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Your ref: Docket No. 52-006
Our ref: DCP/NRC2112

March 31, 2008

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

Westinghouse is submitting responses to the NRC requests for additional information (RAIs) on AP1000 Standard Combined License Technical Report (TR) 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 response is generic and is expected to apply to all COL applications referencing the AP1000 Design Certification and the AP1000 Design Certification Amendment Application.

Responses are provided for RAI-TR85-SEB1-05,-07,-17,-19,-32, and -36, as sent in an email from Dave Jaffe to Sam Adams dated August 9, 2007. These responses complete 39 of 40 requests received to date for TR 85. A response to RAI-TR85-SEB1-40 is scheduled to be submitted on April 11, 2008. Responses to RAI-TR85-SEB1-03, -13, -27, and -38 were submitted under Westinghouse letter DCP/NRC1999 dated September 18, 2002. Responses to RAI-TR85-SEB1-01, -09, and -16 were submitted under Westinghouse letter DCP/NRC2002 dated September 21, 2007. Responses to RAI-TR85-SEB1-02 and -21 were submitted under Westinghouse letter DCP/NRC2006 dated September 28, 2007. Responses to RAI-TR85-SEB1-12, -14, -18, -20, -26, -31, and -33 were submitted under Westinghouse letter DCP/NRC2022 dated October 19, 2007. Responses to RAI-TR85-SEB1-10, -11, -15, -24, -25, -28, -29, -30, -34, -35, -37, and -39 were submitted under Westinghouse letter DCP/NRC2025 dated October 19, 2007. Responses for RAI-TR85-SEB1-04 -06, -08, -22, - and -23 and a revised response for RAI-TR85-SEB1-34 were submitted under Westinghouse letter DCP/NRC2050 dated December 5, 2007.

Pursuant to 10 CFR 50.30(b), the responses to the requests for additional information on SRP Section 5.3.1, are submitted as Enclosure 1 under the attached Oath of Affirmation.

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.

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D063
NRO

Very truly yours,

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

/Attachment

1. "Oath of Affirmation," dated March 31, 2008

/Enclosure

1. Responses to Requests for Additional Information on Technical Report 85

cc:	M. Miernicki	-	U.S. NRC	1E	1A
	E. McKenna	-	U.S. NRC	1E	1A
	P. Ray	-	TVA	1E	1A
	P. Hastings	-	Duke Power	1E	1A
	R. Kitchen	-	Progress Energy	1E	1A
	A. Monroe	-	SCANA	1E	1A
	J. Wilkinson	-	Florida Power & Light	1E	1A
	C. Pierce	-	Southern Company	1E	1A
	G. Zinke	-	NuStart/Entergy	1E	1A
	R. Grumbir	-	NuStart	1E	1A
	E. Schmiech	-	Westinghouse	1E	1A
	B. LaPay	-	Westinghouse	1E	1A

ATTACHMENT 1

"Oath of Affirmation"

ATTACHMENT 1

UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

In the Matter of:)

AP1000 Design Certification Amendment Application)

NRC Docket Number 52-006)

APPLICATION FOR REVIEW OF
"AP1000 GENERAL INFORMATION"
FOR DESIGN CERTIFICATION AMENDMENT APPLICATION REVIEW

W. E. Cummins, being duly sworn, states that he is Vice President, Regulatory Affairs & Standardization, for Westinghouse Electric Company; that he is authorized on the part of said company to sign and file with the Nuclear Regulatory Commission this document; that all statements made and matters set forth therein are true and correct to the best of his knowledge, information and belief.

W E Cummins

W. E. Cummins
Vice President
Regulatory Affairs & Standardization

Subscribed and sworn to
before me this 31st day
of March 2008.

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

Patricia S Aston
Notary Public

ENCLOSURE 1

Responses to Requests for Additional Information on Technical Report 85

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

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

Question:

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

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

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

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

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

Westinghouse Response:

a) Subgrade modulus is used in the following analyses:

- Subgrade moduli of 6267, 3200, 1000, and 300 kcf were used for hard rock, soft rock, soft to medium soil and soft soil sites in the 2D ANSYS parametric linear dynamic analyses described in Section 2.4.2 of the report. The results of the analyses for soft rock and soft soil were not used.
- Subgrade moduli of 6267 kcf and 1000 kcf were used for the hard rock and soft to medium soil sites in the 2D ANSYS non-linear dynamic analyses described in Section 2.4.2 of the report.
- A subgrade modulus of 6267 kcf was used for hard rock in the 3D ANSYS Equivalent Static Non-Linear Analysis for design of the basemat as described in Section 2.3.1 of the report
- A subgrade modulus of 520 kcf was used for soil sites in the 3D ANSYS Equivalent Static Non-Linear Analysis for design of the basemat as described in Section 2.6.1 of the report.
- A subgrade modulus of 260 kcf was used in the 3D ANSYS Equivalent Static Non-Linear Parametric Analysis for evaluation of the effect of a lower subgrade modulus as described in Section 2.7.1.1 of the report.

Table RAI-TR85-SEB1-01-1 shows the subgrade modulus used in the 2D ANSYS analyses for the AP1000 hard rock and soil cases. The hard rock value was calculated for a uniform half space using the formula given in ASCE-4 (Reference 1). The soft rock, upper bound soft to medium, soft to medium and soft soil cases were calculated using the Steinbrenner formula for the degraded soil profiles used in the AP1000 seismic analyses. These profiles assume 80' 6" of soil below the nuclear island basemat and assume fixed base (very hard rock) at a depth of 120 feet below grade. The properties for each layer in the soil profile are shown in DCD Rev 16 Table 3.7.1-4. The values shown in the middle column of Table RAI-TR85-SEB1-01-1 are those reported in TR85 Rev 0. Subgrade modulus was not calculated for the firm rock site since no analyses were performed requiring the subgrade modulus at a firm rock site.

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

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

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

Westinghouse has expanded the number of soil cases it evaluates in its 3D SASSI generic analyses so that no justification is required using AP1000 2D SASSI sensitivity cases. These six generic cases have been identified for convenience as hard rock (HR), firm rock (FR), soft rock (SR), upper bound soft to medium (UBSM), soft to medium (SM), and soft soil (SS). This is shown in the proposed revisions to DCD, Rev 16, Appendix 3G as described in TR03, Rev 1 and TR-134, Rev 0.

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

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

The 2D ANSYS nonlinear analyses analyzed two cases (hard rock, soft to medium soil) to evaluate the effect of lift-off and the maximum bearing pressure. These two cases were selected based on linear analyses that also included the soft rock and soft soil profiles. The analyses of the soft to medium soil case used a subgrade modulus of 1000 kcf which was subsequently determined to be too high for this soil condition. In addition the nuclear island seismic analyses show that the upper bound soft to medium soil case is very close to that of the soft to medium soil case. The non-linear analyses are being supplemented by two additional cases:

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

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

Table RAI-TR85-SEB1-05-1

Subgrade modulus for AP1000 Soil Cases

Soil case	Subgrade modulus (TR85, Rev 0)	Revised subgrade modulus (TR85, Rev 1)
	kcf	kcf
Hard rock	6267	6267
Firm rock		3760
Soft rock	3230	1630
Upper bound soft to medium soil (water table to grade)		1340
Upper bound soft to medium soil (dry)	2334	1320
Soft to medium soil (water table to grade)	1280	780
Soft to medium soil (dry)	963	580
Soft soil (dry)	312	170

Reference:

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

Design Control Document (DCD) Revision:

Revise Table 3.7.1.4 to Table 3.7.1-4 on four sheets. On sheet 4 revise the column headings to be the same as those on sheets 1 to 3.

PRA Revision:

None

Technical Report (TR) Revision:

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

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR85-SEB1-07
Revision: 0

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.

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.

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:

Tables 2.4-1 and 2.4-2 are changed to reflect the 2D analyses with the enhanced shield building.

Table 2.4-1

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

Units: 1000 kips & 1000 ft-kip

	North-South model		East-West model	
Soil case	**North-South Shear**	**Moment about E-W axis**	**East-West Shear**	**Moment about N-S axis**
	F_X	M_{YY}	F_Y	M_{XX}
Hard rock (HR)	52.86	6934	46.78	6085
Firm rock (FR)	49.82	6837	48.07	6118
Soft rock (SR)	50.59	6586	51.63	6554
Upper bound soft to medium soil (UB)	52.22	6416	55.33	7084
Soft to medium soil (SM)	53.41	6810	61.81	7621
Soft soil (SS)	26.65	3683	28.68	4649

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

Units: 1000 kips & 1000 ft-kip

Seismic Reactions	HR	FR	SR	UBSM	SM	SS
Shear NS, F_X	123.75	116.49	118.65	121.48	113.61	73.11
Shear EW, F_Y	112.31	113.55	121.88	128.11	124.94	74.34
Vertical, F_Z	98.76	98.65	99.63	104.55	112.30	94.48
Moments Relative to Centerline of Containment						
M_{XX} EW Excitation	10,916	10,900	11,471	12,229	12,607	7,653
M_{XX} Vertical Excitation	1,660	1,693	1,715	2,017	1,913	1,459
M_{XX} SRSS	11,042	11,031	11,598	12,394	12,751	7,791
M_{YY} NS Excitation	12,184	11,659	11,390	11,274	11,173	6,300
M_{YY} Vertical Excitation	918	935	946	997	1059	829
M_{YY} SRSS	12,218	11,697	11,429	11,318	11,223	6,354

Notes:

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

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR85-SEB1-17
Revision: 0

Question:

In Section 2.5, the first paragraph (Page 19 of 83) states that in the expected basemat construction sequence, concrete for the mat is placed in a single placement. The last sentence of the same paragraph states that once the shield building and auxiliary building walls are completed to Elevation 82'-6", the load path changes and loads are resisted by the basemat stiffened by the shear walls. The staff identified the following issues:

a. Since the size of the basemat is 256 feet by 161 feet, provide a detailed description of how the single placement is to be placed (e.g., by layers or by areas, time period between pouring of layers or areas, if by areas - type of joint detail to ensure proper connection, etc.).

b. Explain how the “single placement” can be completed and considered as a “single placement,” if any unexpected incidents (such as malfunction of concrete mixer, etc.) occur.

c. Provide the basis of how the residual stress at the junction between the shear walls and the shield building is calculated (detailed calculation procedure needs to be provided) and designed for, if the auxiliary building shear walls are to be constructed up to Elevation 82'-6" first and then construction of the shield building.

d. Describe what construction techniques and design provisions are needed to address issues related to the use of a single massive concrete pour of the entire basemat. The response should also address concerns related to the effects of heat generation, restraint, and volume changes associated with a large single massive pour, and how the cracking of the concrete basemat will be avoided.

e. Where in the DCD is the requirement for the COL applicant to follow the construction sequences considered by Westinghouse in the design of the NI structures? If the COL applicant proposes to use a construction sequence that is substantially different than that studied by Westinghouse, the COL applicant should be required to demonstrate that their proposed sequence does not cause a problem.

Westinghouse Response:

a. Site specific placement plans will be developed to address the placement of concrete for the NI basemat. Those plans will address the conditions outlined below:

The concrete for the NI basemat will be placed in a single continuous placement operation. It is expected that the batch plant equipment and materials on site (site dependent) for this operation will consist of the following equipment or equal in order to support this placement:

- 12 cubic yard central mix batch plant (main plant)
- 10 or 12 cubic yard backup/auxiliary batch plant
- All coarse and fine aggregates stockpiled on-site to support the placement
- All admixtures (water reducer, plasticizer, air entraining agent, etc.) on-site to support the placement.
- All cement and fly ash stored on-site (batch plant silos and supplemental storage blimps) or reliability of re-supply during the placement verified.
- If ice is required, adequate supplies will be stored on-site or reliability of re-supply during the placement verified.
- Adequate concrete trucks including back-up trucks on-site to support the placement.
- Adequate personnel and truck drivers assigned to the batch plants to support multiple shift operations.

For the main batch plant, sustained maximum production is expected to reach 250 cubic yards per hour and average production is expected to exceed 200 cubic yards per hour allowing for decreased production periods at the beginning and at the end of the concrete placement. The placement plan shall be based on the use of one plant being able to successfully complete the placement, however the back-up plant may be used during the placement. Initial plans indicate that the placement will take approximately 36 hours.

Concrete will be placed by conventional placement equipment (i.e., pumps, conveyors, buckets, etc.) suitable for the site conditions. Telebelts (conveyors mounted on hydraulic cranes) or conventional conveyors may be used in concert with concrete pumps dependent on the site. Back-up equipment will be provided. Concrete will be placed in a "stair-step" pattern to minimize the exposed working face. Multiple concrete placing crews will be used to balance the concrete placement with the expected rate of concrete supply.

b. In theory a single placement could be interrupted for any one of several reasons. Possible causes of placement interruption based on experience at other projects are listed below together with the associated preventative or mitigating action being planned in each case for the AP1000 NI basemat.

Reason for Interruption	Preventative or Mitigating Action
Bad Weather	Placement to be made only after comprehensive site specific favorable weather forecast. Contingency plans will be in place for unexpected weather conditions.

Breakdown of Batch Plant	Back-up Batch Plant capacity on or nearby the site that satisfies Quality Control and Quality Assurance requirements of the Project. Critical system such as power supply to the batch plant will also have backup.
Breakdown of Concrete Trucks	Backup trucks will be provided.
Breakdown of Concrete Placement Equipment	Backup equipment will be provided.
Inadequate Quantities of Batch Constituents	Sufficient materials will be stored on site to provide for the required concrete quantity plus allowances for extra concrete that may be required for rejected concrete, waste and spillage and low estimated quantities.
Power Failure – unable to operate batch plant	Redundant source of power on site such as a portable diesel generator
Failure of Formwork	Field Engineers will check the formwork prior to the placement. Carpenters will be assigned to monitor the formwork during the placement.
Construction Accident	Enhanced Safety training and briefing of all supervisors and craft labor prior to the placement

In the unlikely event that a major interruption occurs in spite of the above cited Preventative or Mitigating Actions, the duration and cause of the delay and the associated effect on the integrity of the NI basemat will be evaluated. Depending on the level of the impact on the integrity, remediation actions could range from (a) removal, cleaning and green cutting of a new mating surface to (b) complete removal and subsequent placement of a portion of the placement and insertion of a new unplanned construction joint to be designed at the time of the occurrence.

c. The “residual stresses” are evaluated as “locked-in” stresses considering the immediate and long term settlements, the loading history consistent with the construction sequence, and the increasing foundation mat and superstructure stiffness as construction elements are placed and integrated into the structure.

The response to RAI-TR85-SEB1-19 presents details of the computational process and how the resulting forces and moments are considered in the design. The generic analysis includes the effects of three construction sequences, namely, a base case, a delayed Auxiliary building case and a delayed Shield building case.

d. While the quantity of concrete in the NI basemat is relatively large when compared to walls and floor slabs throughout the Nuclear Island, it is not large by normal modern construction practices. The American Concrete Institute (ACI) Code, including ACI

207.1R-05, "Guide to Mass Concrete" and ACI 207.2R-95 (reapproved 2002) "Effect of Restraint, Volume Change and Reinforcement on Cracking of Mass Concrete," has been considered in the design and planning of the NI basemat placement. The most significant issue is the heat of hydration associated with large placement which, in theory, could lead to deleterious cracking if not addressed in the design and construction operation. Depending on the site location and conditions, the concrete temperature will be monitored and the concrete mix will be designed to minimize the heat of hydration, associated temperature rise and subsequent drop and the related tendency for cracking. Measures available for dealing with the heat of hydration, to be worked out on a site by site basis depending on the time of the year and location of the site, include the following:

Aggregate Size and cement fineness
Overall placement procedure
Use of chilled water and/or ice
Enhanced quantity of flyash (pozzolanic)
Use of chilled aggregate
Immediate commencement of curing after finishing
Use of misting equipment
Additives such as water reducers & retarders
Evaporative cooling (water spray) of aggregates

e. DCD 3.8.5.4.2 describes three construction sequences that were evaluated for a soft soil site to demonstrate construction flexibility within broad limits. The acceptability of the construction sequence used by the COL applicant is addressed by an ITAAC.

- A base construction sequence which assumes no unscheduled delays.
- A delayed shield building case which assumes a delay in the placement of concrete in the shield building while construction continues in the auxiliary building.
- A delayed auxiliary building case which assumes a delay in the construction of the auxiliary building while concrete placement for the shield building continues.

The analyses of alternate construction scenarios show that member forces in the basemat are acceptable subject to the following limits imposed for soft soil sites on the relative level of construction of the buildings prior to completion of both buildings at elevation 82′ -6″:

- Concrete may not be placed above elevation 84′ -0″ for the shield building or containment internal structure.

- Concrete may not be placed above elevation 117′ -6″ in the auxiliary building, except in the CA20 structural module where it may be placed to elevation 135' - 3".

References:

ACI 207.1R-05, "Guide to Mass Concrete"
ACI 207.2R-95 (Re-approved 2002), "Effect of Restraint, Volume Change and Reinforcement on Cracking of Mass Concrete"

Design Control Document (DCD) Revision:

Revise Tier 2 subsection 3.8.5.4.2 as follows:

3.8.5.4.2 Analyses of settlement during construction

Construction loads are evaluated in the design of the nuclear island basemat. This evaluation is performed for soil sites meeting the site interface requirements of subsection 2.5.4 at which settlement is predicted to be maximum. In the expected basemat construction sequence, concrete for the mat is placed in a single placement. <u>This placement includes the first 6 feet of the thicker basemat below the containment vessel and shield building but excludes the central zone directly below the bottom of the containment vessel.</u> Construction continues with a portion of the shield building foundation and containment internal structure and the walls of the auxiliary building. The critical location for shear and moment in the basemat is around the perimeter of the shield building. Once the shield building and auxiliary building walls are completed to elevation 82′ -6″, the load path changes and loads are resisted by the basemat stiffened by the shear walls.

The analyses account for the construction sequence, the associated time varying load and stiffness of the nuclear island structures, and the resulting settlement time history. To maximize the potential settlement, the analyses consider a 360 feet deep soft soil site with soil properties consistent with the soft soil case described in subsection 2A.2. Two soil profiles are analyzed to represent limiting foundation conditions, and address both cohesive and cohesionless soils and combinations thereof:

- A soft soil site with alternating layers of sand and clay. The assumptions in this profile maximize the settlement in the early stages of construction and maximize the impact of dewatering.

- A soft soil site with clay. The assumptions maximize the settlement during the later stages of construction and during plant operation.

The analyses focus on the response of the basemat in the early stages of construction when it could be susceptible to differential loading and deformations. As subsequent construction incorporates concrete shear walls associated with the auxiliary building and the shield building, the structural system significantly strengthens, minimizing the impact of differential settlement. The displacements, and the moments and shear forces induced in the basemat are calculated at various stages in the construction sequence. These member forces are evaluated in accordance with ACI 349 using the load factors given in Table 3.8.4-2. Three construction sequences are examined to demonstrate construction flexibility within broad limits.

- A base construction sequence which assumes no unscheduled delays. The site is dewatered and excavated. Concrete for the basemat is placed in a single pour. Concrete for the exterior walls below grade is placed after the basemat is in place. Exterior and interior walls of the auxiliary building are placed in 16 to 18-foot lifts.

- A delayed shield building case which assumes a delay in the placement of concrete in the shield building while construction continues in the auxiliary building. This bounding case maximizes tension stresses on the top of the basemat. The delayed shield building case assumes that no additional concrete is placed in the shield building after the pedestal for the containment vessel head is constructed. The analysis incorporates construction in the auxiliary building to elevation 117′-6″ <u>and filling the CA20 module with concrete to elevation 135’ 3”,</u> and thereafter assumes that construction is suspended.

- A delayed auxiliary building case which assumes a delay in the construction of the auxiliary building while concrete placement for the shield building continues. This bounding case maximizes tension stresses in the bottom of the basemat. The delayed auxiliary building case assumes that no concrete is placed in the auxiliary building after the basemat is constructed. The analysis incorporates construction in the shield building to elevation 84′-0″ and thereafter assumes that construction is suspended.

For the base construction sequence, the largest basemat moments and shears occur at the interface with the shield building before the connections between the auxiliary building and the shield building are credited. Once the shield building and auxiliary building walls are completed to elevation 82′ -6″, the load path for successive loads changes and the loads are resisted by the basemat stiffened by the shear walls. Dewatering is discontinued once construction reaches grade, resulting in the rebound of the subsurface.

Of the three construction scenarios analyzed, the delayed auxiliary building case results in the largest demand for the bottom reinforcement in the basemat. The delayed shield building results in the largest demand for the top reinforcement in the basemat. The analyses of the three construction sequences demonstrate the following:

- The design of the basemat and superstructure accommodates the construction-induced stresses considering the construction sequence and the effects of the settlement time history.

- The design of the basemat can accommodate delays in the shield building so long as the auxiliary building construction is suspended at elevation 117′ -0″. Resumption in construction of the auxiliary building can proceed once the shield building is advanced to elevation 100′ 0″.

- The design of the basemat can accommodate delays in the auxiliary building so long as the shield building construction is suspended at elevation 84′ -0″ feet. Resumption in construction of the shield building can proceed once the auxiliary building is advanced to elevation 100′ 0″.

- After the structure is in place and cured to elevation 100′ -0″, the basemat and structure act as an integral 40 foot deep structure and the loading due to construction above this elevation is not expected to cause significant additional flexural demand with respect to the basemat and the shield building concrete below the containment vessel. Accordingly, there is no need for placing constraints on the construction sequence above elevation 100′ 0″.

The site conditions considered in the evaluation provide reasonable bounds on construction induced stresses in the basemat. Accordingly, the basemat design is adequate for practically all soil sites and it can tolerate major variations in the construction sequence without causing excessive deformations, moments and shears due to settlement over the plant life.

The analyses of alternate construction scenarios show that member forces in the basemat are acceptable subject to the following limits imposed for soft soil sites on the relative level of construction of the buildings prior to completion of both buildings at elevation 82′ -6″:

- Concrete may not be placed above elevation 84′ -0″ for the shield building or containment internal structure.

- Concrete may not be placed above elevation 117′ -6″ in the auxiliary building, except in the CA20 structural module where it may be placed to elevation 135’ - 3”.

Member forces in the basemat considering settlement during construction differ from those obtained from the analyses on uniform elastic soil springs described in subsection 3.8.5.4.1. Although the bearing pressures at the end of construction are similar in the two analyses, the resulting member forces differ due to the progressive changes in structural configuration during construction. The design using the results of the analyses of subsection 3.8.5.4.1 provides sufficient structural strength to resist the specified loads including bearing reactions on the underside of the basemat. However, this may require redistribution of stresses locked in during early stages of construction. A confirmatory evaluation was performed to demonstrate that the member forces due to design basis loads, including locked-in forces due to construction settlement, remain within the capacity of the section. The evaluation was performed for critical locations which were selected as locations where the effect of locked in member forces were judged to be most significant.

The governing scenario is the case with a delay in the auxiliary building construction for the soft soil site with alternating layers of sand and clay. The delay is postulated to occur just prior to the stage where the auxiliary building walls are constructed. Member forces at the end of construction are calculated considering the effects of settlement during construction. The difference in these member forces from those calculated for dead load in the analyses on soil springs are added as additional dead loads in the critical safe shutdown earthquake load combination.

The member forces for the load combination of dead load plus safe shutdown earthquake, including the member forces locked-in during various stages of plant construction, are within the design capacity for the five critical locations. The evaluation demonstrates that the member forces including locked-in forces calculated by elastic analyses remain within the capacity of the section.

PRA Revision:

None

Technical Report (TR) Revision:

Delete Section 5, DCD markup from the Technical Report. Revisions to DCD Revision 16 are now tracked in TR 134 pending inclusion in DCD Revision 17.

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR85-SEB1-19
Revision: 0

Question:

For the analyses of settlement during construction summarized in Section 2.5 of the Technical Report:

a. Describe the analysis and design approach used to calculate the basemat and foundation wall member forces considering settlement during construction for the three alternative construction sequences.

b. Describe how the confirmatory evaluation was performed to demonstrate that the member forces due to design basis loads, including locked-in forces due to construction settlement, remain within the capacity of the section.

c. For the critical sections, provide a comparison how the member forces and design margins for the three construction sequences, including the consideration of all applicable design basis loads, compare to the design analyses based on the uniform elastic soil springs.

d. The last paragraph of Section 2.5 states that "The design using the results of the design analyses on uniform elastic soil springs provides sufficient structural strength to resist the specified loads including bearing reactions on the underside of the basemat. However, this may require redistribution of the stresses locked in during early stages of construction." Explain the meaning of the second sentence quoted above since it is not clear how stresses locked-in during construction can be assumed to be redistributed for design purposes.

Westinghouse Response:

a. Settlement during construction is evaluated in non-linear analyses of a finite element model of the soil and nuclear island. The soil model is shown in Figure RAI-TR85-SEB1-19-1. Immediate settlements are based on elastic properties of the foundation medium, while the time-related settlements use creep parameters established by comparison against one-dimensional consolidation theory. The nuclear island uses the NI05 building model described in DCD Appendix 3G, subsection 3G.2.3. The construction sequence and history of loading are represented by finite time steps (the analysis uses 12 to 16 steps). Concrete placed during a time step is not considered to have structural strength until the beginning of the next time step. Structural forces and moments are extracted from the last iteration of each time step of the analysis.

b. Dead load member forces including the effect of settlement are those from the last step of the non-linear analyses. These member forces are compared to those from a Winkler spring analysis similar to the design analyses for the nuclear island basemat described in DCD subsection 3.8.5.4.1. Differences in member forces were evaluated for the following representative structures:

- 6' thick auxiliary building basemat
- Wall-1
- Wall-7.3
- Wall-M

The dead load member forces including the effect of settlement were added to other design loads and evaluated against the acceptance criteria for each loading combination. The evaluation demonstrates that the forces and moments due to various load combinations of dead, live and operating loads remain within the capacity of the respective structural elements.

c. The analyses described in parts (a) and (b) will be available for audit. These calculations show the effect of the three construction sequences, including the consideration of all applicable design basis loads.

d. The last paragraph of Section 2.5 will be revised as shown below.

Reference:

None

Design Control Document (DCD) Revision:

Revise eighth paragraph of subsection 3.8.5.3.2 as follows:

Member forces in the basemat considering settlement during construction differ from those obtained from the analyses on uniform elastic soil springs described in subsection 3.8.5.4.1. Although the bearing pressures at the end of construction are similar in the two analyses, the resulting member forces differ due to the progressive changes in structural configuration during construction. The design using the results of the analyses of subsection 3.8.5.4.1 provides sufficient structural strength to resist the specified loads including bearing reactions on the underside of the basemat. <u>The member forces in these analyses are those due to primary externally applied loads and do not consider secondary</u> stresses <u>and strains</u> locked in during early stages of construction. A confirmatory evaluation was performed to demonstrate that the member forces due to design basis loads, including locked-in forces due to construction settlement, remain within the capacity of the section. The evaluation was performed for critical locations which were selected as locations where the effect of locked in member forces were judged to be most significant.

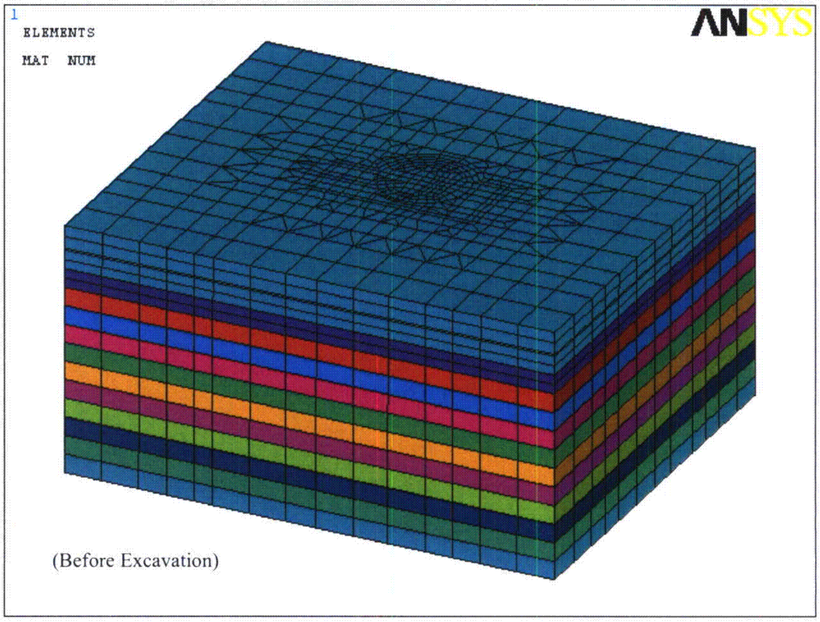
PRA Revision:

None

Technical Report (TR) Revision:

Revise last paragraph of section 2.5 as follows:

Member forces in the basemat considering settlement during construction differ from those obtained from the design analyses on uniform elastic soil springs. Although the bearing pressures at the end of construction are similar in the two analyses, the resulting member forces differ due to the progressive changes in structural configuration during construction. The design using the results of the design analyses on uniform elastic soil springs provides sufficient structural strength to resist the specified loads including bearing reactions on the underside of the basemat. The member forces in these analyses are those due to primary externally applied loads and do not consider secondary ~~However, this may require redistribution of~~ stresses and strains locked in during early stages of construction. A confirmatory evaluation was performed to demonstrate that the member forces due to design basis loads, including locked-in forces due to construction settlement, remain within the capacity of the section. The evaluation was performed for critical locations which were selected as locations where the effect of locked in member forces were judged to be most significant. The member forces for the load combination of dead load plus safe shutdown earthquake, including the member forces locked-in during various stages of plant construction, were within the design capacity for the critical locations. The evaluation demonstrated that the member forces including locked-in forces calculated by elastic analyses remain within the capacity of the section.



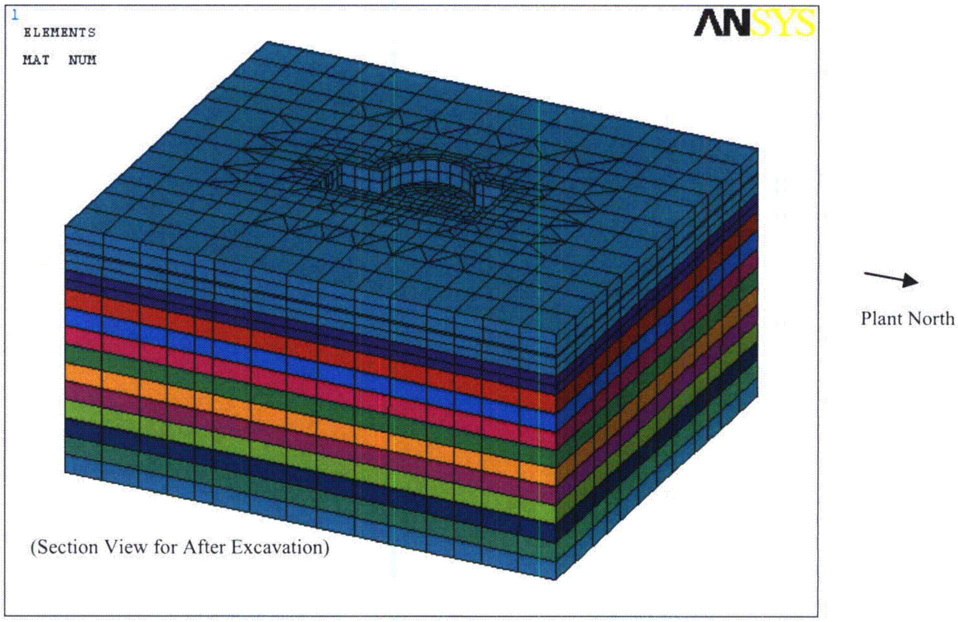


Figure RAI-TR85-SEB1-19-1 Analysis Soil Model (Sand&Clay Site)

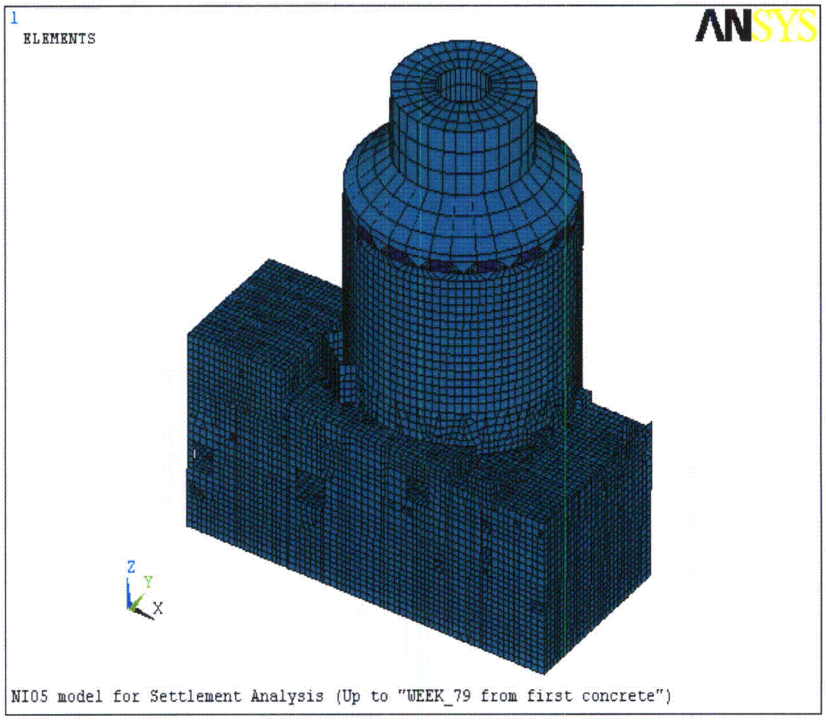


Figure RAI-TR85-SEB1-19-2 Nuclear Island Building Model

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR85-SEB1-32
Revision: 0

Question:

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

Westinghouse Response:

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

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

Design Control Document (DCD) Revision:

Revise first paragraph of subsection 2.5.4.2 as follows:

2.5.4.2 Bearing Capacity

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

The evaluation of the allowable capacity of the soil is based on the properties of the underlying materials (see subsection 2.5.4.5.2), including appropriate laboratory test data to evaluate strength, and considering local site effects, such as fracture spacing, variability in properties, and evidence of shear zones. The allowable bearing capacity should provide a factor of safety appropriate for the design load combination, including safe shutdown earthquake loads.

If the shear wave velocity or the allowable bearing capacity is outside the range evaluated for AP1000 design certification, a site-specific evaluation can be performed using the AP1000 basemat model and methodology described in subsection 3.8.5. The safe shutdown earthquake loads are those from the AP1000 analyses described therein. Alternatively, bearing pressures may be determined from a site-specific analysis using site-specific inputs as described in subsection 2.5.2.3. For the site to be acceptable, the bearing pressures from the site-specific analyses, including static and dynamic loads, need to be less than the capacity of each portion of the basemat.

Revise Tier 1 Table 5.0-1 and Tier 2 Table 2-1 as follows:

Soil		
	Average Allowable Static Bearing Capacity	Greater than or equal to 8,600 lb/ft^2 over the footprint of the nuclear island at its excavation depth
	Maximum Allowable Dynamic Bearing Capacity for Normal Plus SSE	Greater than or equal to 35,000 lb/ft^2 at the edge of the nuclear island at its excavation depth or, Site specific analyses demonstrate factor of safety appropriate for normal plus safe shutdown earthquake loads.

PRA Revision:

None

Technical Report (TR) Revision:

None

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR85-SEB1-36
Revision: 0

Question:

Section 5.1 presents the proposed revisions to DCD Tier 2, Table 2-1, which contains the Site Parameters including those for the soil media. Section 5.2 presents the proposed revisions to DCD Tier 1, Table 5.0-1, which also contains the Site Parameters for the soil. Considering that now the foundation of the AP1000 design has been extended to soil sites, both tables should include the additional items listed below or justification provided for not including the items.

a) Minimum required soil friction angle for soils below and adjacent to the NI. A minimum value of 35 degrees was used in the foundation stability calculations.

b) Settlement Criteria - maximum settlement at key locations (e.g., the corners of the basemat and west side of the shield building), maximum average settlement considering these key locations, maximum differential displacement (e.g., between key locations), and maximum differential displacement between any adjacent structures. Considering the relatively thin 6'-0" basemat for the NI, this criteria is considered important to ensure that there will not be significant settlement which might compromise the structural integrity of the NI basemat and foundation. Also, meeting differential settlement criteria would maintain adequate gap with adjacent structures under seismic loadings to preclude impact. The approach or basis for the selected settlement values should be described.

Westinghouse Response:

a) The minimum required soil friction angle has been added to both Tables 2-1 and 5.0-1 in accordance with Westinghouse's response to RAI-TR85-SEB1-37.

b) DCD subsection 2.5.4.6.11 requires the Combined License applicant to evaluate settlement at soil sites. These evaluations may be performed assuming rigid basemat behavior of the nuclear island and the adjacent buildings.

The effect of settlement on the nuclear island basemat during construction has been considered in the design of the nuclear island as described in Section 2.5 of the report and in DCD subsection 3.8.5.4.2. These analyses consider the flexibility of the basemat during construction. They consider a soft soil site with properties selected to maximize the settlement during construction. These analyses show total settlements of about one foot. The analyses demonstrate that even this significant settlement does not compromise the structural integrity of the NI basemat and foundation.

Westinghouse has established guidance on settlement for the Combined License applicant shown in Table TR85-SEB1-36-1. If site specific settlement analyses predict settlement below the values in this table, the site is acceptable without additional evaluation. If the analyses predict greater settlement, additional evaluation will be performed. This may include specification of the initial building elevations, specification of the stage of construction and settlement for making connections of systems between buildings, etc. It would also include review of the effect of the rotation of buildings and its effect on the gap between adjacent structures. These analyses would provide the basis for review of settlement measurements during construction and subsequent operation.

Acceptable differential settlement between buildings without additional evaluation is identified as 3 inches between the Nuclear Island and the Turbine Building, the Annex Building, and the Radwaste Building. The 3 inches is measured from the center of the Containment Building to the center of the Turbine Building, center of the Annex Building, or the center of the Radwaste Building. Each building, including the Nuclear Island, also has a settlement criterion of no more than ½ inch in 50 feet in any direction. The Nuclear Island has a maximum absolute settlement value of 3 inches.

TABLE TR85-SEB1-36-1
Limits of Acceptable Settlement Without Additional Evaluation

Differential Across Nuclear Island Foundation Mat	Total for Nuclear Island Foundation Mat	Differential between Nuclear Island and Turbine Building.	Differential between Nuclear Island and Other Buildings
½ inch 50 ft	3 inches	½ inch	½ inch

References: None

Design Control Document (DCD) Revision:

Revise subsections 2.5.4.3 and 2.5.4.6.10 and add Table 2.5-1 as follows:

2.5.4.3 Settlement

The Combined License applicant will address short-term (elastic) and long-term (heave and consolidation) settlement for soil sites for the history of loads imposed on the nuclear island foundation and adjacent buildings consistent with the construction sequence. The resulting time-history of settlements includes construction activities such as dewatering, excavation, bearing surface preparation, placement of the basemat, and construction of the superstructure. The settlement under the ~~nuclear island~~building footprint is represented in the distribution of subgrade stiffness.

The AP1000 does not rely on structures, systems, or components located outside the nuclear island to provide safety-related functions. Differential settlement between the nuclear island foundation and the foundations of adjacent buildings does not have an adverse effect on the safety-related functions of structures, systems, and components. Differential settlement under the nuclear island foundation could cause the basemat and buildings to tilt. Much of this settlement occurs during civil construction prior to final installation of the equipment. Differential settlement of a few inches across the width of the nuclear island would not have an adverse effect on the safety-related functions of structures, systems, and components. Table 2.5-1 provides guidance to the Combined License applicant on predictions of absolute and differential settlement that are acceptable without further evaluation. When the predicted settlement exceeds these values, the Combined License applicant will describe any special construction provisions to accommodate the predicted settlement.

2.5.4.6.11 Settlement of Nuclear Island – Data will be provided on short-term (elastic) and long-term (heave and consolidation) settlement for soil sites for the history of loads imposed on the nuclear island foundation and adjacent buildings consistent with the construction sequence. The resulting time-history of settlements includes construction activities such as dewatering, excavation, bearing surface preparation, placement of the basemat, and construction of the superstructure. Special construction requirements will be described, if required, to accommodate settlement predicted to exceed the values shown in Table 2.5-1.

TABLE 2.5-1
Limits of Acceptable Settlement Without Additional Evaluation

DIFFERENTIAL ACROSS NUCLEAR ISLAND FOUNDATION MAT	TOTAL FOR NUCLEAR ISLAND FOUNDATION MAT	DIFFERENTIAL BETWEEN NUCLEAR ISLAND AND TURBINE BUILDING.	DIFFERENTIAL BETWEEN NUCLEAR ISLAND AND OTHER BUILDINGS
½ inch 50 ft	3 inches	½ inch	½ inch

PRA Revision:

None

Technical Report (TR) Revision:

None