Westinghouse

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

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

Direct tel: 412-374-6206
Direct fax: 412-374-5005
e-mail: sisk1rb@westinghouse.com

Your ref: Docket No. 52-006
Our ref: DCP_NRC_002523

June 10, 2009

Subject: AP1000 Response to Request for Additional Information (TR 85)

Westinghouse is submitting responses to NRC requests for additional information (RAI) on Technical Report No. 85. 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-TR85-SEB1-10 R2
RAI-TR85-SEB1-37 R2

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

Very truly yours,

D. A. Lindgren /FOR
Robert Sisk, Manager
Licensing and Customer Interface
Regulatory Affairs and Standardization

/Enclosure

1. Response to Request for Additional Information on Technical Report No. 85

0184ljb.doc

D063
NRO

cc:	D. Jaffe	-	U.S. NRC	1E
	E. McKenna	-	U.S. NRC	1E
	B. Gleaves	-	U.S. NRC	1E
	T. Spink	-	TVA	1E
	P. Hastings	-	Duke Power	1E
	R. Kitchen	-	Progress Energy	1E
	A. Monroe	-	SCANA	1E
	P. Jacobs	-	Florida Power & Light	1E
	C. Pierce	-	Southern Company	1E
	E. Schmiech	-	Westinghouse	1E
	G. Zinke	-	NuStart/Entergy	1E
	R. Grumbir	-	NuStart	1E
	D. Lindgren	-	Westinghouse	1E

ENCLOSURE 1

Response to Request for Additional Information on Technical Report No. 85

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR85-SEB1-10
Revision: 2

Question:

Section 2.4.1 indicates that "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 horizontal analyses..." The following items need to be addressed:

a. What is the technical basis that these results are considered to be conservative?

b. What is the technical basis for combining the M_{xx} EW seismic load with the vertical load by SRSS and similarly for the M_{yy} NS excitation load and the vertical load? (Normally SRSS is applicable to the use of three directional load combination. Since these loads are being used for the NI stability evaluation, normal practice is to utilize the summation of one horizontal load and vertical load, both acting in the worst direction. This would be repeated for the other horizontal load and vertical load.)

c. Footnote 2 of Table 2.4-2 (Page 13 of 83) states that reactions for horizontal input are calculated from the 2D SASSI analyses. Reactions due to vertical input are calculated from the maximum accelerations in 3D ANSYS or SASSI analyses for hard rock (HR), firm rock (FR), upper bound of soft medium soil (UBSM), and soft to medium soil (SM), and from 2D ANSYS analyses for soft rock (SR) and soft soil (SS). Was the 2D ANSYS analyses, referred to here, based on the linear or nonlinear ANSYS analyses? Also, why wasn't one consistent set of analyses (say 2D SASSI) used for both horizontal and vertical input in this evaluation?

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 the items listed below.

a. With the changes made to a number of seismic analyses, explain whether the maximum seismic reactions in Table 2.4-2, developed from the 2D SASSI analyses, are still relied upon for any purpose. If so, then explain where they are utilized and why combining the member forces above grade with the inertia forces below grade, using absolute sum, is considered to be conservative.

b. The use of the SRSS or the 100/40/40 combination method is only acceptable for combining the co-directional responses such as Mxx due to NS, EW, and vertical, in order to obtain a combined Mxx. However, it is not clear from TR 85, DCD Section 3.8.5, nor from the RAI response, how the stability calculations are performed once the individual three loads Mxx, Myy, and vertical (each of these already combined by SRSS or 100/40/40 due to the three earthquake inputs) are determined. DCD 3.8.5.5.4, for example, discusses the overturning evaluation and presents the equation for the factor of safety as the resisting moment divided by the overturning moment. However, this does not explain how the vertical seismic force is considered. The traditional method for evaluating stability (sliding and overturning) of nuclear plant structures in accordance with SRP 3.8.5 is to perform two separate 2-D evaluations, one for the N-S and vertical directions and one for the E-W and vertical directions. Thus, for overturning evaluation as an example, the minimum vertical downward load (deadweight minus upward buoyancy force minus upward vertical seismic force) is considered in calculating the resisting moment and this is then compared to the overturning moment about one horizontal direction (i.e., EW axis); then a similar comparison is made for the same minimum downward vertical load with the overturning moment about the other perpendicular horizontal direction (i.e., NS axis). Westinghouse is requested to clarify if they follow this analytical method for the stability evaluations (sliding and overturning) and document the approach in TR85 and the DCD. If not, then Westinghouse is requested to justify any other alternative method used. Note, with the changes recently made in the various seismic analyses, explain whether the maximum seismic reactions in Table 2.4-2, developed from the 2D SASSI analyses, are still relied upon for use in the stability evaluations performed in Section 2.9 of TR85.

Note: that the issues described above are applicable to all stability evaluations including the new 3D NI20 model using response spectrum analysis with ANSYS, which is used for stability evaluation.

c. With the changes made to a number of seismic analyses, explain whether the results from Table 2.4-2 and footnote 2, developed from the 2D SASSI analyses, are still relied upon for any purpose. If so, then Westinghouse is requested to provide the technical basis for the statement "...different models give consistent results and use of results from different analyses is acceptable."

Additional Request (Revision 2):

In the response for item b of the RAI, Westinghouse indicated that the analysis for stability has been revised to utilize the 3D ANSYS finite element NI20 model using a mode superposition time history analysis (linear with no lift-off). A separate 2D ANSYS lift-off analysis demonstrated that the minor lift-off is negligible. Since the 3D ANSYS NI20 model analysis using three input motions applied simultaneously is utilized for the stability evaluation, the concern raised by the directional combination methods no longer applies. Therefore, this concern has been adequately addressed. However, the RAI response discussed the need to utilize some passive pressure resistance capability of the soil when performing the sliding stability analyses. The passive pressure resistance curve as a function of displacement is based on Reference 1 (Hsai-Yang Fang, "Foundation Engineering Handbook," 1991) given in the RAI response. Westinghouse is requested to provide the complete text in the applicable section or chapter of the referenced book which describes the approach for determining the passive pressure resistance function.

Westinghouse Response:

a. The results in Table 2.4-2 are conservative because of the method of combination of member forces and inertia forces below grade. The maximum member forces at grade are translated down to the underside of the basemat with an absolute combination of the effects of the horizontal shear forces and the moments. The horizontal loads on the portion below grade are added absolutely to the sum of the member forces above grade.

b. As described in DCD subsection 3.7.2.6,

> In analyses with the earthquake components applied separately and in the response spectrum and equivalent static analyses, the effect of the three components of earthquake motion are combined using one of the following methods:
>
> - The peak responses due to the three earthquake components from the response spectrum and equivalent static analyses are combined using the square root of the sum of squares (SRSS) method.
>
> - The peak responses due to the three earthquake components are combined directly, using the assumption that when the peak response from one component occurs, the responses from the other two components are 40 percent of the peak (100 percent-40 percent-40 percent method). Combinations of seismic responses from the three earthquake components, together with variations in sign (plus or minus), are considered. This method is used in the nuclear island basemat analyses, the containment vessel analyses and the shield building roof analyses.

In the combination shown in Table 2.4-2, the moment M_{xx} due to input in the NS direction is zero. Thus the SRSS combination combines two components (EW seismic load and vertical load).

c. The 2D ANSYS analyses referred to in Footnote 2 of Table 2.4-2 were based on linear ANSYS analyses. As described in TR85 many analyses have been performed using a variety of models. At the time of the stability evaluation there was not a consistent set available. However, the different models give consistent results and use of results from different analyses is acceptable.

Westinghouse Response (Revision 1):

a. As discussed in RAI-TR85-SEB1-04, part (2), Revision 1, 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 time history to reflect the seismic response for the other two potential governing soil cases UBSM and SM. The firm rock, soft rock, and soft soil have lower seismic response. Combining the member forces above grade with the inertia forces below grade using absolute sum is conservative since it assumes the structures above grade, and those below grade are in phase (modes closely spaced). Otherwise, one could have used the SRSS method.

b. Westinghouse agrees that the SRSS and 100/40/40 combination method is only acceptable for combining the co-directional responses. When Westinghouse has used this combination method it has been applied only to co-directional responses. The NRC has previously reviewed the acceptable use of the 100/40/40 method as part of the AP600 and the hard rock certification. The NRC in their FSER (NUREG-1793) related to AP1000 hard rock licensing states:

> "As for the suitability of using the 100 percent, 40 percent, 40 percent combination method, the applicant, during audits performed by the staff, provided calculations to demonstrate that the combination method always gives reasonable results by comparing the results with those from the SRSS combination method. From its review of the design calculations, the staff also finds that the difference between results obtained using the two methods was less than 5 percent which is considered insignificant and, therefore, is acceptable."

The NRC review and audit considered stability, and it is further stated in FSER Section 3.7.2.17:

> "... When the equivalent acceleration static analysis method is used, the SRSS method or 100 percent, 40 percent, 40 percent method was used to combine spatial response in conformance with RG 1.92 and consistent with accepted common industry practice. ... Torsional effects and stability against overturning, sliding, and flotation are considered.

When it is necessary to combine co-directional responses, Westinghouse is not using any different methodology that wasn't reviewed and accepted by the NRC previously.

For the seismic stability analysis Westinghouse is using the 3D NI20 model. Time history analyses using ANSYS has been used. This is discussed in RAI-TR85-SEB1-004, part (2). It was not necessary to use the 100 percent, 40 percent, 40 percent method. However, if this method was used the following method would have been used to calculate the co-directional responses:

- The seismic maximum moment about an edge (e.g. column line I) is calculated considering the maximum moment due to the horizontal excitation combined with 40 percent of the moment due to the maximum vertical seismic excitation. (Note that using 100 percent of maximum vertical seismic excitation, and 40 percent of the maximum moment due to horizontal excitation will not control.) This moment is used as the maximum SSE overturning moment in the stability evaluation.

- For sliding 40 percent of the maximum vertical seismic component is considered in the reduction of the normal force in the calculation of the friction force.

Using the maximum time history results a comparison of the stability factors of safety obtained to the 100 percent, 40 percent, 40 percent method to the stability factors of safety obtained from the time history analysis is made. The time history analysis calculates the stability factors of safety at each time step, and the minimum factor of safety used. The coefficient of friction considered is 0.55. This comparison is given in Table RAI-TR85-SEB1-10-01a for sliding in the NS and EW direction, and overturning about the West side of the Shield Building and about column line 11. Also, the comparison is given for the hard rock (HR), upper-bound-soft-to-medium (UBSM) case, and the soft-to medium (SM) case. As seen from this comparison, the 100, 40 percent, 40 percent method is more conservative compared to the time history method for the overturning factors of safety. For sliding partial passive pressure is required to meet the 1.1 limit. To compare the two methods the amount of deflections required to obtain the required passive resistance are compared. This comparison is given in Table RAI-TR85-SEB1-10-01b. As seen from this comparison the NS deflections are essentially the same, and for the EW deflections the 1 x 0.4 x 0.4 method is conservative (larger deflections).

It is noted that Westinghouse has not used response spectrum analysis to perform the stability evaluation.

Table RAI-TR85-SEB1-10-01a: Factor of Safety Comparisons for 1 x 0.4 x 0.4 and TH Methods

Stability Factors of Safety	1 x 0.4 x 0.4 Method			T.H. Method		
	HR	UBSM	SM	HR	UBSM	SM
Sliding N-S SSE $\mu = 0.55$	1.1	1.1	1.1	1.1	1.1	1.1
Sliding E-W SSE $\mu = 0.55$	1.1	1.1	1.1	1.1	1.1	1.1
Overturning WSB SSE	1.31	1.17	1.17	1.62	1.44	1.46
Overturning Col. 11 SSE	1.78	1.77	1.79	2.06	2.00	1.92

Table RAI-TR85-SEB1-10-01b: Displacement Comparisons for 1 x 0.4 x 0.4 and TH Methods

Units: inches

Stability Factors of Safety	1 x 0.4 x 0.4 Method			T.H. Method		
	HR	UBSM	SM	HR	UBSM	SM
Sliding N-S SSE $\mu = 0.55$	0.11	0.10	0.07	0.12	0.12	0.08
Sliding E-W SSE $\mu = 0.55$	0.10	0.79	0.65	0.09	0.50	0.49

Provided below is a summary of the stability evaluation performed using the 3D NI20 model and ANSYS time history seismic analyses. Three cases are considered: HR, UBSM, and SM. The other three cases firm rock, soft rock, and soft soil do not control the stability evaluation.

Seismic Overturning Stability Evaluation

It is not necessary to consider passive pressure in the overturning evaluation. Therefore, in the calculation of the factor of safety for overturning the resistance moment associated with passive pressure is zero (Mp = 0). In Table RAI-TR85-SEB1-~~01~~10-02 is given the factors of safety associated with overturning about column lines 11, 1, I and west side of shield building. All of the factors of safety are above the established limit of 1.1.

Table RAI-TR85-SEB1-~~01~~10-02: Overturning Factors of Safety

Column Line / Wall	HR F.S.	UBSM F.S.	SM F.S.
Column Line 11 (North)	2.06	2.00	1.92
Column Line 1 (South)	1.83	1.79	1.77
Column Line I (East)	1.31	1.18	1.17
West side of Shield Building (West)	1.62	1.44	1.46

Seismic Sliding Evaluation

In the evaluation of sliding different coefficients of friction are considered. They are 0.7, 0.6, and 0.55. Also, it is necessary to rely on passive pressure. Using Case 15 (RAI-TR85-SEB1-35, R1, Table RAI-TR85-SEB1-35-1), and the methodology given in Reference 1 using a soil friction angle of 35°, a relationship between passive pressure and displacement at grade elevation can be defined. This relationship is shown in Figures RAI-TR85-SEB1-~~35~~10-1 and RAI-TR85-SEB1-~~35~~10-2 for the first 5 inches of deflection. Curves are given for the North-South and East-West directions. The passive pressure at zero deflection is equal to the at rest pressure. The total passive soil pressure resistance force is 43,500 kips for the North-South direction, and 69,100 kips for the East-West direction. It is noted that to achieve the full passive pressure displacements in excess of 10 inches are required.

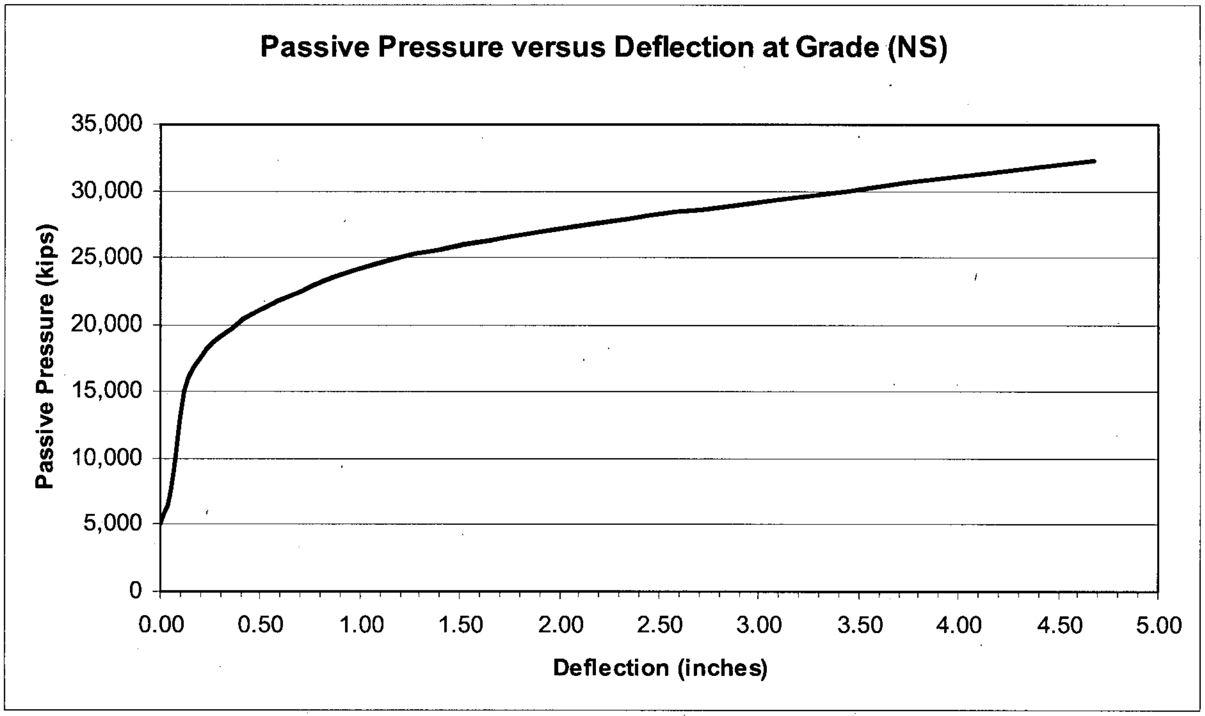

Figure RAI-TR85-SEB1-~~35~~10-1 – Passive Pressure versus Deflection at Grade (North-South Excitation)

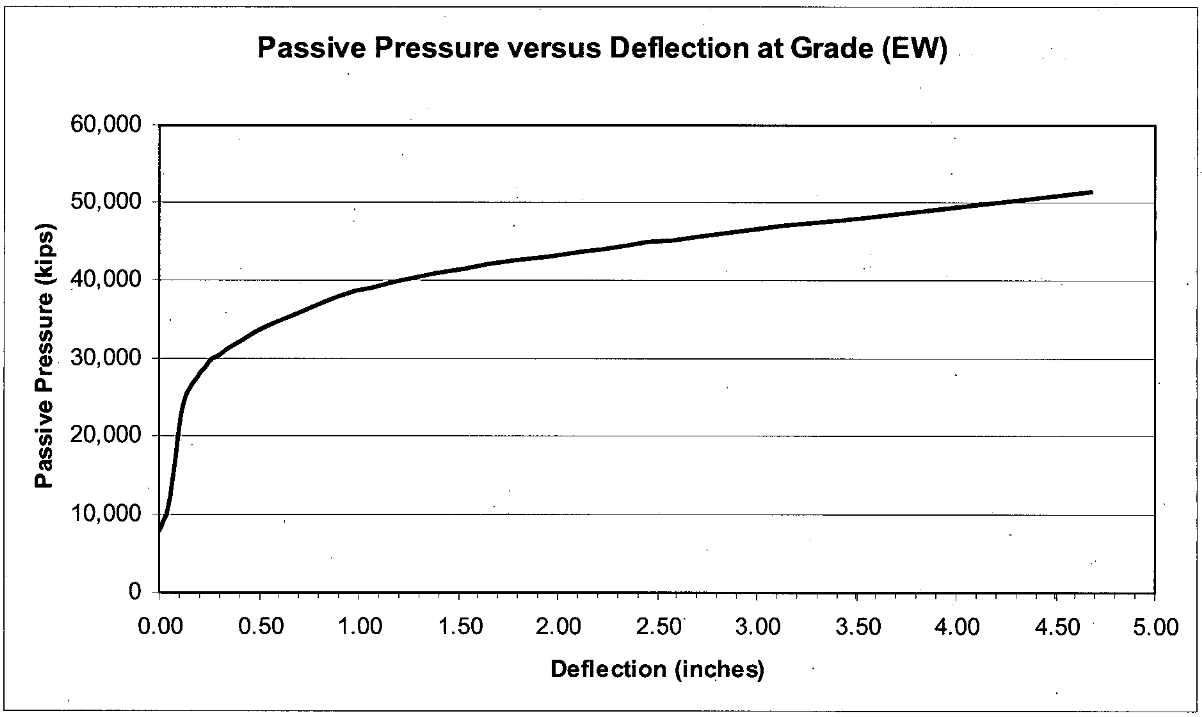

Figure RAI-TR85-SEB1-~~35~~10-2 – Passive Pressure versus Deflection at Grade (East-West Excitation)

During the sliding stability calculation it was determined that the factor of safety for sliding drops below the limit of 1.1 for a very short time if passive pressure is not considered. Plots of the factor of safety (FS) versus time for the hard rock case and the North-South and East-West directions are given in Figures RAI-TR85-SEB1-~~35~~10-3 and RAI-TR85-SEB1-~~35~~10-4 using a coefficient of friction of 0.55. As seen from these figures the time at which the factor of safety drops below 1.1 is very short. This is the only time during the seismic event that this occurs. When the passive pressure is considered, the factor of safety remains above the limit of 1.1.

In Tables RAI-TR85-SEB1-~~35~~10-~~1~~3 to RAI-TR85-SEB1-~~35~~10-~~3~~5 are given a summary of the results for the three coefficient values. Provided is the required passive pressure to maintain the factor of safety equal to or above 1.1. As seen from this summary using a coefficient of friction of 0.55 or higher, deflections less than 0.15 inch for hard rock, less than or equal to 0.5 inch for upper bound soft to medium and soft to medium soil conditions are needed to develop the required amount of passive pressure.

The coefficient of friction is changed from 0.7 to 0.55 for the soils. The coefficient of friction for the waterproofing membrane is also changed from 0.7 to 0.55.

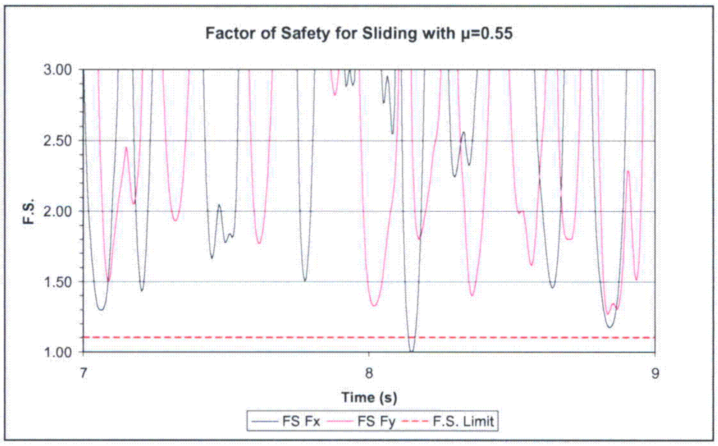

Figure RAI-TR85-SEB1-~~35~~10-3 - North-South FS without Passive Pressure

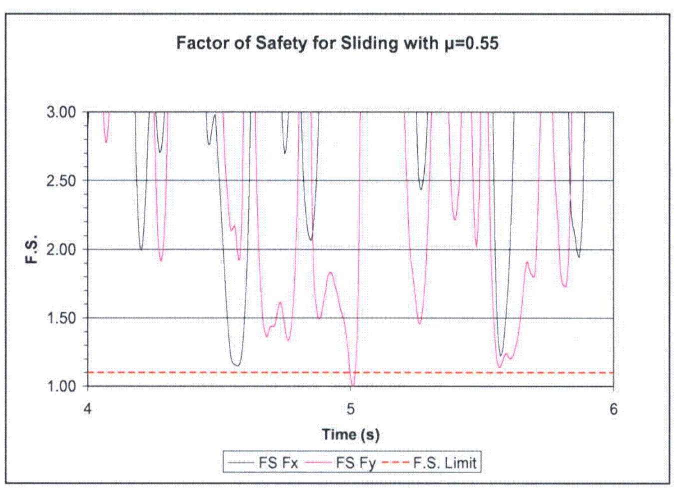

Figure RAI-TR85-SEB1-~~35~~10-4 – East-West FS without Passive Pressure

Table RAI-TR85-SEB1-~~35~~10-~~1~~3 - Factors of Safety against Sliding for Hard Rock

Direction	Coefficient of Static Friction	Passive Pressure Resistance Force Required	Displacement at Grade	F.S.
North – South (Xg)	0.70	(1)	0.00 in	1.24
East – West (Yg)	0.70	(1)	0.00 in	1.23
North – South (Xg)	0.60	7,166 kip	0.05 in	1.10
East – West (Yg)	0.60	10,802 kip	0.04 in	1.10
North – South (Xg)	0.55	15,142 kip	0.12 in	1.10
East – West (Yg)	0.55	18,402 kip	0.09 in	1.10

Note (1) - At rest pressure

Table RAI-TR85-SEB1-~~35~~10-~~2~~4 - Factors of Safety against Sliding for Upper Bound Soft to Medium

Direction	Coefficient of Static Friction	Passive Pressure Resistance Force Required	Displacement at Grade	F.S.
North – South (Xg)	0.70	(1)	0.00 in	1.28
East – West (Yg)	0.70	11,127 kip	0.05 in	1.10
North – South (Xg)	0.60	6,992 kip	0.05 in	1.10
East – West (Yg)	0.60	25,927 kip	0.16 in	1.10
North – South (Xg)	0.55	14,817 kip	0.12 in	1.10
East – West (Yg)	0.55	33,352 kip	0.50 in	1.10

Note (1) - At rest pressure

Table RAI-TR85-SEB1-~~35~~10-~~3~~-5 - Factors of Safety against Sliding for Soft to Medium

Direction	Coefficient of Static Friction	Passive Pressure Resistance Force Required	Displacement at Grade	F.S.
North – South (Xg)	0.70	(1)	0.00 in	1.29
East – West (Yg)	0.70	11,627 kip	0.05 in	1.10
North – South (Xg)	0.60	(1)	0.00 in	1.11
East – West (Yg)	0.60	25,977 kip	0.16 in	1.10
North – South (Xg)	0.55	11,092 kip	0.08 in	1.10
East – West (Yg)	0.55	33,202 kip	0.49 in	1.10

Note (1) - At rest pressure

c. The justification of the statement made that "...different models give consistent results and use of results from different analyses is acceptable." Is given in RAI-TR85-SEB1-04, part (2), Revision 1, where it is shown that the reactions obtained using the 2D SASSI seismic response factor applied to the time history response result in conservative reactions when compared to the 3D SASSI analysis results. Therefore, the acceptability of the seismic response factors developed from the 2D SASSI models for use in the seismic stability evaluations is acceptable.

Westinghouse Response (Revision 2):

In the May 4 to 8, 2009 audit, the NRC reviewed the displacements based on the displacement curves given in Reference 1. The displacements given in the Revision 1 response to this RAI is based on the passive pressures defined using the Case 15 soil parameters as defined in RAI-TR85-SEB1-35. As part of the review of RAI-TR85-SEB1-35, the NRC requested Westinghouse to explain why, for sliding stability evaluation, a high passive pressure was used for resistance of the backfill adjacent to the Nuclear Island (NI) rather than a lower bound value based on soil parameters such as those defined by Case 21 (soil parameters defined in RAI-TR85-SEB1-35). Westinghouse stated that a lower bound was used in for the soil properties similar to Case 21. A comparison of geotechnical parameters and lateral earth pressures was given during the audit and is presented in Table RAI-TR85-SEB1-10-6. Presented in Tables RAI-TR85-SEB1-10-7 to RAI-TR85-SEB1-10-12 are the stability results for Case 15 and the lower bound soil case evaluated. It is noted that the displacements given for Case 15 are slightly different from those given in Revision 1 of this response because the active and dynamic surcharge pressures were slightly modified to be more representative (e.g. dynamic surcharge acting only on one side; active pressure acts below adjacent building foundations). The deflections obtained were discussed. It was stated by Westinghouse that the analysis methodology used was the conservative equivalent static. This will result in large deflections since the seismic loads are considered to be constant and do not reflect the short time duration as shown in Figures RAI-TR85-10-3 and RAI-TR85-10-4. It was requested that Westinghouse perform a more realistic non-linear analysis with sliding friction elements using a 2D ANSYS model.

Westinghouse modified the 2D ANSYS model that was used to study the basemat uplift. This model is described in Subsection 2.4.2 of TR85. This 2D non-linear model is for the East-West direction. There is no need to modify this model for the North-South direction since the NI deflections calculated to maintain a factor of safety of 1.1 is largest in the East-West direction. This model was modified introducing friction elements along the bottom of the basemat that is at the interface of the basemat and soil media.

Direct time integration analysis was performed that is also described in Subsection 2.4.2 of TR85. The three cases that have the lowest factor of safety related to sliding were evaluated. These three cases are HR, UBSM, and SM. The seismic input was increased by 10% so as to maintain the factor of safety against sliding of 1.1. No passive soil resistance is considered in the analyses. The resulting deflections at the base using a coefficient of friction of 0.55 are given in Table RAI-TR85-10-13 for the three cases. As noted above this model did consider vertical uplift in addition to sliding. As seen from this table the Nuclear Island experiences negligible sliding during the seismic event, and no passive soil resistance is necessary from the backfill. This is consistent with the observation made in Revision 1 of this response that:

> *"During the sliding stability calculation it was determined that the factor of safety for sliding drops below the limit of 1.1 for a very short time if passive pressure is not considered. Plots of the factor of safety (FS) versus time for the hard rock case and the North-South and East-West directions are given in Figures RAI-TR85-SEB1-10-3 and RAI-TR85-SEB1-10-4 using a coefficient of friction of 0.55. As seen from these figures the time at which the factor of safety drops below 1.1 is very short."*

Therefore, it can be concluded that the Nuclear Island is stable against sliding, and there is no quality requirement for the backfill material adjacent to the NI (side soil) to remain stable against sliding. Also, as noted in Revision 1 of this response, there is no passive pressure required to maintain stability against overturning.

The factors of safety related to wind and tornado loads have also been revised to remove passive pressure from the calculation of the factor of safety. All of the factors of safety are above the limits established for stability. Changes to the DCD and Technical Report are reflected below under Design Control Document (DCD) Revision and Technical Report (TR) Revision.

During the review of the response given for RAI-TR85-SEB1-04, it was requested that Westinghouse include in the DCD a description of the evaluations performed for the foundation stability which consists of a summary of the analyses presented in TR85, Rev. 1. This request is reflected in this RAI under the DCD revision section below.

Table RAI-TR85-SEB1-10-6 – Comparison of Geotechnical Parameters and Lateral Earth Pressures

Soil Properties/ Parameters	Case 15 Soil	Case 21 Soil	Lower Bound Soil Evaluated
Total Unit Weight (pcf)	150.0	95.0	122.4
Effective Unit Weight (pcf)	87.6	60.0	60.0
Friction Angle (degrees)	35.0	32.0	35.0
-	-	-	-
At-Rest Earth Pressure Coefficient (Ko)	0.426	0.470	0.426
Lateral Ko Earth Pressure at Elev. 60.5 (psf)	1,529	1,147	1,064
Full At-Rest Resistance Force (kips)	7,985 (E-W) 5,022 (N-S)	5,957 (E-W) 3,746 (N-S)	5,635 (E-W) 3,544 (N-S)
-	-	-	-
Passive Earth Pressure Coefficient (Kp)	3.690	3.255	3.690
Lateral Kp Earth Pressure at Elev. 60.5 (psf)	13,229	7,941	9,206
Full Passive Resistance Force (kips)	69,098 (E-W) 43,456 (N-S)	42,244 (E-W) 25,939 (N-S)	48,758 (E-W) 30,664 (N-S)

Table RAI-TR85-SEB1-10-7 – Sliding Factors of Safety with Hard Rock Case 15 Soil Passive Resistance

Direction	Coefficient of Friction	At-Rest Force Applied (kips)	Passive Force Applied (kips)	% of Full Passive Force	Displacement at Grade (inch)	Factor of Safety
North – South	0.70	5,017	N/A	N/A	0.000	1.22
East – West	0.70	7,977	N/A	N/A	0.000	1.24
North – South	0.60	N/A	9,166	21.1	0.065	1.10
East – West	0.60	N/A	10,076	14.6	0.030	1.10
North – South	0.55	N/A	17,116	39.4	0.188	1.10
East – West	0.55	N/A	17,676	25.6	0.082	1.10

Table RAI-TR85-SEB1-10-8 – Sliding Factors of Safety with Upper Bound Soft to Medium Case 15 Soil Passive Resistance

Direction	Coefficient of Friction	At-Rest Force Applied (kips)	Passive Force Applied (kips)	% of Full Passive Force	Displacement at Grade (inch)	Factor of Safety
North – South	0.70	5,017	N/A	N/A	0.000	1.22
East – West	0.70	N/A	10,390	15.0	0.035	1.10
North – South	0.60	N/A	8,910	20.5	0.063	1.10
East – West	0.60	N/A	25,250	36.6	0.145	1.10
North – South	0.55	N/A	16,750	38.5	0.132	1.10
East – West	0.55	N/A	32,610	47.2	0.453	1.10

Table RAI-TR85-SEB1-10-9 – Sliding Factors of Safety with Soft to Medium Case 15 Soil Passive Resistance

Direction	Coefficient of Friction	At-Rest Force Applied (kips)	Passive Force Applied (kips)	% of Full Passive Force	Displacement at Grade (inch)	Factor of Safety
North – South	0.70	5,017	N/A	N/A	0.000	1.27
East – West	0.70	N/A	10,900	15.8	0.042	1.10
North – South	0.60	N/A	5,350	12.3	0.008	1.10
East – West	0.60	N/A	25,300	36.6	0.146	1.10
North – South	0.55	N/A	12,980	29.9	0.099	1.10
East – West	0.55	N/A	32,400	46.9	0.439	1.10

Table RAI-TR85-SEB1-10-10 – Sliding Factors of Safety with Hard Rock Lower Bound Evaluated Soil Passive Resistance

Direction	Coefficient of Friction	At-Rest Force Applied (kips)	Passive Force Applied (kips)	% of Full Passive Force	Displacement at Grade (inch)	Factor of Safety
North – South	0.70	3,544	N/A	N/A	0.000	1.18
East – West	0.70	5,635	N/A	N/A	0.000	1.17
North – South	0.60	N/A	8,200	26.7	0.087	1.10
East – West	0.60	N/A	8,650	17.7	0.052	1.10
North – South	0.55	N/A	16,170	52.7	0.796	1.10
East – West	0.55	N/A	16,250	33.3	0.112	1.10

Table RAI-TR85-SEB1-10-11 – Sliding Factors of Safety with Upper Bound Soft to Medium Lower Bound Evaluated Soil Passive Resistance

Direction	Coefficient of Friction	At-Rest Force Applied (kips)	Passive Force Applied (kips)	% of Full Passive Force	Displacement at Grade (inch)	Factor of Safety
North – South	0.70	3,544	N/A	N/A	0.000	1.18
East – West	0.70	N/A	9,000	18.5	0.055	1.10
North – South	0.60	N/A	8,100	26.4	0.085	1.10
East – West	0.60	N/A	23,900	49.0	0.535	1.10
North – South	0.55	N/A	15,850	51.7	0.711	1.10
East – West	0.55	N/A	31,250	64.1	2.33	1.10

Table RAI-TR85-SEB1-10-12 – Sliding Factors of Safety with Soft to Medium Lower Bound Evaluated Soil Passive Resistance

Direction	Coefficient of Friction	At-Rest Force Applied (kips)	Passive Force Applied (kips)	% of Full Passive Force	Displacement at Grade (inch)	Factor of Safety
North – South	0.70	3,544	N/A	N/A	0.000	1.22
East – West	0.70	N/A	9,500	19.5	0.059	1.10
North – South	0.60	N/A	4,500	14.7	0.031	1.10
East – West	0.60	N/A	23,900	49.0	0.535	1.10
North – South	0.55	N/A	12,100	39.5	0.189	1.10
East – West	0.55	N/A	31,000	63.6	2.24	1.10

Table RAI-TR85-SEB1-10-13 – Seismic Deflections at Bottom of Nuclear Island Basemat due to Sliding (Coefficient of Friction equal to 0.55)

Case	Deflection @ 60.5' El inches
HR	0.003
UBSM	0.016
SM	0.030

Reference: ~~None~~

1. Hsai-Yang Fang, "Foundation Engineering Handbook," Second Edition, 1991, Van Nostrand Reinhold.

Design Control Document (DCD) Revision:

~~None~~ Modify the first sentence in the last paragraph of DCD subsection 3.4.1.1.1.1, Revision 17, to read as follows:

The waterproof function of the membrane is not safety-related; however, the membrane between the mudmats must transfer horizontal shear forces due to seismic (SSE) loading. This function is seismic Category I. The specific ~~static~~ coefficient of friction between horizontal membrane and concrete is ≥~~0.7~~ 0.55.

Modify the following DCD Revision 17 subsections related to seismic stability.

3.8.5.5.3 Sliding

The factor of safety against sliding of the nuclear island (NI) during a tornado or a design wind is shown in Table 3.8.5-2 and is calculated as follows:

$$F.S. = \frac{F_S + F_P}{F_H}$$

where:

$F.S.$ = factor of safety against sliding from tornado or design wind
F_S = shearing or sliding resistance at bottom of basemat
F_P = maximum soil passive pressure resistance, neglecting surcharge effect
F_H = maximum lateral force due to active soil pressure, including surcharge, and tornado or design wind load

The factor of safety against sliding of the nuclear island during a safe shutdown earthquake is shown in Table 3.8.5-2 and is calculated as follows:

$$F.S. = \frac{F_s + F_p}{F_D + F_H}$$

where:

$F.S.$ = factor of safety against sliding from a safe shutdown earthquake
F_S = shearing or sliding resistance at bottom of basemat
F_P = maximum soil passive pressure resistance, neglecting surcharge effect
F_D = maximum dynamic lateral force, including dynamic active earth pressures
F_H = maximum lateral force due to all loads except seismic loads

The sliding resistance is based on the friction force developed between the basemat and the foundation. The governing friction value in the interface zone is a thin soil layer below the mudmat with an angle of internal friction of 35° giving a static coefficient of friction of 0.70. The effect of buoyancy due to the water table is included in calculating the sliding resistance.

3.8.5.5.4 Overturning

The factor of safety against overturning of the nuclear island during a tornado or a design wind is shown in Table 3.8.5-2 and is calculated as follows:

$$F.S. = \frac{M_R}{M_O}$$

where:

$F.S.$ = factor of safety against overturning from tornado or design wind
M_R = resisting moment
M_O = overturning moment of tornado or design wind

The factor of safety against overturning of the nuclear island during a safe shutdown earthquake is shown in Table 3.8.5-2 and is evaluated using the static moment balance approach assuming overturning about the edge of the nuclear island at the bottom of the basemat. The factor of safety is defined as follows:

$$F.S. = (M_R + M_P)/(M_O + M_{AO})$$

where:

F.S. = factor of safety against overturning from a safe shutdown earthquake
M_R = nuclear island's resisting moment against overturning
M_O = maximum safe shutdown earthquake induced overturning moment acting on the nuclear island, applied as a static moment
M_P = Resistance moment associated with passive pressure
M_{AO} = Moment due to lateral forces caused by active and overburden pressures

The resisting moment is equal to the nuclear island dead weight, minus buoyant force from ground water table, multiplied by the distance from the edge of the nuclear island to its center of gravity. The overturning moment is the maximum moment about the same edge from the time history analyses of the nuclear island ~~lumped mass stick~~NI20 model described in subsection 3.7.2 and 3G.2.

3.8.5.5.5 Seismic Stability Analysis

The factors of safety for sliding and overturning for the SSE are calculated for each soil case for the base reactions in terms of shear and bending moments about column lines 1, 11, I and the west side of the shield building at each time step of the seismic time history. 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 time history to reflect the seismic response for the other two potential governing soil cases UBSM and SM. The firm rock, soft rock, and soft soil cases have higher factors of safety against sliding and therefore not considered.

The seismic time history analysis used the ANSYS computer code and the NI20 model. The minimum stability factors of safety values are reported in Table 3.8.5-2. For seismic overturning no passive pressure was considered. For sliding partial passive pressure is considered for sliding. Two soil cases are considered, the soil parameters used for design (friction angle of 35°, and submerged weight of 87.6 pcf), and a lower bound soil density (friction angle of 35°, and submerged weight of 60 pcf). For the design case the amount of passive pressure required to meet the 1.1 factor of safety is 40% for the North-South seismic event, and 47% for the East-West excitation of full passive pressure. For the lower bound case the amount of passive pressure required to meet the 1.1 factor of safety is less than 53% for the North-South seismic event, and 64% of the East-West excitation of full passive pressure. The relationship between passive pressure and displacement at grade is obtained based on the methodology given in Reference 56. The maximum displacement of the Nuclear Island at grade to develop the required passive resistance is 0.5" for the design case, and 2.3" for the lower bound case. These deflections are based on conservative equivalent static analysis with a coefficient of friction of 0.55. This will result in large deflections since the seismic loads are considered to be constant and do not reflect the short time duration that they exist during the seismic event. A more realistic non-linear analysis with sliding friction elements using a 2D ANSYS model was performed. The 2D ANSYS model that was used to study the basemat uplift (see Subsection 3.8.5.5.6 and Appendix 3G). This 2D non-linear model is for the East-West direction. There is no need to consider the North-South direction since the NI deflections calculated to maintain a factor of safety of 1.1 is largest in the East-West direction. This model was modified introducing friction elements along the bottom of the basemat and soil media interface. Direct time integration analysis was performed with vertical uplift and sliding allowed. The three cases that have the lowest factor of safety related to sliding were evaluated. These three cases are HR, UBSM, and SM. The seismic input was increased by 10% to maintain the factor of safety against sliding of 1.1. No passive soil resistance is considered. The resulting maximum deflection using a coefficient of friction of 0.55 is 0.03" at the base of the NI basemat (EL 60.5'). This is negligible sliding during the seismic event, and no passive soil resistance is necessary from the backfill (side soil). Therefore, it can be concluded that the Nuclear Island is stable against sliding, and there is no quality requirement for the backfill material adjacent to the NI (side soil) to maintain stability against sliding.

3.8.5.5.~~5~~6 Effect of Nuclear Island Basemat Uplift on Seismic Response

The effects of basemat uplift were evaluated using an east-west lumped-mass stick model of the nuclear island structures supported on a rigid basemat with nonlinear springs. Floor response spectra from safe shutdown earthquake time history analyses, which included basemat uplift, were compared to those from analyses that did not include uplift. The comparisons showed that the effect of basemat uplift on the floor response spectra is not significant.

3.8.7 References

56. Hsai-Yang Fang, "Foundation Engineering Handbook," Second Edition, 1991, Van Nostrand Reinhold.

Table 3.8.5-2

FACTORS OF SAFETY FOR FLOTATION, OVERTURNING AND SLIDING OF NUCLEAR ISLAND STRUCTURES

Environmental Effect	Factor of Safety[1]
Flotation	
High Ground Water Table	3.7
Design Basis Flood	3.5
Sliding	
Design Wind, North-South	~~23.2~~ 14.0
Design Wind, East-West	~~17.4~~ 10.1
Design Basis Tornado, North-South	~~12.8~~ 7.7
Design Basis Tornado, East-West	~~10.6~~5.9
Safe Shutdown Earthquake, North-South	~~1.28~~1.1[2]
Safe Shutdown Earthquake, East-West	~~1.33~~1.1[2]
Overturning	
Design Wind, North-South	51.5
Design Wind, East-West	27.9
Design Basis Tornado, North-South	17.7
Design Basis Tornado, East-West	9.6
Safe Shutdown Earthquake, North-South	~~1.35~~1.77[3]
Safe Shutdown Earthquake, East-West	~~1.12~~1.17[3]

Note:

1. Factor of safety is calculated for the envelope of the soil and rock sites described in subsection 3.7.1.4.
2. No passive pressure is considered. From non-linear sliding analysis using friction elements the horizontal movement is negligible (< 0.03"). ~~Factor of safety is shown for soils below and adjacent to nuclear island having angle of internal friction of 35 degrees.~~
3. No passive pressure considered. ~~ASCE/SEI 43-05, Reference 42, recognizes that there is considerable margin beyond that given by the moment balance formula and permits a nonlinear rocking analysis. The nonlinear (liftoff allowed) time history analysis described in Appendix 3G.10 showed that the nuclear island basemat uplift effect is insignificant. Further, these analyses were performed for free field seismic ZPA input as high as 0.5g without significant uplift. Therefore the factor of safety against overturning is greater than 1.67 (0.5g/0.3g).~~

APPENDIX 3G NUCLEAR ISLAND SEISMIC ANALYSES

Modify the second paragraph in Section 3.G.1 changing Reference number.

Analyses were performed in accordance with the criteria and methods described in Section 3.7. Section 3G.2 describes the development of the finite element models. Section 3G.3 describes the soil structure interaction analyses of a range of site parameters and the selection of the parameters used in the design analyses. Section 3G.4 describes the fixed base and soil structure interaction dynamic analyses and provides typical results from these dynamic analyses. In Reference ~~3~~6 are provided a summary of dynamic and seismic analysis results (i.e., modal model properties, accelerations, displacements response spectra) and the nuclear island liftoff analyses. The seismic analyses of the nuclear island are summarized in a seismic analysis summary report. Deviations from the design due to as-procured or as-built conditions are acceptable based on an evaluation consistent with the methods and procedures of Sections 3.7 and 3.8 provided the following acceptance criteria are met:

3G.5 References

6. APP-GW-GLR-044, "Nuclear Island Basemat and Foundation," Revision 1, Westinghouse Electric Company LLC

PRA Revision:

None

Technical Report (TR) Revision:

~~None~~

The following modifications are Post Revision 1.

Modify the last paragraph of Section 2.4.1, 2D SASSI Analyses to the following:

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 horizontal analyses also used for the member forces in Table 2.4-1. Horizontal 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 time history to reflect the seismic response for the other two potential governing soil cases UBSM and SM.

Modify Section 2.9 as follows:

2.9 Nuclear island stability

The factors of safety associated with stability of the nuclear island (NI) are shown in Table 2.9-1 for the following cases:

- Flotation Evaluation for ground water effect and maximum flood effect
- The Nuclear Island to resist overturning during a Safe Shutdown Earthquake (SSE)
- The Nuclear Island to resist sliding during the SSE
- The Nuclear Island to resist overturning during a tornado/wind/hurricane condition
- The Nuclear Island to resist sliding during a tornado/wind/hurricane condition.

~~The factors of safety for sliding and overturning for the SSE are calculated for each soil case for the base reactions in terms of shear and bending moments about column lines 1, 11, I and the west side of the shield building at each time step of the seismic time history. The seismic time history analysis used the ANSYS computer code and the NI20 model.~~ The minimum stability factors of safety values are reported in Table 2.9-1. The method of analysis is as described in subsection 3.8.5.5 of the DCD and the coefficient of friction of 0.55 is used. The governing friction value at the interface zone is a thin soil layer (soil on soil) under the mud mat assumed to have a friction angle of 35 degrees. The Combined License applicant will provide the site specific angle of internal friction for the soil below the foundation. ~~For seismic overturning no passive pressure was considered. For sliding partial passive pressure is considered (less than 35% NS and 48% EW). The relationship between passive pressure and displacement at grade is shown in Figures 2.9-1 and 2.9-2. These curves are based on the methodology given in Reference 10.~~

The factors of safety for sliding and overturning for the SSE are calculated for each soil case for the base reactions in terms of shear and bending moments about column lines 1, 11, I and the west side of the shield building at each time step of the seismic time history. 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 time history to reflect the seismic response for the other two potential governing soil cases UBSM and SM. The firm rock, soft rock, and soft soil cases have higher factors of safety against sliding and therefore not considered.

The seismic time history analysis used the ANSYS computer code and the NI20 model. The minimum stability factors of safety values are reported in Table 2.9-1. For seismic overturning no passive pressure was considered. For sliding partial passive pressure is considered. Two soil cases are considered for sliding, the soil parameters used for design (friction angle of 35°, and submerged weight of 87.6 pcf), and a lower bound soil density (friction angle of 35°, and submerged weight of 60 pcf). For the design case the amount of passive pressure required to meet the 1.1 factor of safety is 40% for the North-South seismic event, and 47% of the East-West excitation of full passive pressure. For the lower bound case the amount of passive pressure required to meet the 1.1 factor of safety is less than 53% for the North-South seismic event, and 64% of the East-West excitation of full passive pressure. The relationship between passive pressure and displacement at grade is obtained based on the methodology given in Reference 10. The relationship between passive pressure and displacement at grade is shown in Figures 2.9-1 and 2.9-2. The maximum Nuclear Island displacement of the Nuclear Island at grade to develop the required passive resistance is 0.5" for the design case, and 2.3" for the lower bound case. These deflections are based on conservative equivalent static analysis. This will result in large deflections since the seismic loads are considered to be constant and do not reflect the short time duration that they exist during the seismic event. A more realistic non-linear analysis with sliding friction elements using a 2D ANSYS model was performed. The 2D ANSYS model that was used to study the basemat uplift (see Subsection 2.4.2). This 2D non-linear model is for the East-West direction. There is no need to consider the North-South direction since the NI deflections calculated to maintain a factor of safety of 1.1 is largest in the East West direction. This model was modified introducing friction elements along the bottom of the basemat and soil media interface. Direct time integration analysis was performed with vertical uplift and sliding allowed. The three cases that have the lowest factor of safety related to sliding were evaluated. These three cases are HR, UBSM, and SM. The seismic input was increased by 10% to maintain the factor of safety against sliding of 1.1. No passive soil resistance is considered. The resulting maximum deflection using a coefficient of friction of 0.55 is 0.03" at the base of the NI basemat (EL 60.5'). This is negligible sliding during the seismic event, and no passive soil resistance is necessary from the backfill (side soil). Therefore, it can be concluded that the Nuclear Island is stable against sliding, and there is no quality requirement for the backfill material adjacent to the NI (side soil) to maintain stability against sliding.

Table 2.9-1 – Factors of Safety Related to Stability of AP1000 NI

Load Combination	Sliding		Overturning		Flotation	
	Factor of Safety	Limit	Factor of Safety	Limit	Factor of Safety	Limit
D + H + B + W	Design Wind					
North-South	~~23.2~~14.0	1.5	51.5	1.5	–	–
East –West	~~17.4~~10.1	1.5	27.9	1.5	–	–
D + H + B + W_t	Tornado Condition					
North-South	~~12.8~~7.7	1.1	17.7	1.1	–	–
East –West	~~10.6~~5.9	1.1	9.6	1.1	–	–
D + H + B + W_h	Hurricane Condition					
North-South	~~18.1~~10.3	1.1	31.0	1.1	–	–
East –West	~~14.2~~8.1	1.1	16.7	1.1	–	–
D + H + B + E_S	SSE Event					
North-South	1.1(2)	1.1	–	–	–	–
East-West	1.1(2)	1.1	–	–	–	–
Line 1	–	–	1.77(1)	1.1	–	–
Line 11	–	–	1.~~92~~93(1)	1.1	–	–
Line I	–	–	1.17(1)	1.1	–	–
West Side Shield Bldg	–	–	1.44(1)	1.1	–	–
	Flotation					
D + F	–	–	–	–	3.51	1.1
D + B	–	–	–	–	3.70	1.5

Notes:

(1) No passive pressure is considered.

(2) No passive pressure is considered. From non-linear sliding analysis using friction elements the horizontal movement is negligible (< 0.03"). ~~Factor of safety for sliding considers that the soils below and adjacent to the nuclear island have an angle of internal friction of 35 degrees. Also, the coefficient of friction for soils below the nuclear island is equal to 0.55. The maximum deflection of the nuclear island needed to develop the required passive pressures are less than 0.15 inch for hard rock, less than or equal to 0.5 inch for upper bound soft to medium (UBSM) and soft to medium (SM) soil conditions. The other soil conditions have smaller deflection requirements than the UBSM and SM cases.~~

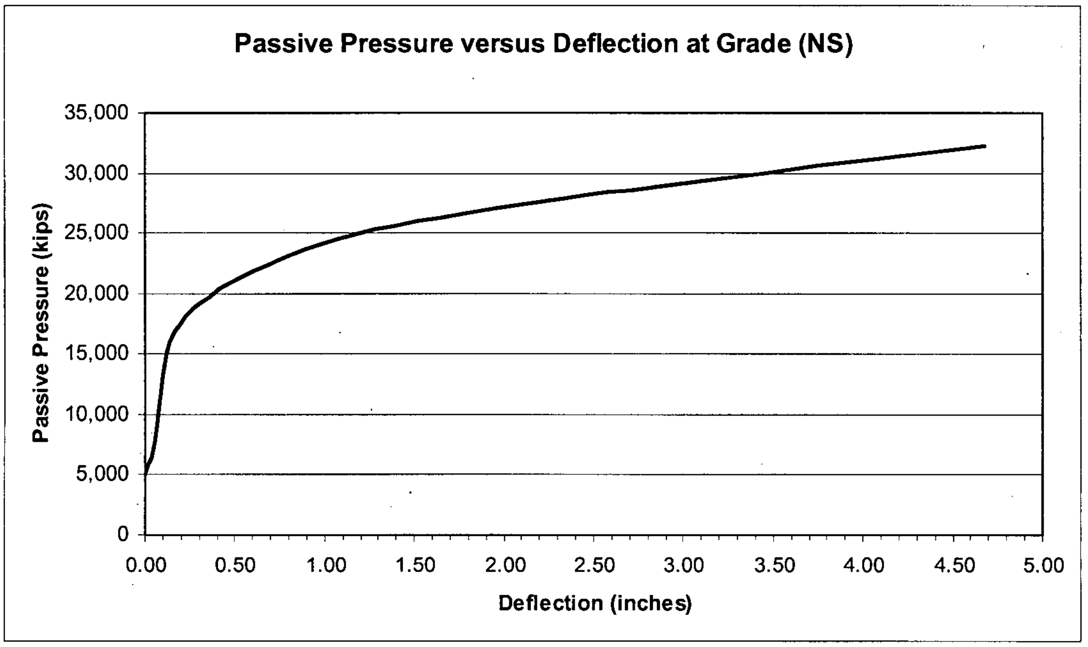

Figure 2.9-1 – Passive Pressure versus Deflection at Grade (North-South Excitation)

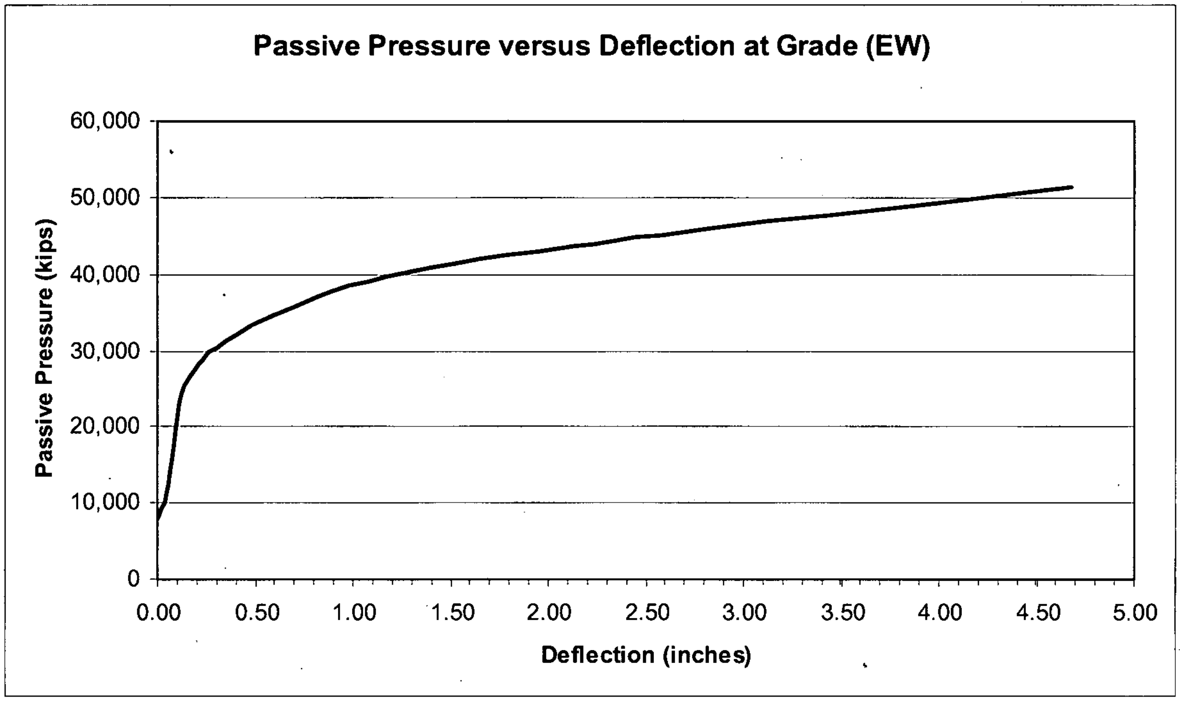

Figure 2.9-2 – Passive Pressure versus Deflection at Grade (East-West Excitation)

4. REFERENCES

10. HSAI-Yang Fang, "Foundation Engineering Handbook," Second Edition, 1991, Van Nostrand Reinhold.

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR85-SEB1-37
Revision: 2

Question:

In Section 5.1, entitled "Proposed Revisions to DCD Section 2.5," the DCD mark up of Section 2.5.4.6.2 states that "Seismic stability requirements are satisfied if the soil layers below and adjacent to the nuclear island foundation are composed predominantly of rock or sand and rock (gravel), or sands that can be classified as medium to dense (standard penetration test having greater than 10 blows per foot)." This criterion of 10 blows per foot, places the soil at the boundary of loose to medium and not medium to dense soils. Also, using the criteria of 10 blows per foot places the soil friction angle below the minimum required 35 degrees for the NI stability calculations. Therefore, provide the technical justification for the adequacy of the blow count criteria and demonstrate that it is consistent with the minimum soil friction angle of 35 degrees used in design and stability calculations. The soil friction angle should also be specified separately as a site interface criteria for soil in DCD Table 2-1 and in DCD Tier 1.

Additional Request (Revision 1)

The RAI response indicates that the phrase "medium to dense" will be revised to read "medium or dense" when describing the sands for which the stability requirements were satisfied. This change addresses the first part of the original RAI. However, this change does not address the second part of the RAI which indicates that using the criterion of 10 blows per foot for medium or dense sands, places the soil friction angle below the minimum required 35 degrees for the nuclear island stability evaluations. Therefore, Westinghouse is requested to revise the blow count criteria or to provide the technical justification for the adequacy of the 10 blows per foot criteria and demonstrate that it is consistent with the minimum soil friction angle of 35 degrees used in the design and stability calculations.

Additional Request (Revision 2)

Based on the information provided in Revision 1 to this RAI response, the remaining concern with the second part of the RAI relates to the acceptable blow count for the soil beneath the basemat and the soil used as backfill at the side of the foundation/walls. The Westinghouse response indicates that a change to the DCD in the second paragraph of subsection 2.5.4.6.2 will be made to indicate that for medium sand a blow count greater than 10 blows per foot, or for dense sand a blow count greater than 30 blows per foot is representative of acceptable backfill. While the blow count of 10 blows per foot has been demonstrated as being acceptable for the backfill material at the side walls of the foundation for the types of soils listed, this criterion has not been demonstrated as being acceptable for the soil beneath the basemat. In the response to other RAIs and as specified in Table 2.0-1 of the DCD, the criterion for the soil beneath the basemat is that a soil internal friction angle of 35 degrees will be demonstrated by the COL applicant. Therefore, the proposed mark-up of Section 2.5.4.6.2 of Revision 17 should be revised to reflect the criterion of 35 degrees for the soil internal friction angle.

In addition, the prior RAI response indicated that the soil friction angle will be specified separately as a site interface criterion for soil in DCD Table 2-1 and in DCD Tier 1 for soils below the NI. Currently, DCD Rev. 17, Tier 1, Table 5.0-1 does not provide this criterion. The RAI response also does not provide the mark-up for this criterion. Therefore, Westinghouse is requested to include the soil friction angle site parameter requirement of 35 degrees beneath the foundation in DCD Tier 1, Table 5.0-1, consistent with criterion in DCD Tier 2, Table 2-1.

Westinghouse Response:

References 1 and 2 provide the technical justification linking the SPT blow count to the internal angle of friction. Table RAI-TR85-SEB1-37-1 (shown below) provides the illustration that a Medium sand with a SPT blow count of 10-30 blows/ft is consistent with an internal angle of friction ranging from 32 to 36 degrees. The NRC is correct that the blow count of greater than 10 places the soil on the boundary of loose to medium. The description "medium to dense" was not intended to define the minimum but rather to state the sands for which the stability requirements were satisfied. The DCD will be clarified to read "medium or dense".

The soil friction angle will be specified separately as a site interface criterion for soil in DCD Table 2-1 and in DCD Tier 1. However, this is limited to soils below the nuclear island. Where side soils do not satisfy the internal friction angle of 35 degrees, DCD subsection 2.2.5.4.6.2 requires the Combined License applicant to evaluate the seismic stability against sliding as described in subsection 3.8.5.5.3 using the site-specific soil properties. In many cases, such as cases where groundwater is significantly below grade, seismic stability can be demonstrated without taking credit for the resistance of the side soils.

Table RAI-TR85-SEB1-37-1 – Soil Properties

Soil Types	Standard Penetration Test N - Blows/ft	Angle of Internal Friction Φ - degrees
Sands (1)	[Ref. 1, Table 10, page 294]	[Ref. 2, Section 5, Table 2] (2)
Very Dense	> 50	41° to 46°
Dense	30-50	36° to 41°
Medium	10-30	32° to 36°

Loose	4-10	28.5° to 32°
Very Loose	0-4	< 28.5°

Notes to Table RAI-TR85-SEB1-37-1:

(1) As stated in Reference 2 "for dry silts and very silty sands values of Φ are usually 2 to 6 deg. less than those shown in Table 2." The Table 2 values are those given in this table for dry sand composed primarily for quartz. Also it is stated in Reference 2, "for silts and very silty sands below the groundwater table, values of Φ are, for the great majority of cases, considerably less (one-third to one-half) than the values for dry material." Reference 1, page 86, states that angle of friction values for silt and silty sand "obtained from slow-shear tests range from about 27° to 30° for the loose state, and 30° to 35° for the dense state. These values are almost as great as those for sand."

(2) Using Table 7, page 82 of Reference 1, a description of the soil can be obtained based on the angle of friction. For the loose sand as well as the combined category of dense/very dense sand, the sand with an angle of friction in the lower range, the sand is made up of uniform round grains, for the upper range it has angular grains that are well graded.

Westinghouse Response (Revision 1):

A change to the DCD in the second paragraph of subsection 2.5.4.6.2 will be made to indicate that for medium sand a blow count greater than 10 blows per foot, or for dense sand a blow count greater than 30 blows per foot is representative of acceptable backfill. The standard penetration test having greater than 10 blows per foot provides the means of assuring that the side soil is competent. There has been no requirement placed on the applicant that the backfill adjacent to the Nuclear Island walls below grade must have a friction angle of 35° or greater. However, it is anticipated that the friction angle will be above 32° for the side soil backfill based on Table RAI-TR85-SEB1-37-1. Recognizing that not all of the passive pressure is required, as discussed in RAI-TR85-SEB1-34, 35, and 40, the sand backfill that ranges from medium to very dense, as well as sand and gravel, and rock provide adequate passive pressures as seen in Table RAI-TR85-SEB1-37-2, noting that Case 15 is used for the AP1000 design. Further, it is noted in the DCD that the COL applicant must do the following:

- Per Subsection 2.5.4.6.2, Revision 17, "If the soil below and adjacent to the exterior walls is made up of clay, sand and clay, or other types of soil other than those classified above as competent, then the Combined License applicant will evaluate the seismic stability against sliding as described in subsection 3.8.5.5.3 using the site-specific soil properties."

- Per Subsection 2.5.4.6.7, Revision 17, "Earth Pressures – The Combined License applicant will describe the design for static and dynamic lateral earth pressures and hydrostatic groundwater pressures acting on plant safety-related facilities using soil parameters as evaluated in previous subsections."

- Per Subsection 2.5.4.6.9, Revision 17, "Static and Dynamic Stability of Facilities – Soil characteristics affecting the stability of the nuclear island will be addressed including foundation rebound, settlement, and differential settlement."

- Per Table 2-1 (Tier 2) and Table 5.0-1 (Tier 1), Revision 17, the minimum soil angle of internal friction must be greater than or equal to 35 degrees below the footprint of the Nuclear Island at its excavation depth.

With these COL required actions, it can be further verified that the backfill is competent and have adequate passive pressure to meet the seismic stability requirements.

Westinghouse Response (Revision 2):

The response given in Revision 1 does indicate the change to the second paragraph of DCD Subsection 2.5.4.6.2. This response should have stated that the change was to the third paragraph. Westinghouse is in agreement that the blow count criterion is not acceptable for the soil beneath the basemat. For the soil beneath the basemat a soil internal friction angle of 35 degrees is specified in Table 2-1. As requested, Westinghouse will revise the second paragraph adding at the end:

> "The Combined License applicant is to demonstrate that the minimum soil angle of internal friction is greater than or equal to 35 degrees below footprint of nuclear island at its excavation depth as specified in Table 2-1. If the minimum soil angle of internal friction is below 35 degrees, then the Combined License applicant will evaluate the seismic stability against sliding as described in subsection 3.8.5.5.3 using the site specific soil properties."

Since it has been shown from non-linear sliding stability analyses that the Nuclear Island has negligible movement at the bottom of the Nuclear Island basemat (see RAI-TR85-SEB1-010, Revision 2) without consideration of passive pressure, it is no longer necessary to define properties for materials adjacent to nuclear island exterior walls to demonstrate they provide passive earth pressures greater than or equal to those used in the seismic stability evaluation for sliding of the Nuclear Island. Therefore, the third paragraph is removed.

For the Tier 1 reference, Revision 0 of the RAI response did appear to call for an update of the "Minimum Soil Angle of Internal Friction" for DCD Rev. 16, Tier 1, Table 5.0-1 as well as Tier 2, Table 2-1. However, this change was not incorporated in DCD Rev 17, Tier 1, Table 5.0-1, and at this time, the Tier 1 table should not be changed to include this site interface criterion for soil.

Referring to the statement added above (*The Combined License applicant …*), this alternative evaluation of the seismic stability against sliding can be made without requiring an exemption; specifying this criterion in the Tier 1 table will require an exemption regardless of the depth or result of the evaluation. Westinghouse would prefer to avoid having Combined License applicants apply for exemptions wherever possible.

Table RAI-TR85-SEB1-37-2 – Passive Pressure, El. 60' 6"

Type of Soil		Case	γ_{sub} #/ft^3	γ_{sat} #/ft^3	ϕ deg	P_P psf
Rock	Hard Rock	1	115	175	46	28563
	Rock	2	100	160	46	24933
	Soft Rock	3	100	160	52	34328
	Soft Rock	4	100	160	43	21527
	Soft Rock	5	85	145	52	29331
	Soft Rock	6	85	145	43	18393
Sand & Gravel		7	80	140	36	12634
		8	80	140	32	10675
Sands	Very Dense	9	100	160	46	24933
		10	100	160	41	19597
		11	70	130	46	17674
		12	70	130	41	13891
	Dense	13	88	150	41	17334
		14	88	150	36	13867
		15	87.6	150	35	13229
		16	65	110	36	10236
		17	65	110	36	10236
	Medium	18	68	130	36	10824
		19	68	130	32	9145
		20	60	95	36	9398
		21	60	95	32	7941

References:

1. Terzaghi, Karl, and Ralph B. Peck, Soil Mechanics in Engineering Practice, John Wiley & Sons, Inc., New York, 1948.

2. Gaylord, E.H., et. al., ed, Structural Engineering Handbook, 4th ed, McGraw-Hill, 1997.

Design Control Document (DCD) Revision:

Revisions identified in Revision 0 of this response have been incorporated in DC Rev 17. The following changes to subsection 2.5.4.6.2 of Revision 17 are to be made:

2.5.4.6.2 The Combined License applicant will establish the properties of the foundation soils to be within the range considered for design of the nuclear island basemat.

Properties of Underlying Materials – A determination of the static and dynamic engineering properties of foundation soils and rocks in the site area will be addressed. This information will include a discussion of the type, quantity, extent, and purpose of field explorations, as well as logs of borings and test pits. Results of field plate load tests, field permeability tests, and other special field tests (e.g., bore-hole extensometer or pressuremeter tests) will also be provided. Results of geophysical surveys will be presented in tables and profiles. Data will be provided pertaining to site-specific soil layers (including their thicknesses, densities, moduli, and Poisson's ratios) between the basemat and the underlying rock stratum. Plot plans and profiles of site explorations will be provided. The Combined License applicant is to demonstrate that the minimum soil angle of internal friction is greater than or equal to 35 degrees below footprint of nuclear island at its excavation depth as specified in Table 2-1. If the minimum soil angle of internal friction is below 35 degrees, then the Combined License applicant will evaluate the seismic stability against sliding as described in subsection 3.8.5.5.3 using the site specific soil properties.

~~Properties of Materials Adjacent to Nuclear Island Exterior Walls – A determination of the static and dynamic engineering properties of the surrounding soil will be made to demonstrate they are competent and provide passive earth pressures greater than or equal to those used in the seismic stability evaluation for sliding of the nuclear island. Seismic stability requirements are satisfied if the soil layers below and adjacent to the nuclear island foundation are composed predominantly of rock, or sand and rock (gravel), or sands that can be classified as medium to dense (standard penetration test having greater than 10 blows per foot). If the soil below and adjacent to the exterior walls is made up of clay, sand and clay, or other types of soil other than those classified above as competent, then the Combined License applicant will evaluate the seismic stability against sliding as described in subsection 3.8.5.5.3 using the site-specific soil properties.~~

Laboratory Investigations of Underlying Materials – Information about the number and type of laboratory tests and the location of samples used to investigate underlying materials will be provided.

Discussion of the results of laboratory tests on disturbed and undisturbed soil and rock samples obtained from field investigations will be provided.

PRA Revision:
None

Technical Report (TR) Revision:

Section 5 of Technical Report 85 is being deleted from TR 85 Revision 1 as stated in the response to RAI-TR85-SEB1-017.