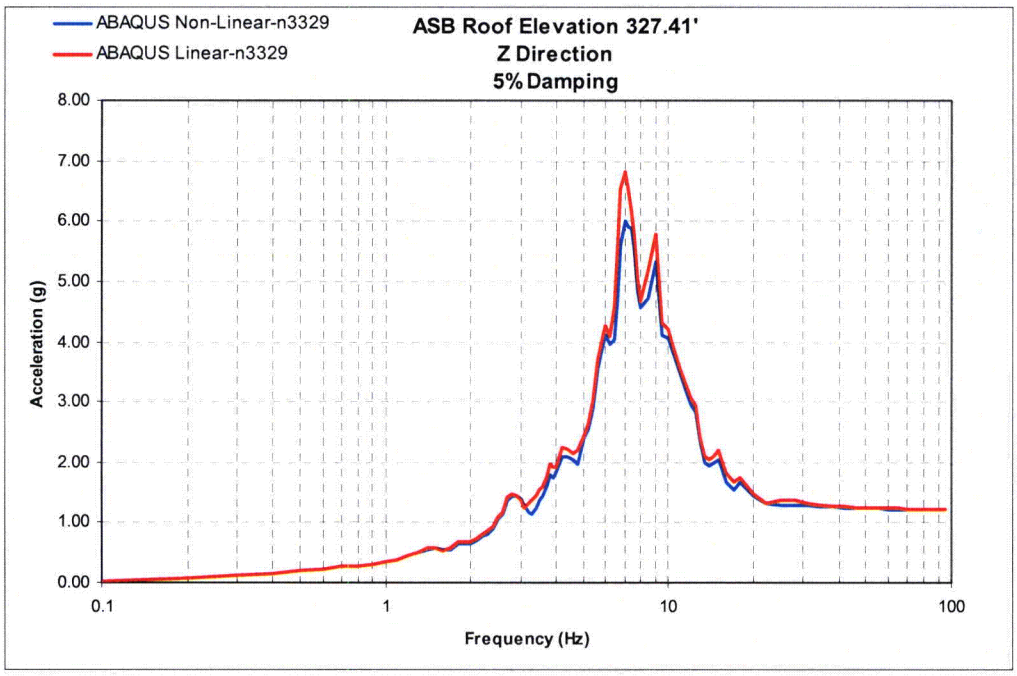

Figure RAI-SRP3.7.1-SEB1-19-13: Z-Direction - ASB Roof Elevation 327.41'

Design Control Document (DCD) Revision:

None

PRA Revision:

None

Technical Report (TR) Revision:

None

RAI Response Number: RAI-SRP3.8.3-SEB1-03
Revision: 2

Question:

DCD Section 3.8.3.4.1 covers the seismic analyses of the containment internal structures (CIS). Subsection 3.8.3.4.1.1 describes the development of the 3D lumped-mass stick model of the CIS based on the structural properties obtained from a 3D finite element model using 3D shell elements. Subsection 3.8.3.4.1.2 describes the stiffness assumptions for local seismic analyses of the in-containment refueling water storage tank (IRWST). No description is provided for the model development and analysis, including the stiffness assumptions, for the global seismic analysis of the CIS. Prior revisions of the DCD did provide a description of this subject in a separate subsection; however, DCD Rev. 16 removed all of this information. Westinghouse is requested to provide a description of the CIS model, the stiffness assumptions utilized, and basis for the selection of the stiffness for the CIS and auxiliary building modules. In addition, DCD Table 3.8.3-2 was revised to utilize the "Monolithic Case 3" concrete stiffness representation of the CIS in the 3D finite element analysis using the equivalent static and response spectra analyses. Westinghouse is requested to explain why the CIS stiffness values were revised from the monolithic case 1 to monolithic case 3, and what is the technical basis for not evaluating the range of possible stiffness values between Cases 1 to 3.

If your response to this request for additional information will reference Revision 17 to the AP1000 DCD, please provide an exact reference.

Additional Request (Revision 1):

NRC Staff requests that in the Structural Modules for CIS DCD section that the information removed from DCD Section 3.8 be returned to the DCD and revised stiffness values be included in the DCD.

Additional Request (Revision 2):

In DCD Revision 16, the applicant removed Section 3.8.3.4.1.2 – Stiffness Assumptions for Global Seismic Analyses in the previous certified DCD. This section discussed the stiffness properties used in the seismic analyses of the containment internal structures (CIS) and the auxiliary building modules. Reference was made to DCD Table 3.8.3-1 which contained the various stiffness cases for the concrete filled steel plate modules used for structures inside containment and the auxiliary building.

The staff reviewed the Westinghouse response to RAI-SRP3.8.3-SEB1-03 Rev.1 and determined that the response did not address the two remaining concerns related to the appropriate concrete stiffness used for reinforced concrete structures as well as the concrete-filled steel plate modules. Provide the following:

1. The information presented in the proposed markup to DCD Table 3.8.3-2 (and associated text in the DCD) does not provide the technical basis for utilizing the stiffness reduction factor of 0.8 for the CIS. As noted in the proposed RAI markup to Section 3.8.3.4, the use of 0.8 is valid

for a relatively small degree of cracking. The level and extent of cracking to justify the use of 0.8 should be provided for the current AP1000 evaluation. Also, the RAI response indicates that the NI model of the concrete structures (beyond the concrete-filled CIS modules) is based on the gross concrete section stiffness reduced by a factor of 0.8 to consider the effect of cracking "as recommended in Table 6-5 of FEMA 356." The staff finds that Table 6-5 of FEMA 356 indicates that the factor of 0.8 is only applicable to flexural rigidity for concrete walls which are uncracked when inspected, and for walls that are cracked the reduction factor is 0.5. For shear rigidity, the FEMA table indicates that the reduction factor is 0.4 for both uncracked and cracked conditions. Therefore, Westinghouse may have inappropriately referenced the FEMA standard for its use of the value of 0.8 for reinforced concrete structures as well as the concrete-filled steel plate module structures. Provide a completed justification supporting your assumptions that stresses in concrete are low and concrete is rarely cracking during SSE, as discussed.

2. State the technical basis for not evaluating the range of possible stiffness values for reinforced concrete structures as well as the concrete-filled steel plate module structures, as requested in the original RAI.

It was requested by the NRC during the June 28 – 30, 2010 audit, that the following information be provided.

1. Describe methodology for generation of FRS at 5 locations (3 applicable key locations (ASB control room floor, ASB fuel building roof at shield building, ASB shield building roof area) and 2 additional locations at Aux building that include highly stressed regions).
   - Abaqus
   - ANSYS
2. Compare the FRS from both methods at the 5 locations (3 applicable key locations and 2 additional locations at Aux building that include some higher stressed region).
3. Provide stress strain time history curves at 2 locations (1 Shield building and 1 Aux building). Show that no significant energy absorption occurs.
4. SC/RC connection
   - Describe methodology for modeling/analysis (accounting for current connection design).
   - Show that varying stiffness values does not significantly affect the response of the SB.
5. Provide discussion of the results and justification.

Westinghouse Response:

Section 3.8.3.4.1.2 was removed since stiffness assumptions for global seismic analyses are part of Section 3.7, Seismic Design. Description is provided for model development and analysis for the containment internal structures (CIS) in DCD Section 3.7.2.3.1 (Rev. 16 &17). Further, Technical Report 03 (Reference 1) was written to provide more details of the seismic analyses for soil sites than provided in the DCD. It is noted that DCD subsection 3.8.3.4.1.1 (Rev. 17), Finite Element Model is not up to date since it discusses 3D lumped-mass stick model of the CIS that is no longer used, a shell model is now used. The first sentence of the

first paragraph of this subsection is removed, and reference is made to DCD Section 3.7 and 3G.

The stiffness assumptions for the global seismic analysis of the CIS and auxiliary building modules is discussed in DCD Section 3.7.2.3 (Rev. 16 & 17) and DCD Section 3.7.2.3.1 (Rev. 16 & 17). It is stated in DCD Section 3.7.2.3:

> "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 reduced by a factor of 0.8 to consider the effect of cracking as recommended in Table 6-5 of FEMA 356 (Reference 5)."

In DCD Section 3.7.2.3 it is stated:

> "The properties of the concrete-filled structural modules are computed using the combined gross concrete section and the transformed steel face plates of the structural modules. The modulus is reduced by a factor of 0.8 to consider the effect of cracking."

The concrete stiffness was changed to Monolithic Case 3 to be consistent with the local seismic analyses of in-containment refueling water storage tank discussed in DCD Section 3.8.3.4.1.2 (Rev. 16 & 17). Foot note 2 was added to Table 3.8.3-2 to refer to DCD Section 3.7 for the specifics related to the global containment internal structures seismic analyses.

Westinghouse Response (Revision 1):

The first paragraph of DCD subsection 3.8.3.4.1.1 is changed in the Revision 0 response to reflect that the three dimensional lumped mass stick model of the CIS is no longer used. The structural modules are modeled within the 3D finite element shell models described in DCD subsection 3.7 and Appendix 3G. For consistency with DCD Section 3.7.2.3 a sentence will be added related to the reduction of concrete modulus by a factor of 0.8 to reflect cracking.

Westinghouse Response (Revision 2):

The information to address items 1 and 2 is provided in the response to RAI-SRP3.7.1-SEB1-19. Provided below is the response to information requested by the NRC during the June audit.

Additional Audit Request:

Westinghouse performed a new time history analysis with the ABAQUS NI20 model in order to demonstrate the stability of the non-linear NI20 model, in which the concrete behavior is described by the Concrete Damaged Plasticity model. The acceleration time histories associated with the envelope of response spectra for all soil sites at foundation level (Elevation 60.5') at 5% critical damping was used as the input. This was performed as an implicit dynamic analysis in ABAQUS using specific alpha-beta damping values for each material property. An ANSYS comparison is not provided in this response since the NI20 model does not typically use the all soil input time history for analysis and has not currently been run. This is a sensitivity study between linear and non-linear concrete material properties.

The response spectra at several locations of the Nuclear Island (NI) were calculated. They agree with and are enveloped by the ones produced by the time history analysis with the linear elastic ABQAUS NI20 model, in which the stiffness is reduced to 80% to reflect the concrete crack effect. It can be concluded that the non-linear ABAQUS NI20 model is capable of better simulating the seismic response of the AP1000 NI to the different levels of the seismic excitation.

The non-linear ABAQUS NI20 model adopts the Concrete Damaged Plasticity (CDP) model to describe the mechanical behavior of the concrete. The material parameters in the CDP model were benchmarked with the testing performed by Purdue University. The linear ABAQUS model used the 80% Young's Modulus of concrete to address the effect of concrete cracking. In both non-linear and linear time history analyses, the input time histories are associated with the envelope of response spectra for all generic soil sites at the foundation level (Elevation 60.5').

The non-linear model includes a benchmarked RC/SC connection using the existing shell and solid elements of the NI20 linear model. This connection was modified to match the testing performed at Purdue University, with the in-plane shear being the governing case. The purpose of including benchmarked non-linear RC/SC connection was to show what effects the RC/SC material properties have on the building response spectra and local stresses.

The all soils input time history was developed by taking the enveloped CSDRS at elevation 60.5', and creating a statistically independent time history. In Figures RAI-SRP3.8.3-SEB1-03-01 through RAI-SRP3.8.3-SEB1-03-03, the enveloped Response Spectra (RS) for all soil sites at Elevation 60.5' are presented and compared against the AP1000 Certified Seismic Design Response Spectra (CSDRS) at 5% damping in the directions of North-South (X), East-West (Y) and Vertical (Z), respectively.

The Response Spectra (RS) at 5% critical damping are produced for 6 locations of the AP1000 NI shown in Table RAI-SRP3.8.3-SEB1-03-01 when subjected to the enveloped RS for all soil sites at El. 60.5'.

Node	Location
1750	Shield Building (SB) West Wall at El. 99'
1574	Auxiliary Building (AB) East Wall at El. 100'
2078	Auxiliary Building (AB) NE Corner at Control Room Floor at El. 116.5'
2505	Auxiliary Building (AB) Roof at El. 160'
2675	Auxiliary Building (AB) Corner of Fuel Building Roof at Shield Building at El. 179.19'
3329	Shield Building (SB) Roof at El. 327.41'

Table RAI-SRP3.8.3-SEB1-03-01

The Response Spectra at 5% critical damping produced by the non-linear and linear time history analyses of ABAQUS NI20 model for the nodes listed above are presented in Figures RAI-SRP3.8.3-SEB1-03-04 through RAI-SRP3.8.3-SEB1-03-21, respectively. It is shown that the RS calculated with the non-linear time history analysis matches and is enveloped by the RS produced by the linear time history analysis. The RC/SC connection effect can be seen at node 1750 and is negligible when modeled with non-linear benchmarked material properties.

A sample load deflection curve is provided in Figure RAI-SRP3.8.3-SEB1-03-22 to show the elastic and plastic behavior of these non-linear material properties. Maximum Principle Stress-strain curves in concrete for two high stress regions of the Auxiliary and Shield buildings have been provided in Figures RAI-SRP3.8.3-SEB1-03-23 through RAI-SRP3.8.3-SEB1-03-26. The plots contain a time history stress-strain curve at the Auxiliary East Wall and Shield Building West Wall for the linear and non-linear model analyses. The testing at Purdue University resulted in complete in-plane yielding in the concrete at a strain of roughly 0.003. These plots show the non-linear elements are acting linearly for the most part, under SSE loading.

The linear plots show higher stresses in these regions because the elements are unable to crack and redistribute stresses to surrounding elements. The non-linear results show lower stresses and that the material property still acting linearly under this loading. High stresses are redistributed in the non-linear concrete elements once these elements begin to crack so the overall magnitude is less than a linear material property with 80% stiffness.

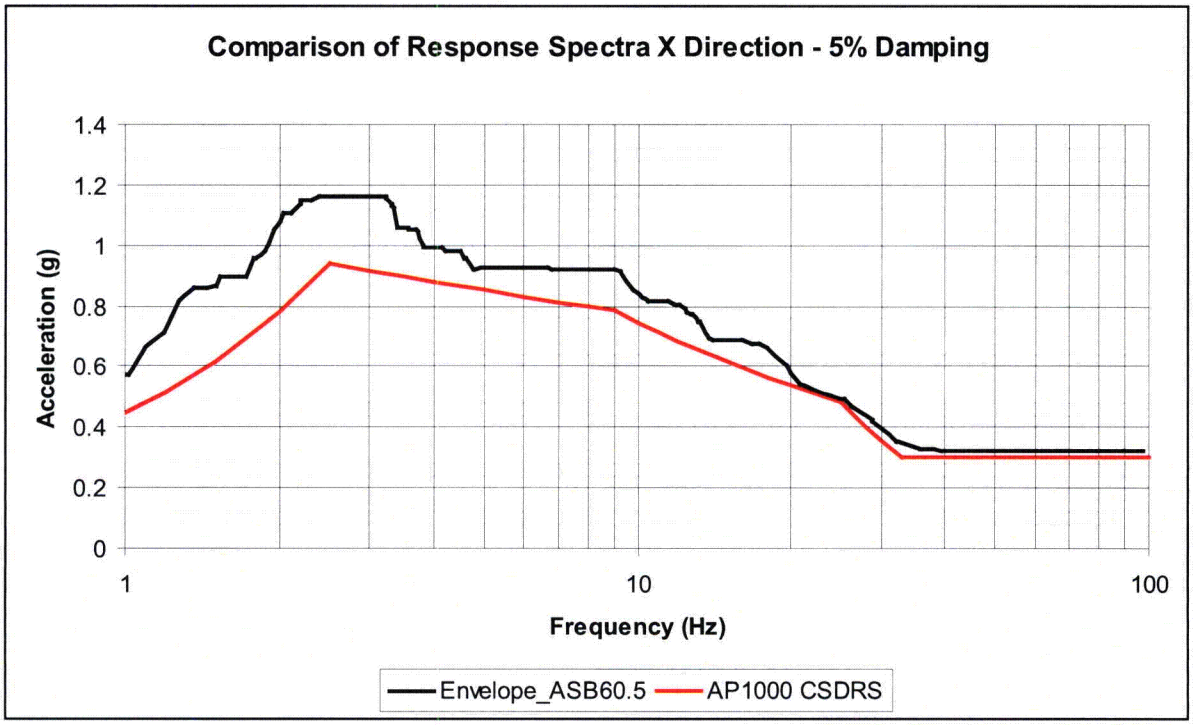

Figure RAI-SRP3.8.3-SEB1-03-01: Enveloped RS at El. 60.5' and AP1000 CSDRS in X direction

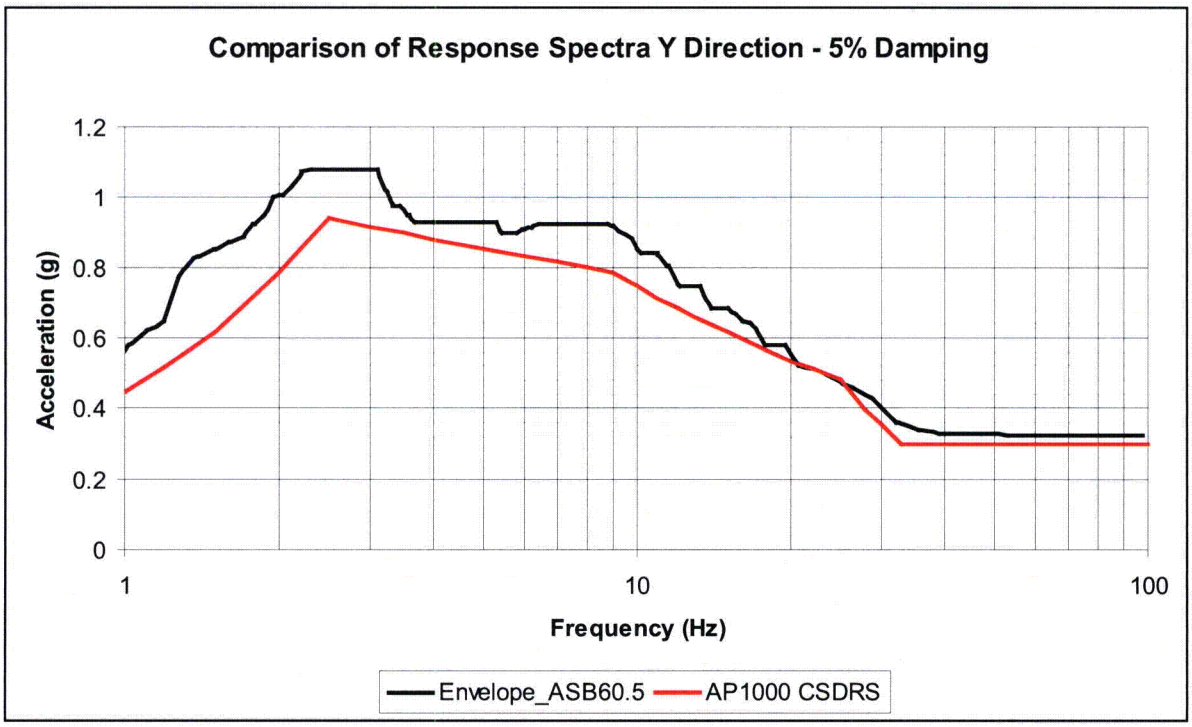

Figure RAI-SRP3.8.3-SEB1-03-02: Enveloped RS at El. 60.5' and AP1000 CSDRS in Y direction

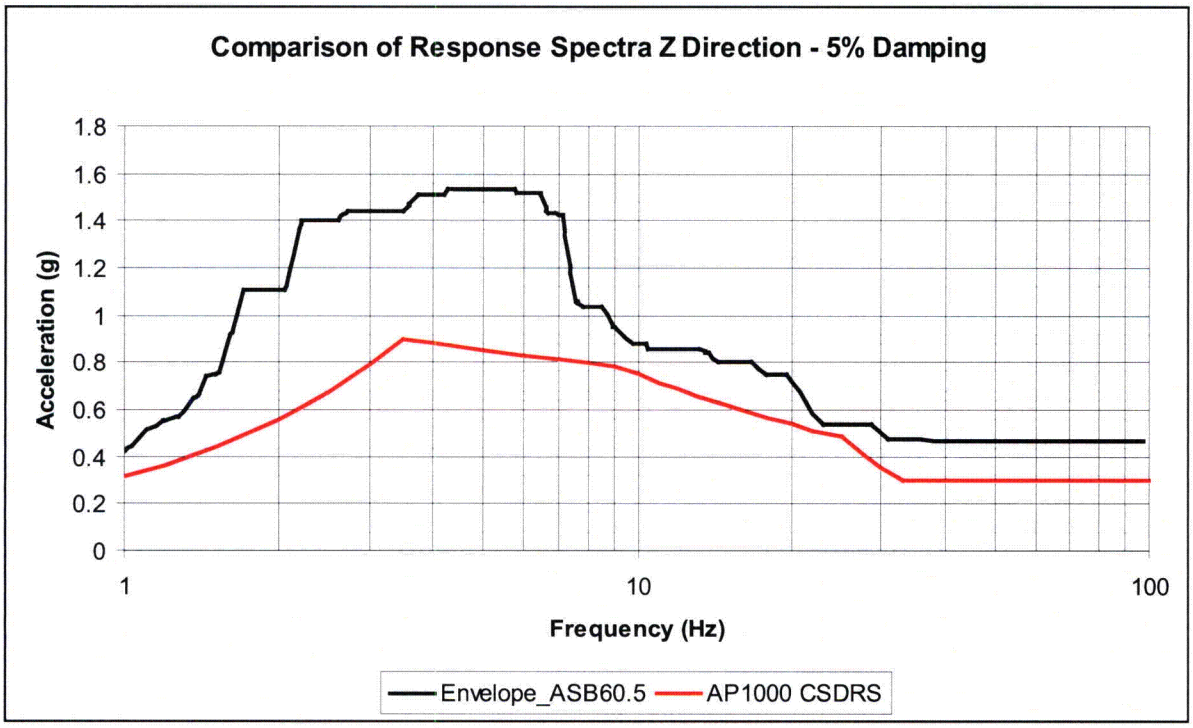

Figure RAI-SRP3.8.3-SEB1-03-03: Enveloped RS at El. 60.5' and AP1000 CSDRS in Z direction

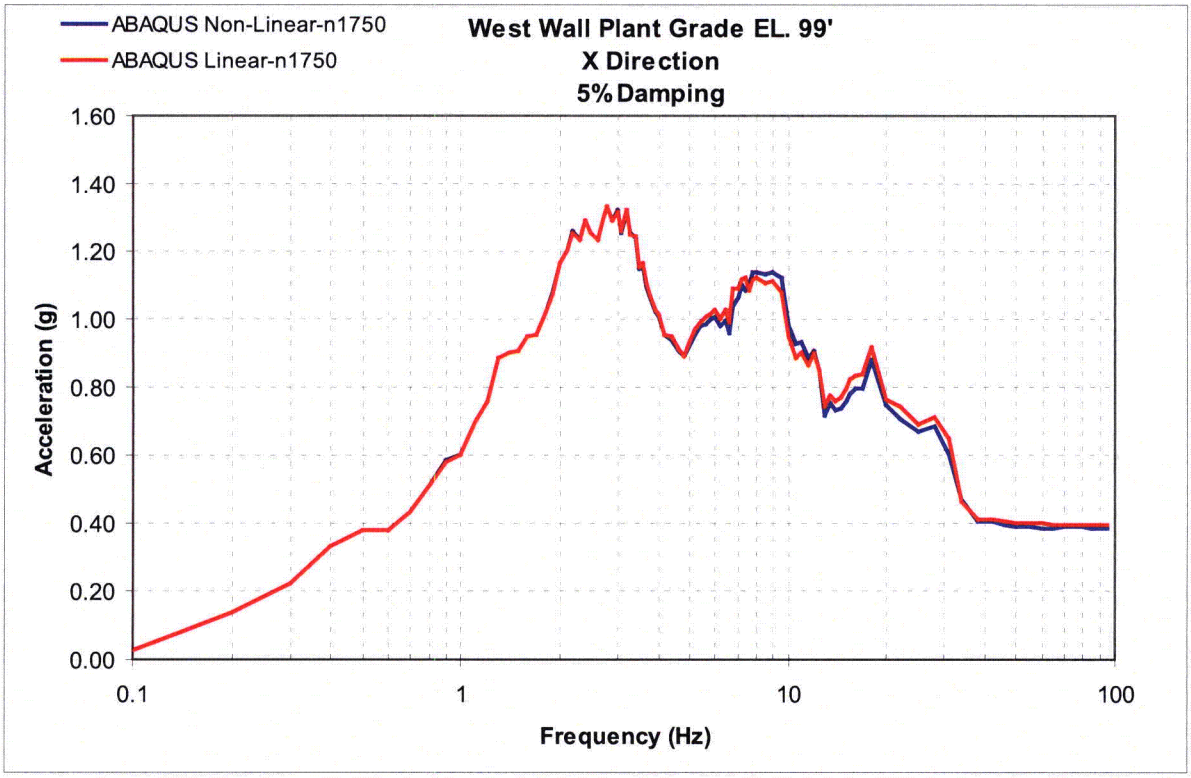

Figure RAI-SRP3.8.3-SEB1-03-04: RS of SB West Wall at El. 99' Subjected to Enveloped RS in X Direction

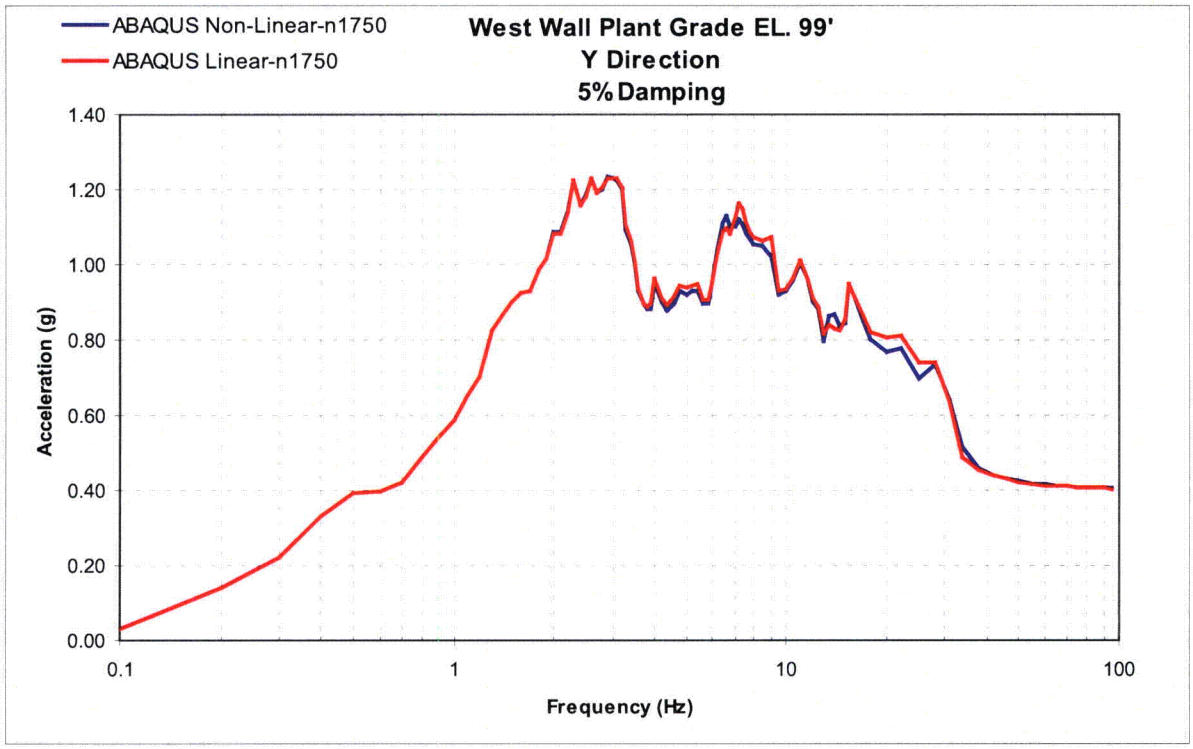

Figure RAI-SRP3.8.3-SEB1-03-05: RS of SB West Wall at El. 99' Subjected to Enveloped RS in Y Direction

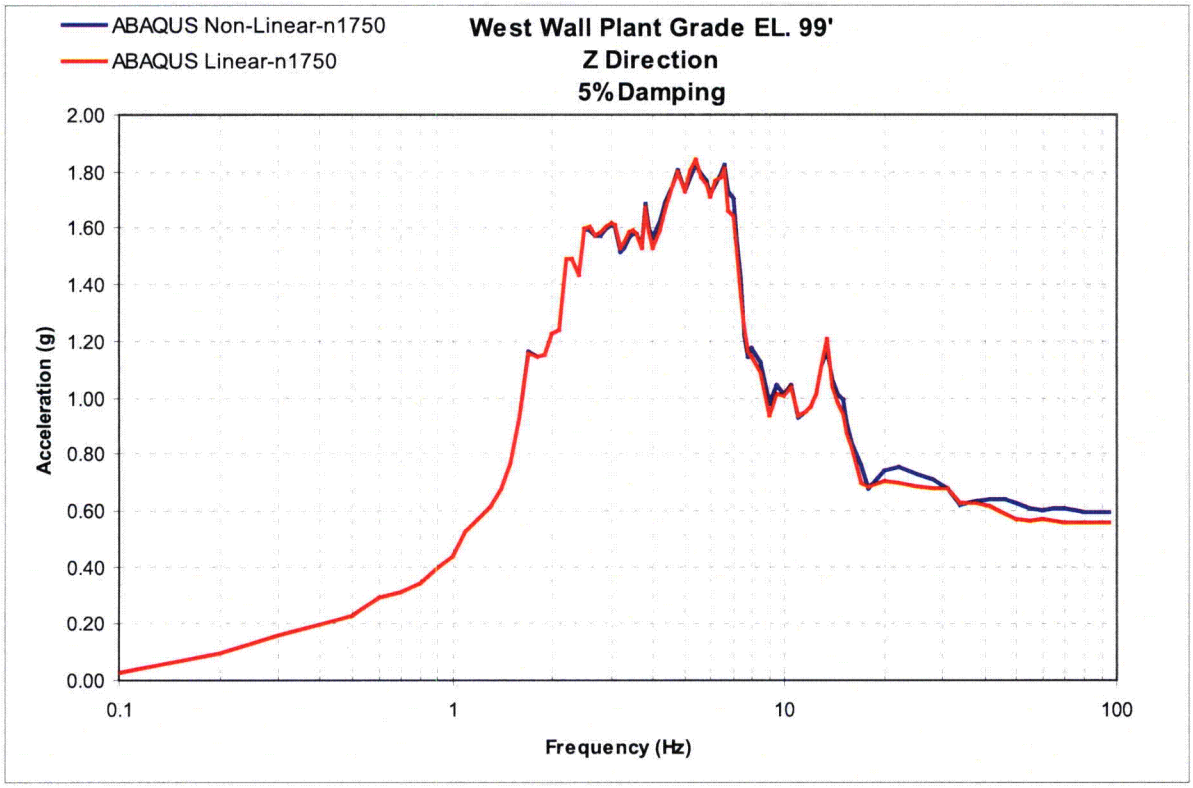

Figure RAI-SRP3.8.3-SEB1-03-06 RS of SB West Wall at El. 99' Subjected to Enveloped RS in Z Direction

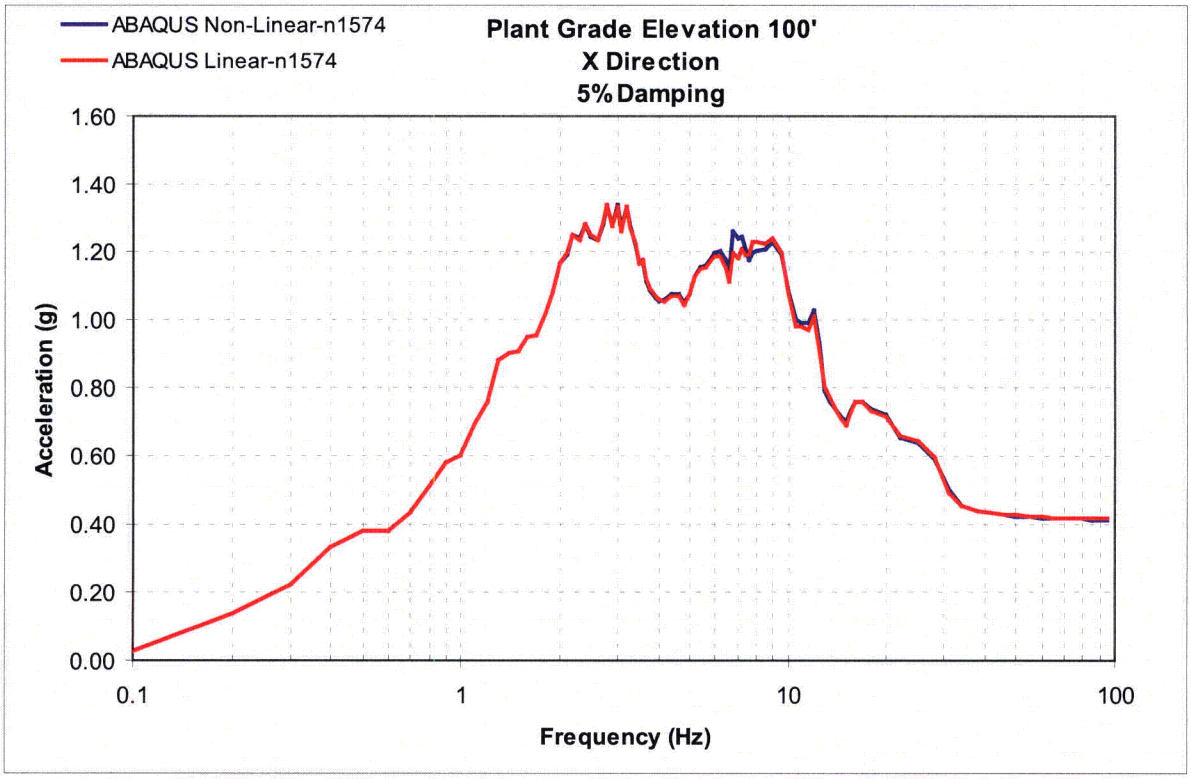

Figure RAI-SRP3.8.3-SEB1-03-07: RS of Aux. Bldg. East Wall at El. 100' Subjected to Enveloped RS in X Direction

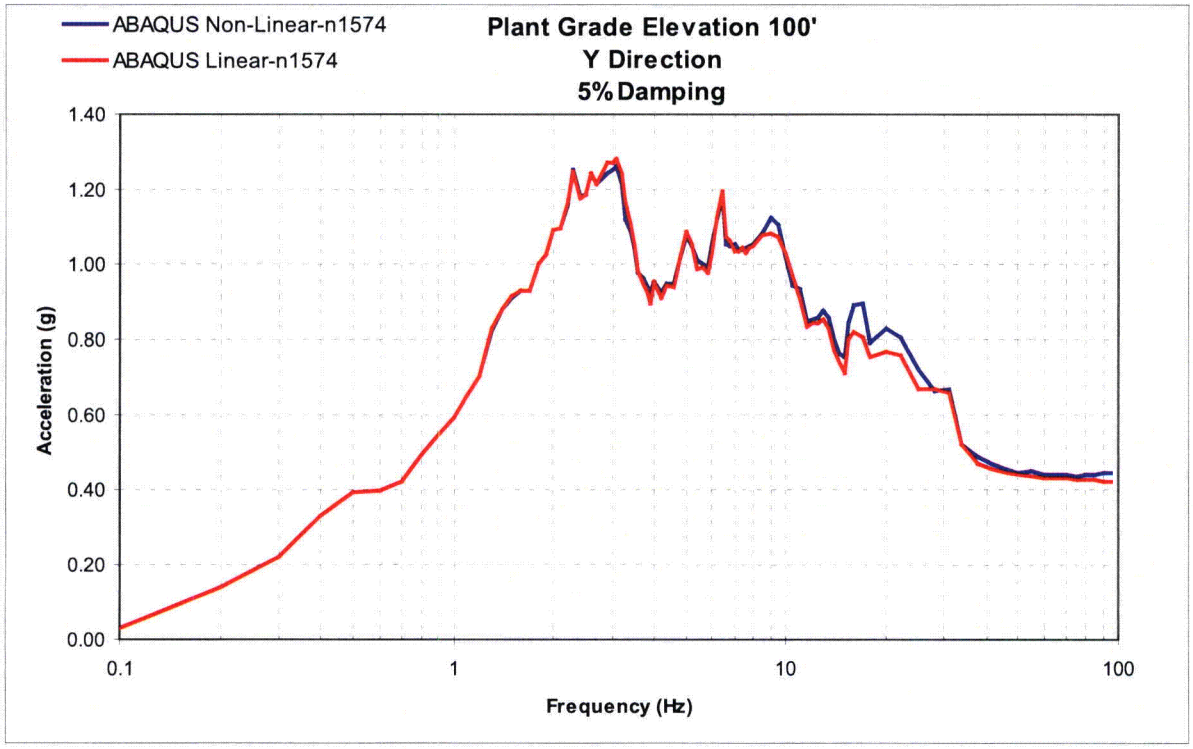

Figure RAI-SRP3.8.3-SEB1-03-08: RS of Aux. Bldg. East Wall at El. 100' Subjected to Enveloped RS in Y Direction

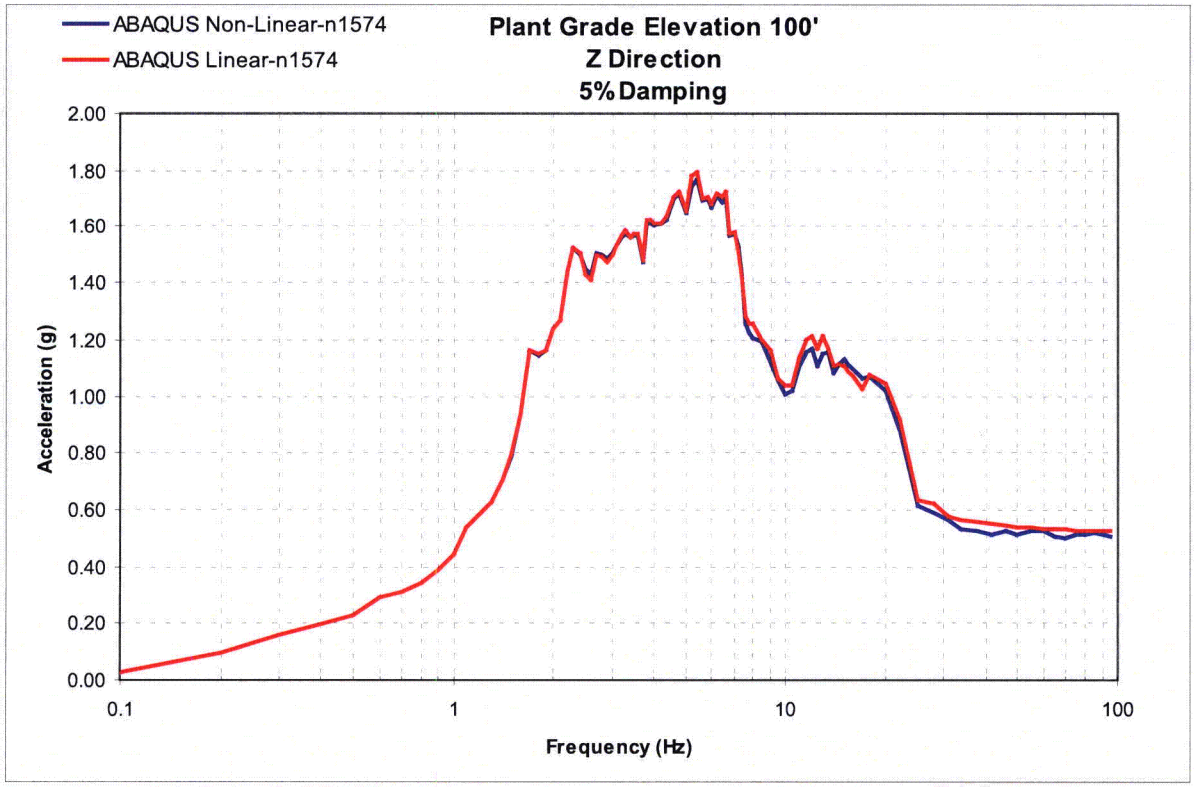

Figure RAI-SRP3.8.3-SEB1-03-09: RS of Aux. Bldg. East Wall at El. 100' Subjected to Enveloped RS in Z Direction

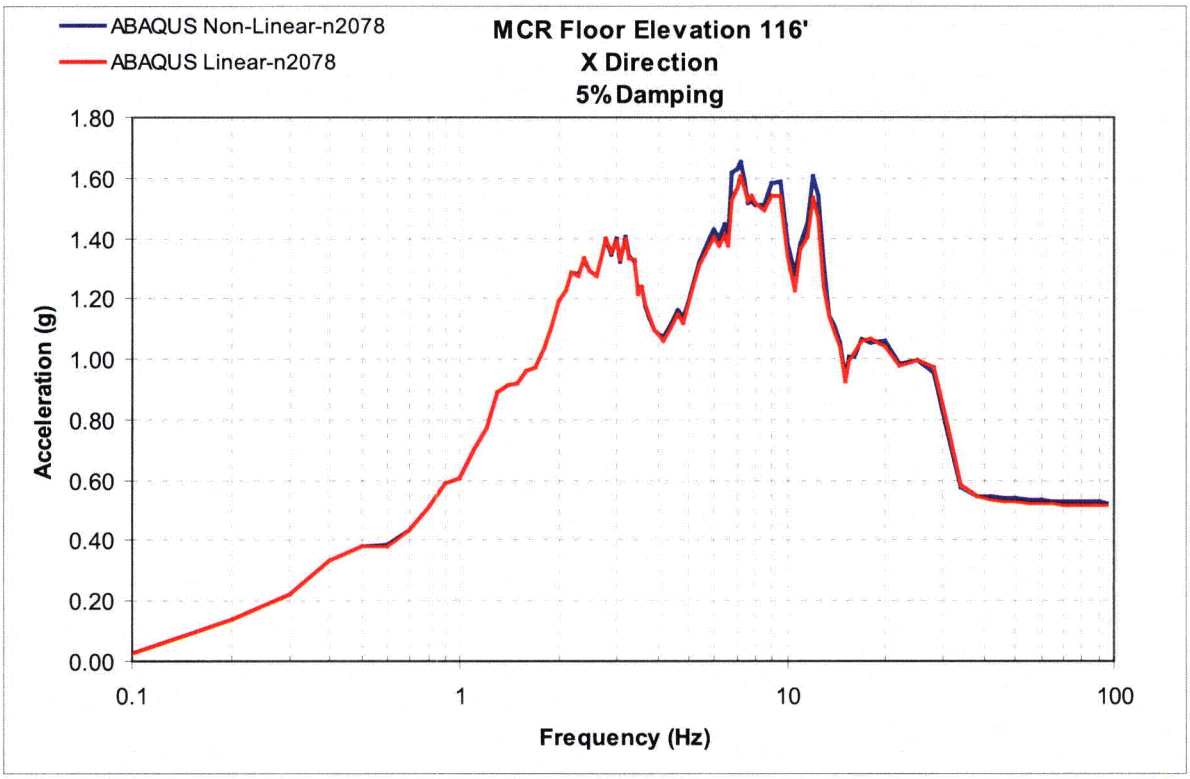

Figure RAI-SRP3.8.3-SEB1-03-10: RS of Aux. Bldg. Main Control at El. 116.5' Subjected to Enveloped RS in X Direction

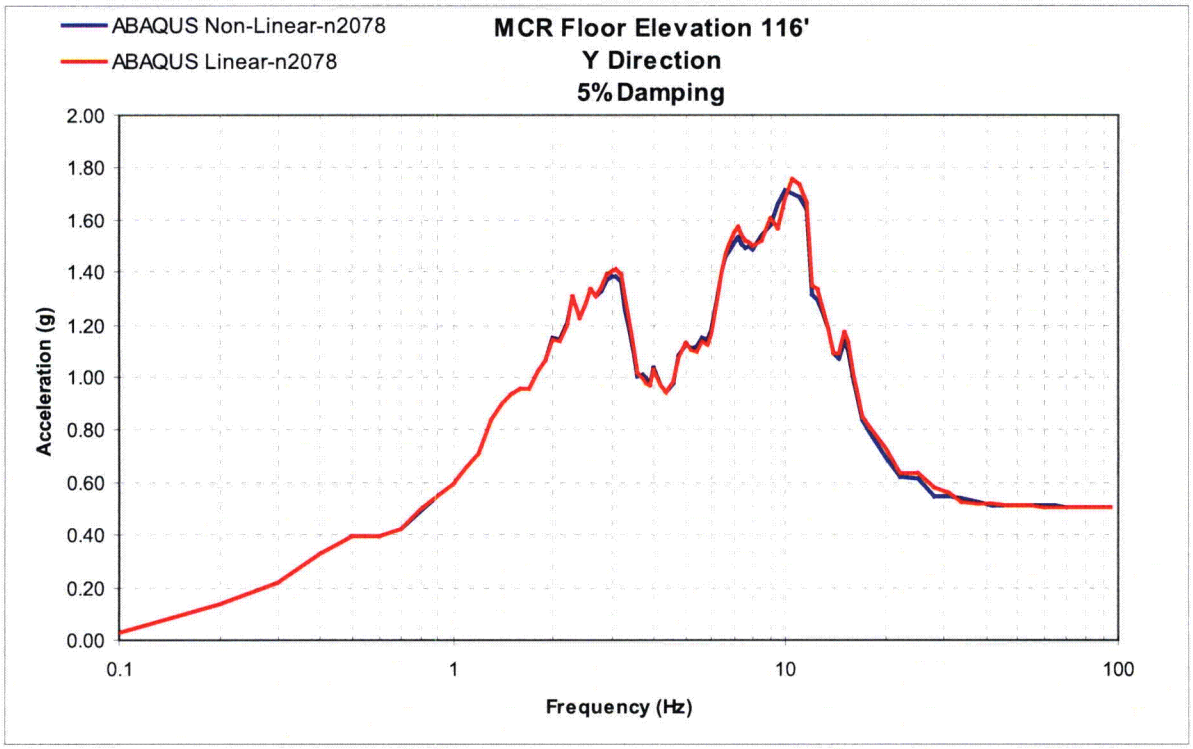

Figure RAI-SRP3.8.3-SEB1-03-11: RS of Aux. Bldg. Main Control Room at El. 116.5' Subjected to Enveloped RS in Y Direction

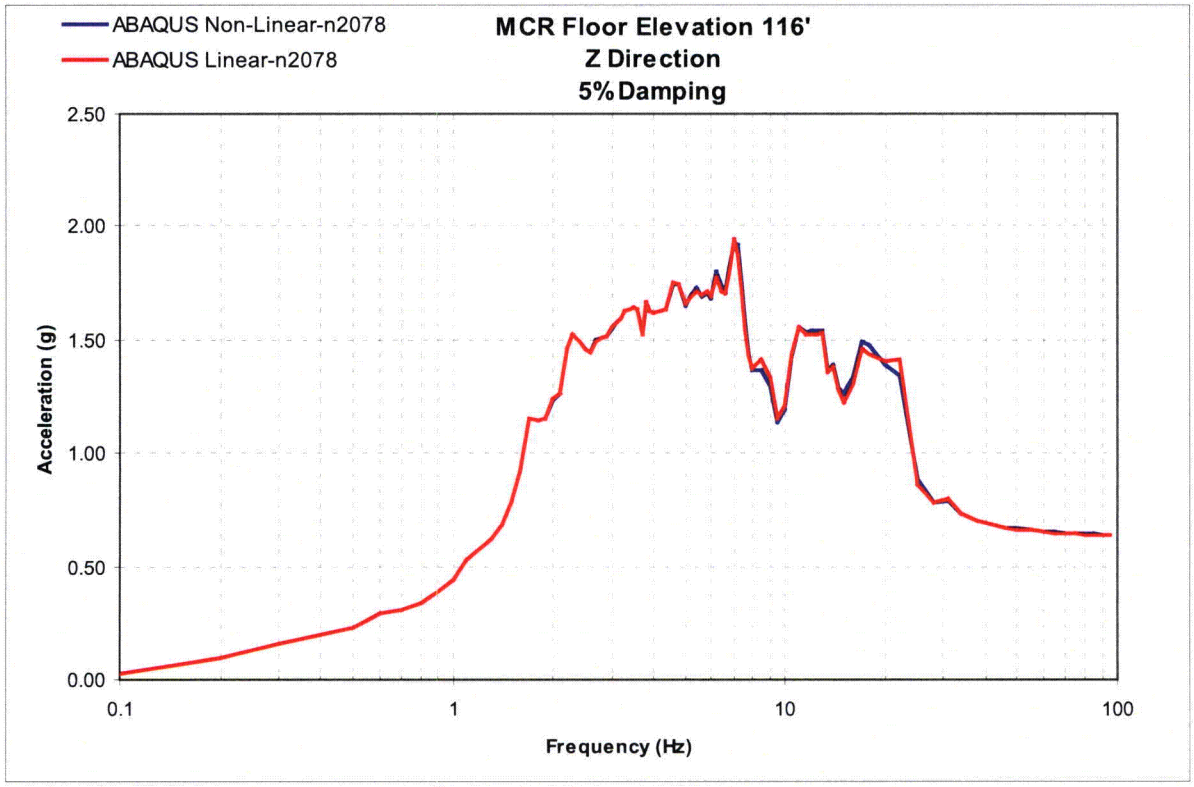

Figure RAI-SRP3.8.3-SEB1-03-12: RS of Aux. Bldg. Main Control Room at El. 116.5' Subjected to Enveloped RS in Z Direction

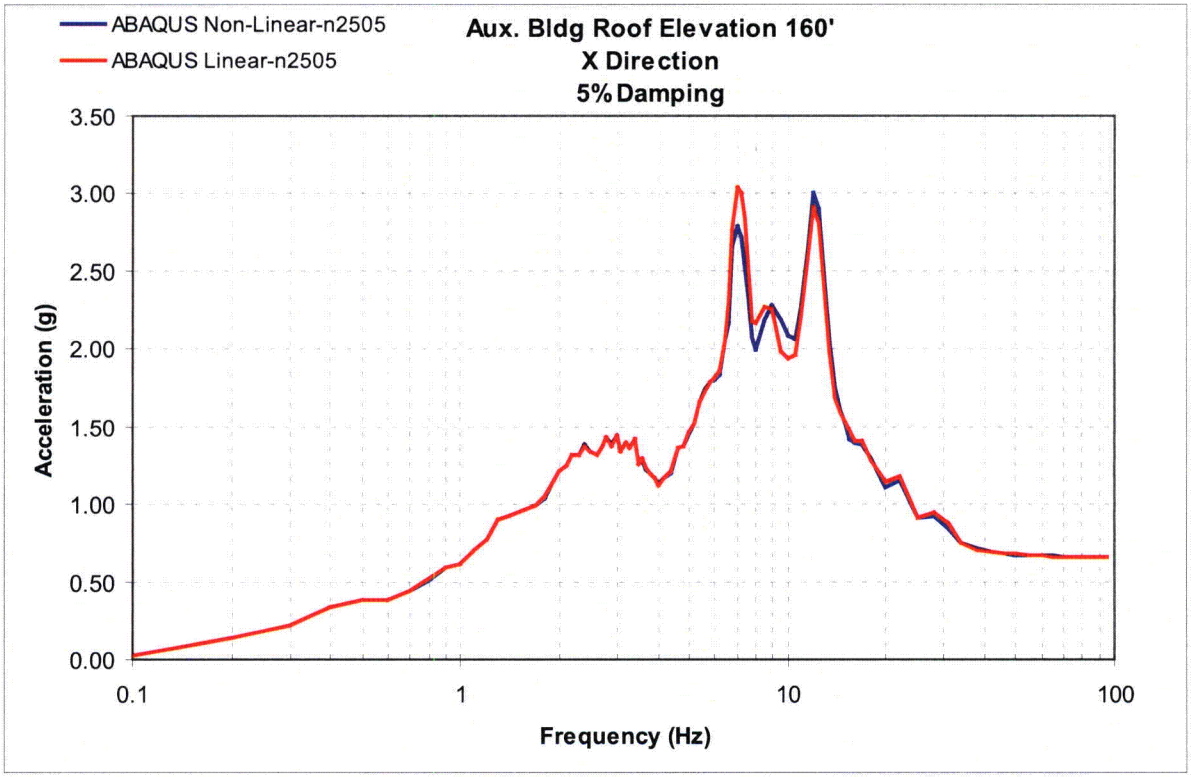

Figure RAI-SRP3.8.3-SEB1-03-13: RS of Aux. Bldg. Roof at El. 160' Subjected to Enveloped RS in X Direction

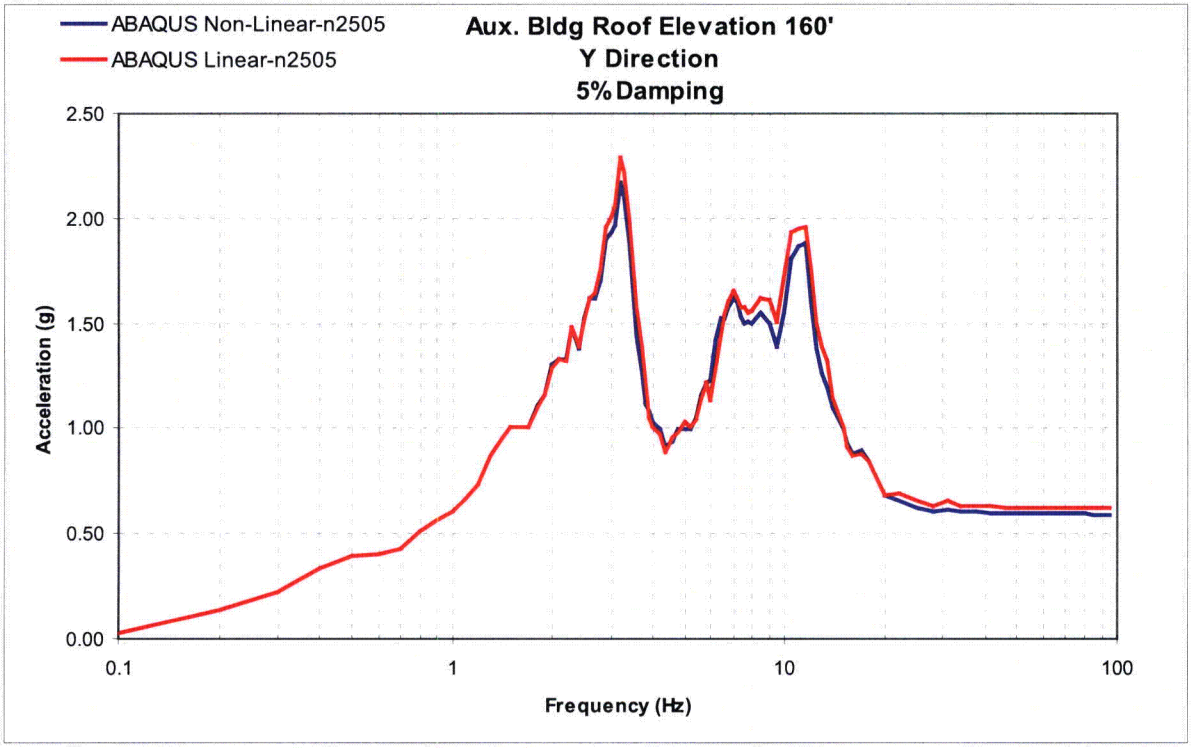

Figure RAI-SRP3.8.3-SEB1-03-14: RS of Aux. Bldg. Roof at El. 160' Subjected to Enveloped RS in Y Direction

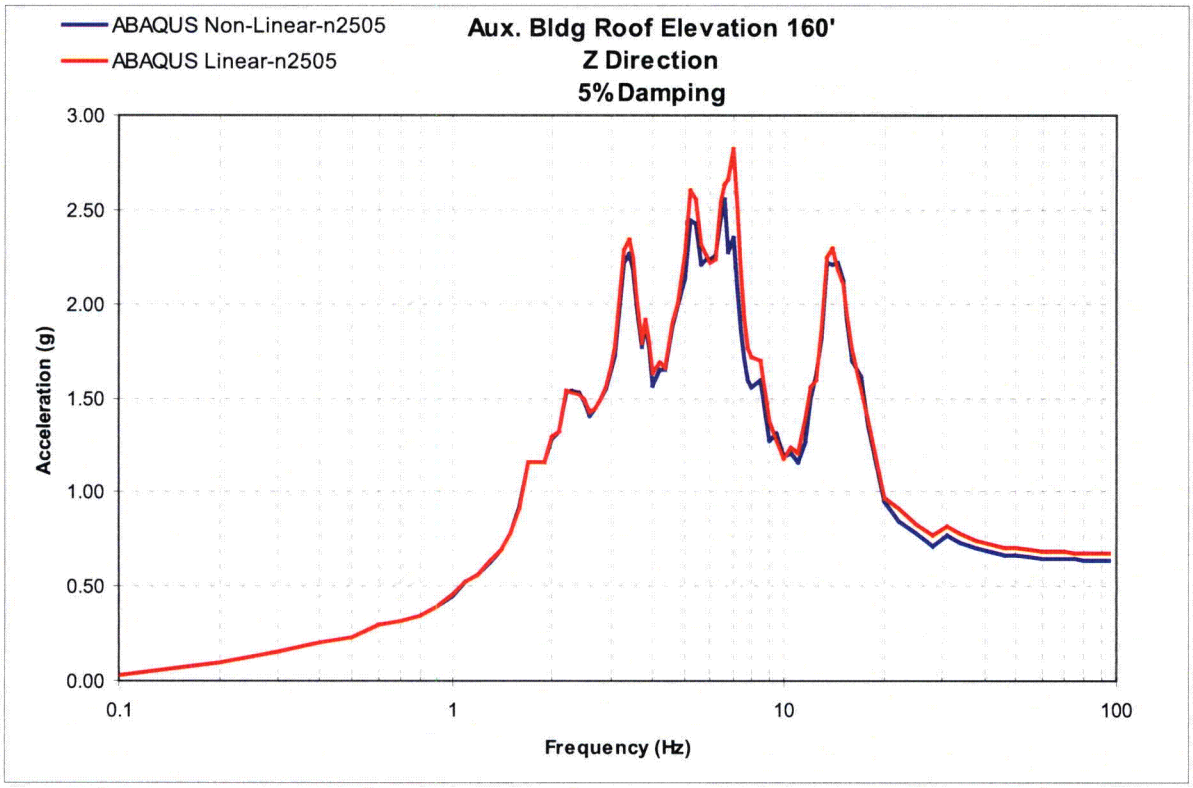

Figure RAI-SRP3.8.3-SEB1-03-15: RS of Aux. Bldg. Roof at El. 160' Subjected to Enveloped RS in Z Direction

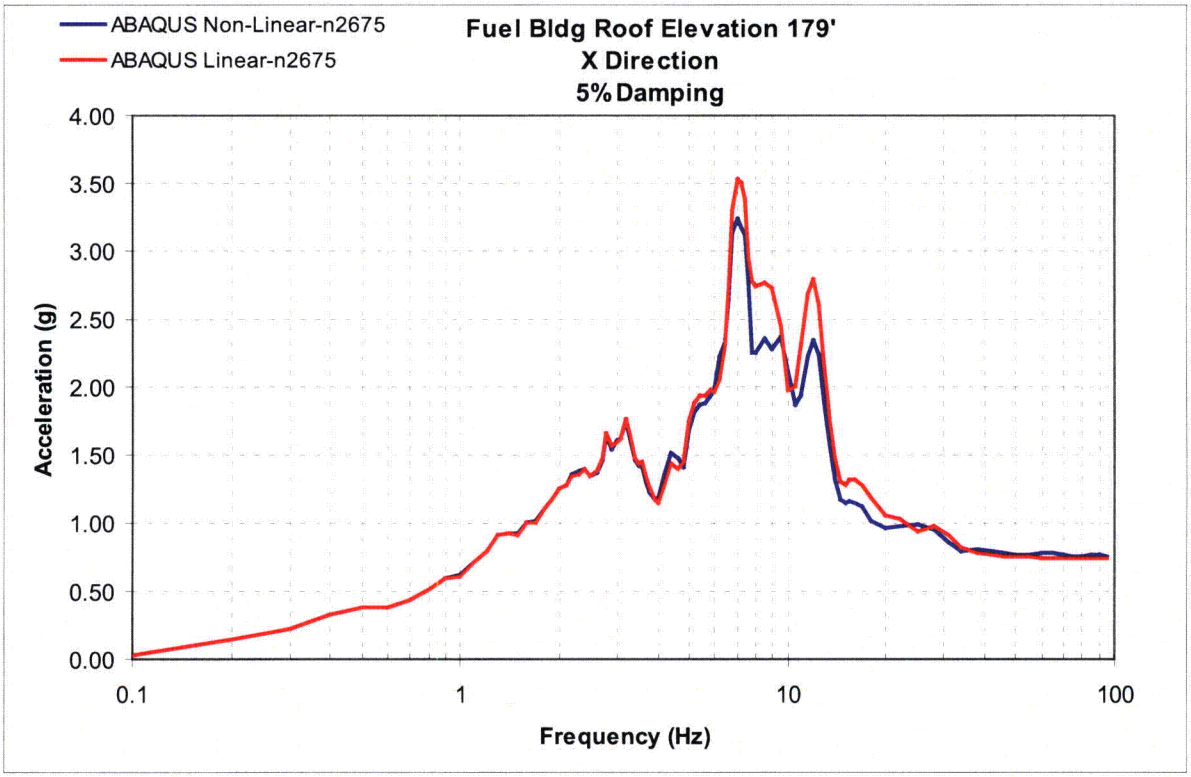

Figure RAI-SRP3.8.3-SEB1-03-16: RS of Aux. Bldg. Corner of Fuel Bldg. Roof at SB at El. 116.5' Subjected to Enveloped RS in X Direction

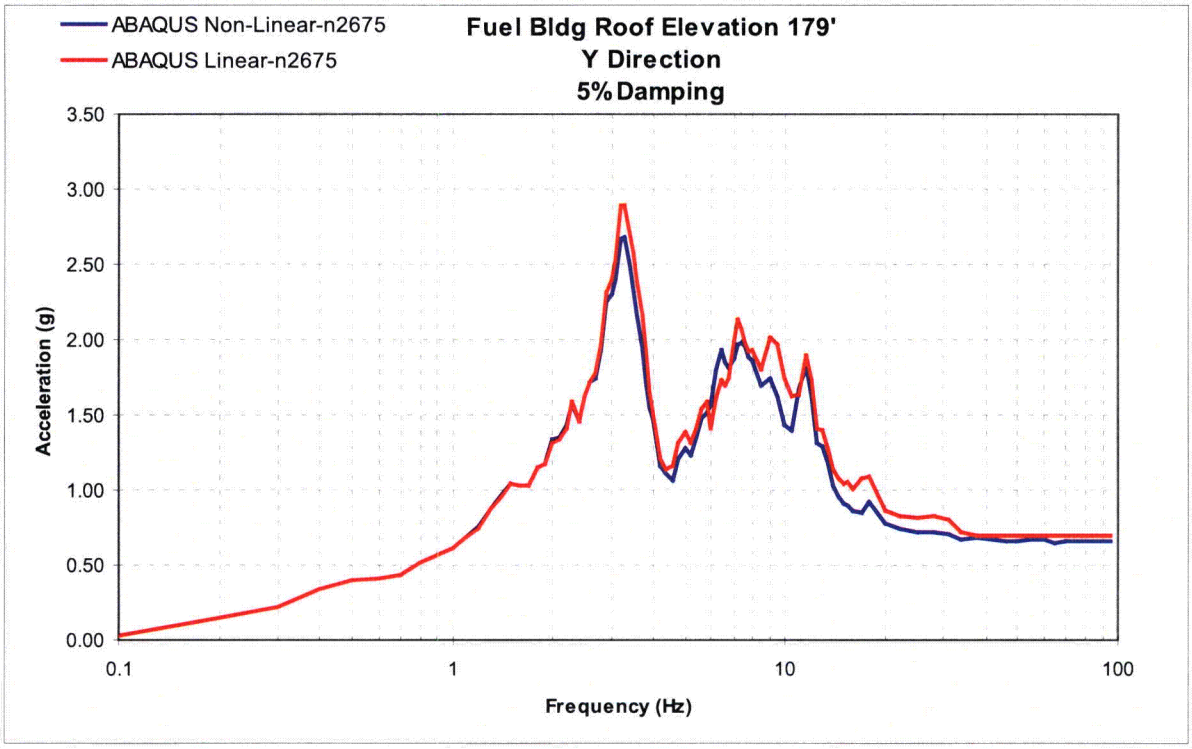

Figure RAI-SRP3.8.3-SEB1-03-17: RS of Aux. Bldg. Corner of Fuel Bldg. Roof at SB at El. 116.5' Subjected to Enveloped RS in Y Direction

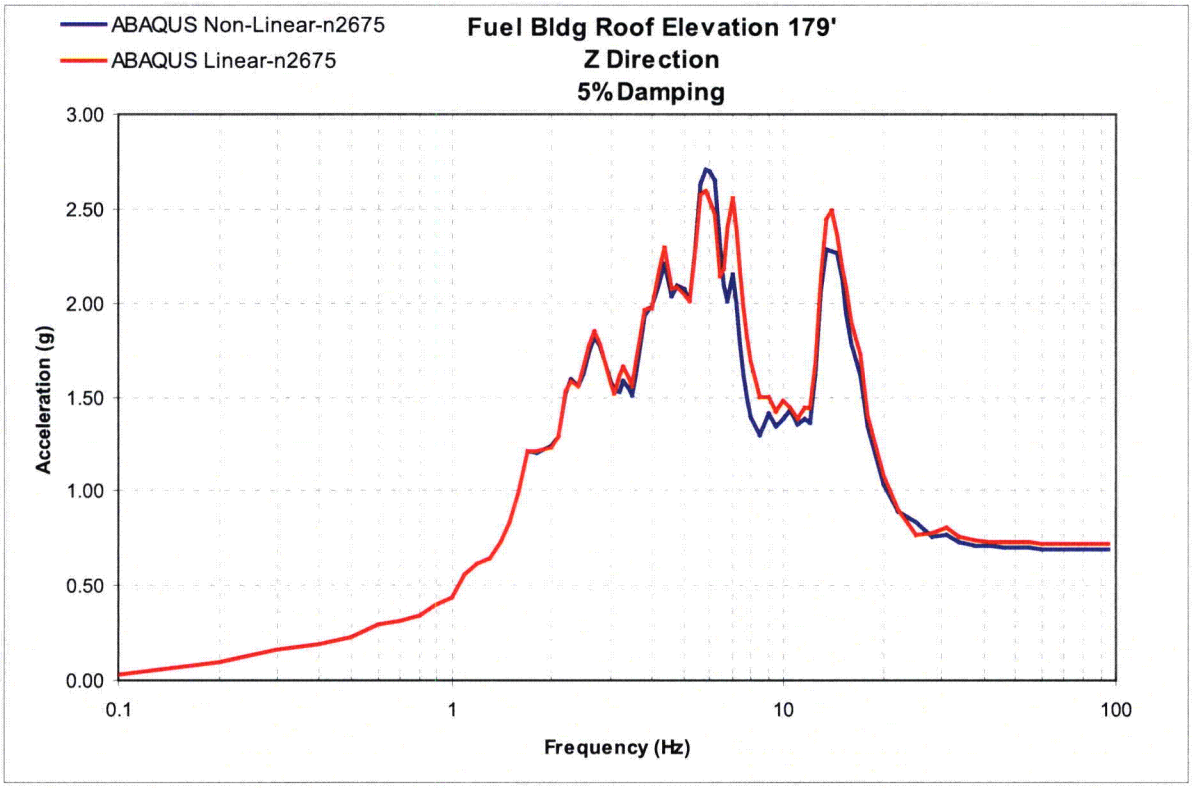

Figure RAI-SRP3.8.3-SEB1-03-18: RS of Aux. Bldg. Corner of Fuel Bldg. Roof at SB at El. 116.5' Subjected to Enveloped RS in Z Direction

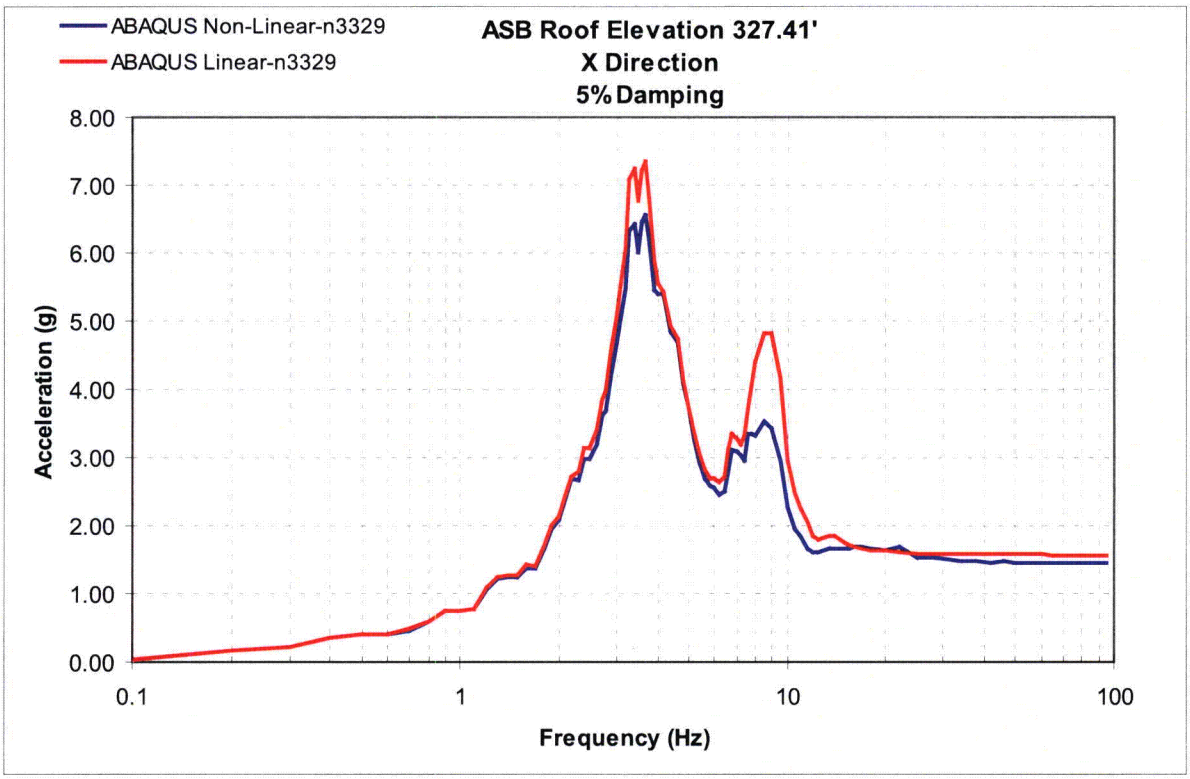

Figure RAI-SRP3.8.3-SEB1-03-19: RS of SB Roof at El. 327.41' Subjected to Enveloped RS in X Direction

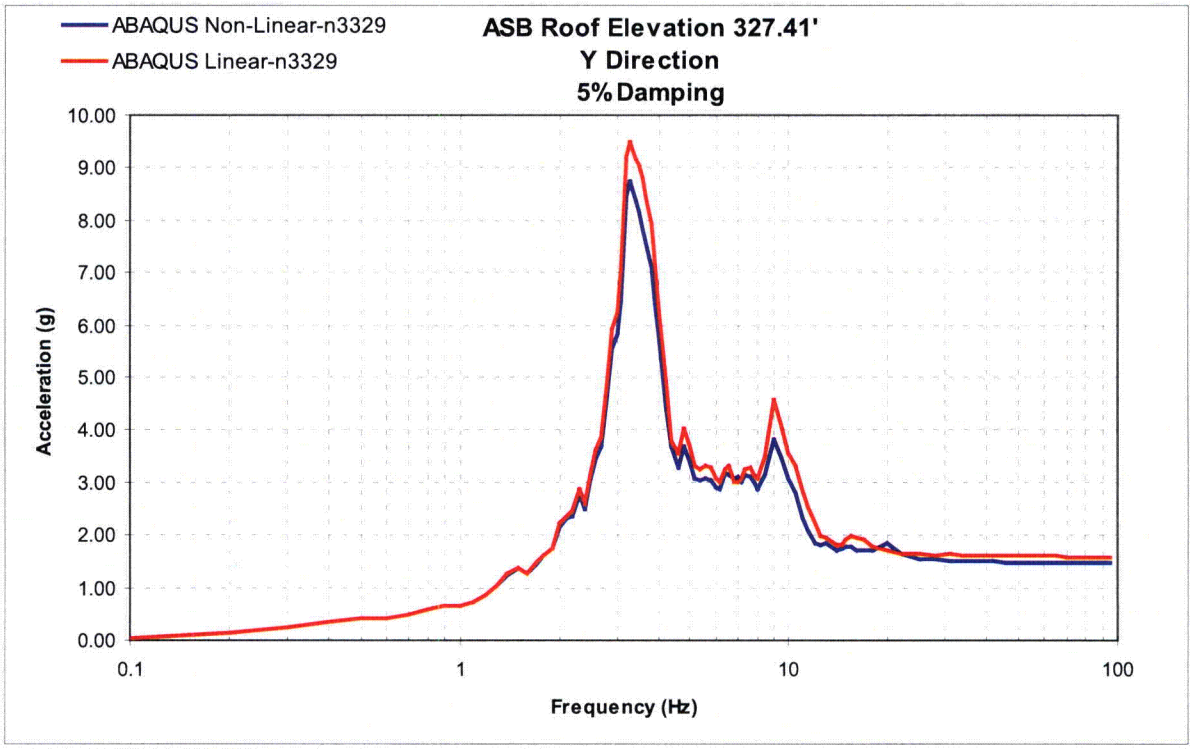

Figure RAI-SRP3.8.3-SEB1-03-20: RS of SB Roof at El. 327.41' Subjected to Enveloped RS in Y Direction

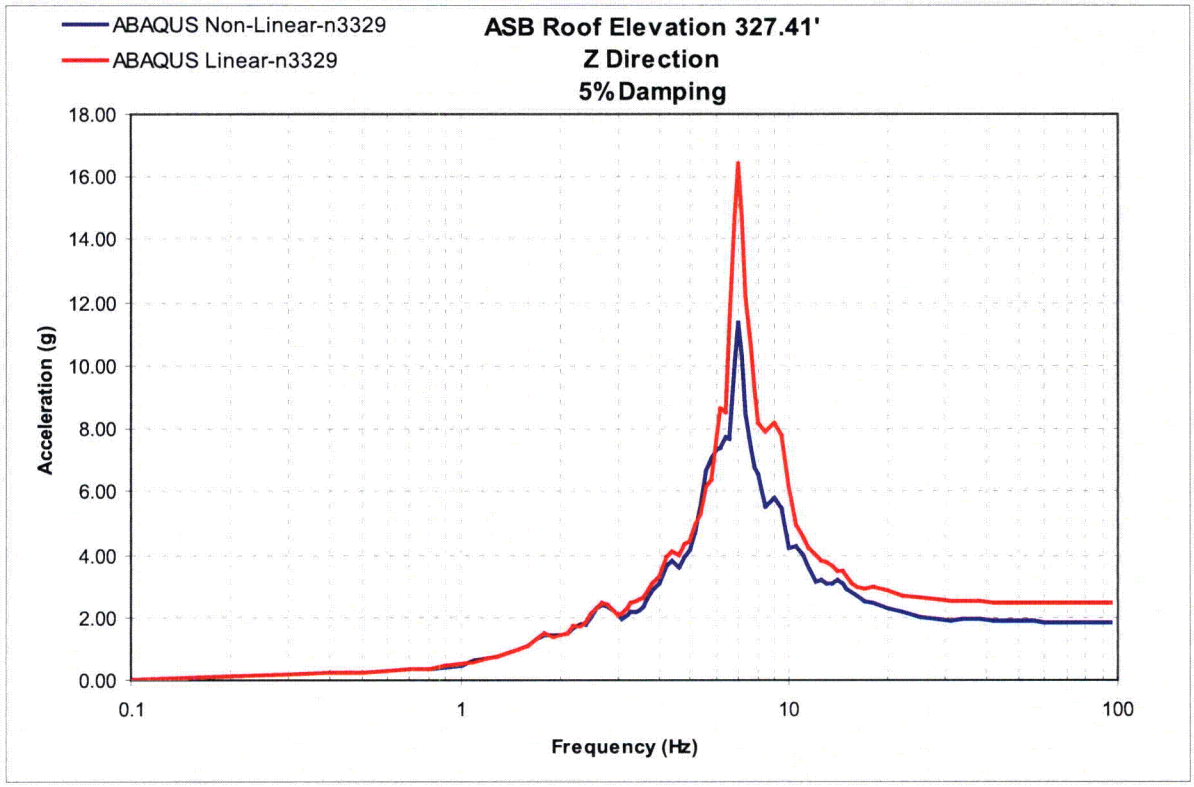

Figure RAI-SRP3.8.3-SEB1-03-21: RS of SB Roof at El. 327.41' Subjected to Enveloped RS in Z Direction

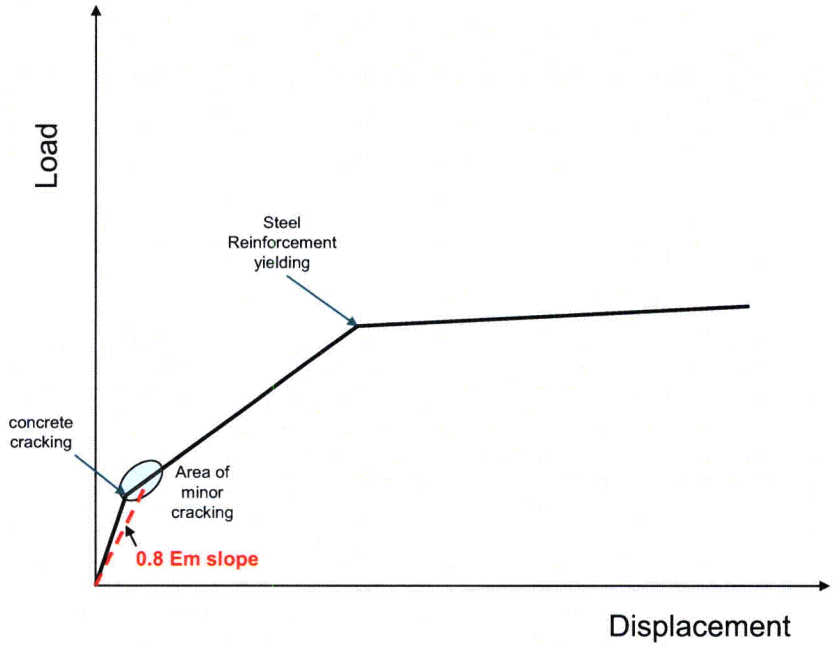

Figure RAI-SRP3.8.3-SEB1-03-22: Sample In-Plane Shear Load Deformation Behavior

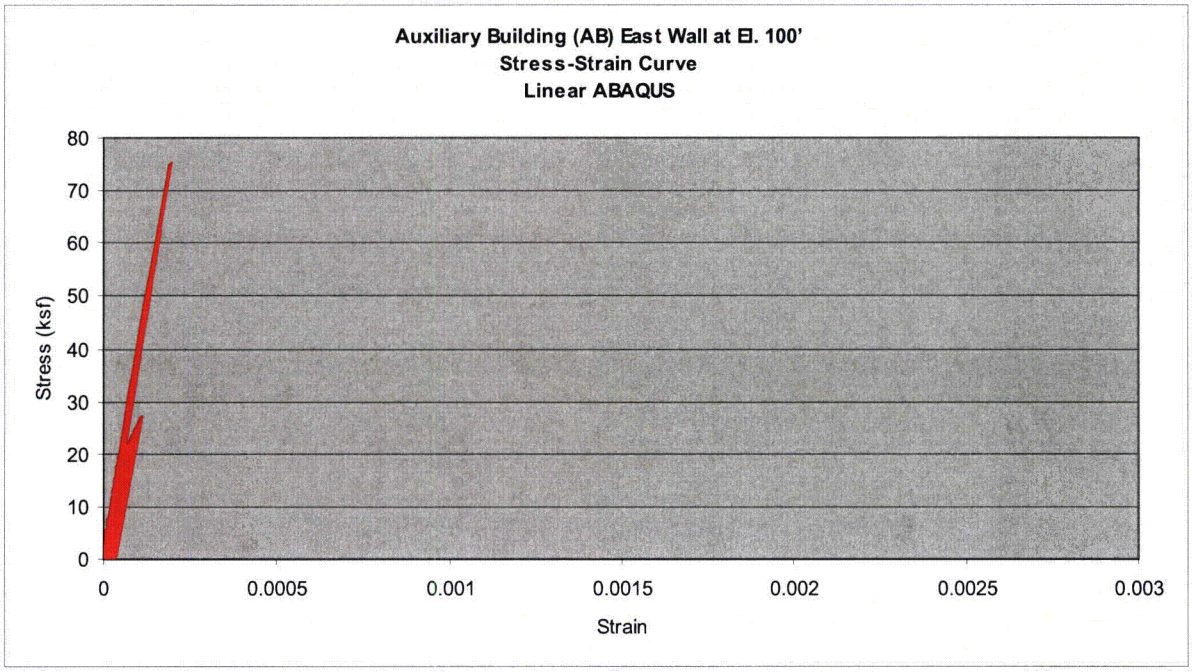

Figure RAI-SRP3.8.3-SEB1-03-23: Linear Stress/Strain Curve at RC AUX East Wall

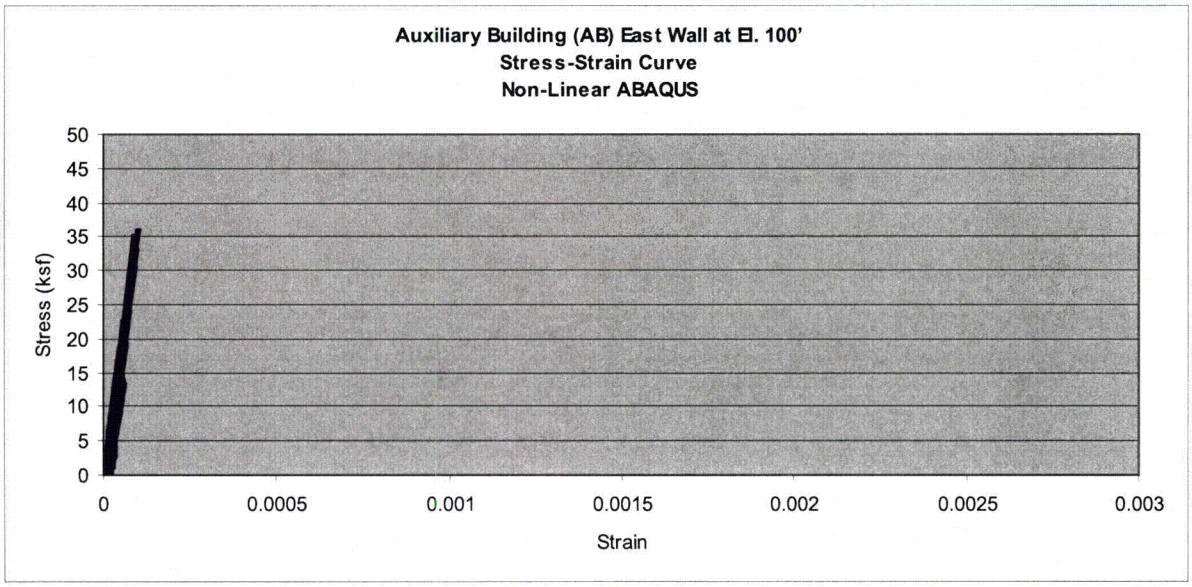

Figure RAI-SRP3.8.3-SEB1-03-24: Non-Linear Stress/Strain Curve at RC AUX East Wall

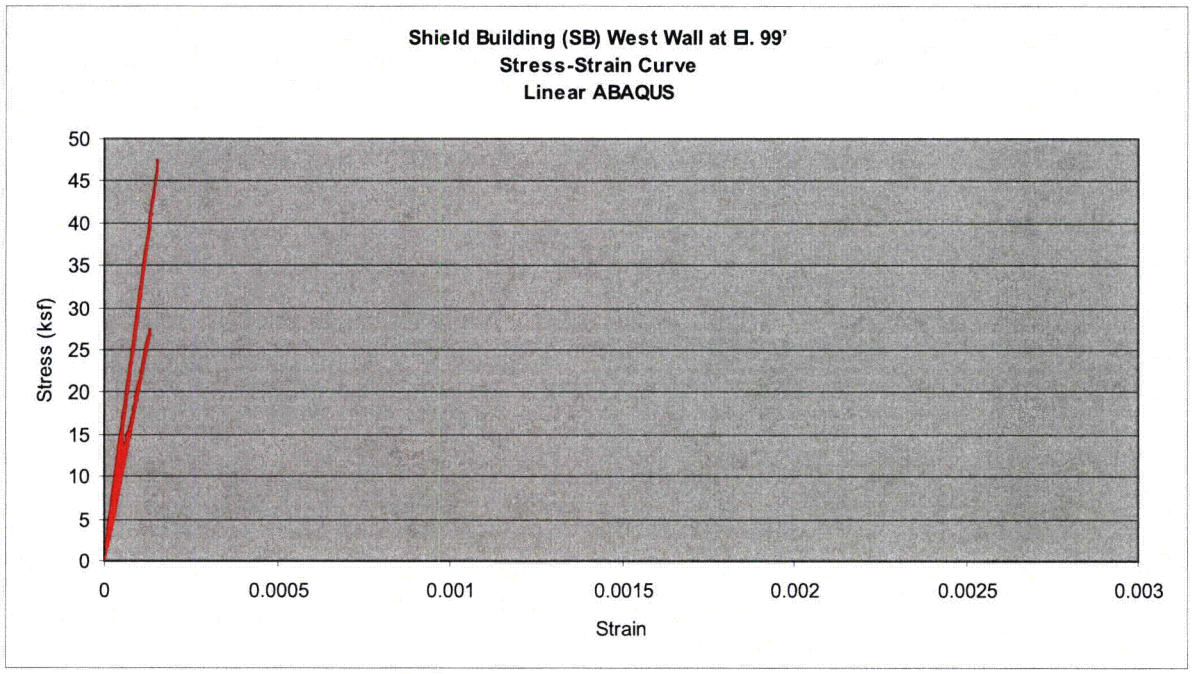

Figure RAI-SRP3.8.3-SEB1-03-25: Linear Stress/Strain Curve at SC Shield Building West Wall

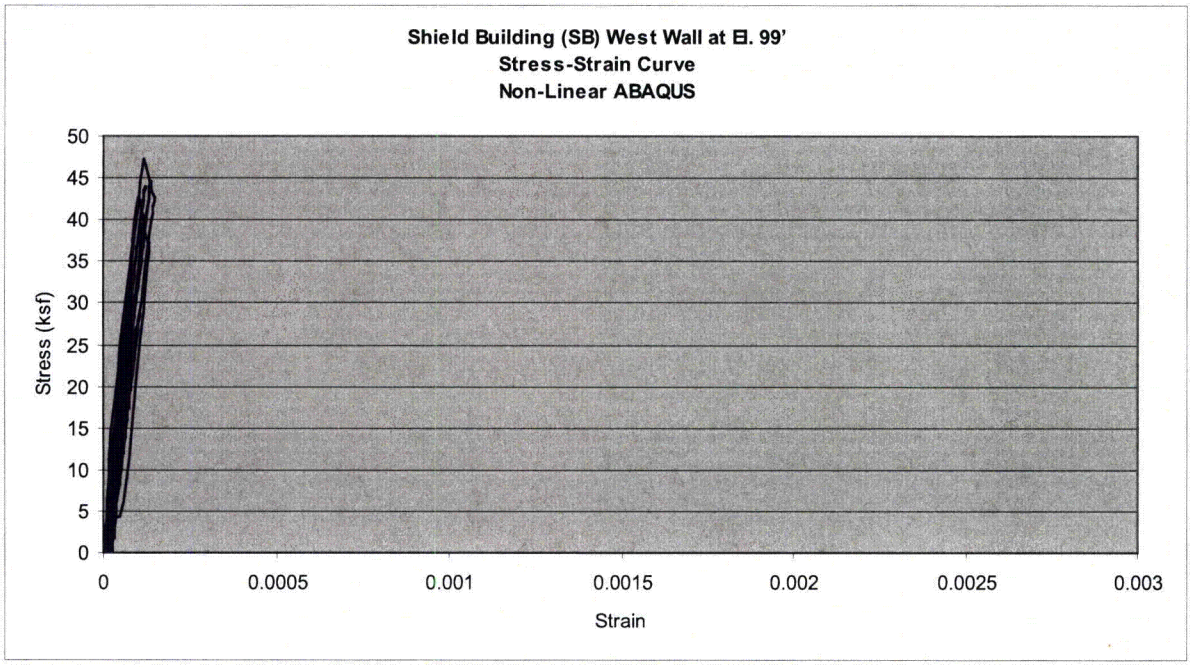

Figure RAI-SRP3.8.3-SEB1-03-26: Non-Linear Stress/Strain Curve at SC Shield Building West Wall

References:

1. APP-GW-S2R-010 (Technical Report 03), "Extension of Nuclear Island Seismic Analyses to Soil Sites.

Design Control Document (DCD) Revision: (Revision 0, 1)

Modify the second paragraph of subsection 3.8.3.4 as shown below:

Methods of analysis for the structural modules are similar to the methods used for reinforced concrete. Table 3.8.3-2 summarizes the finite element analyses of the containment internal structures and identifies the purpose of each analysis and the stiffness assumptions for the concrete filled steel modules. For static loads the analyses use the monolithic (uncracked) stiffness of each concrete element. The elastic modulus is taken as 0.80 times the value calculated based on the ACI Code. This reduced elastic modulus considers a small degree of cracking as described in the seismic analyses in subsection 3.7.2.3. For thermal and dynamic loads the analyses consider the extent of concrete cracking as described in later subsections. Stiffnesses are established based on analyses of the behavior and review of the test data related to concrete-filled structural modules. The stiffnesses directly affect the member forces resulting from restraint of thermal growth. The in-plane shear stiffness of the module influences the fundamental horizontal natural frequencies of the containment internal structures in the nuclear island seismic analyses described in subsection 3.7.2. The out-of-plane flexural stiffness of the module influences the local wall frequencies in the seismic and hydrodynamic analyses of the in-containment refueling water storage tank. Member forces are evaluated against the strength of the section calculated as a reinforced concrete section with zero strength assigned to the concrete in tension.

Modify the second bullet in the last paragraph of subsection 3.8.3.4 as shown below:

- Case 2 considers the full thickness of the wall as uncracked concrete. This stiffness value is shown for comparison purposes. It is applicable for loads that do not result in significant cracking of the concrete and is the basis for the stiffness of the reinforced concrete walls in the nuclear island seismic analyses (prior to the reduction in concrete stiffness by a factor of 0.8). This stiffness was used in the harmonic analyses of the internal structures described in subsection 3.8.3.4.2.2.

Modify the first paragraph of subsection 3.8.3.4.1.1 (Rev. 17) as shown below:

3.8.3.4.1.1 Finite Element Model

~~The three-dimensional (3D) lumped-mass stick model of the containment internal structure is developed based on the structural properties obtained from a 3D finite element model.~~ The structural modules are simulated within the finite element model using 3D shell elements. Equivalent shell element thickness and modulus of elasticity of the structural modules are computed as shown below. The shell element properties are computed using the combined gross concrete section and the transformed steel faceplates of the structural modules. This representation models the composite behavior of the steel and concrete. The modulus of concrete, E_C, is reduced by a factor of 0.8 to consider

the effect of cracking as recommended in Table 6-5 of FEMA 356 (Reference 5 given in DCD Section 3.7.6). See Section 3.7 and Appendix 3G for further discussion of the CIS finite element model.

Modify Table 3.8.3-2 as follows:

Table 3.8.3-2

SUMMARY OF CONTAINMENT INTERNAL STRUCTURES MODELS AND ANALYSIS METHODS

Computer Program and Model	Analysis Method	Purpose	Concrete Stiffness [1]
3D ANSYS finite element of containment internal structures	~~Equivalent static and Response Spectra analyses~~[2]Static	To obtain the in-plane and out-of-plane mechanical ~~seismic~~ forces for the design of floors and walls (dead, live, hydrostatic, pressure)	Monolithic Case ~~3~~1 with Ec reduced by factor of 0.8.
3D ANSYS finite element of containment internal structures ~~fixed at elevation 98′-0″~~	Response Spectra analyses[2]~~Static analyses~~	To obtain the in-plane and out-of-plane seismic forces for the design of floors and walls~~To obtain member forces in boundaries of IRWST for static loads (dead, live, hydrostatic, pressure)~~	Monolithic Case 1 with Ec reduced by factor of 0.8.
3D ANSYS finite element of containment internal structures ~~fixed at elevation 98′-0″~~	Static analyses	To obtain the in-plane and out-of-plane member forces ~~in boundaries of IRWST~~ for thermal loads	Cracked Case 3
The following AP600 analyses are used as background to develop the AP1000 design loads.			
3D ANSYS finite element of containment internal structures fixed at elevation 103′-0″	Harmonic analyses	To evaluate natural frequencies potentially excited by hydrodynamic loads	Uncracked Case 2
	Time history analyses	To obtain dynamic response of IRWST boundary for hydrodynamic loads	Monolithic and cracked Cases 1 & 3

Note:
1. See Table 3.8.3-1 for stiffness case description.
2. See Section 3.7 for discussion of the containment internal structures seismic analyses.

PRA Revision:
None

Technical Report (TR) Revision:
None

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR03-001
Revision: 1

Question: (Revision 0)

The Introduction (p.1, paragraph 5) states "This document addresses seismic response spectra, soil sites, dynamic models, minor structural changes that are significant, seismic results and their impact on seismic design loads for the building structures." The staff notes that only the pressurizer compartment redesign is described in the report. Please describe in detail all the other "minor structural changes that are significant", and why these changes to the AP1000 design are necessary. Also identify the auditable documents that contain the applicable design/analysis calculations for each change.

Additional Question: (Revision 1)

Modeling and analysis of SC wall modules in SB.

TR-03, Rev 4, Section 4.0, does not describe how the SB wall modules are represented in the NI10 and NI20 seismic analysis models. Westinghouse should provide additional information on how the SC modules are characterized in the AP1000 NI analyses.

In TR-03, provide a description of how the SC modules are characterized in the AP1000 NI analyses (e.g., assumptions for "smearing" of SC element properties and for representing the SC/RC connection).

Westinghouse Response: (Revision 0)

The seismic analysis models have been revised from those reviewed during the hard rock design certification for two types of changes. Firstly, there are design changes to the AP1000 which include the shorter pressurizer, an increase in spent fuel storage within the existing pit and a revision to the bracing of the shield slab below the discharge stack. Secondly are changes to the finite element model to better reflect the structural configuration. The changes that have been incorporated into the dynamic models in addition to the redesign of the pressurizer compartment described in the report are discussed further below. All are described in the associated design calculation documenting parts of the finite element model (auxiliary and shield building, containment internal structures, dish below containment vessel, polar crane, core makeup tank).

Design changes

- A design change was made in the spent fuel pool area to permit heavier fuel racks. Masses reflecting the racks and spent fuel were updated. In addition, the water in the fuel pits was modeled as lumped masses instead of solid elements.

  Auditable document: APP-1000-S2C-032, Rev 2, Auxiliary and Shield Building Finite Element Models
  Technical Report: APP-GW-GLR-033, "Spent Fuel Racks Design and Structural Analysis."

- The shield building roof slab bracing was modified from tie rods to cross bracing to improve the seismic response.

  Auditable documents: APP-1000-S2C-032, Rev 2, Auxiliary and Shield Building Finite Element Models

Model improvements

- The dish model was modified to incorporate changes in the annulus configuration included in existing DCD figures. The annulus tunnel on the west side was deleted and replaced by concrete. In addition nodes and elements were modified in the lower shield building and upper CIS basemat to be compatible with the revised Dish model.

  Auditable documents: APP-1010-S2C-002, Rev 3, Finite Element Model of Dish
  Technical Report: APP-GW-GLR-044, "Nuclear Island Basemat and Foundation"

- The core makeup tanks were added as stick models.

  Auditable documents: APP-MV20-S2C-001, Rev 1, Core Makeup Tank Dynamic Model

- Floors in the CIS model were refined to provide better member force results for use in design.

  Auditable document: APP-1100-S2C-034, Rev 4, Finite Element Solid-Shell Model of Containment Internal Structures

- Polar Crane Model – Changes made to the model weight (3% reduction), updated SCV local stiffness, and inclusion of polar crane truck stiffness.

  Auditable documents: APP-MH01-S2C-001, Rev 2.

These changes listed above are considered minor since the nuclear island building basic configuration is not modified. They reflect structural and model changes that are made during design development.

Additional Westinghouse Response: (Revision 1)

The methodology used for modeling the SB wall and SC modules is described in the DCD, Section 3.8.3.4.1.1. The structure is modeled by 3D shell elements using modified stiffness and thickness values to simulate an equivalent response in the structure. This methodology was followed for the SC/RC connection elements as well, and is a conservative approach to the time history analyses.

RAI-SRP3.7.1-SEB1-19 introduced the non-linear ABAQUS analyses performed on the Nuclear Island finite element model. This analysis used benchmarked layered shell elements for the RC/SC connection which transferred all stresses to the steel in the connections once a maximum yield stress in concrete was reached. These elements were included in the comparison made between linear and non-linear models presented in that RAI. Results show the 80% stiffness model response spectra enveloped the non-linear model and provide a conservative approach in terms of response spectra and maximum stresses obtained in the SB wall.

Reference:

None

Design Control Document (DCD) Revision:

DCD revisions are not shown for each RAI. A single set of proposed revisions is given in the response to RAI-TR03-013. The revisions are based on the material in the technical report as well as in the RAI responses. The revisions include changes to Section 3.7 and the addition of a new Appendix 3G providing a summary of the seismic analyses.

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.

The additional structural changes that are described in this RAI will be added to the next revision of the technical report. This addition to the Technical Report will be made at the end of Section 3.0, and is provided below.

Additional structural changes are reflected in the models used for the soil and hard rock cases along with modeling improvements. These are summarized below:

- A design change was made in the spent fuel pool area to permit heavier fuel racks. Masses reflecting the racks and spent fuel were updated. In addition, the water in the fuel pits was modeled as lumped masses instead of solid elements.
- The shield building roof slab bracing was modified from tie rods to cross bracing to improve the seismic response.
- The dish model was modified to incorporate changes in the annulus configuration included in existing DCD figures. The annulus tunnel on the west side was deleted and replaced by concrete. In addition nodes and elements were modified in the lower shield building and upper CIS basemat to be compatible with the revised Dish model.
- The core makeup tanks were added as stick models.
- Floors in the CIS model were refined to provide better member force results for use in design.
- Polar Crane Model – Changes made to the model weight (3% reduction), updated SCV local stiffness, and inclusion of polar crane truck stiffness.

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR03-005
Revision: 2

Question: (Revision 0, 1)

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?

Additional Question: (Revision 2)

TR-03, Rev 4, Section 4.0, states that concrete structures are modeled with linear elastic uncracked properties and that the concrete modulus is reduced to 80% of its value to reduce stiffness to reflect observed behavior of concrete when stresses do not result in significant cracking. Staff notes the SB analyses (Levels 2 &3) have not yet been submitted. The staff reviews procedures used for analytical modeling per the SRP 3.7.2 II 3 staff guidance. To be acceptable, the stiffness, mass, and damping characteristics of the structural systems should be adequately incorporated into the analytical models. The staff's concern is that if the SB has significant cracking, there could be a reduction of the fixed-based SB frequency response leading to an unconservative estimate SSC demands.

The staff has requested that the applicant study the sensitivity of the SB seismic response to a 0.5 stiffness reduction, which is more appropriate when there is significant concrete cracking. Staff review of TR-03, Rev. 4, finds that the 0.8 factor is used for the SB analysis without justification. The staff concern is that if the SB has significant concrete cracking there could be a shift in the fixed-based frequencies of the SB, potentially leading to an increase in the seismic demand on the SB structure and on any systems and components attached to the SB structure.

Based on the SB design changes, provide justification for the assumed 80% reduction in concrete modulus (to account for cracking) given the SB design changes and the more detailed calculations performed (e.g., Level 2 and Level 3 analyses).

Westinghouse Response: (Revision 0, 1)

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.

Additional Westinghouse Response: (Revision 2)

The response is provided in RAI-SRP3.7.1-SEB1-19.

Design Control Document (DCD) Revision:
None

PRA Revision:
None

Technical Report (TR) Revision: (Revision 1)

Revise second paragraph of TR-03, 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.