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Subject: AP1000 Responses to Requests for Additional Information (TR-85)

Westinghouse is submitting responses to the NRC request for additional information (RAI) on AP1000 Standard Combined License Technical Report 85, APP-GW-GLR-044, "Nuclear Island Basemat and Foundation." These RAI responses are submitted in support of the AP1000 Design Certification Amendment Application (Docket No. 52-006). The information included in the responses 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 RAIs:

RAI-TR85-SEB1-03 Rev 1
RAI-TR85-SEB1-04 Rev 1
RAI-TR85-SEB1-05 Rev 1
RAI-TR85-SEB1-11 Rev 1
RAI-TR85-SEB1-15 Rev 1
RAI-TR85-SEB1-32 Rev 1

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,

Robert Sisk

Robert Sisk, Manager
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/Enclosure

1. Responses to Requests for Additional Information on Technical Report No. 85

D063
NRO

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ENCLOSURE 1

Responses to Requests for Additional Information on Technical Report No. 85

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR85-SEB1-03
Revision: ~~0~~1

Question:

In Section 2.3.1, the fourth paragraph (Page 9 of 83) states that, for the AP1000 certified design for hard rock sites, the value of 120,000 pound per square foot was based on the maximum bearing reaction from the (3D FEM) equivalent static nonlinear NI basemat analyses described in Subsection 3.8.5. Provide an explanation for the following:

a. How were the equivalent static nonlinear analyses performed? Were the static forces increased incrementally in an iterative static analysis which released any springs in tension? The description of the analysis method should be described in the technical report and the DCD.

b. Is the same type of analysis performed for the 3D ANSYS equivalent static nonlinear analysis described in Section 2.6.1 of the Technical Report for soil sites?

Explain why the maximum dynamic bearing pressure due to the seismic load reduced from 120,000 psf for the hard rock case in the previous AP1000 certified design to 35,000 psf for the envelope of all rock and soil cases in the current analysis and design?

Additional Request (Revision 1):

The staff reviewed the RAI response provided in Westinghouse letter dated 9/18/07. The response does not adequately demonstrate why the current seismic analysis, using the 2D ANSYS non-linear dynamic analysis, results in such a large reduction of the bearing capacity from 120,000 psf to 35,000 psf, which is a factor of 3.4 times smaller. Westinghouse is requested to
(1) provide the technical basis why this substantial reduction occurred, beyond simply stating it was caused by the difference between an equivalent static analysis and a time history analysis,
(2) describe how the 35 ksf maximum bearing pressure compares to the new 3D NI20 model using ANSYS response spectrum analysis (linear analysis without any liftoff) with input enveloping all 6 soil cases,
(3) describe why the maximum bearing reaction for the current 2D ANSYS analysis only based on 2 directions (EW and vertical) instead of considering the contribution to bearing pressure from all 3 directions, and
(4) describe the technical basis for relying on a very simple single beam model for the NI structure and a horizontal single rigid beam for the basemat, and discuss whether these two items be addressed by comparison to the 3D NI20 model analysis described in item 2 above.

Westinghouse Response:

a. As described in the DCD, the non-linear static analyses were performed using the ANSYS computer code. ANSYS employs the "Newton-Raphson" approach to solve nonlinear problems. ANSYS iterates the analysis until the solution converges.

b. The 3D ANSYS equivalent static nonlinear analysis described in Section 2.6.1 of the Technical Report for soil sites is the same type of analysis as that performed for the hard rock site. The soil springs corresponded to a soil profile rather than to hard rock. There were minor differences to the building models as described and reviewed in Technical Report 03 (Reference 1). The equivalent static accelerations applied in the latest analyses were the envelope of hard rock and soil cases from the nuclear island analyses using shell models. The equivalent static accelerations applied in the hard rock analyses were from the nuclear island analyses using stick models as documented in DCD Rev 15.

The maximum dynamic bearing pressure due to the seismic load reduced from 120,000 psf for the hard rock case in the previous AP1000 certified design to 35,000 psf for the envelope of all rock and soil cases in the current analysis and design due to the use of results from additional non-linear dynamic analyses instead of the more conservative equivalent static analyses. The differences between the maximum dynamic bearing pressure due to the seismic load in the previous AP1000 hard rock certified design and the envelope of all rock and soil cases in the current analysis and design is addressed in Section 2.4.3 which states:

> The AP1000 DCD for hard rock added a requirement of 120,000 lb/ft^2 for dynamic loads. This was based conservatively on the maximum bearing reaction from the equivalent static non-linear nuclear island basemat analyses described in section 2.3. This maximum bearing reaction occurs below the west edge of the thick concrete basemat below the shield building. This value was included in DCD Table 2-1 since it was expected that a hard rock site would be capable of satisfying this bearing demand. The dynamic non-linear analyses described in section 2.4.2 show much lower bearing reactions (27.8 ksf for hard rock) than those from the equivalent static design analyses for the basemat. The 2D ANSYS non-linear analyses show that the soft-to-medium soil case gives higher bearing pressures (34.5 ksf) than the hard rock case. This establishes the soil bearing interface of 35,000 lb/ft^2. The bearing pressures from the ANSYS analyses are conservative because the effect of the side soil is conservatively neglected.

Westinghouse Response (Revision 1):

In response to the many questions in this and other RAIs, Westinghouse has revised the basis for the bearing demand. The demand is now based on 3D SASSI analyses using the 3D NI20 finite element model. These analyses are similar to those described in TR03 which are used as the basis for the seismic response on all sites except hard rock. The modeling of the soil was refined in order to provide bearing pressures in the elements adjacent to the nuclear island.

~~The AP1000 bearing pressures have been calculated by a number of conservative methods. The hard rock certification bearing pressures were performed using acceleration values from the stick models that were greater than the ones obtained from the stick models and using soil springs that allowed no redistribution of peak stresses.~~

~~The values obtained using the ANSYS 2D dynamic analyses are consistent with the 3D SASSI bearing pressures obtained from the generic analyses.~~ The bearing pressures from the 3D SASSI analyses have been obtained by combining the time history results from the North-South, East-West, and vertical earthquakes. The maximum bearing pressures obtained from the various soil cases are listed in Table RAI-TR85-SEB1-03-1. All the maximum bearing pressures occur at the west end of the AP1000 nuclear island. The 3D SASSI results of 38.6 ksf is larger than the 35 ksf limit stated in the DCD but this is a localized stress using no side soil. It is recognized that the maximum peak stress obtained for the hard rock site exceeds the 35 ksf value by 10%. This is a localized stress that is not over all of the foundation, but a very small area of the basemat footprint. It is unrealistic to define the hard rock seismic bearing stress demand based on the localized maximum peak stress. Maintaining a limit of 35 ksf for maximum bearing seismic demand is realistic and conservative because:

- The foundation material directly below the basemat will redistribute this maximum peak stress. The seismic loading will distribute over a larger portion of the rock foundation than where the localized peak stress occurs; therefore reducing the overall maximum seismic stress demand;
- The maximum peak stress is of short duration;
- From the presumptive values of allowable bearing pressures for spread footings (Table 1 from NAVFAC DM-7.02, pg 7.2-142 to -143) for hard, sound rock, an allowable bearing pressure of 80 tsf (160 ksf) can be used for the Hard Rock (HR) soil case. In Table 2, on pg. 7.2-144, item 3, (cited NAVFAC reference) the "bearing pressures up to one-third in excess of the nominal bearing values are permitted for the transient live load from wind or earthquake." The short term, transient earthquake load of 38.3 ksf that is 10% higher than the 35 ksf bearing demand limit is only 24% of the suggested allowable bearing pressure for hard, sound rock without consideration of the one-third increase.

Table RAI-TR85-SEB1-03-1

SASSI 3D Maximum Bearing Pressure

Soil Case	Pressure (Ksf)
Hard Rock (No side soil)	35.0*
Firm Rock	27.9
Soft Rock	24.0
Upper Bound Soft to Medium	25.7
Soft to Medium	23.1
Soft Soil	21.9

* The maximum bearing demand is obtained by averaging over 335 ft^2 of the West Edge of the Shield Building is 33 ksf. The results show small uplift on the east side of the nuclear island similar to those observed in the 2D models. The non-linear analyses using a 2D model in ANSYS are described in section 2.4.2 of TR85. These lift off analyses show that the maximum bearing pressure increases 4 to 6% over the linear results. Thus, the 35 ksf demand specified in the interface parameters envelopes all cases.

(1) The maximum bearing pressure of 120 ksf used as an interface in the hard rock design certification was the maximum bearing pressure from the nuclear island basemat design analyses. These analyses were developed to provide conservative member forces for design of the basemat. The analyses were non-linear static analyses. The applied loads were the maximum acceleration at each elevation obtained from the time history analyses of a nuclear island stick model. This conservatively assumes that the maximum response at each elevation is in phase. Both linear (no lift off) and non-linear (lift off) analyses were performed. The maximum bearing pressure in the linear analyses was 65 ksf; this increased by 82% to 118 ksf in the static lift off analysis. Subsequent liftoff studies using a 2D model in ANSYS are described in section 2.4.2 of TR85 (see response to RAI-TR85-SEB1-05, Rev 1 for the proposed section to be included in TR85 Rev 1). These lift off analyses show that the maximum bearing pressure increases 4 to 6% over the linear results when evaluated in dynamic analyses.

The basemat analyses use a detailed finite element model of the basemat on soil springs. The 2D SASSI models assume a rigid basemat on soil layers. The 2D ANSYS models assume a rigid basemat on soil springs. The west side of the shield building rests on mass concrete modeled with solid elements extending a depth of 39'6" to the underside of the basemat. The auxiliary building rests on a 6' thick basemat modeled with shell elements. These thicknesses provide a near rigid basemat on a soil site. This is demonstrated by the nearly parallel uniformly spaced contours of basemat pressures on a soil site (subgrade modulus, K_v = 520 kcf) shown in Figures 2.6-4 to 2.6-9 of TR85. The basemat is flexible relative to the hard rock foundation. This results in local higher bearing below the walls of the nuclear island.

The bearing results from the nuclear island basemat analyses were replaced in the specification of bearing demand in TR85 by results from the 2D ANSYS lift off analyses. These results are now being replaced by results from 3D SASSI analyses of the NI20 model. This eliminates the conservatisms described above related to the equivalent static seismic analyses model. The model is sufficiently refined below the shield building to include the effects of flexibility of the basemat below the shield building, hence giving reasonable bearing pressure estimate at the west side of the shield building where the bearing demand is highest. The 3D SASSI analyses are linear analyses. The effect of lift off is shown to be small in the 2D ANSYS analyses.

(2) The 3D NI20 model was used in time history analyses (linear analysis without any liftoff) with input enveloping all 6 soil cases. Response spectrum analyses were not performed with this model.

(3) The bearing demand is now based on the results of 3D SASSI analyses and considers the contribution to bearing pressure from all 3 directions

(4) The bearing demand is now based on the results of 3D SASSI analyses of the NI20 finite element model. The simple stick model for the NI structure and a horizontal single rigid beam for the basemat are no longer used as the primary analyses to specify the bearing demand.

References:

1. APP-GW-S2R-010, Revision 1, Extension of Nuclear Island Seismic Analyses to Soil Sites, September, 2007.

Design Control Document (DCD) Revision:
None

PRA Revision:
None

Technical Report (TR) Revision:

~~None~~ The technical report will be revised updating the 2D analyses and adding the 3D SASSI analyses along with the bearing pressures.

AP1000 TECHNICAL REPORT REVIEW
Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR85-SEB1-04
Revision: ~~0~~1

Question:

Sections 2.3.1, 2.4.1, 2.4.2, and 2.6.1 indicate that equivalent static nonlinear analysis (not clear whether 2D or 3D), 2D SASSI analysis, 2D ANSYS linear dynamic analysis, 2D ANSYS nonlinear time history analysis, 3D ANSYS equivalent static non-linear analysis, etc. were performed. Westinghouse needs to develop a table (or tables) similar to AP1000 DCD Tables 3.7.2-14 and 3.7.2-16 to show: (1) the purpose of the analysis; (2) the model type(s); (3) analysis method(s); (4) soil condition(s); (5) loads, load combinations, combination method (for combining loads and directional combination for SSE); (6) governing design loads; and (7) reference location in this technical report or other report for the detailed description.

Additional Request (Revision 1):

The RAI response provided revised DCD Tables 3G.1-1 and 3G.1-2. These tables were revised to address the information requested in this RAI and to reflect the changes in methodology described in other RAI responses. Three entries in these detailed tables, related to the basemat design analyses, soil bearing reactions, and stability evaluation, were also included separately in the RAI response.

Based on this and other RAI responses it appears that a number of the seismic models and analyses have been substantially revised. Therefore, Westinghouse is requested to confirm the staff's understanding that the current seismic analyses of the basemat are based on the following:

(1) <u>Maximum dynamic bearing pressure calculations</u> due to seismic loading are still based on the 2D finite element stick model, using time history analysis with ANSYS, non-linear soil springs (with lift-off), for two soil cases performed previously - hard rock and soft to medium soil (1000 kcf) and two new confirmatory soil cases (1340 kcf and 780 kcf) to be completed. This is further revised by response to RAI TR85-SEB1-22, which states that for this 2D ANSYS analysis, six soil cases shown in the proposed revision to Table 2.6-1 (left hand column) are used. The staff still has concerns with the use of 2D instead of 3D seismic inputs (addressed in RAI TR85-SEB1-03), the use of a simplified stick model (addressed in RAI-TR85-03), and why lower subgrade modulus values of the order of 80 kcf (addressed in RAI-TR85-SEB1-05) were not considered.

(2) <u>Stability evaluations</u> (for sliding and overturning) are based on a new 3D NI20 model response spectrum (linear no lift-off) analysis, enveloping all soil cases, using ANSYS. Westinghouse is requested to provide a full description of this model, range of soil springs used, analysis approach, and results. Since this model assumes no lift-off, Westinghouse is requested to confirm the adequacy of the existing stability evaluations by comparing the set of shear and overturning loads to those from one of the other seismic analyses that include the non-linear soil springs which permit lift-off effects. There is some inconsistency identified with

which model is being used for stability evaluations (addressed under item a below, and RAI TR85-SEB1-34).

(3) New equivalent static accelerations are calculated based on the 3D NI20 model, mode superposition time history analysis, ANSYS, linear, for hard rock and also calculated for the 3D NI20 model, time history analysis, SASSI, for five soil cases (values need to be defined). These are only developed to be used in a confirmatory analysis described below.

(4) The basemat design is still based on the 3D NI05 model (prior to the design change to enhance the shield building), equivalent static analysis, ANSYS, with non-linear soil springs for lift-off from the basemat and for the connections between basemat/containment vessel/CIS basemat, and only one soil case for springs (520 kcf), using the prior equivalent static accelerations from the prior global seismic analyses on hard rock and considering all soil cases. The adequacy of using these accelerations, existing model, and existing design was confirmed by comparing the total base reactions and bearing pressures from the above analysis with a new 3D NI20 updated model for the shield building, fixed base, time history analysis. The time history used for the new fixed base analysis is developed so that it envelops the basemat response given by the 3D SASSI analyses at the corners and center of the basemat for all soil cases.

Since so many of the seismic models and analyses are being substantially updated, it is not clear how the current evaluations and to what extent the previous evaluations will be deleted. Therefore, to facilitate the resolution of this and other RAIs, Westinghouse is requested to provide a revised

This revised technical report should contain in each subsection a complete description of all of the updated models, specific soil cases considered (if qualitative terms are used (e.g., soft soil), then include the corresponding specific soil subgrade modulus values to avoid any misunderstanding), analysis approach, and results, and also should delete the superseded analyses. If any prior analyses remain in TR85, because certain aspects of the design or study are still based on the prior analyses, then the technical report should clearly describe why they remain in the report and should clearly demonstrate that the new evaluations confirm the adequacy of the prior analyses/designs. Note if any soil cases within the entire range of properties are not being considered in all of the analyses, then the technical basis should be provided.

In addition, Westinghouse is requested to clarify the following specific items related to the information presented in Tables 3.G.1-1 and 3.G.1-2 in the RAI response:

a. The 3D finite element analysis model [NI20], listed on page 1 of 7 of the RAI response, indicates that it was used in a response spectrum analysis with seismic input enveloping all soil cases for overturning and stability evaluation. Explain an apparent inconsistency with the analyses in the proposed revision to DCD Table 3G.1-1 in the RAI response.

b. The revised DCD Table 3G.1-2, fifth row, indicates that the "Equivalent static analysis using nodal accelerations from shell model" was used in the "3D finite element model of the nuclear island basemat (NI05)." Explain to what "shell model" this refers and where is it described, and indicate to which specific model these acceleration values are applied in Table 3G.1-1 because no NI basemat NI05 model could be identified. This should be clarified within this Table 3G.1-2. Also, are these the same acceleration values identified in Table 2.6-2(a) in RAI-TR85-SEB1-22? Explain how Westinghouse can derive a single acceleration value at each elevation if it came from a 3D "shell model" that contains many nodes over a range of elevations?

Westinghouse Response:

DCD Tables 3.7.2-14 and 3.7.2-16 in Revision 15 were moved to Appendix 3G and renumbered to Tables 3G.1-1 and 3G.1-2. These Tables were included in TR03, Rev 1 and in TR134. The tables have been edited as shown in the DCD Revisions below to show additional information requested in this RAI as well as revisions due to changes in methodology described in other RAI responses.

Portions of these tables related to the basemat design analyses, soil bearing reactions, and stability evaluation are shown below including reference to the location in this technical report for the detailed description.

3D finite element refined shell model of nuclear island (NI05)	Equivalent static non-linear analysis using accelerations from time history analyses;	ANSYS	To obtain SSE member forces for the nuclear island basemat. <u>See section 2.6 as modified by response to RAI-TR85-SEB1-21</u>
3D finite element coarse shell model of auxiliary and shield building and containment internal structures [NI20] (including steel containment vessel, polar crane, RCL, and pressurizer)	Response spectrum analysis with seismic input enveloping all soils cases	ANSYS	To obtain total basemat reactions for overturning and stability evaluation. To obtain total basemat reactions for comparison to reactions in equivalent static analyses using NI05 model. <u>See section 2.6.1.2 as modified by response to RAI-TR85-SEB1-07 and 22</u>
Finite element lumped mass stick model of nuclear island	Time history analysis	ANSYS	Performed 2D linear and non-linear seismic analyses to evaluate effect of lift off on Floor Response Spectra and bearing. <u>See section 2.4.2</u>

Westinghouse Response (Revision 1):

(1) Westinghouse will base its 35 ksf limit on the SASSI 3D results given in RAI-TR85-SEB1-3. The ANSYS 2D analyses will be used to support that the 35 ksf limit is a reasonable value. The bearing pressures have been obtained from the 3D SASSI analyses. The maximum bearing pressures obtained from the various soil cases are listed in Table RAI-TR85-SEB1-03-1. See RAI-TR85-SEB1-05, Rev.1, for a discussion of the 80 kcf subgrade modulus.

(2) The stability evaluations (for sliding and overturning) are based on a 3D NI20 model time history analysis (linear no lift-off), and not a response spectrum modal analysis. This is consistent with Table 3G1-1 given in DCD Appendix 3G, Revision 17 (see also response to RAI-TR85-SEB1-34). The model used is shown in Figure RAI-TR85-SEB1-04-1. As noted in Section 2.4.2 to Technical Report 85, "Comparison of floor response spectra and the maximum member forces and moments for these two cases show that the liftoff has insignificant effect on the SSE response." Therefore, it has been concluded that liftoff will have insignificant effect on the forces and moments that are being used for seismic stability evaluation.

In an effort to reduce the reliance on passive pressure to resist sliding, Westinghouse is no longer using a time history that envelopes all of the soil cases since it is too conservative. The seismic analysis was performed using the time history inputs as defined in DCD subsection 3.7.1.2, Revision 17. The analysis considered the hard rock case. All the base nodes were constrained to a single node, which in turn was fixed. This allowed the total Nuclear Island reaction forces to be taken from a single node location (node 1153). Node 1153 was selected as the central location, because it is located under the Center of Gravity (CG) for the NI structure. Shown in Figure RAI-TR85-SEB1-04-2 are the elements in the basemat at elevation 60'-6". Key nodes at the Elevation 60' 6" for the basemat of the NI20 model are shown for reference. Node 1153 is centrally located, and all the nodes in the basemat at this elevation are rigidly fixed to this node for the analysis.

The shear and vertical loads obtained from the 2D SASSI analyses given in the response to RAI-TR85-SEB1-07 were used to adjust the hard rock (HR) forces and moments to reflect the change in seismic response due to the other soil cases. These loads are given in Table RAI-TR85-SEB1-04-1. As seen from this table the upper bound soft to medium (UBSM) and soft to medium (SM) soil cases along with the hard rock case are the controlling generic soil cases. Therefore, it is not necessary to consider the other soil cases. The hard rock time history analysis base reactions are adjusted using the factors shown in Table RAI-TR85-SEB1-04-2. In order to confirm the adequacy of the loads being used for the stability evaluations, a comparison is made of the shear and vertical reactions for the UBSM soil case using 3D SASSI results. This comparison is given in Table RAI-TR85-SEB1-04-3. The UBSM case is used since it has the largest shear loads. As seen from this comparison the 3D SASSI results are lower.

(3) See Table 3G.1-1, DCD Revision 17, along with response given in RAI-TR85-SEB1-03.

(4) See Table 3G.1-1, DCD Revision 17, along with response given in RAI-TR85-SEB1-03.

The technical report is being revised to reflect the models and analyses being used. See RAI-TR85-SEB1-03 and RAI-TR85-SEB1-05.

a. As noted in item (2) above, time history analyses are used and not seismic response spectrum analyses for the overturning and stability evaluation. This is consistent with Table 3G1-1 given in DCD Appendix 3G, Revision 17.

b. In Table 3G.1-2, DCD Revision 17, the NI05 model is identified in the third and fourth rows. The basemat is modeled in the NI05 model. In Table 3G.1-1, DCD Revision 17. NI05 is identified on sheet 1, 5th row, and on sheet 3 the 7th row.

   Single acceleration values at each elevation are an average of the accelerations of each node at an elevation from the 3D shell model. This is acceptable since this will result in representative load acting on the basemat.

Table RAI-TR85-SEB1-04-1 - 2D Shears and Vertical Loads
Units: 1000 kips

Seismic Reaction	2D SASSI Hard Rock	2D SASSI Firm Rock	2D SASSI Soft Rock	2D SASSI Upper Bound Soft to Medium	2D SASSI Soft to Medium	2D SASSI Soft
Shear NS	123.75	116.49	118.65	121.48	113.61	73.11
Shear EW	112.31	113.55	121.88	128.11	124.94	74.34
Vertical	98.76	98.65	99.63	104.55	112.30	94.48

Table RAI-TR85-SEB1-04-2 – Factors to Apply to Hard Rock Analysis Base Reactions

Seismic Excitation	Upper Bound Soft to Medium	Soft to Medium
NS	0.98	0.92
EW	1.14	1.11
Vertical	1.06	1.14

Table RAI-TR85-SEB1-04-3 – Shear and Vertical Load Comparisons
Units: 1000 kips

Seismic Reactions at Base	3D ANSYS UBSM	3D SASSI UBSM
Shear NS	91.7	73.7
Shear EW	108.4	95.9
Vertical	111.3	83.9

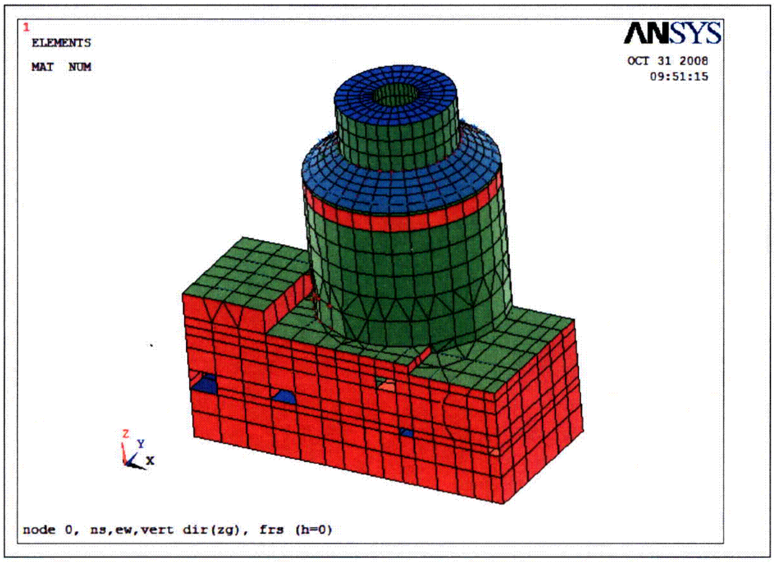


Figure RAI-TR85-SEB1-04-1 – ANSYS NI20 Model

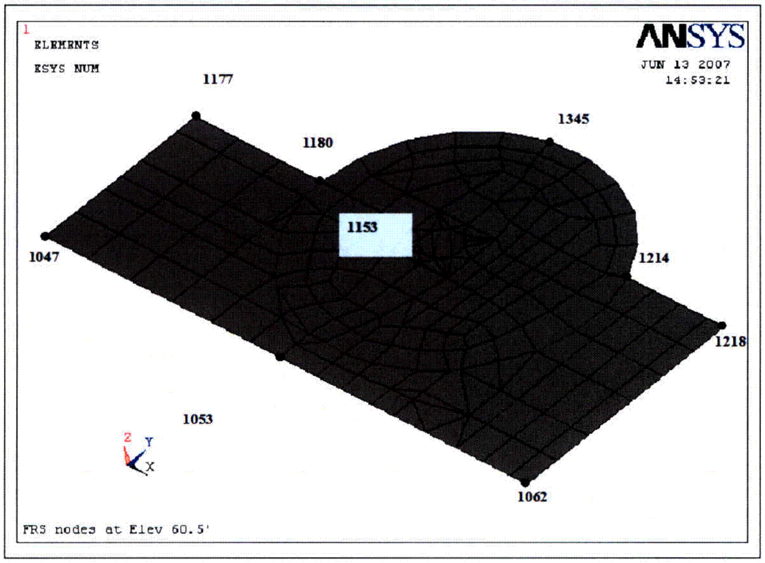


Figure RAI-TR85-SEB1-04-2 – Basemat Elements at Elevation 60'6"

Design Control Document (DCD) Revision:

The revisions to the DCD identified in Revision 0 of this response have been incorporated in DCD Revision 17.

Table 3G.1-1, Sheet 2 and Sheet 3, are modified Post Revision 17 as shown below.

Table 3G.1-1 (Sheet 2 of 4)

SUMMARY OF MODELS AND ANALYSIS METHODS

Model	Analysis Method	Program	Type of Dynamic Response/Purpose
Finite element lumped-mass stick model of nuclear island	Time history analysis	SASSI	Performed 2D parametric soil studies to help establish the bounding generic soil conditions~~.~~, and to develop adjustment factors to reflect all generic site conditions for seismic stability evaluation.
Finite element lumped-mass stick model of nuclear island	Direct integration time history analysis	ANSYS	Performed 2D linear and non-linear seismic analyses to evaluate effect of lift off on Floor Response Spectra and bearing.
3D finite element coarse shell model of auxiliary and shield building and containment internal structures [NI20] (including steel containment vessel, polar crane, RCL, and pressurizer)	Time history analysis	SASSI	Performed for the five soil profiles of firm rock, soft rock, upper bound soft-to-medium soil, soft-to-medium soil, and soft soil. To develop time histories for generating plant design floor response spectra for nuclear island structures. To obtain maximum absolute nodal accelerations (ZPA) to be used in equivalent static analyses. To obtain maximum displacements relative to basemat. To obtain SSE bearing pressures for all generic soil cases. To obtain maximum member forces and moments in selected elements for comparison to equivalent static results.

3D shell model of auxiliary and shield building and containment internal structures [NI20] (including steel containment vessel)	Mode superposition time history analysis	ANSYS	Performed to develop loads for seismic stability evaluation.
3D shell of revolution model of steel containment vessel	Modal analysis; equivalent static analysis using accelerations from time history analyses	ANSYS	To obtain dynamic properties. To obtain SSE stresses for the containment vessel.
3D lumped-mass stick model of the SCV	-	ANSYS	Used in the NI10 and NI20 models.

Table 3G.1-1 (Sheet 4 of 4)

SUMMARY OF MODELS AND ANALYSIS METHODS

Model	Analysis Method	Program	Type of Dynamic Response/Purpose
3D finite element coarse shell model of auxiliary and shield building and containment internal structures [NI20] (including steel containment vessel, polar crane, RCL, and pressurizer)	Mode superposition time history analysis with seismic input enveloping all soil cases	ANSYS	~~To obtain total basemat reactions for overturning and stability evaluation.~~ To obtain total basemat reactions for comparison to reactions in equivalent static linear analyses using NI05 model.

PRA Revision:

None

Technical Report (TR) Revision:

~~None~~ The following paragraph is added at the beginning of Section 2.9. See also Modifications made to Section 2.9 in RAI-TR85-SEB1-10.

The 2D SASSI reactions (NS and EW shear, and vertical) are used to obtain seismic response factors between the hard rock case to the upper-bound-soft-to-medium (UBSM) soil case, and the soft-to-medium (SM) soil case. These factors are used to adjust the hard rock (fixed base) NI20 ANSYS seismic time history analysis base reactions to reflect the seismic response for the other two potential governing soil cases UBSM and SM. The shear and vertical loads obtained from the 2D SASSI analyses given in Table 2.4-2 are used to adjust the hard rock (HR) reaction forces and moments obtained from the time history ANSYS analysis to reflect the change in seismic response due to the other soil cases. As seen from this table the upper bound soft to medium (UBSM) and soft to medium (SM) soil cases along with the hard rock case are the controlling generic soil cases. Therefore, it is not necessary to consider the other soil cases. The hard rock time history analysis base reactions are adjusted using the NS, EW, and vertical factors shown in Table 2.9-2.

Passive soil resistance is not considered for overturning seismic stability evaluation. For sliding, the amount of passive soil resistance, if required, is calculated to obtain the minimum factor of safety of 1.1. The deflection necessary to obtain the required passive pressure is then determined to show that it is reasonable (e.g., less than 2").

Table 2.9-2 – Factors to Apply to Hard Rock Analysis Base Reactions

Seismic Excitation	Upper Bound Soft to Medium	Soft to Medium
NS	0.98	0.92
EW	1.14	1.11
Vertical	1.06	1.14

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR85-SEB1-05
Revision: 1~~0~~

Question:

In Section 2.4.1, the first paragraph (Page 10 of 83) states that the 2D SASSI linear elastic analyses were performed for a variety of soil conditions as described in Section 4.4.1.2 of Westinghouse Technical Report TR-03, Revision 0. Six soil cases with shear wave velocity profiles shown in Figure 2.4-1 were analyzed. These soil cases range from firm rock to soft-to-medium soil. According to Table 2.6-1, the subgrade moduli for AP1000 soil cases range from 3,230 kcf for soft rock down to 312 kcf for soft soil. However, it is not clear from the technical report what modulus values are used for the firm rock or hard rock case for the current AP1000 analyses. For the 2D ANSYS nonlinear analyses, only the hard rock and the soft-to-medium soil cases were considered. For the 3D ANSYS analysis only the soft-to-medium soil case was considered. Section 2.6.1.1 indicates that although the subgrade modulus calculated for the AP1000 soil cases could have justified a subgrade modulus of 1,000 kcf for dry soft-to-medium soil, it was decided to retain the 520 kcf used in the AP600 analyses. This section of the technical report also indicates that this is conservative since it maximizes the bending moments in the slabs. Based on the above, the following information is requested relating to the soil moduli to be used for the various analyses:

a) Provide a complete set of soil subgrade modulus values used for the AP1000 rock and soil cases. Currently the only definition of soil modulus values are presented in Table 2.6; however, it lacks the modulus values for firm rock and hard rock.

b) Section 2.4.1 indicates that six soil cases with shear wave velocity profiles shown in Figure 2.4-1 were analyzed. However, Figure 2.4-1 only shows five soil cases. Furthermore, Section 4.4.1.2 of TR-03, Revision 0, indicates that four design soil profiles were used, while Table 4.4.1-1B of that report shows six soil cases. Explain all of these differences.

c) The staff notes that 520 kcf is generally considered to be appropriate for stiff soils. At the Savannah River Site, a deep soil site, subgrade moduli of the order of 40 kcf are used to evaluate foundations of buildings of similar dimension and contact pressure. Was such a subgrade modulus also used for the design of the AP1000 basemat when located at soil sites; if not, then explain why?

d) From the limited information provided in the technical report, it is not clearly evident that the two soil cases for the 2D ANSYS nonlinear analyses and the one soft-to-medium soil case for the 3D ANSYS analysis adequately envelope the entire range of rock and soil properties. Provide technical basis for the very limited cases considered or extend the analyses to other rock/soil cases.

Additional Request (Revision 1)

Westinghouse is requested to address the following items:

Item a: Clarify if the reference to Table RAI-TR85-SEB1-01-1 should be RAI-TR85-SEB05-1.

Item c: (1) At the Savannah River Site, a deep soil site with average shear wave velocity in the upper several hundred feet of well over 1,000 fps, empirical data based on measured settlement responses from facilities of similar plan area as AP1000 having dead weight pressures of similar magnitude indicate subgrade moduli of about 40 kcf as being appropriate for these static dead loads. Considering that corresponding moduli appropriate for the dynamic loading case are typically of the order of two times the static, the appropriate subgrade modulus for the soil case should be about 80 kcf, not 520 kcf as used in the design reported in TR85. Furthermore, the staff cannot identify the technical basis for the statement made in the RAI response that the "studies showed that the design of the basemat using soil springs with a subgrade modulus of 520 kcf would bound other soil profiles." Therefore, Westinghouse is requested to explain the above statement and why the subgrade modulus as low as 80 kcf has not been considered. In addressing the impact of the use of a subgrade modulus as low as 80 kcf, Westinghouse should consider the impact of this reduced modulus on all aspects of the basemat evaluation - bearing pressure calculations, stability evaluations, and design of the basemat itself (i.e., reinforcement).

(2) The RAI response states that "it was found that local effects of the soil directly below the basemat were significant. This is not included in the subgrade model." Explain the meaning of this statement and the acceptability of not including the local effects of the soil directly below the basemat.

Item d: (1) It appears that the 2D ANSYS stick model has been updated and also the number of soil cases considered have been expanded to consider the following cases: hard rock (no numerical value for subgrade modulus given), UBSM (1,340 kcf), SM (1,000 kcf), and SM (780 kcf). The response to RAI-TR85-SEB1-22 (Table 2.6-1) indicates that the subgrade modulus values used in the 2D ANSYS stick model analyses consist of 6 soil cases (including hard rock) with subgrade modulus values different from those stated in the response to this RAI-TR85-SEB1-05. Westinghouse is requested to explain the inconsistencies.

Westinghouse Response:

a) The following has been revised in the Revision 1 response to address the additional request and also to address updates in the AP1000 analyses.

Subgrade modulus is used in the following analyses:

- Subgrade moduli of 6300~~267~~, ~~3200, 1000, and 300~~2800, 1700, 1500, 900 and 300 kcf were used for hard rock, firm rock, soft rock, upper bound soft to medium soil, soft to medium soil and soft soil sites in the 2D ANSYS parametric linear dynamic analyses described in Section 2.4.2 of the report. The results of the analyses for soft rock and soft soil were not used.

- Subgrade moduli of 6300~~267~~ kcf, 1500 kcf and 900~~1000~~ kcf were used for the hard rock, upper bound soft to medium soil, and soft to medium soil sites in the 2D ANSYS non-linear dynamic analyses described in Section 2.4.2 of the report.
- A subgrade modulus of 6267 kcf was used for hard rock in the 3D ANSYS Equivalent Static Non-Linear Analysis for design of the basemat as described in Section 2.3.1 of the report
- A subgrade modulus of 520 kcf was used for soil sites in the 3D ANSYS Equivalent Static Non-Linear Analysis for design of the basemat as described in Section 2.6.1 of the report.
- A subgrade modulus of 260 kcf was used in the 3D ANSYS Equivalent Static Non-Linear Parametric Analysis for evaluation of the effect of a lower subgrade modulus as described in Section 2.7.1.1 of the report.

Table RAI-TR85-SEB1-05~~1~~-1 shows the subgrade modulus used in the 2D ANSYS analyses for the AP1000 hard rock and soil cases. The hard rock value was calculated for a uniform half space using the formula given in ASCE-4 (Reference 1). The soft rock, upper bound soft to medium, soft to medium and soft soil cases were calculated using the Steinbrenner formula for the degraded soil profiles used in the AP1000 seismic analyses. These profiles assume 80’ 6” of soil below the nuclear island basemat and assume fixed base (very hard rock) at a depth of 120 feet below grade. The properties for each layer in the soil profile are shown in DCD Rev 17~~6~~ Table 3.7.1-4. The values shown in the middle column of Table RAI-TR85-SEB1-05~~1~~-1 are those reported in TR85 Rev 0. Subgrade modulus was not calculated for the firm rock site since no analyses were performed requiring the subgrade modulus at a firm rock site.

Subsequent to issue of TR85, Rev 0, analyses were performed on an ANSYS 2D plane strain model of the soft to medium soil profile for comparison against the Steinbrenner formula as described in Reference 2. The comparison to the values quoted in TR85, Rev 0 was not very good. It was found that the assumption made in the calculation that the center deflection was twice the corner deflection was not supported by the ANSYS results. This assumption is suggested in the literature and is appropriate for deeper soils. The ANSYS analyses and additional calculations at the center using the Steinbrenner formula showed the assumption is not appropriate for the case of the nuclear island footprint on a soil depth of 80’ 6”. The center deflection for such a shallow case is up to 4 times the corner deflections.

The Steinbrenner calculation was revised to calculate the center deflection directly (the common corner of four quarter rectangular mats) as recommended in Reference 2. The average deflection of the mat was then taken as 0.80 times the center deflection based on comparisons to the ANSYS results. The revised average stiffness for each soil profile is shown in the right hand column of the table. The subgrade modulus of 1000 kcf used in the non-linear lift off analyses on soil reported in TR85, Rev 0 is stiffer than the ~~780~~ 867 kcf recalculated for the design soft to medium soil profile (with water table to grade) and less than the ~~1340~~ 1509 kcf for the design upper bound soft to medium soil profile (with water table to grade). The revised values are being used in the updated ~~confirmatory~~ analyses as described in ~~the response to part (d) below~~the proposed revision to TR85 included in this response.

b) As described in Section 2.4.1, six soil cases with shear wave velocity profiles shown in Figure 2.4-3~~1~~ were analyzed in 2D SASSI. Figure 2.4-3~~1~~ shows the five soil cases with shear wave velocity up to 3500 fps. The hard rock is shown in the footnote with shear wave velocity of 8000 fps. This was done to show the differences in the lower shear wave velocity cases more clearly.

   Westinghouse has expanded the number of soil cases it evaluates in its 3D SASSI generic analyses so that no justification is required using AP1000 2D SASSI sensitivity cases. These six generic cases have been identified for convenience as hard rock (HR), firm rock (FR), soft rock (SR), upper bound soft to medium (UBSM), soft to medium (SM), and soft soil (SS). This is shown in ~~the proposed revisions to~~ DCD, Rev 17~~6~~, Appendix 3G~~ as described in TR03, Rev 1 and TR-134, Rev 0~~.

c) A subgrade modulus of the order of 40 kcf was not used for the design of the AP1000 basemat. Studies of the effect of various soil conditions are described in Section 2.7 of the report. Subsection 2.7.1.1 describes the effect of reducing the subgrade modulus from 520 kcf to 260 kcf. Subsection 2.7.1.2 describes 3D analyses with finite element models of the soil. Subsection 2.7.2 describes 2D analyses with finite element soil models. Based on these studies, it was found that local effects of the soil directly below the basemat were significant. This is not included in a subgrade modulus model. The studies showed that the design of the basemat using soil springs with a subgrade modulus of 520 kcf would bound other soil profiles.

d) The design of the nuclear island basemat used results from two analyses (hard rock, soft to medium soil) to size the required reinforcement. Parametric studies described in Section 2.7 of the report investigate a wide range of soil parameters and justify the adequacy of the two cases used in the design analyses.

   The 2D ANSYS nonlinear analyses analyzed ~~two~~ three cases (hard rock, upper bound soft to medium soil, soft to medium soil) to evaluate the effect of lift-off and the maximum bearing pressure. These ~~two~~ cases were selected based on linear analyses that also included the firm rock, soft rock and soft soil profiles. These analyses are described in the proposed revision to TR 85.~~The analyses of the soft to medium soil case used a subgrade modulus of 1000 kcf which was subsequently determined to be too high for this soil condition. In addition the nuclear island seismic analyses show that the upper~~

~~bound soft to medium soil case is very close to that of the soft to medium soil case. The non-linear analyses are being supplemented by two additional cases:~~

- ~~Subgrade modulus of 780 kcf corresponding to the revised modulus for the soft to medium soil with water table to grade~~
- ~~Subgrade modulus of 1340 kcf corresponding to the revised modulus for the upper bound soft to medium soil with water table to grade~~

~~These confirmatory cases also include an update of the 2D stick model to be consistent with the various design changes incorporated in the latest design (e.g. the enhanced shield building and the lower pressurizer doghouse). Results of these confirmatory analyses will be available for audit in April, 2008.~~

Westinghouse Response (Revision 1):

a) The reference to Table RAI-TR85-SEB1-01-1 has been corrected to read RAI-TR85-SEB05-1.

c) (1) For a deep soil site such as at the Savannah River Site, a subgrade modulus of about 40 kcf would be appropriate for determining settlement under these static dead loads. This subgrade modulus is applicable for a uniformly loaded foundation of similar size to the AP1000. Considering that corresponding moduli appropriate for the dynamic loading case are typically of the order of two times the static, the appropriate subgrade modulus elastic settlement for dynamic analyses should be about 80 kcf. This is not the analysis for which the subgrade modulus of 520 kcf is being used. The 520 kcf is being used in a static analysis to estimate member forces in the basemat under combined dead and SSE loads. These member forces are in portions of the 6 foot thick basemat that span about 20 feet between shear walls and are a function of the subgrade modulus appropriate for vertical loads on the walls being distributed into the 6 foot slab spanning 20 feet. The Steinbrenner formula for subgrade modulus results in a subgrade modulus per unit area that is inversely proportional to the span. Hence for a span of 20 feet the modulus would be about 10 times higher than that for a 200 foot footprint on a deep soil site. The soil behavior under the basemat has been investigated with finite element soil models as described in Sections 2.7.1 (3D models) and 2.7.2 (2D models) of the TR85 report. These studies confirmed that bearing pressure distribution was much influenced by the soil elements directly below the basemat (the deep portion of the soil only affects total deflections and has little influence on the local deflections which are more closely related to member forces).

The statement made in the RAI response that the "studies showed that the design of the basemat using soil springs with a subgrade modulus of 520 kcf would bound other soil profiles" is based on the studies described in Sections 2.7.1 and 2.7.2. A subgrade modulus as low as 80 kcf has not been considered because such a value, as discussed in the previous paragraph, is not applicable for the member forces in the basemat. The lower value would be applicable to evaluation of overall settlement by a subgrade modulus approach; overall settlement on a very soft deep site is addressed separately in Section 2.5 of the report. Bearing pressures and stability are addressed separately using the results of the SASSI analyses.

(2) The local effects of the soil directly below the basemat were found to be significant in the analyses described in Section 2.7 using finite element models of the soil. These studies showed reduction of basemat member forces due to the local effect of the soil below the basemat and concluded that the design analyses using a subgrade modulus of 520 kcf were conservative and concluded the acceptability of not including the local effects of the soil directly below the basemat.

d) (1) The 2D ANSYS stick model has been updated and the number of soil cases considered has been expanded to consider the following cases: hard rock (6300 kcf), FR (2800 kcf), SR (1700 kcf), UBSM (1,500 kcf), SM (900 kcf), and SS (300 kcf). These analyses are described in the proposed revision to TR 85. The values in this proposed revision supersede those shown in the response to RAI-TR85-SEB1-22 (Table 2.6-1).

Table RAI-TR85-SEB1-05-1

Subgrade modulus for AP1000 Soil Cases

Soil case	Subgrade modulus (TR85, Rev 0)	Revised subgrade modulus (TR85, Rev 1)
	kcf	kcf
Hard rock	6267	6267
Firm rock		~~3760~~2833
Soft rock	3230	1661~~30~~
Upper bound soft to medium soil (water table to grade)		~~1340~~1509
Upper bound soft to medium soil (dry)	2334	~~1320~~1508
Soft to medium soil (water table to grade)	1280	~~780~~867
Soft to medium soil (dry)	963	~~580~~670
Soft soil (water table to grade)		276
Soft soil (dry)	312	170

Reference:

1. ASCE 4-98, Seismic Analysis of Safety Related Nuclear Structures
2. Bowles, "Foundation Aanalysis and Design" Fifth Edition

Design Control Document (DCD) Revision:

The ~~Revise Table 3.7.1.4 to Table 3.7.1-4 on four sheets. On sheet 4 revise the column headings to be the same as those on sheets 1 to 3.~~
DCD revisions described in Revision 0 of this response have been incorporated in DCD Rev 17.

PRA Revision:

None

Technical Report (TR) Revision:

~~Revisions to Section 2.6.1 and Table 2.6-2 are shown in the response to RAI-TR85-SEB1-22.~~
Revisions to Section 2.4 and 2.6 ~~will be identified once the confirmatory analyses have been completed~~are identified on the following pages.

2.4.1 2D SASSI analyses

Parametric 2D SASSI linear elastic analyses were performed for a variety of soil conditions as described in Section 4.4.1.2 of Reference 3. The SASSI model in the east west direction is shown in Figures 2.4-1 and 2.4-2. These analyses used AP1000 building models comprising 3 sticks (ASB, CIS and SCV). Six soil cases with shear wave velocity profiles shown in Figure 2.4-3 were analyzed in each direction. Bedrock with shear wave velocity of 8000 fps was assumed at a depth of 120' below grade. Thus the depth of soil below the foundation mat is 80.5'. The building models used in the parametric analyses were updated to include changes to the nuclear island such as the change to the enhanced shield building. The properties of the ASB and CIS in the NI combined stick model are developed to match the properties of the nuclear island shell models.

Bending moments in the building sticks for the six AP1000 cases are shown in Figure 4.4.1-5 of Reference 3. The ASB and CIS sticks are coupled below grade. The bending moments in the ASB stick above grade are shown in Table 2.4-1 from the analyses of the updated model. These bending moments provide a direct measure of the effect of soils on the total overturning moment. These overturning moments lead to the maximum bearing pressures which control design of the basemat and the demand on the soil.

Table 2.4-2 shows the reactions at the underside of the basemat for each soil case. These are conservative estimates using the results of the 2D SASSI analyses also used for the member forces in Table 2.4-1. Loads on the portion below grade are added absolutely to the sum of the member forces above grade. The 2D SASSI reactions (Fx, Fy, and Fz) are used to obtain seismic response factors between the hard rock case to the upper-bound-soft-to-medium (UBSM) soil case, and the soft-to-medium (SM) soil case. These factors were used to adjust the hard rock base reaction time history to reflect the seismic response for the other two potential governing soil cases UBSM and SM.

The soft-to-medium soil case and the upper bound soft to medium soil case result in the largest bending moments in the ASB stick at grade for seismic input in the east west direction. The AP1000 footprint is shorter along the east west axis than along the north south axis. Softer sites typically have lower soil strength than the firmer sites. From review of the member forces in Table 2.4-1, and the bearing reactions in Table 2.4-2, the soft to medium soil case and the upper bound soft to medium soil case are selected as the basis for the bearing demand. The effect of lift off is investigated for these cases as described in the following section. The hard rock case was also analyzed since this case had been included in the hard rock design certification.

2.4.2 2D ANSYS non-linear dynamic analyses

The SASSI analyses described in section 2.4.1 are linear elastic analyses. They permit tension to be carried across the interface between the soil and the basemat. Dead and live load bearing pressures from the ANSYS analyses on soil springs are shown in Figure 2.6-3. The bearing pressures vary from about 6 ksf on the east side to 14 ksf below the edge of the shield building on the west side. The absolute value of some of the seismic bearing pressures calculated by SASSI exceed the dead load bearing pressures giving a resultant tension uplift. The effect of lift off was analyzed in ANSYS. Linear seismic analyses were performed on the ANSYS models to confirm similar behavior to the SASSI analyses. Non-linear analyses were then performed for dead plus seismic loads with compression only contact elements.

Lift off was evaluated using an East-West lumped-mass stick model of the nuclear island structures supported on a rigid basemat with nonlinear springs. The liftoff analysis model is shown in Figure 2.4-4 and consists of the following elements:

- The nuclear island (NI) combined stick model (ASB, CIS and SCV). The three sticks are concentric and the reactor coolant loop is included as mass only. This model is the same model as was used in the updated 2D SASSI analyses described in Section 2.4.1.
- The rock and soil were modeled as horizontal and vertical spring elements with viscous damping at each node of the rigid beam. The vertical soil spring at each node is the subgrade modulus shown in Table 2.4-3 multiplied by the area of the footprint associated with each node. The horizontal spring is calculated from that in the vertical direction assuming that the ratio of horizontal and vertical stiffness for the layered site has the same relationship as for a semi-infinite medium. Soil damping is included in the soil spring element and is calculated to give the percentage shown in Table 2.4-3 at the fundamental frequency of the building soil system.
- The rigid basemat model with a footprint area that varies along the East-West (Y) axis of the model matching the footprint of the nuclear island. The NI combined stick is attached to the rigid basemat at the NI gravity center, which is about 9 feet from the center of the rigid basemat. In the north-south direction, the stick is fixed at the bottom (EL. 60.5').

Direct integration time history analyses were performed. Time histories were applied at the underside of the foundation (elevation 60.5'). These time histories were foundation level inputs calculated from the AP1000 time histories at grade using a SHAKE analysis with the degraded properties shown in Figure 2.4-3(b). Structural damping was included as mass and stiffness proportional Rayleigh damping matching the modal damping of 7% at the fundamental frequency and at 25 Hertz. The first ANSYS analyses used Rayleigh damping matching 7%. Floor Response Spectra (FRS) and member force results were compared to those from 2D SASSI. The bending moment on the Auxiliary Shield Building (ASB) stick at grade is used as a measure of the overturning which is of greatest significance in the lift off analyses. The SASSI and ANSYS results showed the largest overturning for the UBSM and SM soil cases. For these soil cases the ANSYS results of both the FRS and member forces were lower than the SASSI results. The Rayleigh damping was reduced from 7% to 5 % for UBSM and SM so that the FRS and ASB bending moment at grade matched those from SASSI. Typical FRS are compared in Figure 2.4-5 for the soft to medium soil case. In the horizontal direction the FRS compare very well. In the vertical direction the ANSYS analyses show higher values than SASSI making the ANSYS analyses slightly more conservative; this is partially due to the Rayleigh damping which is selected to give appropriate damping for the horizontal frequency around 2.5 Hz and gives much lower damping at the fundamental vertical frequency of about 6 Hz. The ASB bending moments at grade are compared in Table 2.4-4. These show a good match between the ANSYS and SASSI models.

Time history analyses were run by direct integration for dead load plus the east west and vertical components of the safe shutdown earthquake for two cases:

- linear soil springs able to take both tension and compression. This case was run to compare against the linear results from the 2D SASSI analyses to confirm the soil springs and damping properties.
- non-linear soil springs where the vertical springs act in compression only and the horizontal springs are active when the vertical spring is closed and inactive when the vertical spring lifts off.

Comparison of floor response spectra for these two cases show that the liftoff has insignificant effect on the SSE floor response spectra. Thus, the superstructure may be designed neglecting liftoff. Only the basemat design need consider the effects of liftoff as described in Section 2.6.

Figure 2.4-6 shows the time history of the deflection and pressure at the west and east edge around the time that the peak pressure occurs at the west edge. The three sheets show results for hard rock (HR), upper bound soft to medium (UBSM) and soft to medium (SM). The linear results show maximum bearing pressures on the west side of 31 to 33 ksf. Lift off increases the subgrade pressure close to the west edge by 4 to 6% with insignificant effect beneath most of the basemat. The effect on the pressure at the west edge is significantly less than that calculated in the non-linear basemat analyses using equivalent static accelerations.

Table 2.4-1

Maximum member forces in ASB stick at elevation 99' from 2D SASSI analyses

Units: 1000 kips & 1000 ft-kip

	North-South model		**East-West model**	
Soil case	**North-South Shear**	**Moment about E-W axis**	**East-West Shear**	**Moment about N-S axis**
	F_X	M_{YY}	F_Y	M_{XX}
Hard Rock (HR)	52.85	6934	46.77	6085
Firm Rock (FR)	49.81	6837	48.05	6118
Soft Rock (SR)	50.54	6586	51.58	6554
Upper Bound Soft to Medium Soil (UB)	52.12	6416	55.24	7084
Soft to Medium Soil (SM)	53.24	6810	61.67	7621
Soft Soil (SS)	26.01	3683	28.08	4649

Table 2.4-2 – Maximum Seismic Reactions at Center Line of Containment

Units: 1000 kips & 1000 ft-kip

Seismic Reactions	HR	FR	SR	UBSM	SM	SS
Shear NS, F_X	123.75	116.49	118.65	121.48	113.61	73.11
Shear EW, F_Y	112.31	113.55	121.88	128.11	124.94	74.34
Vertical, F_Z	98.76	98.65	99.63	104.55	112.30	94.48
Moments Relative to Centerline of Containment						
M_{XX} EW Excitation	10,916	10,900	11,471	12,229	12,607	7,653
M_{XX} Vertical Excitation	1,660	1,693	1,715	2,017	1,913	1,459
M_{XX} SRSS	11,042	11,031	11,598	12,394	12,751	7,791
M_{YY} NS Excitation	12,184	11,659	11,390	11,274	11,173	6,300
M_{YY} Vertical Excitation	918	935	946	997	1,059	829
M_{YY} SRSS	12,218	11,697	11,429	11,318	11,223	6,354

Notes:

1. HR = Hard Rock, FR = Firm Rock, SR = Soft Rock, UBSM = Upper Bound Soft to Medium Soil, SM = Soft to Medium Soil, SS = Soft Soil.
2. Reactions for horizontal input are calculated from member forces at grade in 2D SASSI analyses plus maximum acceleration times mass below grade. Reactions due to vertical input are calculated from maximum accelerations in 3D ANSYS or SASSI analyses for HR, FR, UBSM and SM and from 2D ANSYS analyses for SR and SS.

Table 2.4-3

Soil Properties in ANSYS Model

	Assumption of Soil Conditions				
	Soil Material Property		ANSYS Soil Spring Property		
	Density pcf	Poisson's Ratio	Stiffness kcf		Damping %
			Vertical	East-West	
Hard Rock	150	0.250	6300	5477	2
Firm Rock	150	0.250	2800	2434	5
Soft Rock	150	0.250	1700	1478	5
Upper-Bound Soft-to-Medium Soil	110	0.35 / 0.383 (2)	1500	1187	5
Soft-to-medium Soil	110	0.35 / 0.450 (2)	900	666	5
Soft Soil	110	0.40 / 0.483 (2)	300	213	20

Notes:

1. Soil conditions are identified using the same notation as in Reference 3.
2. Poisson's ratio is shown for dry soils. The second value is the average value over the depth of the soil column accounting for ground water. This value is used in establishing horizontal springs.
3. Soil spring damping is applied as damping element to give specified damping at the first frequency.

Table 2.4-4

Comparison of member forces in ASB stick at elevation 99' from 2D SASSI and ANSYS analyses

Units: 1000 kips & 1000 ft-kip

	SASSI			ANSYS		
Soil case	**Axial**	**East-West Shear**	**Moment about N-S axis**	**Axial**	**East-West Shear**	**Moment about N-S axis**
	F_Z	F_Y	M_{XX}	F_Z	F_Y	M_{XX}
Hard Rock (HR)	47.72	46.77	6085	52.95	52.01	6330
Firm Rock (FR)	48.67	48.05	6118	54.78	53.84	6428
Soft Rock (SR)	49.48	51.58	6554	57.34	53.68	6592
Upper Bound Soft to Medium Soil (UB)	52.20	55.24	7084	61.14	60.18	7581
Soft to Medium Soil (SM)	54.78	61.67	7621	63.80	58.65	7311
Soft Soil (SS)	37.96	28.08	4649	52.39	32.63	4009

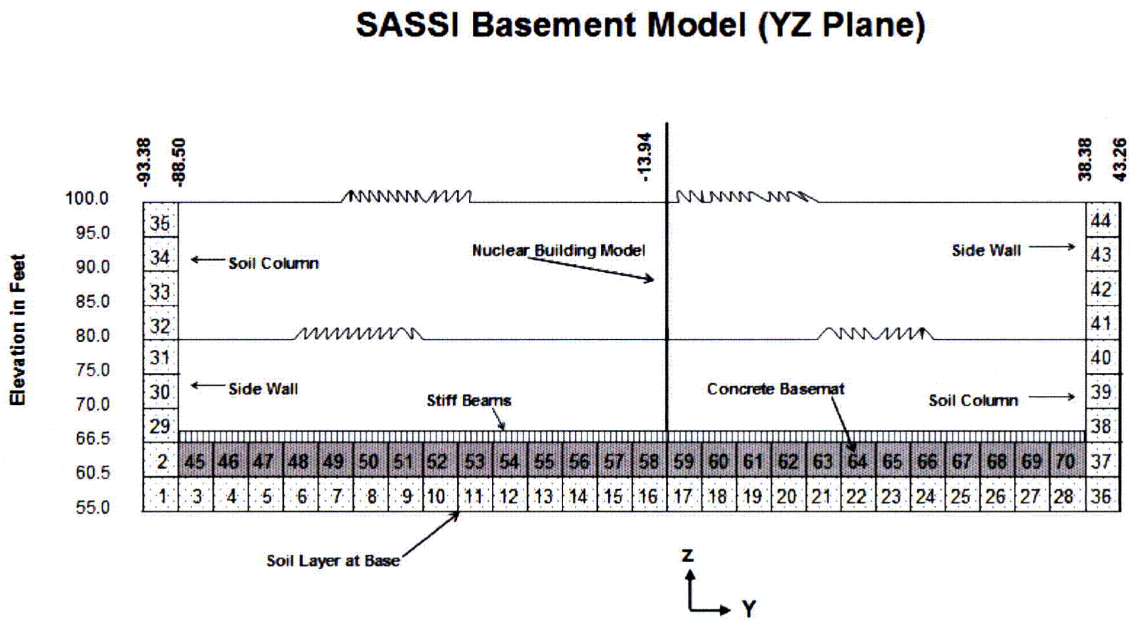


Figure 2.4-1: SASSI Basement Model (YZ Plane)

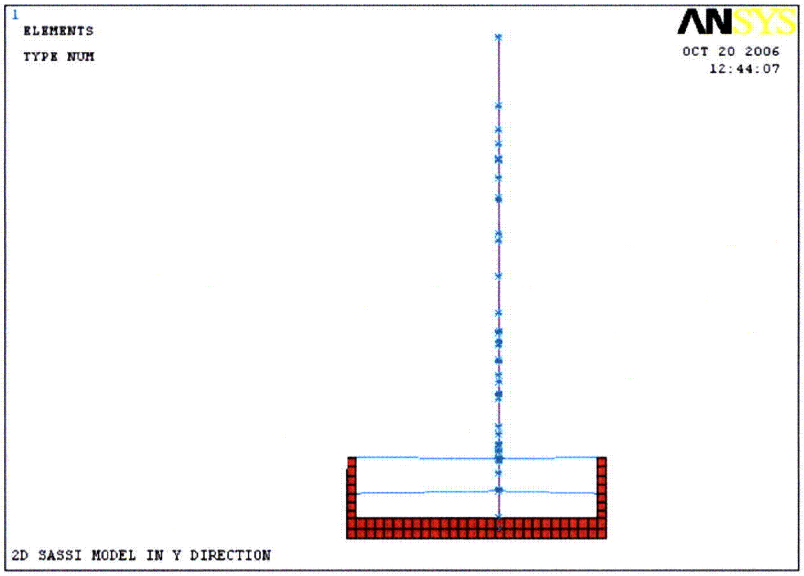
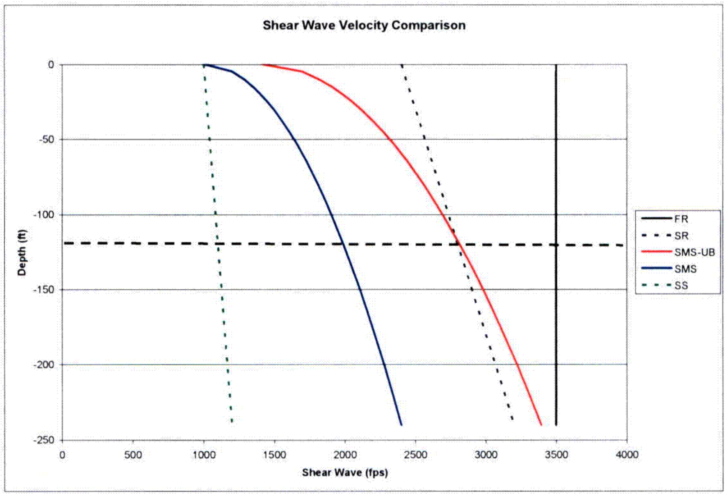


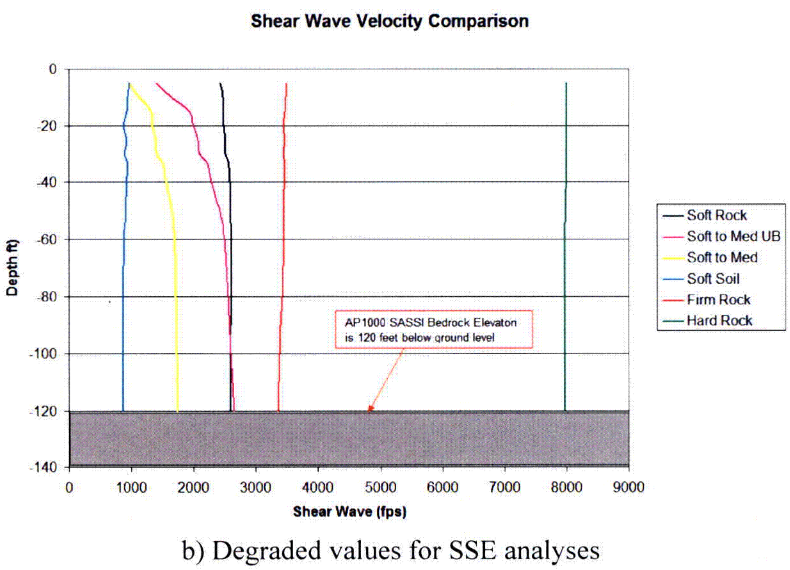
Figure 2.4-2: East-West 2D SASSI Model in Y Direction



a) Low strain values

Notes:

Fixed base analyses were performed for hard rock sites. These analyses are applicable for shear wave velocity greater than 8000 feet per second.
Design analyses have soil to depth of 120' with rock below having shear wave velocity of 8000 feet per second.



b) Degraded values for SSE analyses

Figure **2.4-3** Generic Soil Profiles

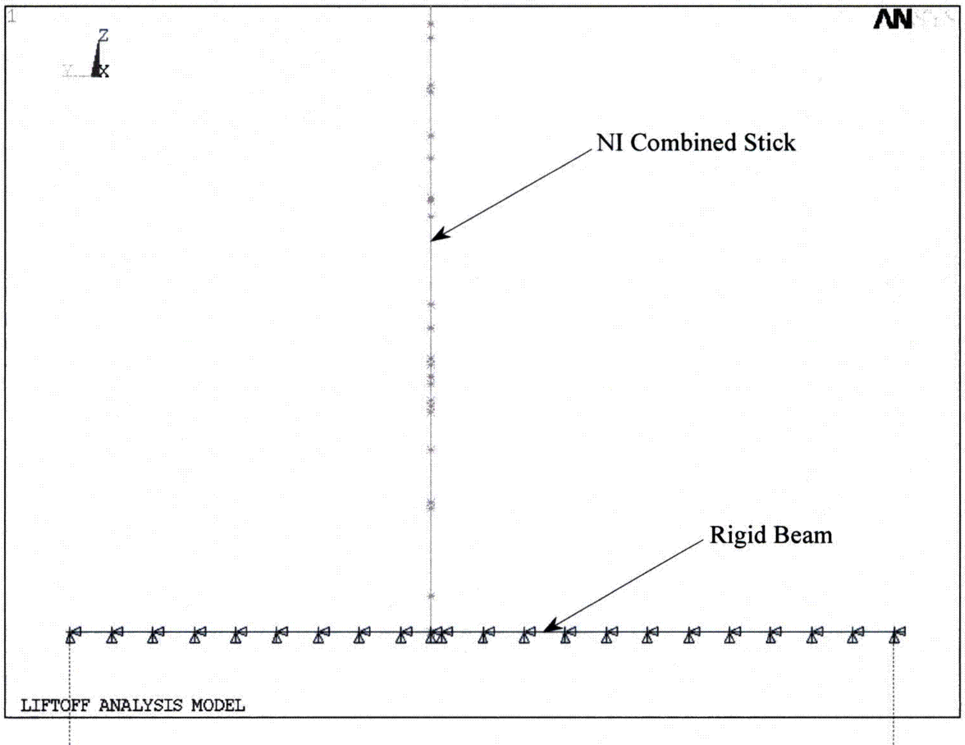


Figure 2.4-4 – ANSYS Lift Off Model

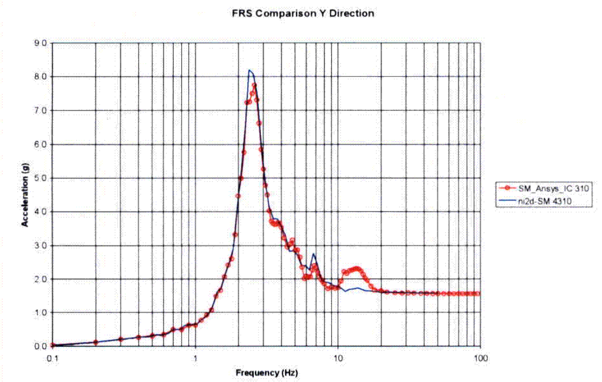
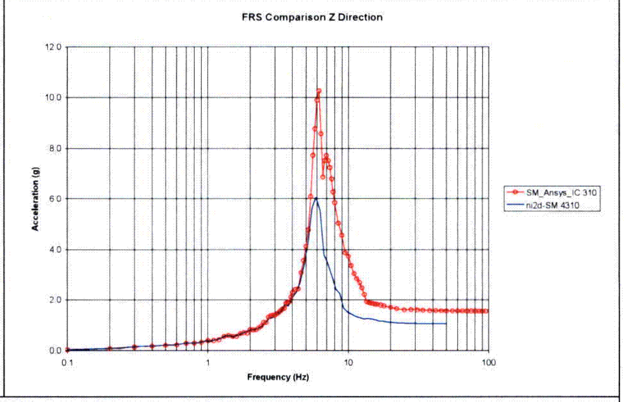
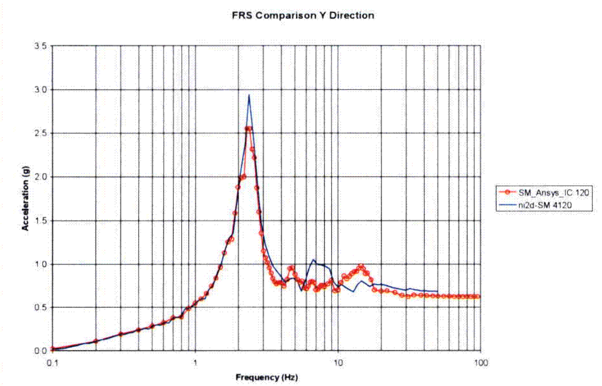
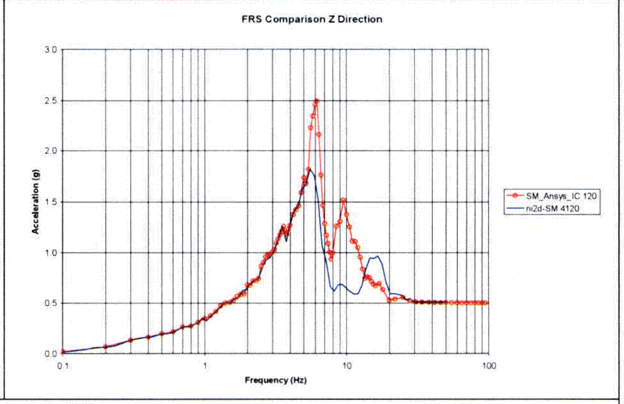
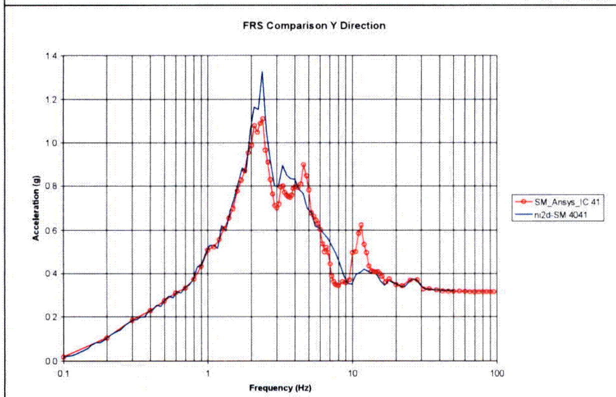
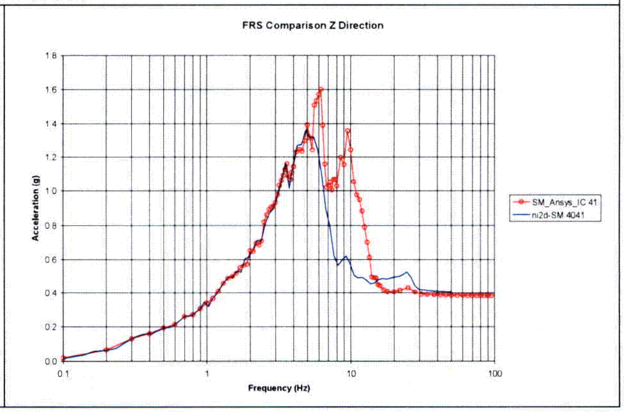
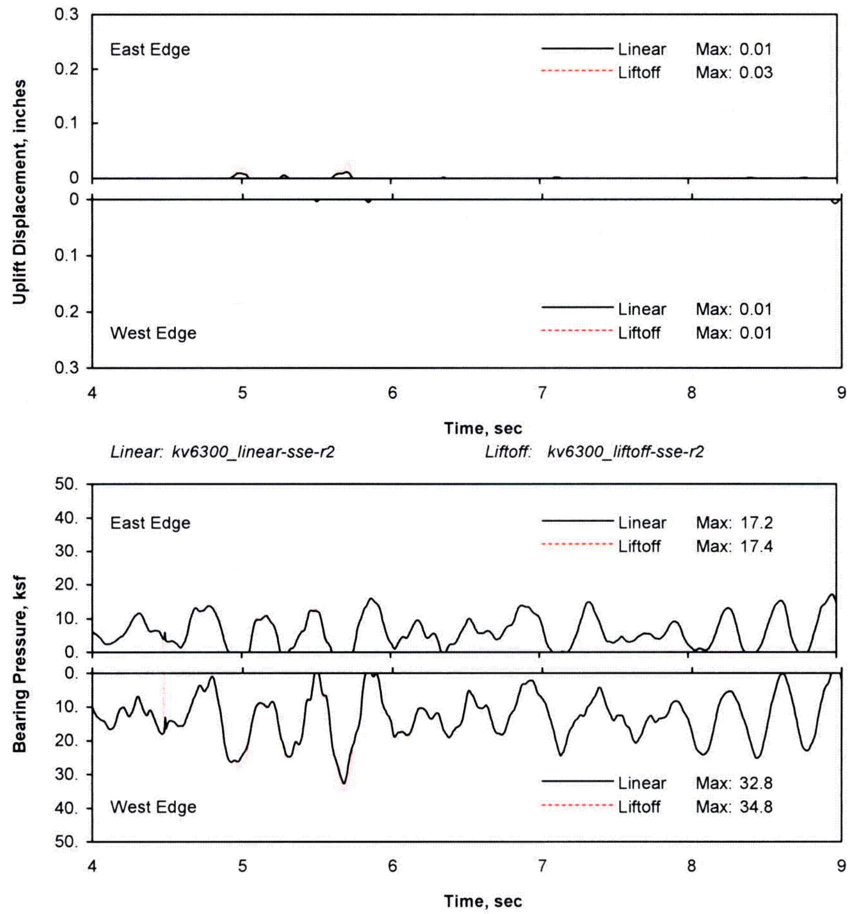
Horizontal	Vertical
ASB - Top of Shield Building – Elevation 327’	
	
ASB - Top of Fuel Building – Elevation 180’	
	
ASB Elevation 99’	
	

Figure 2.4-5 Comparison of SASSI and ANSYS FRS for Soft to Medium Soil

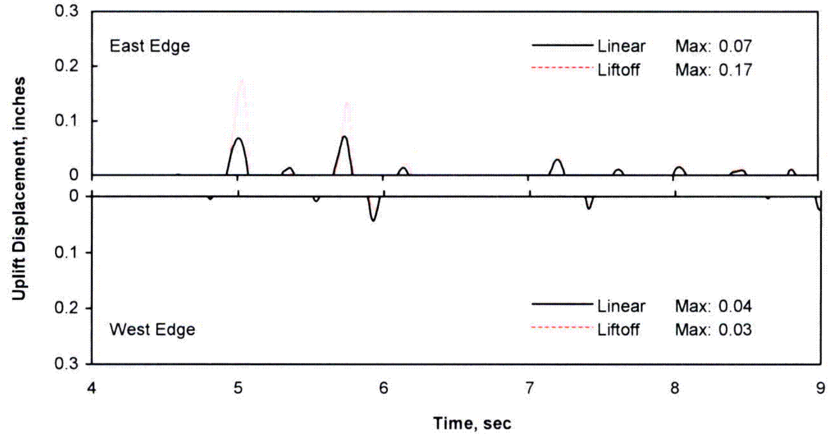


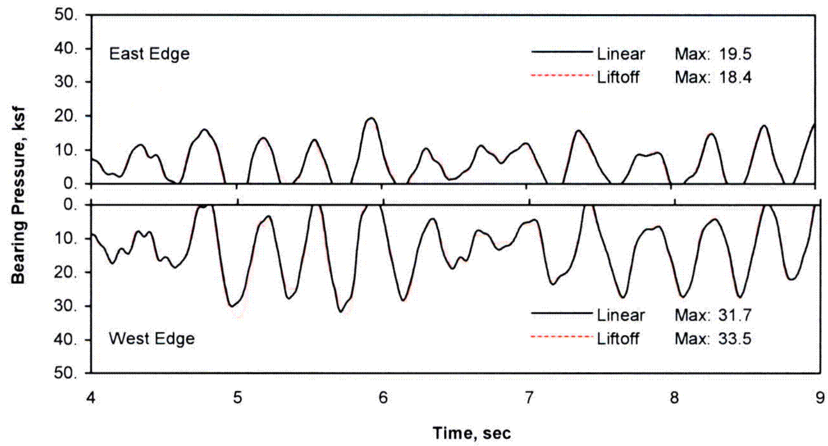
Maximum uplift Displacements					
East			West		
	inches	Time(s)		inches	Time(s)
Linear	0.01	5.71	Linear	0.01	8.94
Liftoff	0.03	5.72	Liftoff	0.01	8.96

Maximum Bearing Pressure					
East			West		
	ksf	Time(s)		ksf	Time(s)
Linear	17.18	8.97	Linear	32.77	5.68
Liftoff	17.38	8.97	Liftoff	34.85	5.68

Figure 2.4-6 – 2D ANSYS Time History of Basemat Edges

Sheet 1 of 3 –Hard Rock Case



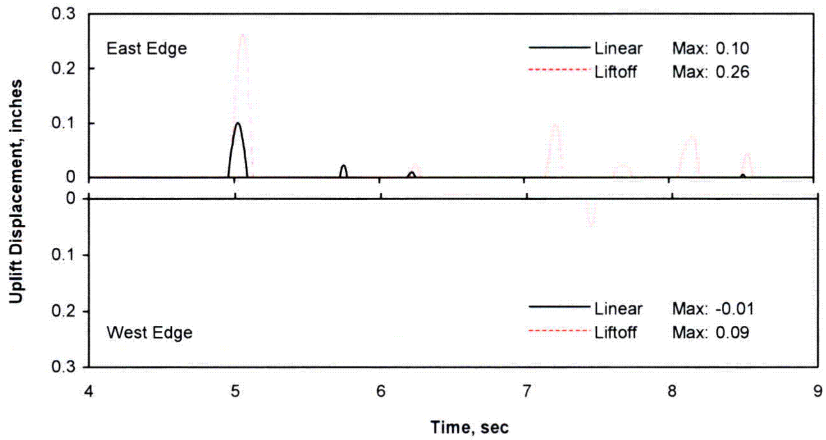


Maximum uplift Displacements					
East			West		
	inches	Time(s)		inches	Time(s)
Linear	0.07	5.74	Linear	0.04	5.92
Liftoff	0.17	5.03	Liftoff	0.03	9.00

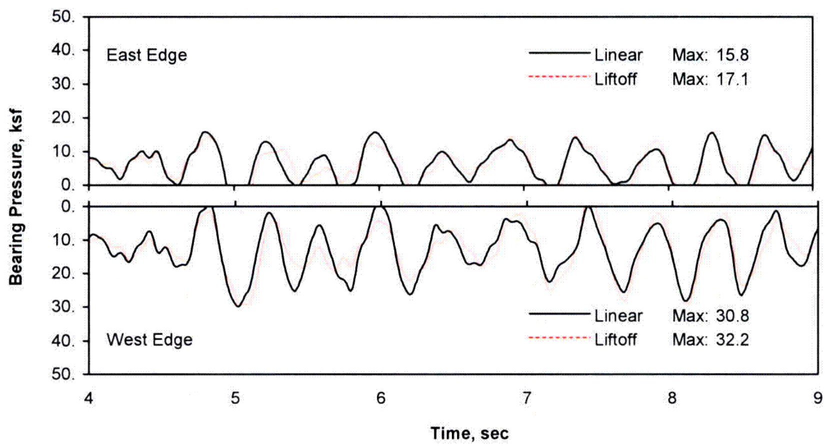
Maximum Bearing Pressure					
East			West		
	ksf	Time(s)		ksf	Time(s)
Linear	19.46	5.93	Linear	31.69	5.71
Liftoff	18.42	5.90	Liftoff	33.51	5.01

Figure 2.4-6 2D ANSYS Time History of Basemat Edges

Sheet 2 of 3 –Upper-Bound Soft-to-Medium Soil



Linear: kv900_linear-sse-r2_ic *Liftoff: kv900_liftoff-sse-r2_ic*



Maximum uplift Displacements					
East			West		
	inches	Time(s)		inches	Time(s)
Linear	0.10	5.03	Linear	-0.01	10.81
Liftoff	0.26	5.06	Liftoff	0.09	10.85

Maximum Bearing Pressure					
East			West		
	ksf	Time(s)		ksf	Time(s)
Linear	15.84	4.80	Linear	30.82	10.60
Liftoff	17.06	10.41	Liftoff	32.18	10.61

Figure 2.4-6 2D ANSYS Time History of Basemat Edges

Sheet 3 of 3 –Soft-to-Medium Soil

Revise second paragraph of Section 2.6 text and Table 2.6-1 as shown below. Note that this revision includes additional changes to those shown in the response to RAI-TR85-SEB1-22, Rev 0.

Table 2.6-1 shows the subgrade modulus calculated for each of the ~~2D SASSI~~generic soil cases using the Steinbrenner method previously used for the AP600. These calculations used the same degraded shear modulus properties in each layer as used in the SASSI analyses.~~They used a constant Poisson's ratio and do not consider the effect of the water table up to grade.~~ The subgrade moduli shown in Table 2.6-1 were used in the 2D ANSYS analyses described in section 2.4.2. The subgrade moduli were confirmed by results of an ANSYS study. Floor response spectra from the ANSYS analyses compared well in the frequency range of soil structure interaction to the results of 2D SASSI. These comparisons confirmed that the subgrade moduli provide a close match for the overall dynamic response.

Table 2.6-1

Subgrade modulus for AP1000 Soil Cases

Soil case	Subgrade modulus
	kcf
Hard rock	6267
Firm rock	2833
Soft rock	1661
Upper bound soft to medium soil (water table to grade)	1509
Upper bound soft to medium soil (dry)	1508
Soft to medium soil (water table to grade)	867
Soft to medium soil (dry)	670
Soft soil (water table to grade)	276
Soft soil (dry)	170

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR85-SEB1-11
Revision: 1~~0~~

Question:

Section 2.4.1 indicates that the "Horizontal loads on the portion below grade are added absolutely to the sum of the member forces above grade. The reactions in this table (Table 2.4-1) are used in the evaluation of nuclear island stability described in section 2.9." Why is this calculation performed rather than using the resultant forces at the base of the 2D SASSI model directly as shown in Figures 4.4.1-4 and 4.4.1-5 in TR-03, Revision 0? Also, since the 2D ANSYS results give higher bearing pressures in the soil and also greater uplift of the foundation from the soil, explain whether the 2D SASSI results bound the 2D ANSYS results for evaluation of the nuclear island stability calculations.

Additional Request (Revision 1)

The staff reviewed the RAI response provided in Westinghouse letter dated 10/19/07. If the 2D SASSI analyses are still applicable, then in order to understand the approach described in the RAI response, Westinghouse is requested to:

1. Provide a detailed figure of the 2D SASSI model and a description of the analysis approach. The figure should include all of the information referred to in the RAI response.

2. The RAI response did not address the request made in the RAI. Westinghouse is requested to show whether the 2D SASSI results bound the 2D ANSYS results. TR85, Rev. 0, stated that the 2D SASSI results were used for evaluation of the nuclear island stability calculations. This could be done for the governing stability cases for overturning and sliding. When this comparison is made, it should be done for the same soil profile(s) in both analyses in order to have a consistent comparison.

Westinghouse Response:

Reactions were calculated as described in the report because the resultant forces at the base of the stick are not the total reactions on the soil. The 2D SASSI stick model includes representation of the exterior walls below grade. The walls are connected to the ASB stick by stiff horizontal elements. Resultant forces at the base of the sticks do not include the reactions transmitted into the side soils. Hence the results just above grade are used for the structures above grade and the structures below grade are added using the mass below grade multiplied by the maximum acceleration at each elevation.

The non-linear ANSYS analyses give higher bearing pressures and lift off. However, this is due to the local effects of lift off and does not have much effect on the total base reactions. Note that the assumption implicit in the nuclear island stability calculation is that the nuclear island is supported at a line along one edge.

Westinghouse Response (Revision 1):

1. A detailed figure of the 2D SASSI model and a description of the analysis approach is included in the proposed revision to TR85 provided in the Revision 1 response to RAI-TR85-SEB1-05.

2. The proposed revision to TR85 provided in the Revision 1 response to RAI-TR85-SEB1-05 updates Sections 2.4.1 and 2.4.2 on the 2D SASSI and 2D ANSYS analyses. As described therein the purpose of the ANSYS analyses is to evaluate the effect of lift off on the bearing pressures. For this purpose the ANSYS models were compared to the SASSI models and damping was adjusted to match the SASSI results. Typical comparisons are shown in the proposed revision to TR 85 (see RAI-TR85-SEB1-05, Rev 1). Neither model is bounding. For discussion of stability see response to RAI-TR85-SEB1-04, Rev 1.

Design Control Document (DCD) Revision:
None

PRA Revision:
None

Technical Report (TR) Revision:

None

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR85-SEB1-15
Revision: ~~0~~1

Question:

Section 2.4.3 indicates that the AP1000 site interface requirements for soil to be included in DCD Table 2-1 include an average allowable static bearing capacity greater than or equal to 8.6 ksf and a maximum allowable dynamic bearing capacity for normal plus SSE greater than or equal to 35 ksf at the edge of the NI at its excavation depth. The maximum allowable dynamic bearing capacity is based on the 2D ANSYS nonlinear dynamic analyses. Westinghouse needs to address the following:

a. Since the 2D ANSYS nonlinear model and results (for EW and vertical) are used for the final determination of the maximum allowable bearing capacity needed for the site soil conditions, explain why the effect of the third earthquake direction (NS) is not also considered.

b. Since only EW and vertical SSE earthquake loadings were considered, explain whether the two time histories were input simultaneously or analyzed separately, and how the responses from the two directional earthquake analyses were combined.

c. The site interface criteria of 35 ksf is applicable to "normal" plus SSE; however, the 35 ksf appears to be based on dead load and SSE. Clarify whether the term "normal" is intended to include other normal loads such as live load; fluid loads; weight and pressure of soil, water in the soil, and surcharge loads; and any other applicable normal loads. If so, then the bearing pressure calculation should consider these loads. If normal load was not intended to include all of these loads, then explain why not.

d. Explain why the other load combinations such as those that include live load, accident pressure and accident temperature, or wind instead of earthquake were not considered.

Additional Request (Revision 1):

The staff reviewed the RAI response provided in Westinghouse letter dated 10/19/07. Based on the information provided, Westinghouse is requested to address:

a. As explained by the RAI response, the maximum bearing pressure is close to the EW center liner of the nuclear island so that the contribution of the NS earthquake is expected to be small. Westinghouse is requested to identify the magnitude of the bearing pressure contribution in the NS direction and if it has some contribution, then it should be added.

b. If the EW and vertical SSE earthquake loadings were input simultaneously in the 2D ANSYS time history analysis, then explain why the RAI response indicated that the responses were added algebraically. In a time history analysis, with the EW and vertical input motions simultaneously applied, there is only one analysis performed; therefore, explain the algebraic combination.

c. Explain the loads that are included in the "average allowable static bearing capacity" identified in TR85, Section 2.4.3 - Site Interface for Soil.

d. The RAI response indicates that the other load combinations such as those that include live load, accident pressure and accident temperature, or wind instead of earthquake, is addressed in the response to RAI TR85-SEB1-28. The response to RAI TR85-SEB1-28, however, does not explain why the design pressure which is treated as the accident pressure inside containment (Pa) and accident temperature (Ta) are not considered for calculation of the soil bearing pressure requirement. Westinghouse is requested to explain why the load combinations that include these loads are not considered with and without the SSE, when determining the maximum soil bearing pressure requirements.

Westinghouse Response:

a. The maximum bearing pressure occurs below the west side of the shield building. This is shown by the results of the equivalent static non-linear basemat analyses in Table 2.6-2 with the bearing pressures plotted in Figures 2.6-7. It is also shown by the results of the non-linear 2D ANSYS analyses in Figures 2.4-5 where the maximum bearing pressure occurs below the west edge of the shield building. The location of maximum bearing pressure is close to the east-west center line of the nuclear island so the contribution of the north south earthquake is small.

b. The EW and vertical SSE earthquake loadings were input simultaneously and responses were added algebraically.

c. Normal loads are those defined for inclusion as mass in the global seismic analyses of the nuclear island. They include equipment and fluid loads. They also include 25% of the specified floor live loads. The loads do not include the weight and pressure of soil, water in the soil, or surcharge loads.

d. The other load combinations such as those that include live load, accident pressure and accident temperature, or wind instead of earthquake are discussed in the response to RAI-TR85-SEB1-28.

Westinghouse Response (Revision 1):

a. The values obtained using the ANSYS 2D dynamic analyses are consistent with the 3D SASSI bearing pressures obtained from the generic analyses. The bearing pressures from the 3D SASSI analyses have been obtained by combining the time history results from the North-South, East-West, and vertical earthquakes. The maximum bearing pressures obtained from the various soil cases are listed in Table RAI-TR85-SEB1-03-1.

Westinghouse will base its 35 ksf limit on the SASSI 3D results given in RAI-TR85-SEB1-3. The ANSYS 2D analyses will be used to support that the 35 ksf limit is a reasonable value.

b. Agreed. Delete "and responses were added algebraically" from response.

c. The loads that are included in the "average allowable static bearing capacity" identified in TR85, Section 2.4.3 - Site Interface for Soil are the normal loads. The average load is the total load divided by the footprint area.

d. The non-linear analyses of the basemat were performed for dead and live load with 16 combinations of seismic loads (1.0, 0.4, 0.4). In addition, for a critical direction combination of seismic inputs, a non-linear analysis was performed with containment pressure. This showed that the containment pressure had only small effect on the bearing pressures. The soil bearing requirement is established from 3D SASSI analyses. The basemat analyses demonstrate that the effect of pressure is small and does not need to be considered in the maximum bearing demand. Accidental thermal does not occur concurrent with the design pressure and is not included as a design case.

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-TR85-SEB1-32
Revision: ~~0~~1

Question:

As shown by the studies in Section 2.7.1.2.1, when the soil is represented as solid elements rather than Winkler soil springs, higher bearing pressures occur at the edges and lower bearing pressures away from the edges. This is referred to as the effects of the Boussinesq distribution. Although this indicates that the basemat slab away from the walls would have higher bearing pressures using the Winkler soil spring approach (see Figure 2.7-2), the calculation of the maximum bearing pressure would still exist at the building edges if the soil is modeled as solid elements. Therefore, explain why the maximum bearing pressure for the AP1000 design, discussed in Section 2.4.2, should be based on the 2D ANSYS nonlinear dynamic analysis using Winkler soil springs rather than solid soil elements?

Additional Request (Revision 1):

The staff reviewed the RAI response submitted in Westinghouse letter dated March 31, 2008, and notes that the outstanding issues raised by this RAI are considered to be very significant. The RAI response states that the DCD "revision now indicates the line of lift-off, thereby defining the maximum total load applied to the foundation at the time of maximum demand...the dynamic bearing capacity is related to the overall loading on the foundation and to the shear strength mobilized over a failure surface in the foundation soils. The local maximum values close to the edge are not significant to this capacity and will redistribute if local stresses in the soil are excessive. This total load rather than a peak stress below an edge is to be considered by the Combined License applicant in demonstrating stability of the foundation material." Westinghouse is requested to address the following:

1. The above statements are not consistent with the criteria in the DCD because the statements indicate that the total load is used by the Combined License applicant to demonstrate the adequacy of the soil whereas, the DCD requires comparison of the maximum bearing pressure demand to bearing pressure capacity (e.g., DCD Tier 2, Section 2.5.4.2 and DCD Tier 1, Chapter 5.). Explain this inconsistency.

2. As noted in the original RAI, the studies in Section 2.7.1.2.1 demonstrate that when the soil is represented as solid elements, higher bearing pressures occur at the edges than when uniform Winkler type soil springs are used. This is a well known behavior in soil mechanics and is referred to as the Boussinesq effect. Since the current dynamic soil bearing pressure demand criterion of 35 ksf is still based on the 2D ANSYS stick model analysis, Westinghouse is requested to either (1) justify the statement that the localized peak soil pressures will redistribute if local stresses in the soil are excessive and the NI will still be stable or (2) explain what is the technical basis for using a uniform soil spring representation rather than soil brick

finite element or a soil spring distribution which more accurately captures the actual pressure distribution beneath the basemat.

3. The proposed revision to DCD Section 2.5.4.2 - Bearing Capacity, states that the "The maximum demand of 35 ksf occurs under the west edge of the shield building and is primarily due to the response to the east-west component of the earthquake. The east edge of the nuclear island lifts off the soil. The Combined License applicant will verify that the site specific allowable soil bearing capacities for static and dynamic loads at the site will exceed this demand. The evaluation may be limited to response in the east-west direction since the bearing demand is lower in the north-south direction." Explain what is meant by the statement that an "evaluation" may be limited to response in the east-west direction, because no "evaluation" or analysis to be performed by the applicant can be located in the DCD; instead the allowable soil bearing capacity needs to be shown to be greater than the bearing demand under static and dynamic loads.

Westinghouse Response:

Subsection 2.5.4.2 is being revised to clarify the maximum bearing pressure of 35 ksf, As stated in the DCD, it is obtained from analyses using uniform soil springs. The revision now indicates the line of lift off, thereby defining the maximum total load applied to the foundation at the time of maximum demand. Unlike the static case, where the allowable bearing capacity is controlled by settlements, the dynamic bearing capacity is related to the overall loading on the foundation and to the shear strength mobilized over a failure surface in the foundation soils. The local maximum values close to the edge are not significant to this capacity and will redistribute if local stresses in the soil are excessive. This total load rather than a peak stress below an edge is to be considered by the Combined License applicant in demonstrating stability of the foundation material.

Various analyses described in the report investigate the effect of modeling the soil with uniform spring and solid element representations. Comparisons are made in linear analyses using SASSI and ANSYS. Comparisons are made in ANSYS linear and non-linear analyses to show the effect of lift off. The analyses show small differences in the distribution of the bearing pressures but good agreement in the total loads imposed on the foundation material. The small differences in distribution (the Boussinesq effect) are not significant to the evaluation of the stability of the foundation material.

Westinghouse Response (Revision 1):

The maximum seismic bearing pressure demand defined for comparison to the subgrade pressure capacity is consistent with the DCD. See RAI-TR85-SEB1-03, Rev. 1 for discussion of the 35 ksf maximum bearing seismic demand.

In response to the many questions in this and other RAIs, Westinghouse has revised the basis for the bearing demand. The demand is now based on 3D SASSI analyses using the 3D NI20 finite element model as described in the response to RAI-TR85-SEB1-03, Rev 1. This change to use of the 3D SASSI results addresses the original question in this RAI. The additional questions in Rev 1 of this RAI apply to the Rev 0 response which has now been superseded.

The statement in the DCD Section 2.5.4.2, "The evaluation may be limited to response in the east-west direction since the bearing demand is lower in the north-south direction" has been removed. See DCD revision section below.

Design Control Document (DCD) Revision:

The changes to the DCD shown in Rev 0 of this RAI response have been implemented in DCD Rev 17. Revise first paragraph of DCD Rev 17 subsection 2.5.4.2 as follows:

2.5.4.2 Bearing Capacity

The maximum bearing reaction determined from the 3D SASSI analyses described in Appendix 3G is less than 35,000 lb/ft^2 under all combined loads, including the safe shutdown earthquake. ~~These analyses use uniform soil springs below the basemat.~~ The maximum dynamic bearing demand of 35 ksf occurs under the west edge of the shield building and is primarily due to the response to the east-west component of the earthquake. The east edge of the nuclear island lifts off the soil. The Combined License applicant will verify that the site-specific allowable soil bearing capacities for static and dynamic loads at the site will exceed the static and dynamic bearing demand given in Table 2-1. ~~The evaluation may be limited to response in the east-west direction since the bearing demand is lower in the north-south direction.~~

PRA Revision:

None

Technical Report (TR) Revision:

None