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December 22, 2008

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-SEB1-07, Rev. 1
RAI-TR85-SEB1-10, Rev. 1
RAI-TR85-SEB1-12, Rev. 1
RAI-TR85-SEB1-34, Rev. 2
RAI-TR85-SEB1-35, Rev. 1
RAI-TR85-SEB1-40, 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,

for

Robert Sisk, Manager
Licensing and Customer Interface
Regulatory Affairs and Standardization

/Enclosure

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

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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-SEB1-07
Revision: ~~0~~1

Question:

Section 2.4.1 discusses the 2D SASSI analyses performed to obtain loads for the NI dynamic stability evaluation and to determine the governing soil cases. Since the 2D SASSI analyses are only two dimensional and lack the effect of all three dimensions, provide a comparison of the soil bearing pressures and shear/overturning moments from the 2D SASSI results to the 3D SASSI results for the same soil conditions.

Additional Request (Revision 1):

In view of the changes made to many of seismic models and analyses, Westinghouse is requested to confirm (1) that the AP1000 design is now relying on the updated 3D NI20 response spectrum analysis, which envelopes all soil cases, using ANSYS, for stability evaluations (sliding and overturning). For stability evaluations, the 2D SASSI analysis is no longer used. For the bearing pressure calculation, now only the 2D ANSYS stick model analysis is used and it is performed for all 6 soil cases, and so the 2D SASSI analysis is no longer needed for this case as well, because previously it was needed to identify the governing soil case. It appears that the 2D SASSI analysis is only being utilized to confirm the adequacy of the 2D ANSYS stick model analysis which is used to calculate the maximum soil bearing pressure demand.

The RAI response states that "The 2D SASSI analyses results also show that the seismic interface displacements between the adjacent buildings and the Nuclear Island is less than the 2" gap at foundation level and the 4" gap at superstructure. These cases included a supplemental case for weak top soil (750 fps) over hard rock." Westinghouse is requested to explain in greater detail (2) the meaning of a supplemental case for weak top soil (750 fps) over bedrock, whether this is another soil case in addition to the six cases already being considered for seismic and foundation analyses. Describe how is it considered in all three of the analyses: stability calculations, bearing pressure calculations, and design of the basemat. Also, explain the relationship of the use of 750 fps with the DCD Tier 1 criteria of a minimum shear wave velocity of 1,000 fps for site interface.

Westinghouse Response:

The soil bearing pressures from the 3D SASSI results were reviewed to see if the comparison requested by this RAI could be provided. Due to the coarser modeling of the soil in the 3D model than in the 2D model, soil pressure results from the 3D model show significant variation between adjacent elements and comparisons to the 2D results are not meaningful.

A time history analysis was performed using the AP1000 nuclear island 3D shell model NI20. The time history input is developed from the envelope of the broadened floor response spectra of the six site profiles at the edges, along the side walls, and at the center of the AP1000 nuclear island basemat. The shear/overturning moments from this time history analysis, which is an envelope of all soil cases, are compared to the 2D SASSI analysis results. This comparison is given in Tables RAI-TR85-SEB1-07-1 and RAI-TR85-SEB1-07-2. The individual soil cases are compared to the 3D shell model NI20 in Table RAI-TR85-SEB1-07-1, and the maximum values compared in Table RAI-TR85-SEB1-07-2. As seen from this comparison the shear/overturning moments compare closely between the 3D and 2D analyses.

The 2D SASSI analyses results also show that the seismic interface displacements between the adjacent buildings and the Nuclear Island is less than the 2" gap at foundation level and the 4" gap at superstructure. These cases included a supplemental case for weak top soil (750 fps) over hard rock.

Westinghouse Response (Revision 1):

(1) As stated in RAI-TR85-SEB1-04, Revision 1, the stability evaluations (sliding and overturning) are based on the 3D NI20 time history analysis. This is consistent with DCD Table 3G.1-1, Revision 17. The 2D SASSI analyses were used to generate seismic response factors between hard rock and upper-bound-soft-to-medium, and hard and soft-to-medium soil cases. A response spectrum analysis enveloping all the soil cases was not performed for stability evaluation. The time history analysis described in Revision 0 of this response using the 3D NI20 model that enveloped all of the soil cases was too conservative, and resulted in an over prediction of the passive pressure required to meet the stability factor of safety limits. See RAI-TR85-SEB1-10, Revision 1, for further discussion of the stability analysis. Also, see RAI-TR85-SEB1-03, Revision 1, for a discussion of the models used in the bearing pressure calculations.

(2) The supplemental case referred to applies only to analyses performed for the adjacent building structures. For these structures a "weak" top soil (shear wave velocity of 750 fps) over hard rock is considered. The six cases (hard rock and soil cases) considered for the Nuclear Island remain as defined in the AP1000 DCD. The DCD Tier 1 criteria of a minimum shear wave velocity of 1,000 fps for site interface remains unchanged. It applies only to the Nuclear Island.

Table RAI-TR85-SEB1-07-1 – Comparison of 3D and 2D Shears and Moments

Units: 1000 kips & 1000 kip-ft

Seismic Reaction	NI20 Model all Soils Case Forces and Moments	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	116.45	123.75	116.49	118.65	121.48	113.61	73.11
Shear EW	127.51	112.31	113.55	121.88	128.11	124.94	74.34
Vertical	129.68	98.76	98.65	99.63	104.55	112.30	94.48
Moment about Line I	15,178	13,011	12,975	13,320	14,317	14,944	10,115
Moment about SBW side	15,988	14,034	14,038	14,377	15,417	16,125	11,124
Moment about Line 11	19,515	17,506	17,149	16,735	17,461	18,155	13,194
Moment about Line 1	20,149	17,607	17,225	16,754	17,535	18,256	13,342

Notes to Table:

- The shears are at elevation 60.5'
- The overturning moments are about the identified axis at elevation 60.5'

Table RAI-TR85-SEB1-07-2 – Maximum Shear and Moment Comparisons

Units: 1000 kips & 1000 kip-ft

Seismic Reaction	NI20 Model all Soils Case Forces and Moments	2D Enhanced Shield Building
Shear NS	116.45	123.75
Shear EW	127.51	128.11
Vertical	129.68	112.30
Moment about I	15,178	14,944
Moment about SBW	15,988	16,125
Moment about 11	19,515	18,155
Moment about 1	20,149	18,256

Design Control Document (DCD) Revision:
None

PRA Revision:
None

Technical Report (TR) Revision:

See RAI-TR85-SEB1-05, Revision 1, for changes to the technical report.

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR85-SEB1-10
Revision: ~~0~~1

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."

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 Table RAI-TR85-SEB1-01a 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-SEB1-01b. 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-SEB1-01a: 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 μ = 0.55	1.1	1.1	1.1	1.1	1.1	1.1
Sliding E-W SSE μ = 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-SEB1-01b: 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 μ = 0.55	0.11	0.10	0.07	0.12	0.12	0.08
Sliding E-W SSE μ = 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-SEB-01-02 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-SEB-01-02: 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-SEB1-35-1 and RAI-TR85-SEB1-35-2 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.

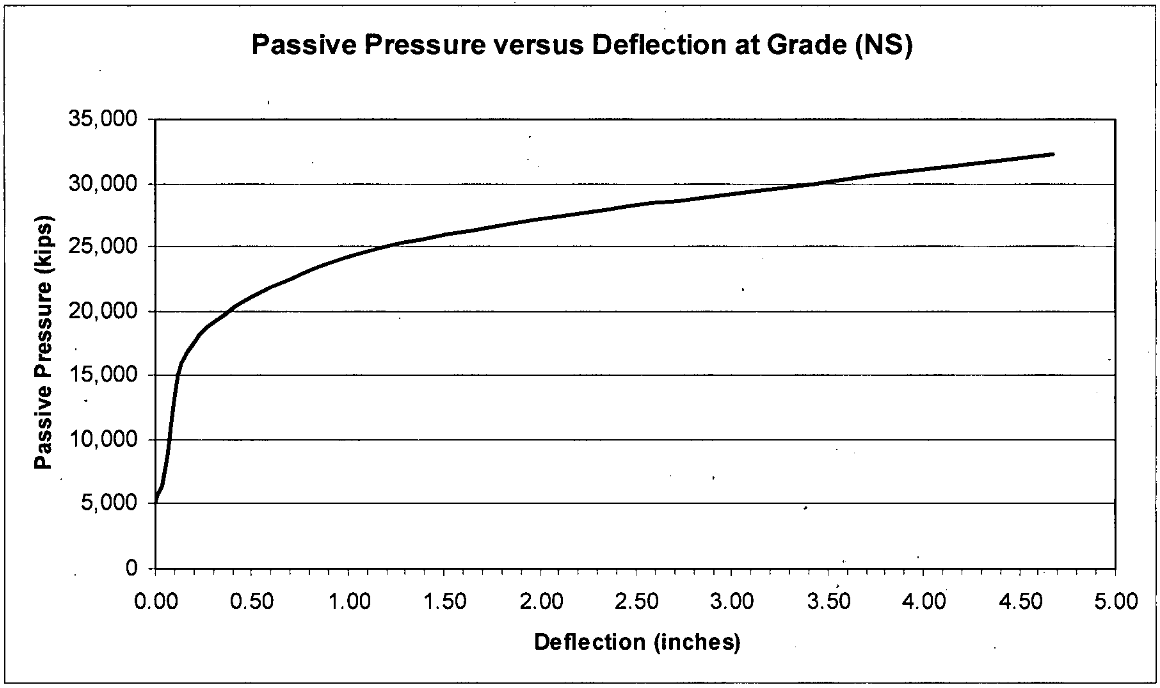

Figure RAI-TR85-SEB1-35-1 – Passive Pressure versus Deflection at Grade (North-South Excitation)

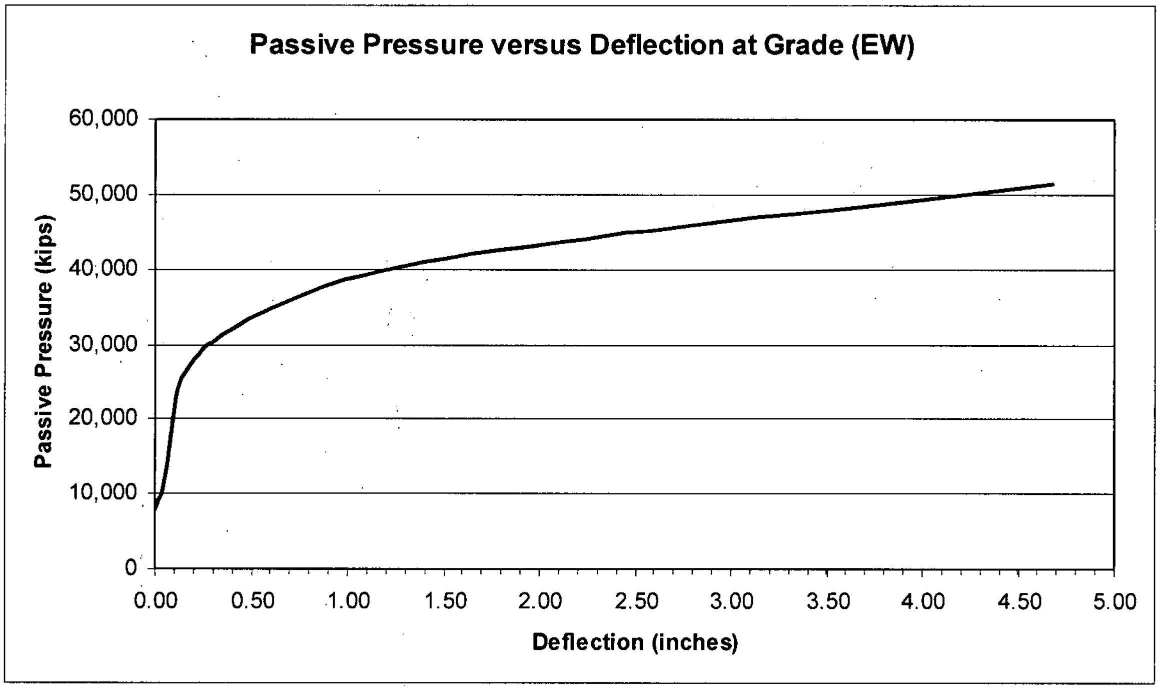

Figure RAI-TR85-SEB1-35-2 – Passive Pressure versus Deflection at Grade (East-West Excitation)

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-SEB1-35-3 and RAI-TR85-SEB1-35-4 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-SEB1-35-1 to RAI-TR85-SEB1-35-3 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.

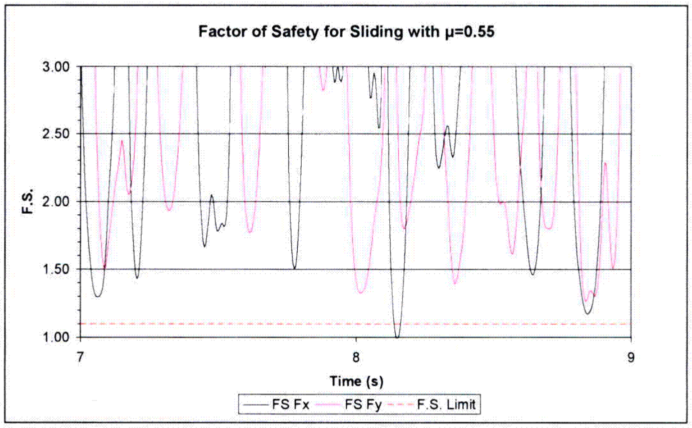

Figure RAI-TR85-SEB1-35-3 - North-South FS without Passive Pressure

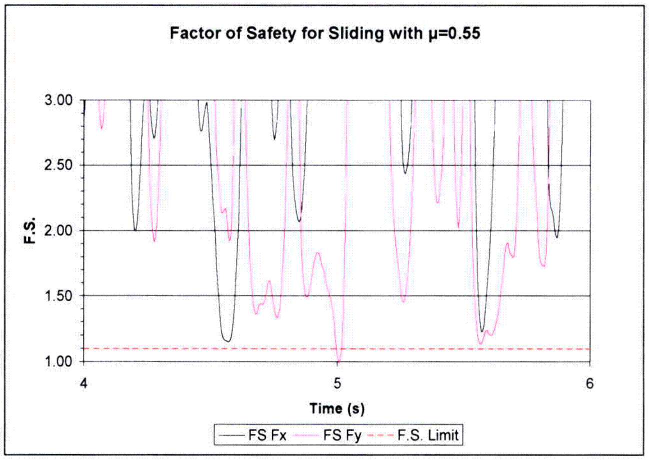

Figure RAI-TR85-SEB1-35-4 – East-West FS without Passive Pressure

Table RAI-TR85-SEB1-35-1 - 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-SEB1-35-2 - 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-SEB1-35-3 - 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.

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 4.4.1.1.1.1, Revision 17, to read as follows:

The waterproof function of the membrane is not safety-related; however, the membrane between the the mudmats must transfer horizontal shear forces due to seismic (SSE) loading. This function is seismic ~~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 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 coefficient of friction of 0.~~70~~55, and internal friction angle of 35° for the soil. 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.

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
Design Wind, East-West	17.4
Design Basis Tornado, North-South	12.8
Design Basis Tornado, East-West	10.6
Safe Shutdown Earthquake, North-South	~~1.28[2]~~1.1[2]
Safe Shutdown Earthquake, East-West	~~1.33[2]~~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[3]~~1.17[3]

Note:

1. Factor of safety is calculated for the envelope of the soil and rock sites described in subsection~~ subsection~~ 3.7.1.4.
2. Factor of safety is shown for soils below and adjacent to nuclear island having 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.

3. ~~ASCE/SEI 43-05, Reference 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).~~No passive pressure is considered.

PRA Revision:

None

Technical Report (TR) Revision:

~~None~~ 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 reactions in this table are used estimate the effect of soils in the evaluation of nuclear island stability described in section 2.9.~~ 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 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~~shown in Table 2.4-2.~~ 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 ~~with the exception that the sliding resistance is based on the friction force developed between the basemat and the foundation using a~~and the coefficient of friction of ~~0.70~~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. ~~In the case of a rock foundation, the mud mat will interlock with the rock, and therefore, the friction angle will be at least 55 degrees.~~ 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.

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	1.5	51.5	1.5	–	–
East –West	17.4	1.5	27.9	1.5	–	–
D + H + B + W_t	Tornado Condition					
North-South	12.8	1.1	17.7	1.1	–	–
East –West	10.6	1.1	9.6	1.1	–	–
D + H + B + W_h	Hurricane Condition					
North-South	18.1	1.1	31.0	1.1	–	–
East –West	14.2	1.1	16.7	1.1	–	–
D + H + B + E_S	SSE Event					
North-South	~~1.28~~1.1[(2)]	1.1	–	–	–	–
East-West	~~1.33~~1.1[(2)]	1.1	–	–	–	–
Line 1	–	–	~~1.39~~1.77	1.1	–	–
Line 11	–	–	~~1.42~~1.92	1.1	–	–
Line I	–	–	~~1.07~~1.17[(1)]	1.1	–	–
West Side Shield Bldg	–	–	~~1.06~~1.44[(1)]	1.1	–	–
	Flotation					
D + F	–	–	–	–	3.51	1.1
D + B	–	–	–	–	3.70	1.5

Notes:

(1) ~~Considering active and passive soil pressures on the external walls below grade, the minimum factor of safety against overturning (1.07 and 1.06) increases to 1.12 (Line I) & 1.10 (West Side of Shield Building). This factor of safety meets the requirement of 1.1 based on the conservative moment balance formula treating the seismic moment as static loads. ASCE/SEI 43-05, Reference 7, recognizes that there is considerable margin beyond that given by the moment balance formula. Reference 7 permits a nonlinear rocking analysis. A nonlinear (liftoff allowed) time history analysis is described in Section 2.4.2 showing 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).~~No passive pressure is considered.

(2) 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.

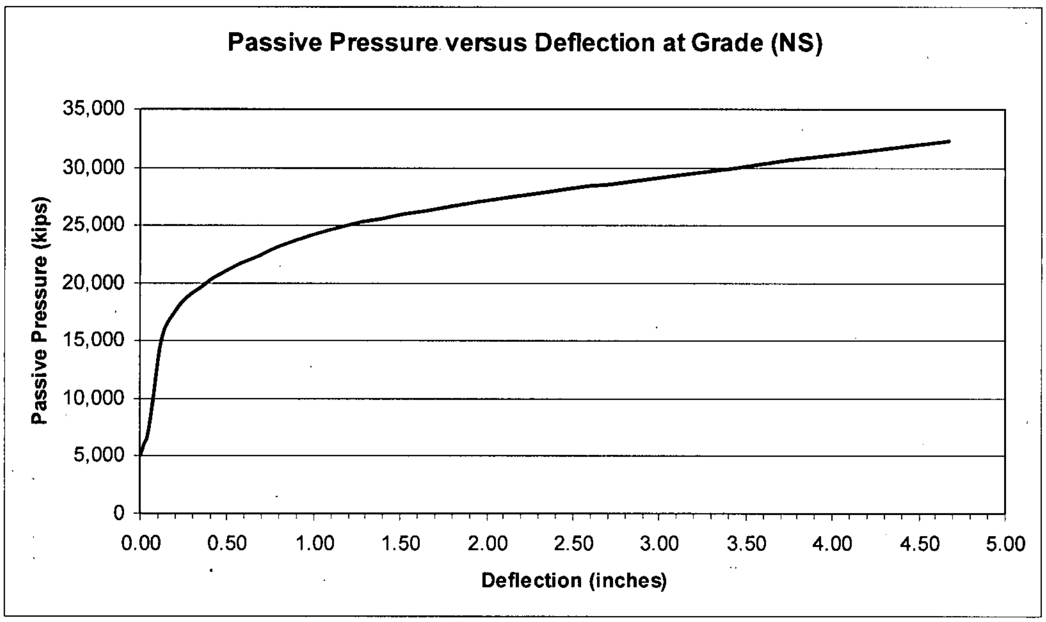

Figure 2.9-1 – Passive Pressure versus Deflection at Grade (North-South Excitation)

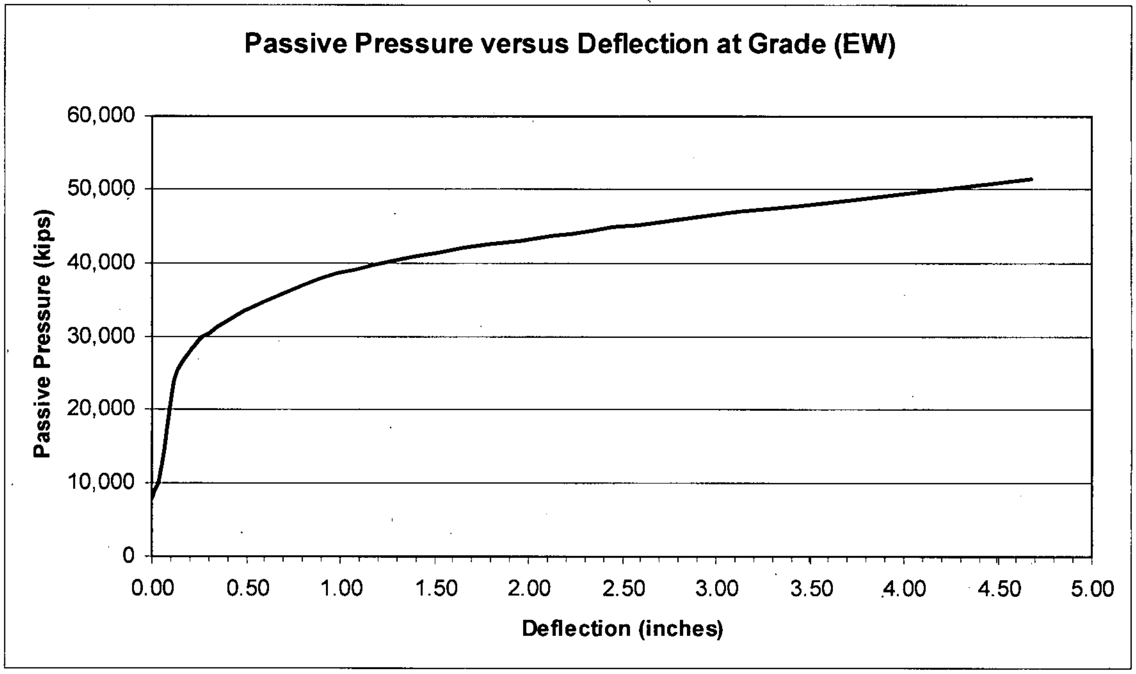

Figure 2.9-2 – Passive Pressure versus Deflection at Grade (East-West Excitation)

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-SEB1-12
Revision: 1~~0~~

Question:

Section 2.4.2 indicates that the 2D ANSYS nonlinear time history analyses were performed for dead load plus EW and vertical SSE loads. The analyses and results are described in greater detail in Section 7.0 of TR-03, Revision 0. The following information is requested relating to these analyses:

a. Section 2.4.2 states that a comparison of floor response spectra and the maximum member forces and moments for the linear and nonlinear soil spring cases show that liftoff has an insignificant effect on the SSE response. This statement needs to be clarified or revised because based on Figure 2.4-5, for the soft to medium soil case, the bearing reactions increase by 16%, which suggest that the nonlinear effect is important and does need to be considered.

b. Based on the 3D ANSYS equivalent static approach, Figure 2.6-7 shows that more than 50% of the basemat lifts up and the bearing pressure for this case is much higher than that for the 2D ANSYS nonlinear case. Since the 2D ANSYS model represents the basemat as a rigid beam and has some other simplifying assumptions, explain why the maximum bearing pressure from the more accurate 3D ANSYS model, which includes the flexibility of the basemat and considers the three dimensional features of the NI structures, is not used to define the maximum bearing pressure in addition to its use for designing the basemat.

c. The 2D ANSYS nonlinear time history analysis only considers uplift between the soil and the NI basemat. Explain how the potential uplift and sliding between the containment internal structures (CIS) concrete base and the steel containment shell is addressed for the various soil conditions. Also, provide the basis for the statement in Section 3.8.2.1.2 of the DCD which indicates that the shear studs provided between the containment and concrete basemat below the containment are not required for design basis loads, but provide additional margin for earthquakes beyond the SSE. Have analyses been performed for the AP1000 design (based on the SSE) to demonstrate that there is no uplift or sliding of the containment with respect to the basemat for the various soil conditions? If not, explain why.

d. ~~d.~~ Section 7.2 of TR-03, Revision 0, indicates that the soil damping is low (2%) for the soft rock case, 5% for the soft-to-medium case and increases to 30% for the soft soil case. Since radiation damping at layered soil sites may in fact be low, provide the technical basis for the use of these damping values. If the conservatism of the selected values is not clearly demonstrated, then these should be considered as site interface parameters to be met by the COL applicant.

Additional Request (Revision 1)

The staff reviewed the RAI response provided in Westinghouse letter dated 10/19/07. As a result of the information provided in this response, the staff requests Westinghouse to address the following items which correspond to parts b through d identified in the original RAI.

Part b. The RAI response provides qualitative information which suggests that the higher bearing pressure loads in the 3D ANSYS equivalent static non-linear analysis versus the 2D ANSYS dynamic time history analysis is only due to the conservatism inherent in the equivalent static non-linear method. The staff requests Westinghouse to confirm this quantitatively.

c. Regarding the question about consideration of various soil sites for the lift-off and sliding analysis of the containment internal structures and the steel containment, the RAI response indicated that the overturning moments for soil sites are similar to those for hard rock. From Table 2.4-2 of TR-85 (2D SASSI) for Mxx, SRSS and from Figures 4.4.1-4 and 4.4.1-5 of TR-03 (2D-ANSYS analysis) this conclusion is not evident. Therefore, Westinghouse is requested to tabulate the overturning moments about N-S and E-W at the bottom of the containment internal structures and the containment for all of the soil cases analyzed and demonstrate that they are indeed bounded by the hard rock case used in the previous hard rock design, or to include all of the other soil cases in updated analyses.

d. The RAI response indicates that Section 7.2 of TR-03, Rev. 1 has been revised to show only the damping value of 5% used in the 2D ANSYS lift-off analyses for the soft-to-medium site. While this was revised in Table 7-2-1 of TR-03, Rev. 1, the paragraph in Section 7.2 that references this table still discusses the acceptability of damping equal to 30% for a soft soil case. Westinghouse is requested to revise the text as well to be consistent with their RAI response. In addition, the staff notes that Section 7.2 of TR-03 has a subsection entitled Basemat Displacements which also refers to a Figure 7.2-1; however, no such figure could be located. Westinghouse is requested to provide this information.

Westinghouse Response:

a. This statement has been clarified as shown in the revisions to the Technical Report shown below. The comparison of the maximum member forces and moments for the linear and nonlinear soil spring cases is discussed in the response to RAI-TR85-SEB1-14.

b. The bearing pressures and lift-off in the 3D ANSYS non-linear analyses exceed those from the 2D ANSYS time history analyses. This is due to the conservatism inherent in the equivalent static method. The 2D ANSYS bearing pressures in Figure 2.6-7 show fairly uniform spacing between contours. In addition the location of maximum bearing pressure in both the 2D and 3D analyses is below the west side of the shield building where the nuclear island is nearly rigid due to the thickness of mass concrete and the stiffening of the superstructure. Thus the flexibility of the basemat and the three dimensional features of the NI structures are not significant in defining the maximum bearing pressure and the results of the dynamic analysis are used rather than the conservative results of the equivalent static analysis.

c. The design analyses of the nuclear island basemat include consideration of sliding and lift-off between the containment internal structures and the containment vessel, and of sliding between the containment vessel and the nuclear island basemat as described in the response to RAI-TR85-SEB1-21. Analyses of stability for the hard rock site demonstrated that there was no uplift or sliding at the interface of the containment internal structures and the containment vessel. These analyses showed potential uplift of the containment vessel and containment internal structures from the nuclear island basemat for the Review Level Earthquake (RLE). Based on these analyses, Westinghouse provided shear studs between the containment vessel and the nuclear island basemat to provide additional margin for the Review Level Earthquake (RLE). These studs were then designed to accommodate pressurization of the containment vessel. The number of studs required for containment pressure was more than double the number required for seismic overturning for the RLE at the hard rock site. Since the overturning moments for soil sites are similar to those for hard rock, the number of shear studs provided is also sufficient to prevent uplift or sliding of the containment vessel with respect to the basemat for the various soil conditions. The NRC staff audited calculation "Containment Vessel Shear Studs", APP-1100-S2C-102, Revision 0, during the December 15–16, 2003 audit.

d. Section 7.2 of TR-03, Revision 1, has been revised to show only the damping value of 5% used in 2D ANSYS lift-off analyses for the soft-to-medium case. This value was based on comparison to the 2D SASSI analyses for the soft-to-medium site.

Westinghouse Response (Revision 1):

b) This request for additional information relates to the higher bearing pressure loads in the 3D ANSYS equivalent static non-linear analysis versus the 2D ANSYS dynamic time history analysis. This is addressed in the Revision 1 response to RAI-TR85-SEB1-03.

c) The bending moments in the sticks at grade from the 2D SASSI analyses are shown in Table RAI-TR85-SEB1-012-001. The CIS moments is the moment at grade considered in the sliding evaluation of the CIS on the containment vessel bottom head. The sum of these moments is the moment at grade considered in the sliding evaluation of the CIS and containment vessel on the dish concrete. These demonstrate that the hard rock case controls.

d) Section 7 of TR-03 has been deleted in Revision 3. Information pertinent to the basemat analyses has been included in the revision to Sections 2.4.1 and 2.4.2 of TR85 as shown in the response to RAI-TR85-SEB1-05, Rev 1. In the revised analyses for the soft soil case damping was limited to 20%. This value provided reasonable comparison to the 2D SASSI results.

Reference: None

Design Control Document (DCD) Revision:

None

PRA Revision:

None

Technical Report (TR) Revision:

Revise fifth paragraph of Section 2.4.2 as follows:

Comparison of floor response spectra ~~and the maximum member forces and moments~~ for these two cases shows that the liftoff has insignificant effect on the SSE floor response spectra. Thus, the superstructure may be designed neglecting liftoff. The basemat design considers the effects of liftoff as described in Section 2.6.

Table RAI-TR85-SEB1-012-001

SASSI Bending Moments at Grade

X direction

Stick	Elev.	Overturning Moment about EW (Y) Axis, 1000 k.ft					
		HR	FR	SR	UBSM	SM	SS
SCV	100.0	753	778	779	808	554	315
CIS	98.0	627	489	437	386	303	237
Sum		1380	1267	1216	1195	857	552

Y direction

Stick	Elev.	Bending Moment about NS (X) Axis, 1000 k.ft					
		HR	FR	SR	UBSM	SM	SS
SCV	100.0	877	822	811	866	684	531
CIS	98.0	497	464	421	402	366	278
Sum		1374	1286	1233	1267	1049	809

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

RAI Response Number: RAI-TR85-SEB1-34
Revision: ~~1~~2

Question:

Section 2.9 describes the nuclear island stability evaluation for various load combinations. Table 2.9-1 lists the load combinations and corresponding factors of safety for sliding, overturning, and flotation. For the overturning case with the SSE loading, factors of safety of 1.06 and 1.07 are shown, both of which are less than the NRC SRP 3.8.5 acceptance criteria of 1.10. Since footnote 1 to the table indicates that the factors of safety for overturning are 1.12 and 1.10 when active and passive pressures on the external walls below grade are considered, then why don't the entries in the table reflect these values which would then meet the acceptance criteria. If the passive soil pressure is relied upon to meet the acceptance criteria for overturning, then DCD Section 3.8.5.5.4, which describes overturning evaluation of the foundation, also needs to be revised to include this effect. Confirm that the foundation walls have been designed for the passive soil pressures as well.

Additional Request (Revision 1):

The staff reviewed the RAI response provided in Westinghouse letter dated December 4, 2007, and as a result, requests Westinghouse to address the following items:

1) The staff notes that the revised RAI response indicates that the nuclear island (NI) stability evaluation has been updated to use the 3D shell model NI20 based on a time history analysis. This does not appear to be consistent with the response to RAI-TR85-SEB1-04, which states that a 3D NI20 model using a response spectrum analysis is used for the stability evaluation. Westinghouse is requested to clarify this inconsistency.

2) The RAI response indicates that for overturning stability analysis, the contribution from passive soil pressure is relied upon to resist overturning. Since passive pressure is also needed to resist sliding, Westinghouse is requested to describe how passive pressure is considered to resist both overturning and sliding at the same time. This should include an explanation of the point of rotation for overturning and the pressure distribution with respect to height that corresponds to the simultaneous consideration of rotation and sliding.

Westinghouse Response:

Westinghouse agrees and will make the change as recommended in the question. It is confirmed that the foundation walls have been designed for the passive soil pressures as well.

The nuclear island seismic stability evaluation has been updated for the envelope of the six site profiles of hard rock, firm rock, soft rock, upper bound soft-to-medium soil, soft-to-medium soil, and soft soil site. A time history analysis was performed using the AP1000 nuclear island 3D shell model NI20. The time history input is developed from the envelope of the broadened floor response spectra of the six site profiles at the edges, along the side walls, and at the center of the AP1000 nuclear island basemat. There was very little change in the seismic factors of safety associated with overturning and sliding. This is seen below comparing minimum safety factors.

Factor of Safety	2D Model	NI20 3D Shell Model
Sliding north-south earthquake	1.28	1.28
Sliding east-west earthquake	1.33	1.33
Overturning NS earthquake	1.39	1.35
Overturning EW earthquake	1.10	1.12

The NI20 model is the model used in the most recent seismic analyses of the nuclear island described in TR03, Rev. 1. The factors of safety obtained from the new analyses are being included in the proposed revisions to the DCD and TR85.

Westinghouse Response to Revision 1:

1) As noted in RAI-TR85-SEB1-04 R1, time history analyses using the NI20 model are used and not seismic response spectrum analyses. This is consistent with Table 3G1-1 given in DCD Appendix 3G, Revision 17.

2) As discussed in RAI-TR85-SEB1-10, Rev. 1, the overturning stability analysis does not rely on passive soil pressure to resist overturning. Therefore, passive pressure is not considered to resist both overturning and sliding at the same time. The passive pressure distribution has been described in RAI-TR85-02 R1.

Design Control Document (DCD) Revision:

The revisions described in Revision 0 of this response are incorporated in DCD Rev 17. The additional changes required related to stability are given in RAI-TR85-SEB1-10, R1.

PRA Revision:

None

Technical Report (TR) Revision:

Changes to Section 2.9 related to Nuclear Island Stability are given in RAI-TR85-SEB1-10, R1.

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR85-SEB1-35
Revision: ~~0~~1

Question:

Section 2.9 indicates that the sliding resistance is based on the friction force developed between the basemat and the foundation using a coefficient of friction of 0.70. This is based on soil having a friction angle of 35 degrees. The Combined License will provide the site specific angle of internal friction for the soil below the foundation. Based on this information, address the following items:

a. Since DCD Section 3.8.5.5.3 indicates that the sliding factor of safety is based on the shearing or sliding resistance at the base of the basemat and the soil passive resistance, the reliance on both of these resisting forces need to be based on a consistent set of assumptions. Since the passive resistance of soil requires sufficient displacement to mobilize the full passive resisting forces at the foundation walls and side of the basemat, provide the technical basis for using a coefficient of friction of 0.70 for the sliding resistance beneath the basemat which is considered to be applicable to the static (not sliding or dynamic) friction resistance of soil.

b. What are the numerical contributions of the sliding frictional resistance and the soil passive pressure resistance for the NS and EW directions?

c. Has Westinghouse confirmed that using a minimum angle of internal friction of 35 degrees is achievable for most soil sites?

d. DCD Tier 1, Section 3.3, states that "Exterior walls and the basemat of the NI have a water barrier up to site grade level." Describe this water barrier and how does this affect the assumed coefficient of friction between the basemat and the soil? Is it high enough to ensure that soil friction would govern?

Additional Request (Revision 1):

Based on the review of the RAI response provided in Westinghouse letter dated 10/19/07, the following items need to be addressed:

a. The RAI response did not address the requested information in the RAI. In calculating the factor of safety for the basemat against sliding during earthquakes, Westinghouse combines the friction force at the bottom of the basemat and the maximum soil passive pressure resistance on the foundation walls and basemat vertical edge in order to obtain the total resistant force. Westinghouse is request to explain the basis for using the static coefficient of friction of 0.70 (which implies essentially no sliding of the basemat will occur) at the same time as the maximum soil passive resistance (which would require sufficient horizontal displacement of the foundation to mobilize the maximum passive resisting forces at the foundation walls and the side of the basemat). In other words, if the maximum soil passive pressure is relied upon to resist sliding, then the full static coefficient of friction cannot be utilized.

b. From the values given for the sliding resistance for the soil passive pressure, it appears that the soil passive resistance is an important contribution to the total sliding resistance of the foundation. Therefore, Westinghouse is requested to (1) describe how the maximum soil passive pressure resistance is calculated and the passive pressure distribution in the vertical direction and (2) describe whether saturated and/or unsaturated soil conditions were considered in your analysis and indicate which case governs.

c. The RAI response indicates that "soils can achieve a friction angle of 35 degrees which is addressed and answered in Question a of TR-85 RAI-35." However, the response to Question a of TR-85 RAI-35 did not demonstrate that a friction angle of 35 degrees can be achieved for a range of common soil profiles expected at various sites. The RAI response also indicates that "the basis being provided in Table RAI-TR85-1, which shows an internal friction angle of 35 degrees being included in the medium soil type (sand)." However the staff cannot identify what Table RAI-TR85-1 this statement is referring to. If the actual reference was intended to be Table RAI-TR85-37-1, then classifying soil as "medium soil type (sand)" by itself does not assure that a soil friction angle of 35 degrees can be achieved. Finally, the RAI response indicates that "Dense soil types and hard rock sites will also meet the minimum soil friction angle of 35 degrees, often proven to have a much higher friction angle." While this may be the case, the staff believes that demonstrating the required soil friction angle should be based on actual testing of the soil, and therefore, the staff is requesting that Westinghouse identify what type of testing will be required to be implemented by the COL applicants to demonstrate that the coefficient of friction for the soil material beneath the foundation at the site meets the required coefficient of friction used in the design. The response to the above items, should clearly demonstrate that the soil friction angle (and thereby the corresponding coefficient of friction) used in design can be achieved for a reasonable range of soil conditions expected to exist at plant sites.

d. The RAI response describes the types of waterproofing membranes that should be placed immediately beneath the upper mud mat, and on top of the lower mud mat or leveling concrete, which has been finished in accordance with ACI 301, Section 5.3.4.2.d. The staff requests that Westinghouse provide technical information which demonstrates that, for the types of waterproofing material proposed in the RAI response, the membranes can (1) achieve a coefficient of friction used in design (at this time specified as 0.70), (2) have the strength, with margin, to withstand the shear and compression loads from the NI, and (3) do not degrade over the design life of the plant under similar loading and environmental conditions in the mud mat concrete. In addition, since this waterproofing membrane is not soil, explain why the RAI response refers to a requirement for this material in terms of a minimum friction angle rather than a coefficient of friction. Describe the location in DCD Tier 1 and DCD Tier 2 for the requirements for the waterproofing membrane where the coefficient of friction and strength are specified.

Westinghouse Response:

a. Using the formula tan(δ) = φ (tan(35°) = 0.70), Terzaghi, Karl, and Ralph B. Peck, Soil Mechanics in Engineering Practice, the technical basis for using a coefficient of friction of 0.70 for the sliding resistance beneath the basemat is justified. Furthermore the coefficient of wall friction, the value of angle δ between concrete and soil, usually is taken as equal to the angle of internal friction φ of the soil medium, with the coefficient of friction being 0.70 for sound rock sites.

b. The sliding frictional resistance is 142,373 kips, with numerical contributions from the plant deadweight = 281,223 kips, buoyant force = 76,003 kips and vertical wind load = 1,830 kips. The soil passive pressure resistances for the NS and EW directions are 43,456 kips and 69,098 kips respectively.

c. The soils can achieve a friction angle of 35 degrees which is addressed and answered in Question A of TR-85 RAI35. The basis being provided in Table RAI-TR85-1, which shows an internal friction angle of 35 degrees being included in the medium soil type (sand). Dense soil types and hard rock sites will also meet the minimum soil friction angle of 35 degrees, often proven to have a much higher friction angle.

d. A Waterproofing Membrane should be placed immediately beneath the upper Mud Mat, and on top of the lower Mud Mat or leveling concrete, which has been finished in accordance with ACI 301, Section 5.3.4.2.d. This bottom (horizontally planar) geosynthetic membrane should be textured on both sides to maintain minimum sliding coefficient requirements. For plasticized polyvinyl chloride (PVC) membranes, a geocomposite should be formed by applying a geotextile to both sides of the PVC geomembrane, such as CARPI USA's "SIBELON 2CNT" liner. For high-density polyethylene (HDPE) membranes, the geosynthetic should be spiked or studded on both sides, such as AGRU-America's "Super Gripnet Liner." As the membrane transitions to the walls of the Nuclear Island (vertically planar), a smooth geosynthetic liner may be used since sliding is not a design concern along these vertical planes.

   A Waterproofing Membrane is to be selected such that for horizontal surfaces, the minimum friction angle achieved at any interface is at least 35 degrees, yielding a friction coefficient of at least 0.7, to be consistent with AP1000 DCD requirements. In order to provide the durability required for construction and implementation, as well as the flexibility for a manageable installation, it has been recommended from multiple vendors that the Waterproofing Membrane have a thickness of 80 mils, both for HDPE or PVC.

Westinghouse Response (Revision 1):

a. As noted in response to RAI-TR85-SEB1-10, Revision 1, the full passive pressure is not being used, only a portion. The static coefficient of friction has been lowered to 0.55. The maximum deflection of the nuclear island needed to develop the needed 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. Therefore, the maximum soil passive pressure is not relied upon to resist sliding.

b. (1) The method used to calculate the passive pressure is given below for the Nuclear Island walls below grade. There is no passive soil pressure resistance component in the vertical direction. Resistance in the vertical direction is supplied by the soil subgrade bearing capacity.

The coefficient of passive pressure (K_P) is determined from the following relationship:

$K_P = \mathrm{Tan}^2(45° + \phi / 2)$ (Rankine Method with no wall friction and horizontal ground surface)

Where, ϕ = angle of internal friction of the granular backfill.

The passive earth pressure is calculated by the following formula:

$P_P = K_P \gamma h$

Where,
h = depth below grade (100' 0")
Above water table: $\gamma = \gamma_s$ = Saturated unit weight of granular back fill above water table.
Below water table: $\gamma = \gamma_s - \gamma_w$

Recognizing that the ground water table is at 98 feet plant elevation, the formula for the passive pressure at the base, 60' 6" can be written as follows:

$$P_P = [\mathrm{Tan}^2(45° + \phi / 2)] \times [(100 - 98) \times \gamma_s + (98 - 60.5)(\gamma_s - \gamma_w)]$$

(2) The passive earth pressures are defined for 21 soil cases in Table RAI-TR85-SEB1-35-1 for soil types of rock, sand and gravel, and sand. As seen from this table, the highest loads are obtained for the rock cases. However, it is unrealistic to consider the properties for in-place rock to be similar to those of for the backfill material. A more representative soil is between dense and medium sand (case 15), which is the same as used for the AP600 plant.

The general geotechnical model for the site contains a static ground water level 2 feet below the horizontal ground surface. The horizontal ground surface is assumed at elevation 100 feet and the static ground water level is at elevation 98 feet. A saturated unit weight density of the soil above and below the ground water level has been assigned a value of 150 pounds per cubic foot (pcf) for the Case 15 granular (dense sand) soil model.

Table RAI-TR85-SEB1-035-1 – 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

c. In Chapter 2 of the AP1000 Design Control Document, Revision 17, Table 2-1site characteristics for which the AP1000 is designed are provided. It is stated:

> "The site is acceptable if the site characteristics fall within the AP1000 plant site design parameters in Table 2-1. Should specific site parameters or characteristics be outside the envelope of assumptions established by Table 2-1, the Combined License applicant referencing the AP1000 will demonstrate that the design satisfies the requirements imposed by the specific site parameters and conforms to the design commitments and acceptance criteria described in the AP1000 Design Control Document."

In Table 2-1 the Minimum Soil Angle of Internal Friction is defined to be Greater than or equal to 35 degrees below footprint of nuclear island at its excavation depth. It should not be Westinghouse's responsibility to identify what type of testing will be required to be implemented by the COL applicants to demonstrate that the coefficient of friction for the soil material beneath the foundation at the site meets the required coefficient of friction used in the design. This is the responsibility of the COL applicants. Therefore, the Combined License applicant must demonstrate that they have a friction angle of 35 degrees using the testing procedures defined by them.

d. Reference to the angle of friction should not have been given in the initial response, only the coefficient of friction for the membrane. In Section 3.4.1.1.1.1, Revision 17 Tier 2, the requirement related to coefficient of friction and the horizontal membrane is given:

   "... 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."

   The requirement for the coefficient of friction will be changed to 0.55. Tests will be performed that demonstrate that this coefficient of friction is achieved.

For strength and durability requirements of the membrane it has been recommended from multiple vendors that the Waterproofing Membrane have a thickness of 80 mils, both for HDPE or PVC. However, it is noted that if the membrane looses strength or degrades over its life, this will not result in a lower coefficient of friction than 0.55 since the surface will be concrete on concrete. In accordance with Reference 2, Section 11.7.4.3, the coefficient of friction shall be taken as a) 1.0 for normal weight concrete placed against hardened surface intentionally roughened as in Section 11.7.9 or b) 0.6 for normal weight concrete placed against hardened surface not intentionally roughed. Case b) could result if the waterproofing membrane looses strength of degrades over time. However, the alkaline (concrete) environment in which the HDPE membrane will be placed is not detrimental to this material.

Reference:

1. Terzaghi, Karl, and Ralph B. Peck, Soil Mechanics in Engineering Practice, John Wiley & Sons, Inc., New York, 1948.
2. Building Code Requirements for Structural Concrete (ACI 318-05), American Concrete Institute, 2005

Design Control Document (DCD) Revision:

~~None~~

Modify Section 3.4.1.1.1.1, Tier 2 to read as follows:

"... the membrane between the ~~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."

PRA Revision:
None

Technical Report (TR) Revision:
None

RAI Response Number: RAI-TR85-SEB1-40
Revision: ~~0~~1

Question:

Regarding the dynamic stability of the NI structures, Westinghouse is requested to provide additional information to demonstrate how the sliding criteria assumed during the APl000 SSI analyses (sliding friction value of 0.7) are in fact to be attained from the interface between the basemat and mudmat and from the interface between the top portion of the mudmat and the lower portion of the mudmat through the waterproofing membrane.

Additional Request (Revision 1):

The staff reviewed the RAI response submitted in Westinghouse letter dated October 19, 2007, and notes that the outstanding issues raised by this RAI are considered to be very significant. Westinghouse has still not demonstrated that all sliding interfaces have been properly considered. Therefore, Westinghouse is requested to demonstrate the technical adequacy of the approach being used in each of the sliding interfaces listed below.

1. Regarding the sliding interface between the bottom of the NI concrete basemat and the top of the upper concrete mudmat, Westinghouse is requested to provide the technical basis for assuming that the coefficient of friction between these two surfaces is at least equal to 0.7. If this cannot be demonstrated then what special interface treatments (e.g., roughened concrete as defined in ACI) will be needed between the mass concrete basemat and the upper concrete mudmat to achieve the 0.7 coefficient of friction, or will a testing program be needed to guarantee that a minimum friction factor of 0.7 can be achieved.

2. Regarding the sliding interface between the upper concrete mudmat and lower concrete mudmat through the waterproofing membrane - issues related to ensuring that such waterproofing membrane is suitable and will have a minimum coefficient of friction of 0.7 are addressed in RAI-TR85-SEB1-35.

3. Regarding the sliding interface between the bottom of the lower concrete mudmat and the soil, Westinghouse is requested to provide the technical basis for assuming that the coefficient of friction between these two surfaces is at least equal to 0.7. The current formulation being used for sliding resistance (i.e., Section 2.9 of TR85 and DCD Section 3.8.5.5.3) is applicable to sliding resistance/shear failure within the soil media not concrete on soil (see item 4 below). Since standard data indicates that effective ultimate friction between concrete and soil is typically less than 0.7 (e.g., Navy design manual DM-7.02), then Westinghouse needs to explain what coefficient of friction is assumed for this interface and the basis for this value.

4. Regarding the sliding interface within the soil media, and as described in Section 2.9 of TR85, and in Section 3.8.5.5.3, "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." Using tan(phi) gives the coefficient of friction of 0.7 which was assumed by Westinghouse to be governing coefficient of friction among the different sliding interfaces. It should be noted that this is the sliding resistance/shear failure capacity within the soil media, not concrete on soil. Also, the technical basis for assuming that the governing coefficient of friction among the different sliding interfaces has not been provided (see Items 1 through 3 above). As noted in RAI-TR85-35, the RAI response did not address the requested information. In calculating the factor of safety for the basemat against sliding during earthquakes, Westinghouse combines the friction force at the bottom of the basemat and the maximum soil passive pressure resistance on the foundation walls and basemat vertical edge as the total resisting force. Westinghouse is request to provide the technical basis for using this approach which utilizes the static coefficient of friction of 0.70 (which implies <u>essentially no horizontal sliding</u> of the basemat) at the same time as the maximum soil passive resistance (which would require <u>sufficient horizontal displacement</u> of the foundation to mobilize the passive resisting forces at the foundation walls and the side of the basemat). Further issues related to utilizing the soil passive pressure for both sliding and overturning are discussed under RAI TR85-SEB1-35.

In addition to the above 4 items, Westinghouse is requested to identify what set or sets of soil parameters were utilized in (a) the seismic analyses for stability evaluations to develop the maximum shear and moments and (b) the sliding and overturning stability calculations (including passive soil pressure calculations). These would include the soil properties and compaction requirements for the backfill material at the side of the embedded foundation walls and the vertical edge of the basemat. Soil properties beneath the basemat which were used to develop the subgrade modulus values should also be defined. These properties should be clearly presented in the DCD so that the Combined License applicants can reference the DCD design without performing additional site-specific stability analyses. Also, describe in the DCD how each of these soil parameters should be obtained (i.e., tested and measured) and verified at the site.

5. Describe how the potential effect of saturated soils from groundwater or water infiltration from the surface has been considered in: all seismic soil structure interaction (SSI) analyses, calculation of the subgrade modulus (used in all seismic analyses for bearing pressure, stability evaluations, and design of the basemat foundation), selecting the coefficient of friction, and calculation of the passive soil pressures used in the stability evaluations and the design of the foundation walls.

Westinghouse Response:

Alternate approaches for waterproofing systems are described in subsection 3.4.1.1.1.1. For each of the waterproofing system selected for a site a test to demonstrate that the interface between the top portion of the mudmat and the lower portion of the mudmat has a sliding coefficient of friction of at least 0.7 will be performed. The Combined License applicant will describe the excavation and backfill methods, along with the description of the waterproofing system selected, and reference the test report that documents that the minimum coefficient of friction of 0.7 is achieved. If there are site conditions that could affect the sliding coefficient of friction, separate from the waterproof membrane, a lower coefficient may be justified by performing site specific analyses and demonstrating sliding stability.

DCD subsections 2.5.4.1, 2.5.4.6 and 3.4.1.1 are revised as shown below to clarify the requirements for the waterproofing membrane and the mudmat.

Westinghouse Response (Revision 1):

1. A test program will be performed that is documented in a report that can be referenced by the COL applicant.

2. As stated in RAI-TR85-SEB1-10, Rev. 1, and RAI-TR85-SEB1-35, Rev. 1, the coefficient of friction has been reduced from 0.7 to 0.55, and a test program will be performed to demonstrate that this coefficient of friction.

3. The coefficient of friction of 0.55 is consistent with what is expected for soil; therefore, the coefficient of friction between the interface of the mudmat and concrete, and the concrete and soil, will be equal to or greater than the coefficient of friction (0.55) between soil interfaces. In accordance with NAVFAC DM 7.02 (Sept. 1986), Table 1, page 7.2-63, the interface friction factor (or coefficient of friction, μ) between mass concrete placed on clean gravel, gravel-sand mixtures or coarse sand subgrade materials ranges from 0.55 to 0.60. The lower coefficient of friction (0.55) that Westinghouse is now using in the sliding stability calculation is consistent with the lower bound coefficient of friction for the waterproofing membrane, and that of the granular, in-situ subgrade material.

4. As noted in RAI-TR85-SEB1-10, Rev. 1, Westinghouse is no longer using the full passive pressure for sliding stability, and has reduced the coefficient of friction to 0.55. In this RAI response lower coefficients of friction below 0.55 are discussed. It is shown that the Nuclear Island remains stable and within the factor of safety of 1.1 without significant displacements. It is also noted, and discussed in RAI-TR85-SEB1-10, Rev. 1, passive pressure is no longer used in the calculation of the overturning factors of safety.

In addition to the above 4 items, responses are given for the supplementary items:

a. To develop the maximum shear and moments for the seismic analyses for stability, six soil types were considered: hard rock, firm rock, soft rock, upperbound soft to medium soil, soft to medium soil, and soft soil. These are described in DCD Section 3.7.1.4, Revision 17.

b. For the sliding and overturning stability calculations the six site characteristics described in item a above are used. The passive soil pressure calculation is described in RAI-TR85-SEB1-35, Rev. 1. The Combined License applicants referencing the AP1000 design must address site specific information related to the geotechnical engineering aspects of the sites. This is discussed in DCD Section 2.5.4.6, Revision 17. Excavation and backfill requirements are described in this section. Further, per DCD 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." No further action is required for sites within the bounds of the AP1000 site parameters as defined in the DCD. The testing programs and measurements is the responsibility of the COL applicants. The strain-dependent shear modulus curves for the foundation materials are shown in Figures 3.7.1-15 and 3.7.1-16 of DCD Revision 17. The different curves for soil in Figure 3.7.1-16 apply to the range of depth within a soil column below grade.

5. In the seismic soil structure interaction (SSI) analyses the effect of saturated soil from groundwater or water infiltration from the surface is considered by setting the pressure wave velocity (Vp) equal to 5,000 ft/sec. The strain-dependent properties used in the SSI analyses for the safe shutdown earthquake are shown in DCD, Revision 17, Table 3.7.1-4 and Figure 3.7.1-17 for the firm rock, soft rock, upper bound soft-to-medium soil, soft-to-medium soil, and soft soil properties. As stated above, the coefficient of friction has been reduced from 0.7 to 0.55 to be consistent with the soil characteristics associated with design condtions. The calculation of the passive soil pressures used in the stability evaluations and the design of the foundation walls are described in RAI-TR85-SEB1-02, Rev. 1, , RAI-TR85-SEB1-10, Rev. 1, and RAI-TR85-SEB1-35, Rev. 1.

DCD Revision:

The revisions described in Revision 0 of this response are incorporated in DCD Rev 17.

PRA Revision: None

Technical Report (TR) Revision: None