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

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

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

RAI-TR85-SEB-1-10
RAI-TR85-SEB-1-27
RAI-TR85-SEB-1-32
RAI-SRP-3.8.3-SEB-1-04

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,

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/Enclosure

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

D063
NRO

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ENCLOSURE 1

Response to Request for Additional Information on SRP Section TR85

RAI Response Number: RAI-TR85-SEB1-10
Revision: 5

Question:

Section 2.4.1 indicates that "Table 2.4-2 shows the reactions at the underside of the basemat for each soil case. These are conservative estimates using the results of the 2D SASSI horizontal analyses..." The following items need to be addressed:

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

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

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

Additional Request (Revision 1):

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

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

b. The use of the SRSS or the 100/40/40 combination method is only acceptable for combining the co-directional responses such as Mxx due to NS, EW, and vertical, in order to obtain a combined Mxx. However, it is not clear from TR 85, DCD Section 3.8.5, nor from the RAI response, how the stability calculations are performed once the individual three loads Mxx, Myy, and vertical (each of these already combined by SRSS or 100/40/40 due to the three earthquake inputs) are determined. DCD 3.8.5.5.4, for example, discusses the overturning evaluation and presents the equation for the factor of safety as the resisting moment divided by the overturning moment. However, this does not explain how the vertical seismic force is considered. The traditional method for evaluating stability (sliding and overturning) of nuclear plant structures in accordance with SRP 3.8.5 is to perform two separate 2-D evaluations, one

for the N-S and vertical directions and one for the E-W and vertical directions. Thus, for overturning evaluation as an example, the minimum vertical downward load (deadweight minus upward buoyancy force minus upward vertical seismic force) is considered in calculating the resisting moment and this is then compared to the overturning moment about one horizontal direction (i.e., EW axis); then a similar comparison is made for the same minimum downward vertical load with the overturning moment about the other perpendicular horizontal direction (i.e., NS axis). Westinghouse is requested to clarify if they follow this analytical method for the stability evaluations (sliding and overturning) and document the approach in TR85 and the DCD. If not, then Westinghouse is requested to justify any other alternative method used. Note, with the changes recently made in the various seismic analyses, explain whether the maximum seismic reactions in Table 2.4-2, developed from the 2D SASSI analyses, are still relied upon for use in the stability evaluations performed in Section 2.9 of TR85.

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

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

Additional Request (Revision 2):

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

Additional Request (Revision 3):

1. Remove the F_P term in the equations and explain the removal in DCD sections 3.8.5.5.3 and 3.8.5.5.4.
2. Check the reference to Table 3.8.5-2 and clarify the reference in the second paragraph of the revised DCD markup in the RAI response.
3. Check DCD Table 3.8.5-2 for use of zero passive pressure and explain or justify use.

Additional Request (Revision 4):

a) The staff reviewed the response provided in Westinghouse letter dated September 22, 2009 and found that insufficient information was provided. In the response, the passive earth pressure was removed from the seismic stability analysis, an explanation was provided why reliance on soil passive pressure is not required for stability evaluation, and related tables were revised in the corresponding subsections of the DCD and TR-85. This information is subject to an audit for its adequacy.

b) As a result of the staff's structural audit conducted during the week of August 10, 2009, the NRC staff requested the justification as to why TR-85 is not identified as Tier 2* since it is referenced in DCD Section 3.8.5 and it contains key details of the design of the foundation. Similarly, justification as to why TR-9 (Containment Vessel Design Adjacent to Large Penetrations) and TR-57 (Nuclear Island: Evaluation of Critical Sections) are not identified as Tier 2* information because they contain key analysis and design information for the containment, and the auxiliary and shield buildings, which are not sufficiently described in the DCD, was not provided. Therefore, the staff requests either that TR-9, TR-57, and TR-85 be identified as Tier 2* information in the DCD, or a justification provided.

Additional Request (Revision 5):

During the NRC seismic audit conducted during the week of June 14 to 18, 2010, the NRC staff discussed with Westinghouse the response to Revision 4 of this RAI. The staff clarified that the request to identify selected technical reports as Tier 2* information was to assure that important information on the design and analysis of structures is captured as information that requires review before it is revised or changed. Key details of the analysis and design of the containment, auxiliary building, shield building, and foundation need to be included in the licensing basis. Westinghouse should provide a means to accomplish this. The technical reports that should be considered when identifying the key details of the analysis and design include TR-03, TR-9, TR-57, TR-85, TR-115, and the shield building report.

It is requested that changes in the nuclear island seismic sliding displacements due to modifications of the non-linear seismic sliding model be reflected in this RAI.

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 <u>absolute</u> combination of the effects of the horizontal shear forces and the moments. The horizontal loads on the portion below grade are added <u>absolutely</u> to the sum of the member forces above grade.

b. As described in DCD subsection 3.7.2.6,

> In analyses with the earthquake components applied separately and in the response spectrum and equivalent static analyses, the effect of the three components of earthquake motion are combined using one of the following methods:

- The peak responses due to the three earthquake components from the response spectrum and equivalent static analyses are combined using the square root of the sum of squares (SRSS) method.

- The peak responses due to the three earthquake components are combined directly, using the assumption that when the peak response from one component occurs, the responses from the other two components are 40 percent of the peak (100 percent-40 percent-40 percent method). Combinations of seismic responses from the three earthquake components, together with variations in sign (plus or minus), are considered. This method is used in the nuclear island basemat analyses, the containment vessel analyses and the shield building roof analyses.

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

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

Westinghouse Response (Revision 1):

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

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

> "As for the suitability of using the 100 percent, 40 percent, 40 percent combination method, the applicant, during audits performed by the staff, provided calculations to demonstrate that the combination method always gives reasonable results by comparing the results with those from the SRSS combination method. From its review of the design calculations, the staff also finds that the difference between results obtained using the

two methods was less than 5 percent which is considered insignificant and, therefore, is acceptable."

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

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

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

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

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

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

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

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

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

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

Units: inches

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

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

Seismic Overturning Stability Evaluation

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

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

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

Seismic Sliding Evaluation

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

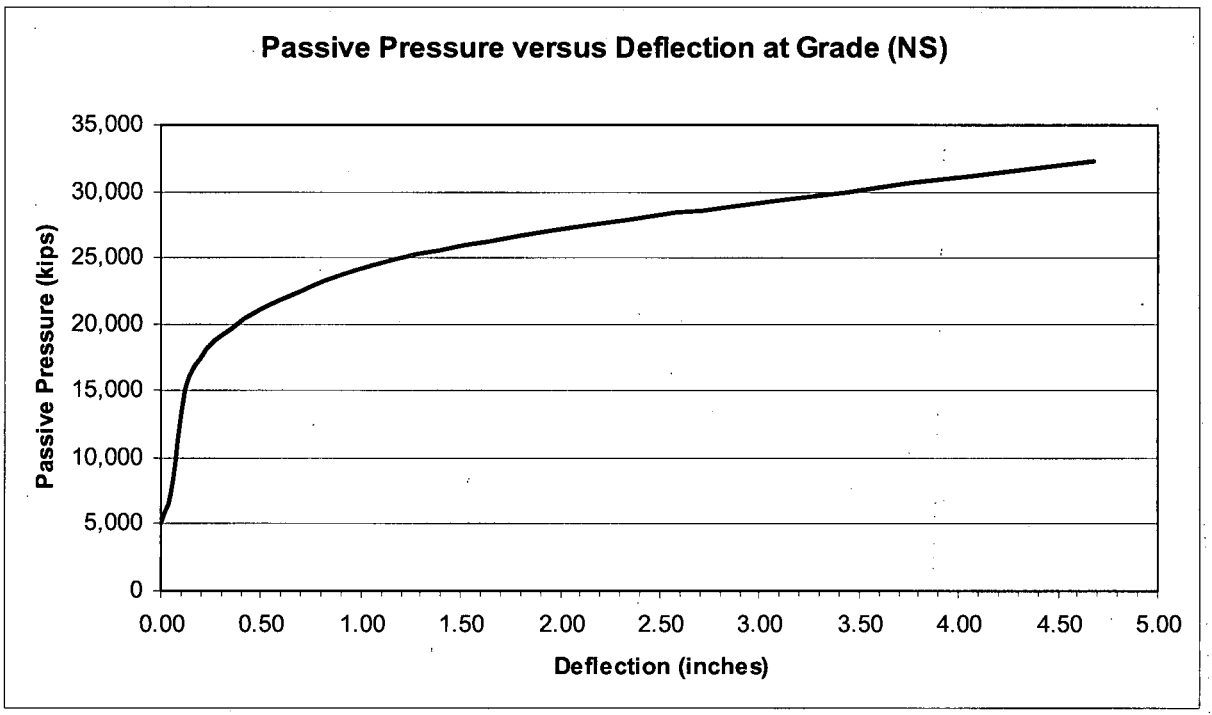


Figure RAI-TR85-SEB1-10-1 – Passive Pressure versus Deflection at Grade (North-South Excitation)

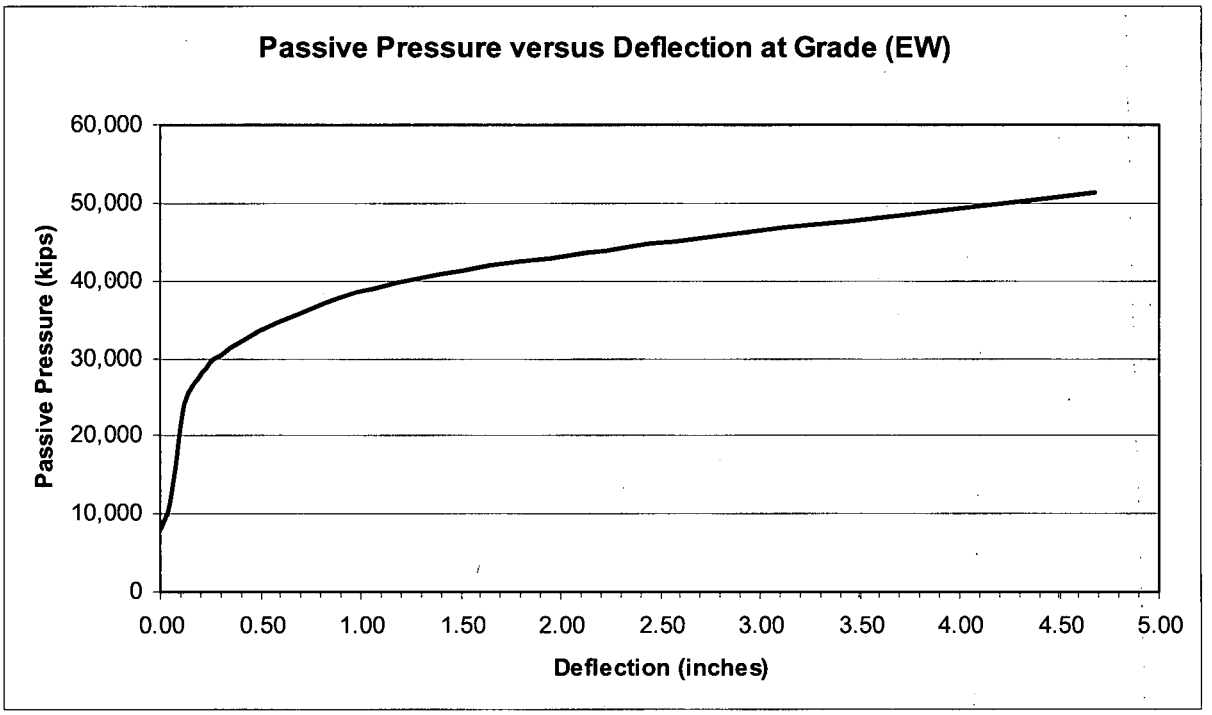


Figure RAI-TR85-SEB1-10-2 – Passive Pressure versus Deflection at Grade (East-West Excitation)

During the sliding stability calculation it was determined that the factor of safety for sliding drops below the limit of 1.1 for a very short time if passive pressure is not considered. Plots of the factor of safety (FS) versus time for the hard rock case and the North-South and East-West directions are given in Figures RAI-TR85-SEB1-10-3 and RAI-TR85-SEB1-10-4 using a coefficient of friction of 0.55. As seen from these figures the time at which the factor of safety drops below 1.1 is very short. 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-SEB1-10-3 to RAI-TR85-SEB1-10-5 are given a summary of the results for the three coefficient values. Provided is the required passive pressure to maintain the factor of safety equal to or above 1.1. As seen from this summary using a coefficient of friction of 0.55 or higher, deflections less than 0.15 inch for hard rock, less than or equal to 0.5 inch for upper bound soft to medium and soft to medium soil conditions are needed to develop the required amount of passive pressure.

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

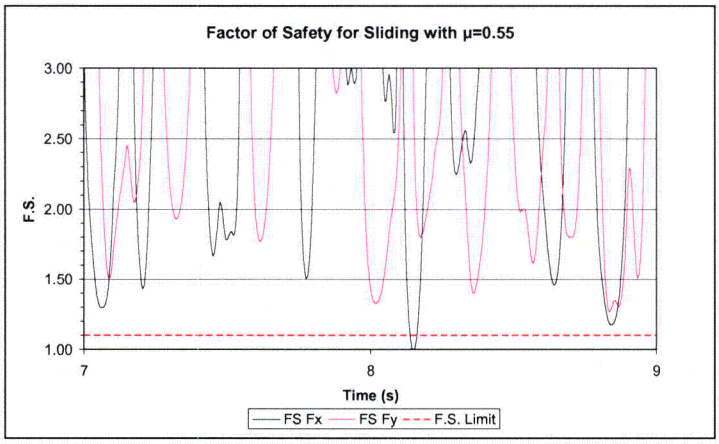


Figure RAI-TR85-SEB1-10-3 - North-South FS without Passive Pressure

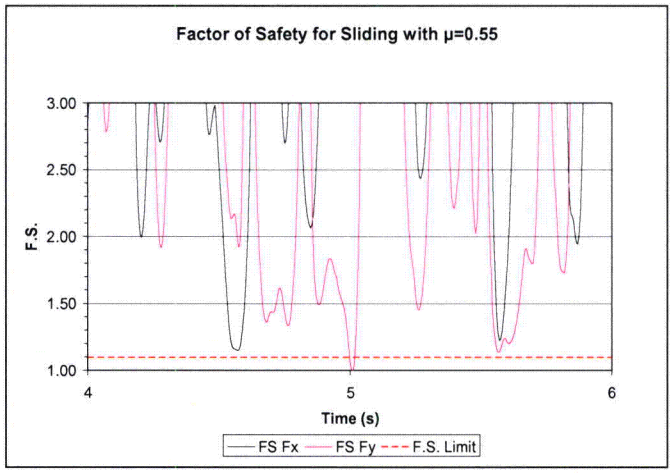


Figure RAI-TR85-SEB1-10-4 – East-West FS without Passive Pressure

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

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

Note (1) - At rest pressure

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

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

Note (1) - At rest pressure

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

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

Note (1) - At rest pressure

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

Westinghouse Response (Revision 2):

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

as shown in Figures RAI-TR85-10-3 and RAI-TR85-10-4. It was requested that Westinghouse perform a more realistic non-linear analysis with sliding friction elements using a 2D ANSYS model.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Case	Deflection @ 60.5' El Without buoyant force inches	Deflection @ 60.5' El With buoyant Force Inches
HR	0.02	0.06
UBSM	0.08	0.15
SM	0.12	0.19

Reference:

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

Westinghouse Response (Revision 3):

Clarified the safe shut down earthquake sliding equation in DCD section 3.8.5.5.3 removing F_P, F_H, and clarifying the definition for F_D. The M_P term in the equation in DCD section 3.8.5.5.4 is removed. A sentence is added to both sections to explain why those terms are not included. In the second paragraph of DCD section 3.8.5.5.4 the phrase "the static moment balance approach" is removed and replaced by "time history analysis."

The second paragraph of the revised DCD markup for Subsection 3.8.5.5.5 is modified to remove confusion related to passive pressure. The reference to Table 3.8.5-2 in Subsection 3.8.5.5.5 is removed.

Seismic deflections at the bottom of the Nuclear Island Basemat due to sliding (coefficient of friction equal to 0.55) are given for both cases with/without buoyant force in the DCD (buoyant force deflections are added to Table RAI-TR85-SEB1-10-13).

Footnote (3) in DCD Table 3.8.5-2 will be removed to clarify that the values in the table use zero passive pressure. .

Westinghouse Response (Revision 4):

a) The response to Revision 3 of this RAI was discussed with the NRC staff prior to submittal. The response to Revision 3 and the associated DCD revisions provided the changes requested by the staff during these conversations. The responses are supported by the analyses that support the stability evaluation. It is not practicable to include the information in the supporting analyses in the RAI response or DCD mark-up. Westinghouse will schedule a time for the staff to audit theses analyses.

b) APP-GW-GLR-044 (TR-85), APP-GW-GLR-005 (TR-09), and APP-GW-GLR-045 (TR-57) were prepared to support and inform the NRC of changes to structural design of the AP1000 in the design certification amendment. These changes included completion and design finalization of portions of the structural design. The design completion addressed COL information items by completion of design activities that were not complete at the time of design certification. Design finalization changes to the DCD included changes due to the expansion of the seismic response spectra in the Design Certification amendment and were incorporated in the critical section information in the DCD. These reports were formally transmitted by letter to the NRC and are included on the AP1000 Design Certification docket.

The design basis for the structural design of the AP1000 is documented in calculations and design reports that are available for review and audit. The information included in the DCD should be sufficient to satisfy the guidance of the Standard Review Plan (SRP). The SRP includes guidance that design criteria and design and analysis methods be included in the DCD. The SRP guidance does not include the inclusion of design details in the DCD.

Documents in the DCD currently identified as Tier 2* are codes and standards or are topical reports that provide methods and criteria. Defining the subject documents as Tier 2* would include excessive detail as Tier 2* information. The subject reports include the results of calculations. Because reanalysis may result in small changes in the results these results should not be identified as Tier 2*. Incorporating the subject reports as Tier 2* would require NRC approval for changes at a level of detail that is not appropriate.

If there is specific information in the subject documents that the staff considers to be in the nature of criteria and methods and should be included in the DCD that information should be identified for inclusion in the DCD. Given the justification in the preceding paragraphs Westinghouse will not be incorporating APP-GW-GLR-044 (TR-85), APP-GW-GLR-005 (TR-09), and APP-GW-GLR-045 (TR-57) as Tier 2* information.

Westinghouse Response (Revision 5):

Westinghouse will review the information in RAI responses and the structural technical reports for the key analysis and design information that should be included in the AP1000 Design Control Document (DCD). Consistent with the Standard Review Plan, this information is expected to include design criteria, design methods, modeling requirements, analysis methods, and limits. This information will be included in the DCD so that the staff will not rely on information in the technical reports to support the SER conclusions. Commitments to the NRC included in the technical reports will be included. Modifications to standard industry methods and computer codes will be identified.

The review will be completed after the responses for the remaining unresolved RAIs are finalized. Westinghouse will provide DCD mark-ups for the complete Sections 3.7 and 3.8 identifying the information to be added or revised. Appendices 3G and 3I will also be reviewed for revisions.

Subsequent to agreement on the content of the DCD, portions of the DCD that should be identified as Tier 2* will be identified.

The revised nuclear island seismic sliding displacements due to modifications of the non-linear seismic sliding model are reflected in this RAI. The revised nuclear island seismic sliding displacement values are reflected in Table RAI-TR85-SEB1-10-13, DCD subsection 3.8.5.5.5, footnote 2 of DCD Table 3.8.5-2, TR85 section 2.9, and footnote (2) of table 2.9-1.

Design Control Document (DCD) Revision:

Modify the first sentence in the last paragraph of DCD subsection 3.4.1.1.1.1, Revision 17, to read as follows:

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

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

3.8.5.5.3 Sliding

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

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

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:

$$F.S. = \frac{F_s}{F_D}$$

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~~ seismic force from safe shutdown earthquake

~~F_H = maximum lateral force due to all loads except seismic loads~~

The sliding resistance is based on the friction force developed between the basemat and the foundation. The governing friction value in the interface zone is a thin soil layer below the mudmat with an angle of internal friction of 35° giving a static coefficient of friction of ~~0.70~~0.55. The effect of buoyancy due to the water table is included in calculating the sliding resistance. ~~F_P~~ Passive soil pressure resistance is not included in the equations above because passive pressure is not considered for sliding stability. Since there is no passive pressure considered, active and overburden soil pressures are also not considered.

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~~time history analysis assuming overturning about the edge of the nuclear island at the bottom of the basemat. The factor of safety is defined as follows:

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

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. ~~M_P~~Resistance moment due to passive pressure is not included in the equation above because passive pressure is not considered for overturning stability.

3.8.5.5.5 Seismic Stability Analysis

The factors of safety for sliding and overturning for the SSE are calculated for each soil case for the base reactions in terms of shear and bending moments about column lines 1, 11, I and the west side of the shield building at each time step of the seismic time history. The 2D SASSI reactions (Fx, Fy, and Fz) are used to obtain seismic response factors between the hard rock case to the upper-bound-soft-to-medium (UBSM) soil case, and the soft-to-medium (SM) soil case. These factors were used to adjust the hard rock time history to reflect the seismic response for the other two potential governing soil cases UBSM and SM. The firm rock, soft rock, and soft soil cases have higher factors of safety against sliding and therefore not considered.

A non-linear analysis with sliding friction elements using a 2D ANSYS model was performed. The 2D ANSYS model that was used to study the basemat uplift (see Subsection 3.8.5.5.6 and Appendix 3G). This 2D non-linear model is for the East-West direction. There is no need to consider the North-South direction since the NI deflections calculated to maintain a factor of safety of 1.1 is largest in the East-West direction. This model was modified introducing friction elements along the bottom of the basemat and soil media interface. Direct time integration analysis was performed with vertical uplift and sliding allowed. The three cases that have the lowest factor of safety related to sliding were evaluated. These three cases are HR, UBSM, and SM. The seismic input was increased by 10% to maintain the factor of safety against sliding of 1.1. No passive soil resistance is considered. The resulting maximum displacement at the base of the NI basemat (EL 60.5') using a coefficient of friction of 0.55 is 0.12" without buoyant force consideration, and 0.19" with buoyant force considered. This is negligible sliding during the seismic event, and no passive soil resistance is necessary from the backfill (side soil). Therefore, it can be concluded that the Nuclear Island is stable against sliding, and there is no quality requirement for the backfill material adjacent to the NI (side soil) to maintain stability against sliding.

The minimum seismic stability factors of safety values are reported in Table 3.8.5-2.

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

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

3.8.7 References

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

Table 3.8.5-2

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

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

Note:

1. Factor of safety is calculated for the envelope of the soil and rock sites described in subsection 3.7.1.4.
2. From non-linear sliding analysis using friction elements the horizontal movement is negligible (0.12"without buoyant force consideration, and 0.19" with buoyant force considered). ~~Factor of safety is shown for soils below and adjacent to nuclear island having angle of internal friction of 35 degrees.~~
3. ~~ASCE/SEI 43-05, Reference 42, recognizes that there is considerable margin beyond that given by the moment balance formula and permits a nonlinear rocking analysis. The nonlinear (liftoff allowed) time history analysis described in Appendix 3G.10 showed that the nuclear island basemat uplift effect is insignificant. Further, these analyses were performed for free field seismic ZPA input as high as 0.5g without significant uplift. Therefore the factor of safety against overturning is greater than 1.67 (0.5g/0.3g).~~

APPENDIX 3G NUCLEAR ISLAND SEISMIC ANALYSES

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

Analyses were performed in accordance with the criteria and methods described in Section 3.7. Section 3G.2 describes the development of the finite element models. Section 3G.3 describes the soil structure interaction analyses of a range of site parameters and the selection of the parameters used in the design analyses. Section 3G.4 describes the fixed base and soil structure interaction dynamic analyses and provides typical results from these dynamic analyses. In Reference ~~36~~ are provided a summary of dynamic and seismic analysis results (i.e., modal model properties, accelerations, displacements response spectra) and the nuclear island liftoff analyses. The seismic analyses of the nuclear island are summarized in a seismic analysis summary report. Deviations from the design due to as-procured or as-built conditions are acceptable based on an evaluation consistent with the methods and procedures of Sections 3.7 and 3.8 provided the following acceptance criteria are met:

3G.5 References

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

PRA Revision:

None

Technical Report (TR) Revision:

~~None~~

The following modifications are Post Revision 1.

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

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

Modify Section 2.9 as follows:

2.9 Nuclear island stability

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

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

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

The factors of safety for sliding and overturning for the SSE are calculated for each soil case for the base reactions in terms of shear and bending moments about column lines 1, 11, I and the west side of the shield building at each time step of the seismic time history. The 2D SASSI reactions (Fx, Fy, and Fz) are used to obtain seismic response factors between the hard rock case to the upper-bound-soft-to-medium (UBSM) soil case, and the soft-to-medium (SM) soil case. These factors were used to adjust the hard rock time history to reflect the seismic response for the other two potential governing soil cases UBSM and SM. The firm rock, soft rock, and soft soil cases have higher factors of safety against sliding and therefore not considered.

The seismic time history analysis used the ANSYS computer code and the NI20 model. The minimum stability factors of safety values are reported in Table 2.9-1. For seismic overturning no passive pressure was considered. For sliding partial passive pressure is considered. Two soil cases are considered for sliding, the soil parameters used for design (friction angle of 35°, and submerged weight of 87.6 pcf), and a lower bound soil density (friction angle of 35°, and submerged weight of 60 pcf). For the design case the amount of passive pressure required to meet the 1.1 factor of safety is 40% for the North-South seismic event, and 47% of the East-West excitation of full passive pressure. For the lower bound case the amount of passive pressure required to meet the 1.1 factor of safety is less than 53% for the North-South seismic event, and 64% of the East-West excitation of full passive pressure. The relationship between passive pressure and displacement at grade is obtained based on the methodology given in Reference 10. The relationship between passive pressure and displacement at grade is shown in Figures 2.9-1 and 2.9-2. The maximum Nuclear Island displacement of the Nuclear Island at grade to develop the required passive resistance is 0.5" for the design case, and 2.3" for the lower bound case. These deflections are based on conservative equivalent static analysis. This will result in large deflections since

the seismic loads are considered to be constant and do not reflect the short time duration that they exist during the seismic event. A more realistic non-linear analysis with sliding friction elements using a 2D ANSYS model was performed. The 2D ANSYS model that was used to study the basemat uplift (see Subsection 2.4.2). This 2D non-linear model is for the East-West direction. There is no need to consider the North-South direction since the NI deflections calculated to maintain a factor of safety of 1.1 is largest in the East West direction. This model was modified introducing friction elements along the bottom of the basemat and soil media interface. Direct time integration analysis was performed with vertical uplift and sliding allowed. The three cases that have the lowest factor of safety related to sliding were evaluated. These three cases are HR, UBSM, and SM. The seismic input was increased by 10% to maintain the factor of safety against sliding of 1.1. No passive soil resistance is considered. The resulting maximum displacement at the base of the NI basemat (EL 60.5') using a coefficient of friction of 0.55 is 0.12" without buoyant force consideration, and 0.19" with buoyant force considered. This is negligible sliding during the seismic event, and no passive soil resistance is necessary from the backfill (side soil). Therefore, it can be concluded that the Nuclear Island is stable against sliding, and there is no quality requirement for the backfill material adjacent to the NI (side soil) to maintain stability against sliding.

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

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

Notes:

(1) No passive pressure is considered.

(2) No passive pressure is considered. From non-linear sliding analysis using friction elements the horizontal movement is negligible (0.12"without buoyant force consideration, and 0.19" with buoyant force considered). ~~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.~~

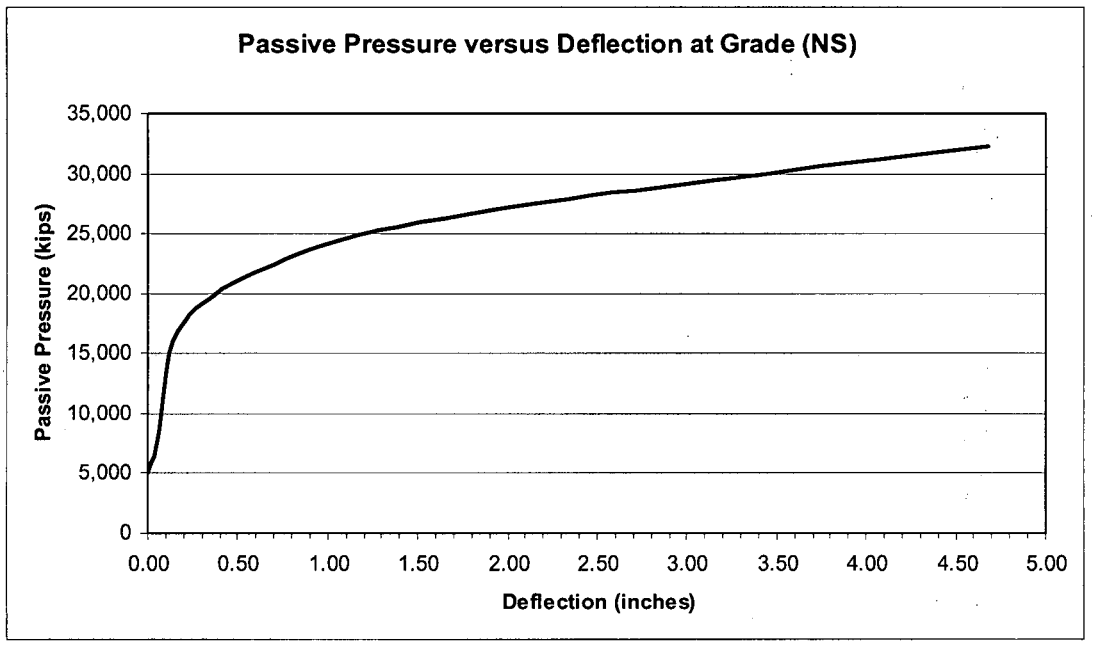


Figure 2.9-1 – Passive Pressure versus Deflection at Grade (North-South Excitation)

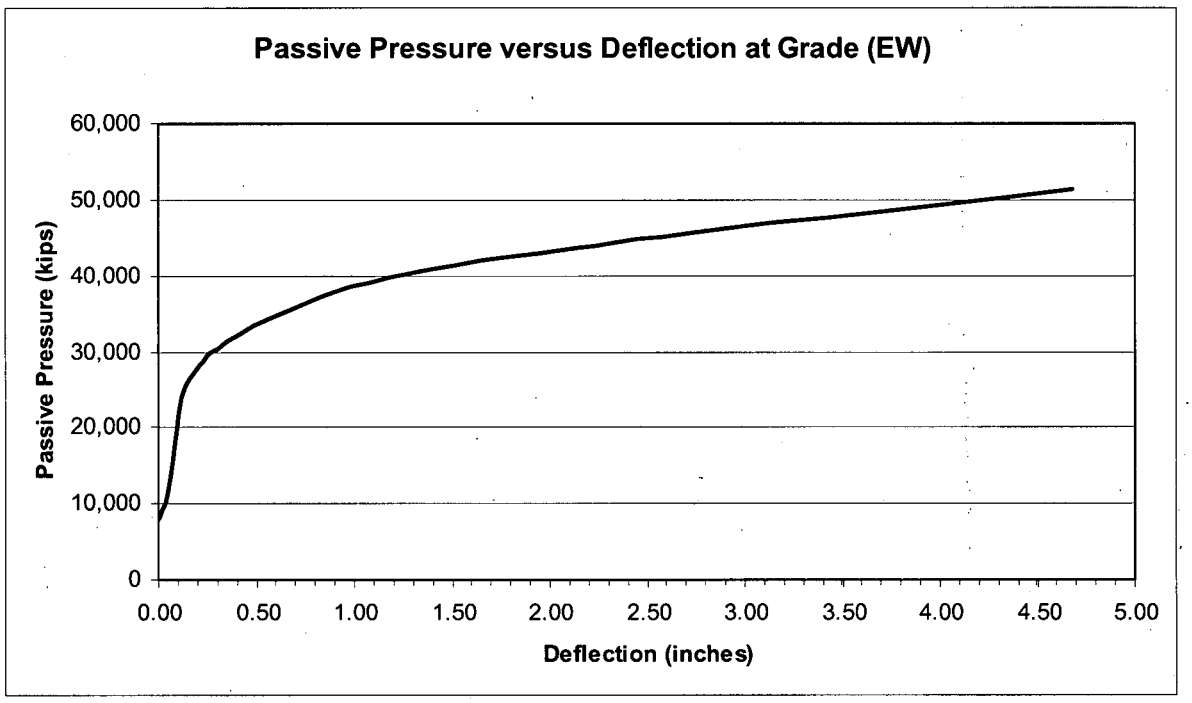


Figure 2.9-2 – Passive Pressure versus Deflection at Grade (East-West Excitation)

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-SEB1-27
Revision: 2

Question:

Section 2.6.1.4 discusses liftoff analyses performed for 16 load cases of dead, live, and seismic loads. The results of the analyses are used for the basemat design. Explain what the 16 load cases correspond to and why weren't all possible load combinations utilized in accordance with the 100/40/40 rule.

Additional Request (Revision 1):

The response explains that the non-linear analyses were not performed for the cases with 1.0 applied in the vertical direction since the maximum bearing demand is due to overturning rather than vertical seismic. It is not evident that this would always be the case. Please explain.

The purpose of the calculation described in Section 2.6 is not to find the maximum bearing pressure under the basemat, but rather to design the entire basemat. Explain your confidence that at other locations away from the maximum bearing pressure location, the combination with 1.0 applied to the vertical direction may not govern (e.g., near the line of rotation where overturning contribution would be very small). The applicant is requested to consider the 100/40/40 combination method for considering the three earthquake directions as it was intended, and not to drop certain combinations based on a judgment that has not been justified.

Additional Request (Revision 2):

- SCV
  - Identify where 100/40/40 was applied
  - Describe methodology for analysis
  - Develop justification for the use of the 100/40/40 method for the SCV
- Roof (Tension ring and air inlet)
  - Identify where 100/40/40 was applied to confirm the tension ring and air inlet were the only two locations.
  - Describe methodology for analyses – Explain in sufficient detail (before proceeding with the evaluation) how the two analyses below are performed (e.g., what models, what seismic inputs, type of analysis, etc.)
    - 100/40/40
    - Linear elastic analysis (ANSYS)
  - Compare member forces at key locations in the tension ring and air inlet for both methods used. The key locations consist of representative locations throughout the two structural regions identified above where the 100/40/40 gave results within 20% of the code limits. Provide discussion of the results and justification of 100/40/40.

Westinghouse Response (Revisions 0 and 1):

Linear analyses were performed for all 24 cases of the 100/40/40 combination method in the initial hard rock analyses and used to select the 16 cases analyzed in subsequent non-linear analyses. Typical results are shown in Figure RAI-TR85-SEB1-027-01. The cases with unit load factor in the vertical direction are load cases 17 to 24. The following 16 cases for the 1.0, 0.4, 0.4 method were used. Non-linear analyses were not performed for the cases with 1.0 applied in the vertical direction since the maximum bearing and member force demand is due to overturning rather than vertical seismic.

Load Case 3-# = D+L+Es where Es takes the following forms for each #.

#=1:	Es= 1.0xSns	+0.4xSew	+0.4xSvt
#=2:	Es= 1.0xSns	+0.4xSew	-0.4xSvt
#=3:	Es= 1.0xSns	-0.4xSew	+0.4xSvt
#=4:	Es= 1.0xSns	-0.4xSew	-0.4xSvt
#=5:	Es= -1.0xSns	+0.4xSew	+0.4xSvt
#=6:	Es= -1.0xSns	+0.4xSew	-0.4xSvt
#=7:	Es= -1.0xSns	-0.4xSew	+0.4xSvt
#=8:	Es= -1.0xSns	-0.4xSew	-0.4xSvt
#=9:	Es= 0.4xSns	+1.0xSew	+0.4xSvt
#=10:	Es= 0.4xSns	+1.0xSew	-0.4xSvt
#=11	Es= 0.4xSns	-1.0xSew	+0.4xSvt
#=12	Es= 0.4xSns	-1.0xSew	-0.4xSvt
#=13:	Es= -0.4xSns	+1.0xSew	+0.4xSvt
#=14:	Es= -0.4xSns	+1.0xSew	-0.4xSvt
#=15:	Es= -0.4xSns	-1.0xSew	+0.4xSvt
#=16:	Es= -0.4xSns	-1.0xSew	-0.4xSvt

where,

Sns element forces due to SSE acceleration in X (NS)
Sew element forces due to SSE acceleration in Y (EW)
Svt element forces due to SSE acceleration in Z (VT)

Westinghouse Response (Revision 2):

DCD subsection 3.7.2.6 describes the use of the 100-40-40 method as follows:

> 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:
>
> - For seismic analyses with the statistically independent earthquake components applied separately, the time-history responses from the three earthquake components are combined algebraically at each time step to obtain the combined response time-history. This method is used in the SASSI analyses.
> - 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 from the equivalent static analyses 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.

The third bullet is clarified to show that the 100/40/40 method is only used to combine the results of linear and non-linear equivalent static analyses.

As stated in the third bullet, the 100/40/40 method is used in the nuclear island basemat analyses, the containment vessel analyses and the shield building roof analyses.

For AP1000 equivalent static analyses Westinghouse carried the 24 combinations of 100/40/40 through the detailed design calculations and then enveloped the results. This is similar to the approach for other load combinations. For the containment vessel detail evaluation, stresses were combined to calculate stress intensity and critical load cases were evaluated for buckling. For the reinforced concrete structures, the required reinforcement was calculated for the combination of member forces. Typical comparisons against time history or response spectrum analyses are provided below for the containment vessel and the shield building roof to demonstrate that the approach taken is conservative. Application to the nuclear island basemat is addressed in the response to RAI-TR85-SEB1-32, Rev 4.

The 100/40/40 method is a conservative extension of methodology used for many years in seismic design. Design for two horizontal direction of input is specified in the UBC as follows:

> The requirement that orthogonal effects be considered may be satisfied by designing such elements for 100 percent of the prescribed design seismic forces in one direction plus 30 percent of the prescribed design seismic forces in the perpendicular direction. The combination requiring the greater component strength shall be used for design. Alternatively, the effects of the two orthogonal directions may be combined on a square-root-sum-of-the squares (SRSS) basis. When the SRSS method of combining directional effects is used, each term computed shall be assigned the sign that will result in the most conservative design.

This methodology was expanded to three directions and the 30% increased to 40% in nuclear plant applications. Note that the UBC recognizes that the 100/40/40 method provides appropriate relative signs for the member forces so that the strength can be evaluated for each of the cases and the maximum value used. The SRSS method requires consideration of the worst combination of signs for each term (or member force).

The combination of the three directions of seismic input in equivalent static analyses is not addressed in either R.G 1.92 or ASCE 4-98. Both R.G 1.92 and ASCE 4-98 provide guidance for response spectrum analyses. This guidance applies to cases where seismic results have already been combined by SRSS so the signs of the individual components have been lost. As a result the variation of the signs needs to be addressed in the detailed design calculation as described in paragraph 3.2.7.1.3 of ASCE 4-98. Part (a) takes the SRSS or envelope of 100/40/40 of all member forces and then all possible signs which is conservative. Part (b) provides a fairly complicated way to reduce this conservatism. This alternate method leads to realistic multiple responses that could occur simultaneously. Such an approach is not necessary for equivalent static analyses combined by the 100/40/40 method since each combination includes appropriate signs on the member force components.

In the Shield Building report Westinghouse did measure the conservatism of the SRSS approach used in design against an application of the 100/40/40 method to the response spectrum analyses. While not directly in accord with the ASCE 4-98 guidance, the degree of conservatism could have been measured against the methods of ASCE 4-98 subsection 3.2.7.1.3 party (b) and a similar degree of conservatism would be expected.

Steel Containment Vessel (SCV)

The design of the steel containment vessel for the analysis of the large penetrations (personnel locks and equipment hatches) utilizes the 100/40/40 combination method. A 3-D finite element model of the steel containment vessel with polar crane is used. For the regions surrounding the large penetrations, the mesh size is very small, for regions away from the major penetrations, the mesh is much coarser.

Seismic analysis is performed in two parts, global equivalent static accelerations, and local equivalent static accelerations. For the global loading, elevation dependent static accelerations are applied to the model. For the local loading, rotational and radial accelerations caused by

the eccentricity of the weight and c.g. locations of the equipment hatches and personnel locks are applied. The combination of the two seismic input loads are algebraically summed (local vertical bending plus North-South direction earthquake, radial acceleration plus East-West direction earthquake, local horizontal bending plus Vertical direction earthquake) for each of the component directions. The resulting stresses from the three directions of seismic loading are combined by the 100/40/40 method and are then combined with dead, pressure, and thermal loading. The critical case is considered for the local buckling evaluation.

The appropriateness of the 100/40/40 combination method is evaluated for the SSE loading condition by a direct comparison of the combined seismic stress results against those from a time history analysis for the regions surrounding the major penetrations. Confirmatory time history and equivalent static analyses were performed on a coarse mesh version of the same finite element model. The time history analysis used 3 orthogonal time history inputs with mode superposition. The resulting stress intensities for both time history and equivalent static with local penetration accelerations were compared from the same model. The results for the 4 major areas are shown in Figures RAI-TR85-SEB1-27-4 through RAI-TR85-SEB1-27-7. The results confirm that the equivalent static accelerations, when applied separately and then combined by 100/40/40 produce conservative results to the more generally accepted time history analysis results.

Shield Building Roof (SBR)

The design of the shield building roof uses results from equivalent static analyses of a detailed finite element (FE) model. The equivalent static accelerations applied to this model were developed from analyses of the NI20 nuclear island model such that the member forces from the equivalent static analysis would envelope those from the response spectrum analysis performed for input motion applied at the foundation level enveloping all the soil cases. The maximum member forces from these analyses in the elements representing the air inlet region (see Figure RAI-TR85-SEB1-27-8) are summarized in Table RAI-TR85-SEB1-27-1. In this table the results of the three direction of input are combined by the SRSS method.

The member forces in a single element on the north azimuth (element 2911) are summarized in Table RAI-TR85-SEB1-27-2. The upper portion of the table shows the results for each direction of input from the response spectrum analyses as well as the SRSS combination thereof. The lower portion of the table shows the results for each direction of input from the equivalent static analyses as well as the SRSS combination thereof and the maximum of the 100/40/40 combination thereof. The detailed 100/40/40 combination of 24 cases is shown in Table RAI-TR85-SEB1-27-3. Both the response spectrum and equivalent static analyses show that at this location the north-south input causes hoop forces (TX) and overturning of the roof (TY, MY) while the east-west input causes in-plane shear (TXY).

The ACI design evaluation of the shield building roof determines the reinforcement required at each section to resist the combination of member forces. This ensures that the stress in the reinforcement is within the code design strength. The maximum demand is determined from the

24 100/40/40 cases. The reinforcement in the X direction includes demand for the TX, TXY, and MX member forces; the reinforcement in the Y direction includes demand for the TY, TXY, and MY member forces; the through wall reinforcement includes demand for the NX and NY shear forces where the design strength may be affected by membrane forces TX and TY.

As shown in Table RAI-TR85-SEB1-27-3, one of the individual cases is always close to including the maximum values of the axial force and moment (TX, MX or TY, MY). The controlling case is that where the axial force is in tension. The relative sign of the moment is not significant since the reinforcement is equal on the two faces. The evaluation does not combine the maximum axial tension (TX or TY) with the maximum in-plane shear (TXY). These forces are clearly caused by different directions of horizontal input so the assumptions of the 100/40/40 combination are realistic. In the conservative SRSS combination method for response spectrum analyses described in paragraph 3.2.7.1.3 of ASCE 4-98, the maximum of each of these member forces would be assumed in the detailed evaluation. As described in the ASCE 4-98 commentary, this method is overly conservative.

Design Control Document (DCD) Revision:

Revise third bullet of subsection 3.7.2.6 as follows:

- The peak responses due to the three earthquake components from the equivalent static analyses 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.

PRA Revision:
None

Technical Report (TR) Revision:

Revise section 2.6.1.4 as follows: The underlined portion is added by Revision 1 of this response.

2.6.1.4 Normal plus seismic reactions

Liftoff analyses were performed for 16 load cases of dead, live and seismic loads for the soil site with subgrade modulus of 520 kcf. Seismic loads are applied with unit factor in one direction and with 0.4 factor in the other two directions. The 16 cases were those having the unit factor applied in the horizontal direction in order to maximize the overturning. Cases with unit factor in the vertical direction <u>were also analyzed in linear analyses and</u> do not control. Maximum bearing reactions at the corners of the auxiliary building and at the west side of the shield building are shown in Table 2.6-3. Bearing pressure contours are shown in Figures 2.6-4 to 2.6-8 for the five load cases resulting in these maximum bearing reactions. The seismic load combination is shown for each figure. Note that the bearing pressures reduce rapidly away from the corners. These figures show lift off for equivalent static loads which are higher than the maximum time history loads as discussed in section 2.4.2. This is particularly the case for load combinations with unit seismic load in the Y direction (East-West) where the footprint dimension is smaller. The results of the equivalent static analyses are used for basemat design. The maximum bearing capacity reactions for defining minimum dynamic soil bearing capacity are based on time history analyses as discussed in Section 2.4.2.

Table RAI-TR85-SEB1-27-1

Maximum Member Forces in Air Inlet Region

Comparison of Equivalent Static Acceleration Results to Response Spectrum Results

	TX (kip/ft)	TY (kip/ft)	TXY (kip/ft)	MX (kip-ft/ft)	MY (kip-ft/ft)	MXY (kip-ft/ft)
Acceleration	247.03	137.09	168.73	36.48	161.87	34.50
Response Spectrum	230.71	115.02	117.24	36.25	149.29	40.09
Percent Difference	7.07%	19.18%	43.92%	0.64%	8.42%	-13.93%

Table RAI-TR85-SEB1-27-2

Member Forces in Element 2911 in Air Inlet Region

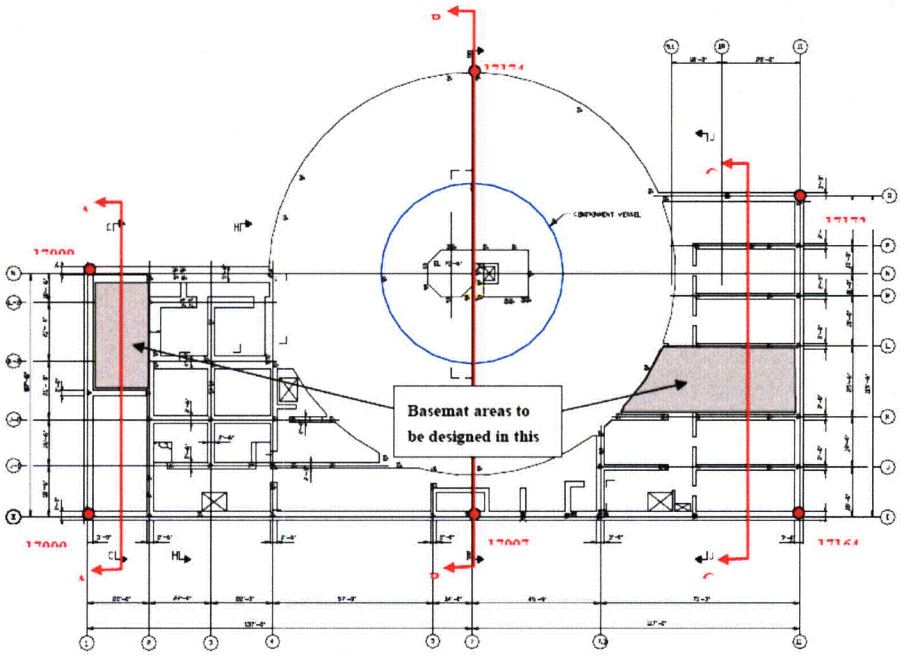
Comparison of Equivalent Static Acceleration Results to Response Spectrum Results

Input	TX (kip/ft)	TY (kip/ft)	TXY (kip/ft)	MX (kip-ft/ft)	MY (kip-ft/ft)	MXY (kip-ft/ft)
Response Spectrum						
North-south	155.29	73.96	14.58	11.09	85.16	8.84
East-west	15.56	7.8	105.98	6.16	8.87	15.8
Vertical	158.75	78.30	3.46	18.04	113.84	4.79
SRSS	222.62	107.99	107.03	22.05	142.44	18.73
Equivalent static						
North-south	-194.41	98.11	-6.65	-16.74	-111.71	2.70
East-west	-8.77	2.99	126.49	-1.25	-3.02	-10.61
Vertical	148.10	-90.85	0.80	18.28	112.76	-0.45
SRSS	244.55	133.75	126.67	24.82	158.75	10.96
Max 100/40/40	257.16	135.65	129.47	25.48	158.65	11.87

Table RAI-TR85-SEB1-27-3

Member Forces in Element 2911

Ens	Eew	Ev	TX	TY	TXY	MX	MY	MXY
1			-194.41	98.11	-6.65	-16.74	-111.71	2.70
	1		-8.77	2.99	126.49	-1.25	-3.02	-10.61
		1	148.10	-90.85	0.80	18.28	112.76	-0.45
1	0.4	0.4	-138.68	62.97	44.27	-9.93	-67.81	-1.72
1	0.4	-0.4	-257.16	135.65	43.63	-24.55	-158.02	-1.36
1	-0.4	0.4	-131.66	60.57	-56.93	-8.93	-65.40	6.76
1	-0.4	-0.4	-250.14	133.25	-57.57	-23.55	-155.61	7.12
-1	0.4	0.4	250.14	-133.25	57.57	23.55	155.61	-7.12
-1	0.4	-0.4	131.66	-60.57	56.93	8.93	65.40	-6.76
-1	-0.4	0.4	257.16	-135.65	-43.63	24.55	158.02	1.36
-1	-0.4	-0.4	138.68	-62.97	-44.27	9.93	67.81	1.72
0.4	1	0.4	-27.29	5.89	124.15	-0.63	-2.60	-9.71
0.4	1	-0.4	-145.77	78.57	123.51	-15.26	-92.81	-9.35
0.4	-1	0.4	-9.75	-0.09	-128.83	1.87	3.44	11.51
0.4	-1	-0.4	-128.23	72.59	-129.47	-12.76	-86.77	11.87
-0.4	1	0.4	128.23	-72.59	129.47	12.76	86.77	-11.87
-0.4	1	-0.4	9.75	0.09	128.83	-1.87	-3.44	-11.51
-0.4	-1	0.4	145.77	-78.57	-123.51	15.26	92.81	9.35
-0.4	-1	-0.4	27.29	-5.89	-124.15	0.63	2.60	9.71
0.4	0.4	1	66.83	-50.41	48.74	11.08	66.87	-3.61
0.4	0.4	-1	-229.37	131.29	47.14	-25.48	-158.65	-2.71
0.4	-0.4	1	73.84	-52.80	-52.46	12.08	69.28	4.87
0.4	-0.4	-1	-222.36	128.90	-54.06	-24.48	-156.24	5.77
-0.4	0.4	1	222.36	-128.90	54.06	24.48	156.24	-5.77
-0.4	0.4	-1	-73.84	52.80	52.46	-12.08	-69.28	-4.87
-0.4	-0.4	1	229.37	-131.29	-47.14	25.48	158.65	2.71
-0.4	-0.4	-1	-66.83	50.41	-48.74	-11.08	-66.87	3.61
Max (1,0 0.4, 0.4)			257.16	135.65	129.47	25.48	158.65	11.87
Min (1,0 0.4, 0.4)			-257.16	-135.65	-129.47	-25.48	-158.65	-11.87
SRSS			244.55	133.75	126.67	24.82	158.75	10.96



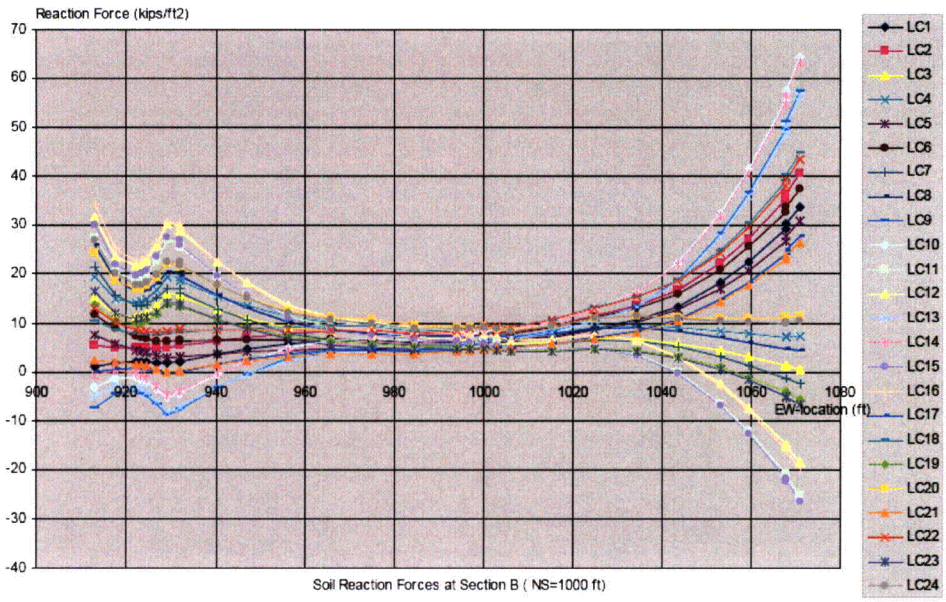


Figure RAI-TR85-SEB1-027-01 Soil Reaction Forces on Section B-B

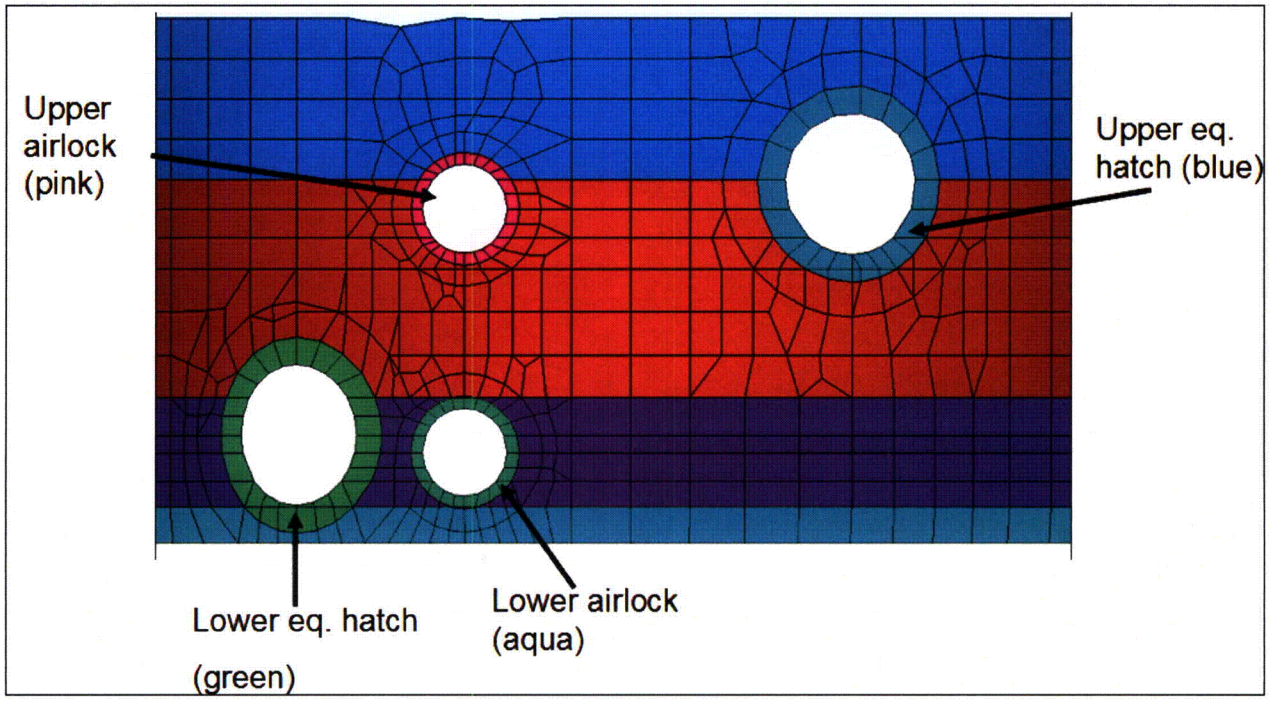


Figure RAI-TR85-SEB1-27-3 SCV 3-D Finite Element Model Large Penetrations

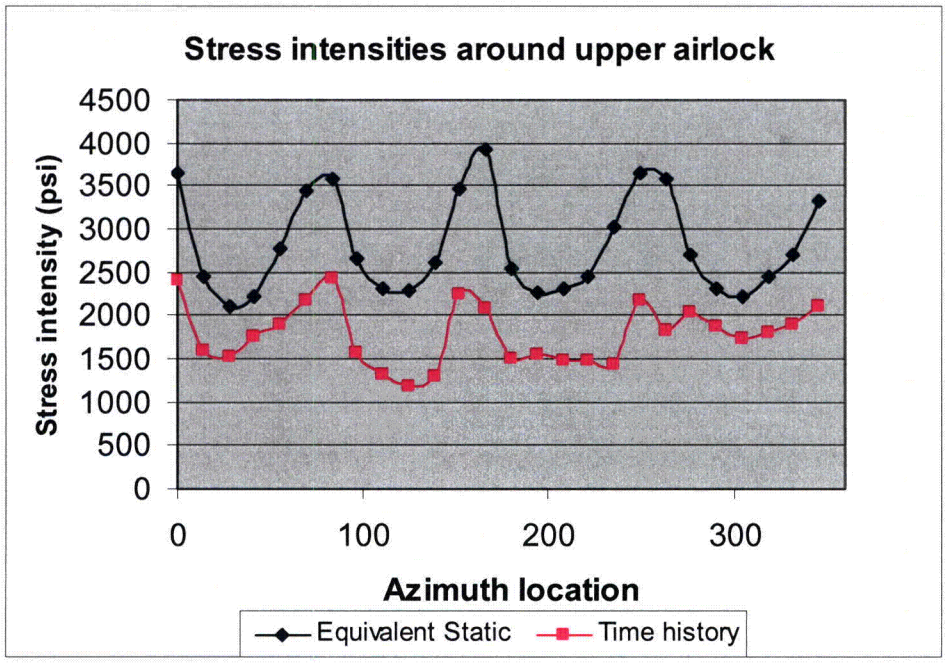


Figure RAI-TR85-SEB1-27-4 SCV 3-D Stress Intensity Comparison for Upper Airlock

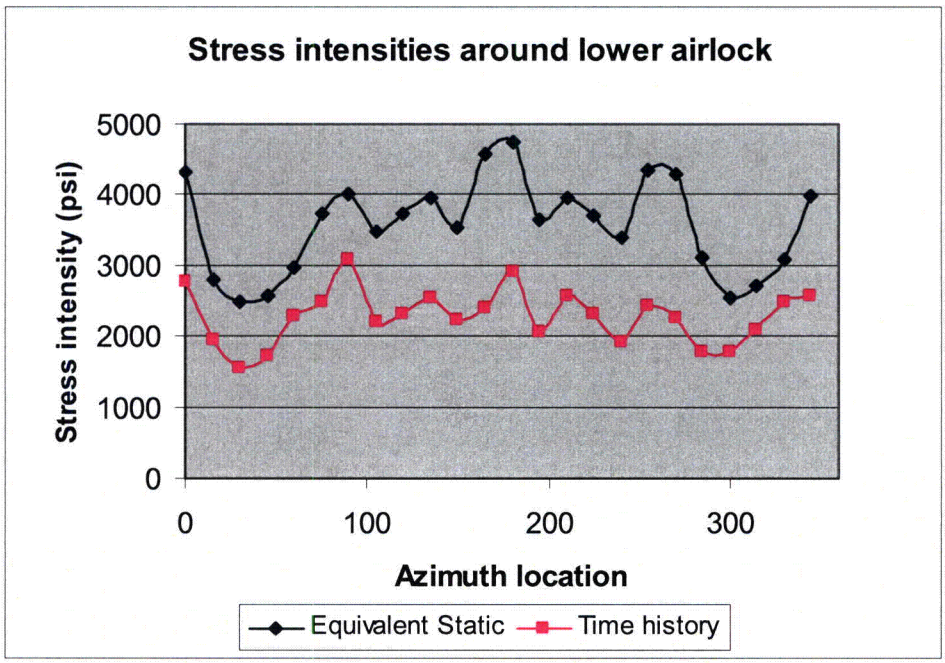


Figure RAI-TR85-SEB1-27-5 SCV 3-D Stress Intensity Comparison for Lower Airlock

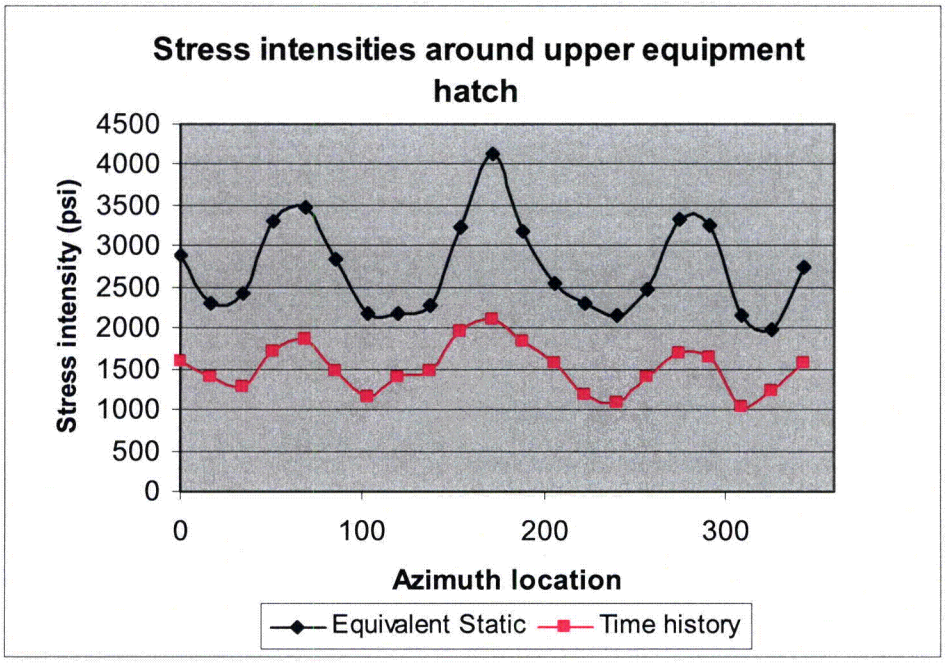


Figure RAI-TR85-SEB1-27-6 SCV 3-D Stress Intensity Comparison for Upper Equipment Hatch

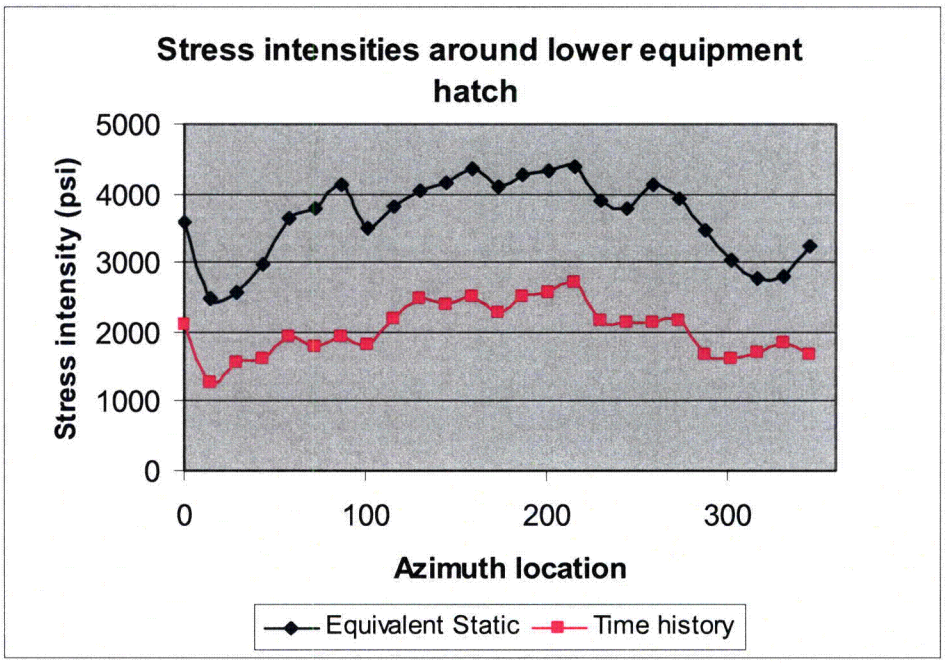


Figure RAI-TR85-SEB1-27-7 SCV 3-D Stress Intensity Comparison for Lower Equipment Hatch

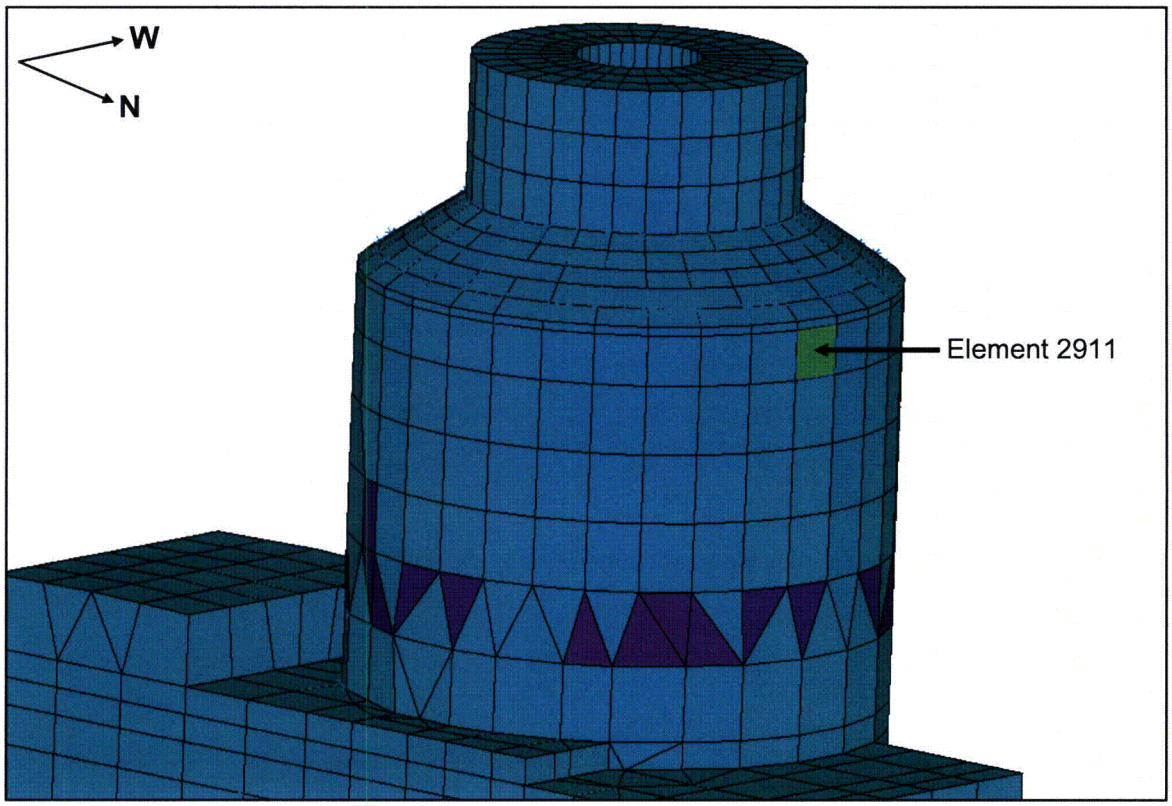


Figure RAI-TR85-SEB1-27-8 NI20 Finite Element Model

AP1000 TECHNICAL REPORT REVIEW

Response to Request For Additional Information (RAI)

RAI Response Number: RAI-TR85-SEB1-32
Revision: 4

Question:

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

Additional Request (Revision 1):

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

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

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

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

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

Additional Request (Revision 2):
(Follow-up RAIs dated 4/27/09)

The RAI response indicates that the seismic analysis for determining the soil bearing pressure demand has been revised to utilize the SASSI 3D finite element NI20 model using a seismic time history soil-structure interaction analysis. This analysis was performed for the hard rock case and five soil conditions. The model includes a surrounding layer of excavated soil and the existing soil media. The soil media in SASSI is an idealization of the various horizontal soil layers. This representation of the soil in SASSI is considered to be more realistic and accurate than the uniform Winkler type soil springs used in the 2D ANSYS analyses, and thus addresses the concern regarding the calculation of soil bearing pressure demand. However, for the design of the basemat, Westinghouse has not demonstrated that its use of uniform Winkler type soil springs is adequate. As noted in the prior RAI, due to the Boussinesq effect in soil, the distribution of stiffness would be higher at the edges and lower away from the edges. Therefore, Westinghouse is requested to demonstrate that the use of the uniform soil springs for the design of the foundation is acceptable, when it is known that the actual distribution of the soil stiffness would not be uniform.

Additional Request (Revision 3):
NRC Meeting August 10-14

1. Clarify the Model identification in Table RAI-TR85-SEB1-32-1, Line 2 and Line 3. Consider use of description in Appendix 3G.
2. In the second paragraph on page 4 of 13 of the RAI response how does the referenced report from section 2.7.2 of TR85 demonstrate the acceptability of using Winkler springs instead of the method using Boussinesq effect.

Additional Request (Revision 4):

The staff reviewed the Westinghouse response to RAI-TR85-SEB1-32, Rev. 3 and determined that Item 2 of the response did not adequately address staff's concerns. The information provided does not clearly demonstrate that the bending moments and shear forces in the basemat using the Winkler soil springs are acceptable. For example, as shown in Figures RAI-TR85-SEB1-32-2 and 3 provided in Revision 2 to this RAI, there are locations where the bending moments using the finite element model are larger than the results for the Winkler soil spring model. Therefore, provide the technical basis for using the Winkler soil spring model. The technical basis should include tabulated or plotted curves (not contour plots) showing moments (positive and negative) and shear forces at locations in the basemat that govern the design for the rest of basemat.

Westinghouse Response:

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

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

Westinghouse Response (Revision 1):

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

In response to the many questions in this and other RAIs, Westinghouse has revised the basis for the bearing demand. The demand is now based on 3D SASSI analyses using the 3D NI20

finite element model as described in the response to RAI-TR85-SEB1-03, Rev 1. This change to use of the 3D SASSI results addresses the original question in this RAI. The additional questions in Rev 1 of this RAI apply to the Rev 0 response which has now been superseded.

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

Westinghouse Response (Revision 2):

Table RAI-TR85-SEB1-32-1 shows the summary of the maximum reactions of the nuclear island for various soil and analysis methods. The results from Table 2.4-5 of the technical report, APP-GW-GLR-044, R1 (TR85), are shown for Item 1. Two other sources are contained in the table as comparison. The results of the linear analyses show consistent results demonstrating that the equivalent static analyses of the basemat result in bearing pressures similar to a more realistic model represented in the 3D SASSI analyses presented in Table 2.4-5.

Section 2.7.1 of the technical report describes studies performed to evaluate the effect of different soil modeling. These studies analyzed a 3D finite element model of the complete nuclear island on soil finite elements or soil springs. Additional comparisons are provided in this response for the following soil models:

- Winkler soil springs of 520 kcf similar to the design analyses of the basemat (Model W)
- Finite element model of 80-foot deep soil layer below the nuclear island. The properties for this soil were selected to match the 520 kcf soil (Model L080).

Figure RAI-TR85-SEB1-32-1 shows bearing pressures due to dead load. The sections are located as shown in Figure 2.7-2 of the technical report. The RAI figure only shows the two cases identified above. The finite element soil model shows high local bearing pressures close to the edge. This is due to the Boussinesq effect. The higher pressures at the edge result in lower pressures away from the edge. Figures RAI-TR85-SEB1-32-2 and 3 show bending moment contours MX and MY for these two cases. It is seen that the bending moments for the soil spring case are higher than those for the soil finite element model even in the bays immediately adjacent to the edge.

Section 2.7.2 of the technical report describes an additional study performed to evaluate the basemat soil interaction. The north-west corner of the AP1000 shown in Figure RAI-TR85-SEB1-32-4 was modeled and analyzed in two dimensions using the non-linear VECTOR2 structural analysis program. The model of the basemat and soil is shown in Figure RAI-TR85-SEB1-32-5. The model of the basemat is 20269.2 mm (66.5') long, simulating 3 bays of 18' and ½ bay of 25', and 1828.8 mm (6') high. The element size is 152.4 mm (6") x 304.8 mm (12") for the first 55' and 152.4 mm (6") x 292.1 mm (11.5") for the last 11.5'. The total number of nodes

and elements for the basemat is 884 and 804, respectively. The model of the soil is 23104.8 mm (75.8' depth) and 61069.2 mm (200.3'). The soil model extends almost twice the soil depth beyond the end of the basemat. The total number of nodes and elements for the soil is 4794 and 4642, respectively. The soil properties are chosen to give the same vertical displacement of the nuclear island under dead load as the 520 kcf soil springs. X-direction is horizontal and Y is vertical. The nodes on the last column are restrained in the X-direction to simulate symmetry. The VECTOR2 program considers cracking of the concrete and non-linear behavior of the reinforcement. Structural response is calculated up to failure for monotonically applied vertical displacement of the shear walls. Contact pressures on the soil are shown in Figure RAI-TR85-SEB1-32-6 (Figure 2.7-10, TR85) as a function of the applied displacement. They indicate substantial Boussinesq effect with high bearing pressures below the edge of the basemat. The analyses showed significant redistribution of soil bearing pressures as the load increased. The basemat withstood loading about three and a half times the design loads with final failure occurring in shear close to the exterior wall.

The studies documented in section 2.7 of the technical report and summarized above show the Boussinesq effect in rock and soil with an effective stiffness that is higher at the edges and lower away from the edges. This distribution is presented on Figure RAI-TR85-SEB1-32-7. The influence values represent the effect of the contact on a half space beneath a rigid circular footing. The Boussinesq equation does not account for the influence of the basemat's embedment depth of 39.5 feet which would increase the bearing capacity of the foundation. Because of the basemat dimensions (plan and thickness), the foundation would be considered flexible. The studies demonstrate that the use of uniform Winkler type soil springs is adequate for a flexible mat foundation.

Westinghouse Response (Revision 3):

1. The 2D SASSI model listed at Line 2 of Table RAI-TR85-SEB1-32-1 refers to the East-West 2D stick model described in section 2.4.1 of APP-GW-GLR-044, and also shown in Figure 2.4-2 of that report. The 2D Time History model listed at Line 3 of the table refers to the lift-off analysis performed in ANSYS described in section 2.4.2 of APP-GW-GLR-044, and shown in Figure 2.4-4 of that report.

2. The referenced report, from section 2.7.1 of TR85, demonstrates the acceptability of using Winkler springs instead of the method using Boussinesq effect. As concluded in section 2.7.1.2.4, the analyses with finite element models of the soil were performed as linear elastic analyses. They require much greater computer running time and do not lead to significantly better results. The design analyses are non-linear to consider lift off. They require a more detailed model of the nuclear island than that used in the studies. They must address more combinations of seismic input than used in the studies. Hence Winkler springs were selected for use in the design analyses similar to those used in the AP600 analyses.

As discussed in section 2.7.1.2.1, the models with finite element representation of the soils show larger bearing reactions at the edges than the Winkler spring model. These higher reactions at the edges give a corresponding reduction of bearing reactions and member forces away from the edges. Hence the uniform Winkler springs are conservative for design of the basemat since reinforcement in the basemat is controlled by member forces below the center of each panel and the interior walls (the exterior wall acts more like a simple support than a fixed support).

The referenced report from section 2.7.2 of TR85 does not directly demonstrate the acceptability of using Winkler springs instead of the method using Boussinesq effect. However, it shows substantial margin (about three and a half times the design loads) in the design with a more detailed soil and structure model that also shows bearing pressures consistent with the Boussinesq distribution.

Westinghouse Response (Revision 4):

Moments (positive and negative) and shear forces are shown in Table RAI-TR85-SEB1-32-2 in elements in the middle of the two critical sections identified in the DCD. The element locations are shown in Figure RAI-TR85-SEB1-32-8. Positive moments put the top surface of the mat in tension. The critical sections were selected as representative of the most limiting portions of the basemat and are described in DCD subsection 3.8.5.4.4 as follows:

Basemat between column lines 9.1 and 11 and column lines K and L (Bay 2 in Table)

This portion of the basemat is designed as a two way slab with the shorter directions spanning a distance of 23′ 6″ between the walls on column lines K and L. The slab is continuous with the adjacent slabs to the east and west. The critical loading is the bearing pressure on the underside of the slab due to dead and seismic loads. This establishes the demand for the top flexural reinforcement at mid span and for the bottom flexural and shear reinforcement at the walls. The basemat is designed for the member forces from the analyses described in subsection 3.8.5.4.1. The top and bottom reinforcement in the east west direction of span are equal. The reinforcement provided is shown in sheets 1, 2 and 5 of Figure 3.8.5-3. Typical reinforcement details showing use of headed reinforcement for shear reinforcement are shown in Figure 3H.5-3.

Basemat between column lines 1 and 2 and column lines K-2 and N (Bay 1 in Table)

This portion of the basemat is designed as a two way slab with the shorter direction spanning a distance of 22′ 0″ between the walls on column lines 1 and 2. The slab is continuous with the adjacent slabs to the north and with the exterior wall to the south. The critical loading is the bearing pressure on the underside of the slab due to dead and seismic loads. This establishes the demand for the top flexural reinforcement at mid span and for the bottom flexural and shear reinforcement at wall 2. The basemat is designed for the member forces from the analyses on uniform soil springs described in subsection 3.8.5.4.1. The reinforcement provided is shown in sheets 1, 2 and 5 of Figure 3.8.5-3. Typical reinforcement details showing use of headed reinforcement for shear reinforcement are shown in Figure 3H.5-3.

Table RAI-TR85-SEB1-32-2 shows the bending moments and shear forces in the principal span direction (X direction for Bay 1 and Y direction for Bay 2). Bay 2 is away from the edges of the basemat. The maximum positive moment and shear forces in Bay 2 are lower with the soil model (L-080) than the Winkler spring model. The minimum (maximum negative) moment is about equal with the soil model (L-080) and the Winkler spring model.

Bay 1 is at the south west corner of the basemat. In this bay the maximum positive moment and shear forces are also lower with the soil model (L-080) than the Winkler spring model. The mid span positive moment reduces by 45%. However, the minimum (maximum negative) moment with the soil model (L-080) is 4861% greater than the Winkler spring model. This increase was evaluated as follows:

> The bottom reinforcement provided in Bay 1 in the north south direction is #14 bars at 12" centers as shown in Table 2.6.9 of the TR85 report. This provides an area of 2.25 in^2/ft. The design calculation shows that the reinforcement required in the highest loaded element at this location is 2.00 in^2/ft. This demand is calculated for the design loads and includes the additional 20% margin described in section 2.6.2 of the report. The governing load case is Load Case 3-6 (Es= -1.0xSns+0.4xSew-0.4xSvt) which maximizes the bearing pressure in the south-west corner. An increase in the demand by 48% would increase the reinforcement required to 2.96 in^2/ft which is 32% greater than that provided. The ACI Code permits redistribution of negative and positive moments by up to 20% for a flexural member with equal top and bottom reinforcement. The slab has a clear span of 19' in the north-south direction and 40' in the east-west direction so the moment at the section shown in Table RAI-TR85-SEB1-32-2 is behaving similar to a one way flexural member. Since the mid span moment reduces by 45% in the L-080 model, and the required positive reinforcement based on the design analyses is also less than the reinforcement provided, 20% of the negative moment can be redistributed. Thus, the reinforcement required for the negative moment at the end span is 0.8 x 2.96 = 2.37 in^2/ft, which is 5% greater than that provided.
>
> The bearing reactions from the equivalent static non-linear lift off analyses of the basemat are conservative relative to the linear elastic SASSI analyses which show that the lift off is negligible. This conservatism is shown in Tables 2.6.2 (c) and (d) of TR 85 showing that the non-linear lift off analyses are 20% higher at the south west corner than the results of analyses with input enveloping all the soil cases (33.1 ksf versus 27.1 ksf).
>
> The L080 soil model has relatively large element sizes for the soil below the edge of the nuclear island. This would be expected to overestimate the soil stiffness below the edge of the nuclear island thus overestimating the Boussinesq effect. Even if the member forces were to be amplified by 48% shown in the comparison study of the two models, the span has sufficient margin to accommodate the greater negative moment demand shown by the L080 soil model.

A detailed study of the south-west bay on soil was also performed using the non-linear Vector analysis methodology similar to that described for the north end of the nuclear island in section 2.7.2 of the report. In these 2D analyses the soil was represented by a much more detailed model than was possible in the 3D L-080 model. The results of this analysis are shown below in the same format as that in Table 2.7-2 of the report for the north portion. This shows a significantly increased capacity for the south west bay when analyzed in the detailed Vector 2 analyses relative to the Winkler soil spring case (100 ksf versus 41 ksf).

Table 6.2: Summary of results for VECTOR2 SW runs

Run ID	Elastic limit	Initial strain hardening[1]							90 % of ultimate[1]						
	Average Contact pressure	Average Contact Pressure	Max. Contact Pressure	Min. Contact Pressure	Support Displ.	Max. Vert. Def.	Max. Horiz. Def.	Max. Crack size	Average Contact Pressure	Max. Contact Pressure	Min. Contact Pressure	Support Displ.	Max. Vertical Def.	Max. Horiz. Def.	Max. Crack size
SWUDL	0.53 Mpa (11 Ksf)	1.30Mpa (27 Ksf)	N/A	N/A	N/A	5.3 mm (0.21in)	2.6 mm (0.10in)	2.2 mm (0.09in)	1.72Mpa (36 Ksf)	N/A	N/A	N/A	12.2mm (0.48in)	6.6 mm (0.26in)	6.1 mm (0.24in)
SWSPR	0.53 Mpa (11 Ksf)	1.44Mpa (30 Ksf)	1.91Mpa (40 Ksf)	1.34Mpa (28 Ksf)	20.3mm (0.8 in)	5.4 mm (0.22in)	2.7 mm (0.11in)	2.4 mm (0.09in)	2.0 Mpa (41 Ksf)	2.96Mpa (62 Ksf)	1.72Mpa (36 Ksf)	30.6mm (1.2 in)	11.0mm (0.47in)	6.5 mm (0.26in)	5.8 mm (0.23in)
SWHALFSP	0.72 Mpa (15 Ksf)	Same as 90 % of ultimate							4.8 Mpa (100Ksf)	11.0Mpa (230Ksf)	2.0 Mpa (42 Ksf)	61.0mm (2.4 in)	9.4 mm (0.37in)	2.2 mm (0.09in)	2.3 mm (0.09in)

Thus, the increase in negative moments showing up in the L-080 model analyses do not affect the overall strength of the nuclear island basemat. The analyses using the Winkler springs provide a design with substantial margin for the design loading.

The two critical bays were selected in the AP600 design certification to demonstrate design methodology. They include a bay with maximum span between the shear walls and a bay at the edge with maximum span and maximum bearing demand. These bays bound the results for other bays. The bottom reinforcement in both directions is uniform (#14 bars at 12" centers each way). Other bays have the same reinforcement and lower demand so similar amplification of the negative moment demand (tension on the bottom face) would be within the reinforcement provided.

The analyses are non-linear analyses performed using 16 combinations (1.0 on horizontal input) of the 100/40/40 equivalent static seismic directions of input. The other 8 combinations (1.0 on vertical input) have been shown not to govern (see response to RAI-TR85-SEB1-27). Each of the 16 combinations results in maximum bearing reactions below a different part of the basemat. Bearing reactions for representative cases are shown in Figures 2.6.4 to 2.6-8 in the TR85 report. For each analysis member forces are combined following the ACI Code equations to develop the required reinforcement in each direction. An example of the member forces in a critical element, TX, TXY, and MX and the required reinforcement, AXTOP and AXBOT is shown in Table RAI-TR85-SEB1-32-3. These are the member forces that affect the reinforcement demand in the north south direction. The reinforcement demand is primarily associated with the bending moment MX which is largest in cases LC06 and LC07 with the seismic input towards the south. The reinforcement demand from the 16 cases is enveloped.

Table RAI-TR85-SEB1-32-1

Maximum Basemat Bearing Pressure (Summary)

No.	Method of Analysis	Foundation Conditions					
		HR (ksf)	FR (ksf)	SR (ksf)	UBSM (ksf)	SM (ksf)	SS (ksf)
1	3D SASSI, NI20 Model, TH + vertical earthquake	35.0 *	27.9	24.0	25.7	23.1	21.9
2	2D SASSI, ni2D** model	29.1	24.0	24.5	27.4	30.2	20.2
3	2D ANSYS Time History, ni2D** model: Linear	32.8	N/A	N/A	31.7	30.8	N/A
	:Lift off	34.9	N/A	N/A	33.5	32.2	N/A

Notes:

* 38.3 ksf was the maximum localized peak calculated; a limit of 35 ksf for maximum bearing seismic demand is obtained by averaging the soil pressure about the West edge of the shield building where the maximum stress occurs.

** The ni2D model refers to the East-West 2D stick model of the Nuclear Island used for the SASSI and ANSYS analyses. The SASSI analysis used the model embedded in soil 120' below grade. The ANSYS linear lift off model was a non-linear analysis on a rigid basemat. The SASSI and ANSYS model representations are shown in TR85, Figures 2.4-2 and 2.4-4 respectively.

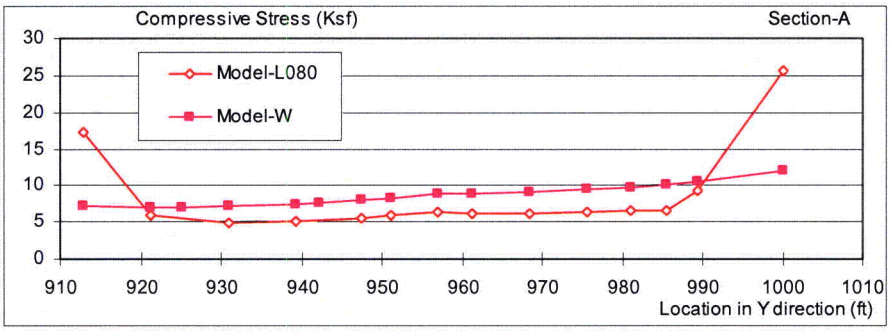
Table RAI-TR85-SEB1-32-2
Bending Moments and Shears in Critical Elements

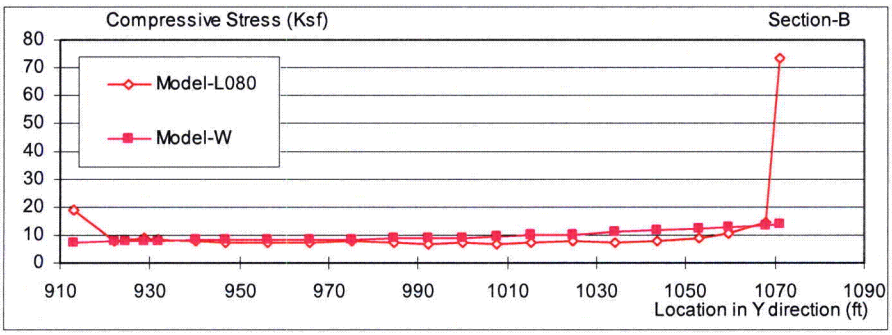
	Model W		Model L-080		Ratio	L-080/W
ELEM	MX	NX	MX	NX	MX	NX
Bay 1	kip.ft/ft	kip/ft	kip.ft/ft	kip/ft		
427	148.73	38.44	104.65	18.58	0.70	0.48
428	146.25	39.52	107.74	21.23	0.74	0.54
442	248.89	-31.84	123.03	-27.65	0.49	0.87
443	251.95	-29.42	137.34	-27.28	0.55	0.93
457	-93.01	-84.24	-137.56	-59.28	1.48	0.70
458	-83.81	-86.78	-134.98	-65.96	1.61	0.76
Max	251.95	39.52	137.34	21.23	0.55	0.54
Min	-93.01	-86.78	-137.56	-65.96	1.48	0.76
	MY	NY	MY	NY	MY	NY
Bay 2	kip.ft/ft	kip/ft	kip.ft/ft	kip/ft		
776	-69.67	45.92	-65.82	20.64	0.94	0.45
777	46.23	29.80	8.29	23.66	0.18	0.79
778	129.31	7.58	70.40	4.09	0.54	0.54
779	128.48	-13.64	68.83	-8.04	0.54	0.59
780	-32.80	-39.68	-35.86	-25.98	1.09	0.65
795	-66.58	43.90	-70.83	23.01	1.06	0.52
796	43.55	29.44	7.87	24.90	0.18	0.85
797	121.48	7.19	73.49	4.58	0.60	0.64
798	120.12	-14.14	72.31	-9.36	0.60	0.66
799	-26.55	-37.40	-33.09	-26.88	1.25	0.72
Max	129.31	45.92	73.49	24.90	0.57	0.54
Min	-69.67	-39.68	-70.83	-26.88	1.02	0.68

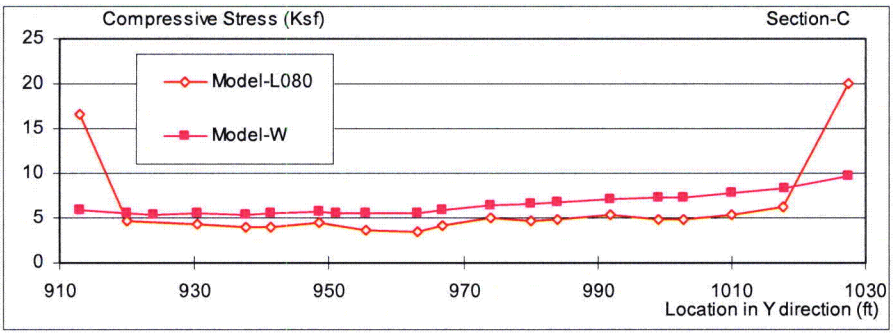
Note: See Figure RAI-TR85-SEB1-32-8 for location of elements

Table RAI-TR85-SEB1-32-3
Design Member Forces and Required Reinforcement in Critical Element

Element	3835										
	D	L	Ens	Eew	Ev	TX	TXY	MX	AXTOP	AXBOT	AXYSH
LC01	1.4	1.7				-17.8	2.5	-328.0	1.307	1.595	0.000
LC02	1.0	1.0	1.0	0.4	0.4	-31.3	-13.8	103.5	1.307	1.595	0.000
LC03	1.0	1.0	1.0	0.4	-0.4	57.0	44.6	58.8	1.307	1.595	0.000
LC04	1.0	1.0	1.0	-0.4	0.4	-8.8	7.2	79.8	1.307	1.595	0.000
LC05	1.0	1.0	1.0	-0.4	-0.4	-2.4	25.2	104.5	1.307	1.595	0.000
LC06	1.0	1.0	-1.0	0.4	0.4	-103.6	-8.2	-752.3	1.307	1.866	0.000
LC07	1.0	1.0	-1.0	0.4	-0.4	-77.8	-8.8	-738.6	1.307	1.996	0.000
LC08	1.0	1.0	-1.0	-0.4	0.4	-83.9	-67.9	-579.8	1.307	1.663	0.000
LC09	1.0	1.0	-1.0	-0.4	-0.4	-72.7	-61.7	-631.7	1.307	1.663	0.000
LC10	1.0	1.0	0.4	1.0	0.4	23.5	64.6	-1.8	1.307	1.595	0.000
LC11	1.0	1.0	0.4	1.0	-0.4	26.8	82.5	-198.2	1.307	1.595	0.000
LC12	1.0	1.0	0.4	-1.0	0.4	4.3	-9.2	52.1	1.307	1.595	0.000
LC13	1.0	1.0	0.4	-1.0	-0.4	14.8	-40.3	-6.5	1.307	1.595	0.000
LC14	1.0	1.0	-0.4	1.0	0.4	-59.2	116.7	-529.7	1.307	1.866	0.102
LC15	1.0	1.0	-0.4	1.0	-0.4	-45.3	70.2	-599.0	1.307	1.866	0.000
LC16	1.0	1.0	-0.4	-1.0	0.4	-47.7	-96.6	-219.8	1.307	1.595	0.000
LC17	1.0	1.0	-0.4	-1.0	-0.4	-36.8	-84.4	-326.9	1.307	1.595	0.000







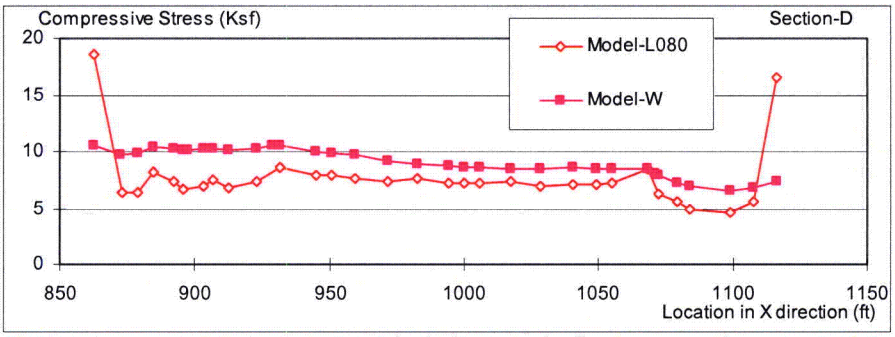
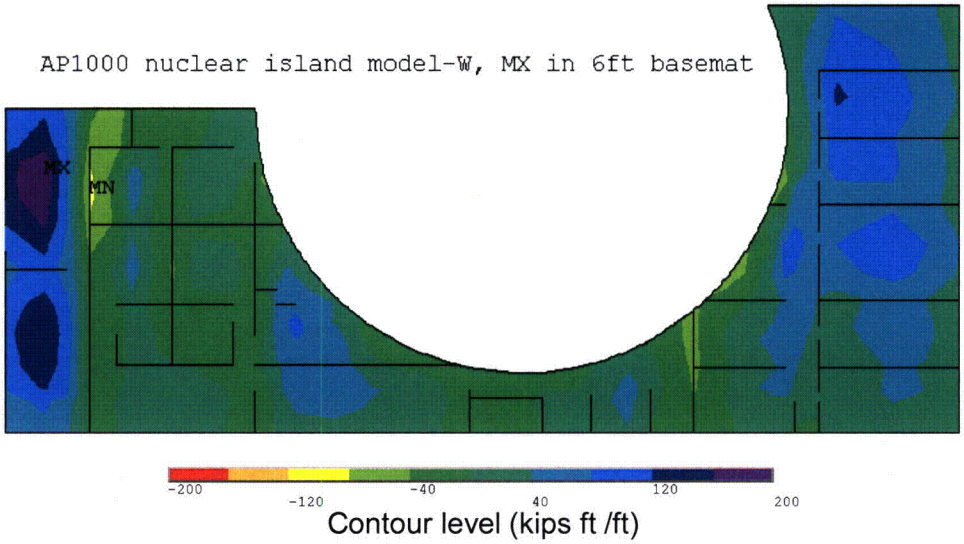


Figure RAI-TR85-SEB1-32-1
Comparison of Vertical Stress at Basemat Bottom Node – No embedment



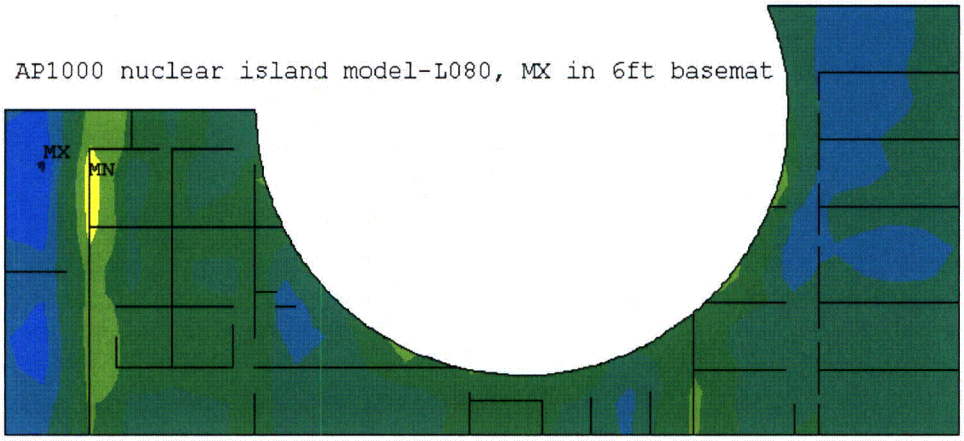
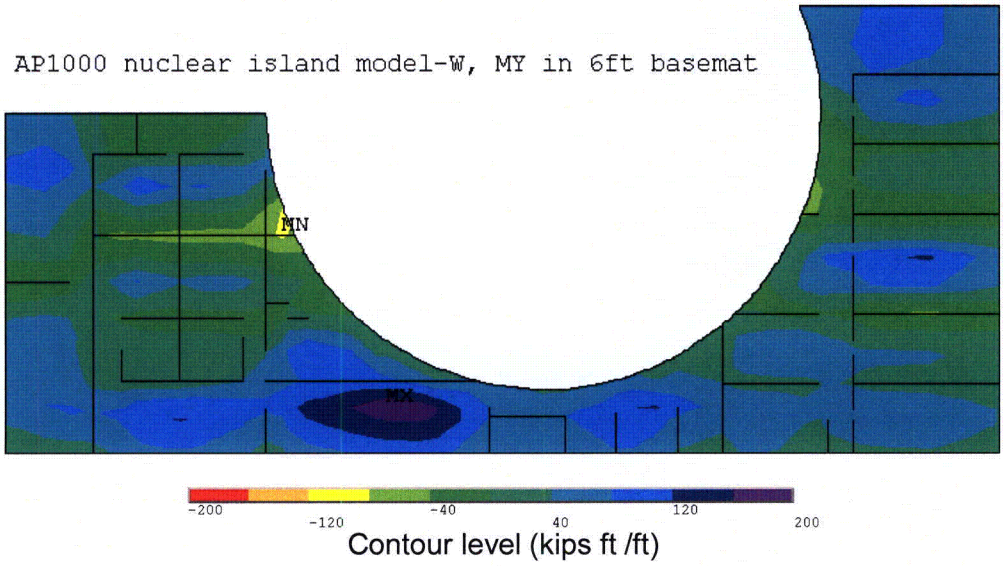


Figure RAI-TR85-SEB1-32-2 Bending Moment MX for Model-W and Model-L080



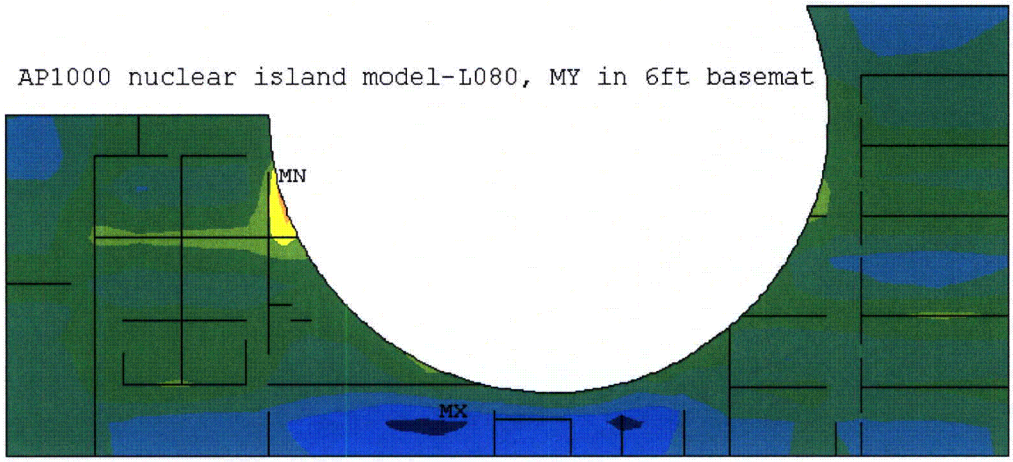


Figure RAI-TR85-SEB1-32-3 Bending Moment MY for Model-W and Model-L080

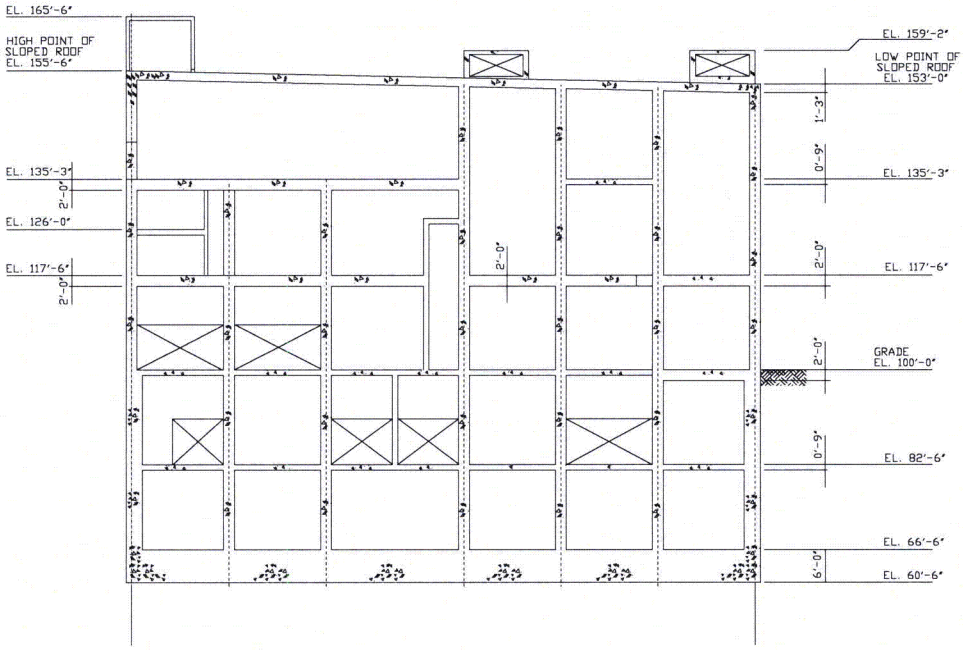


Figure RAI-TR85-SEB1-32-4

Cross section through north end of auxiliary building looking south

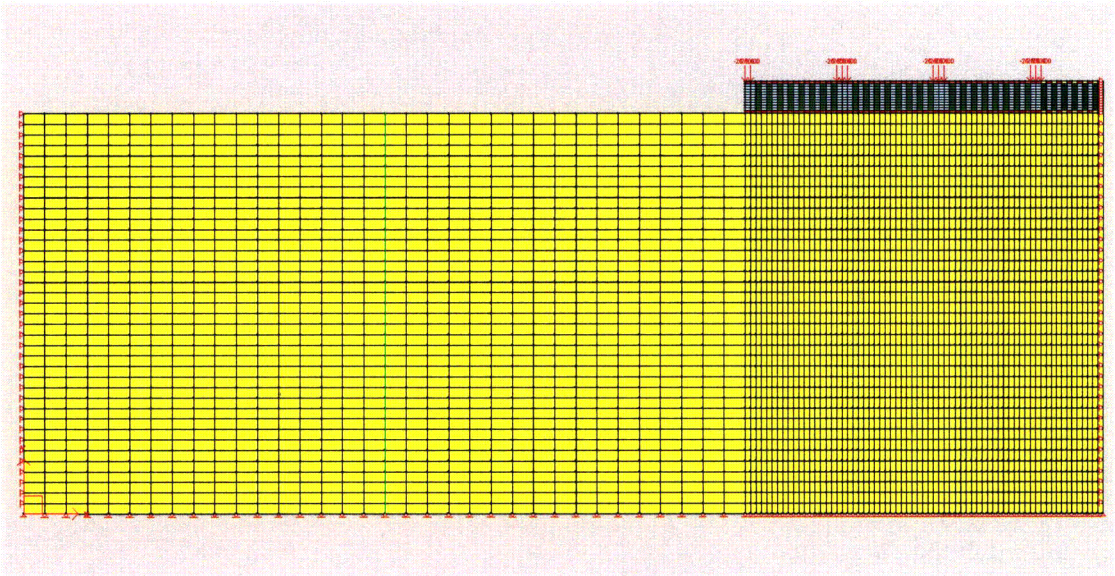


Figure RAI-TR85-SEB1-32-5

Vector2 model looking north with Soil Elements

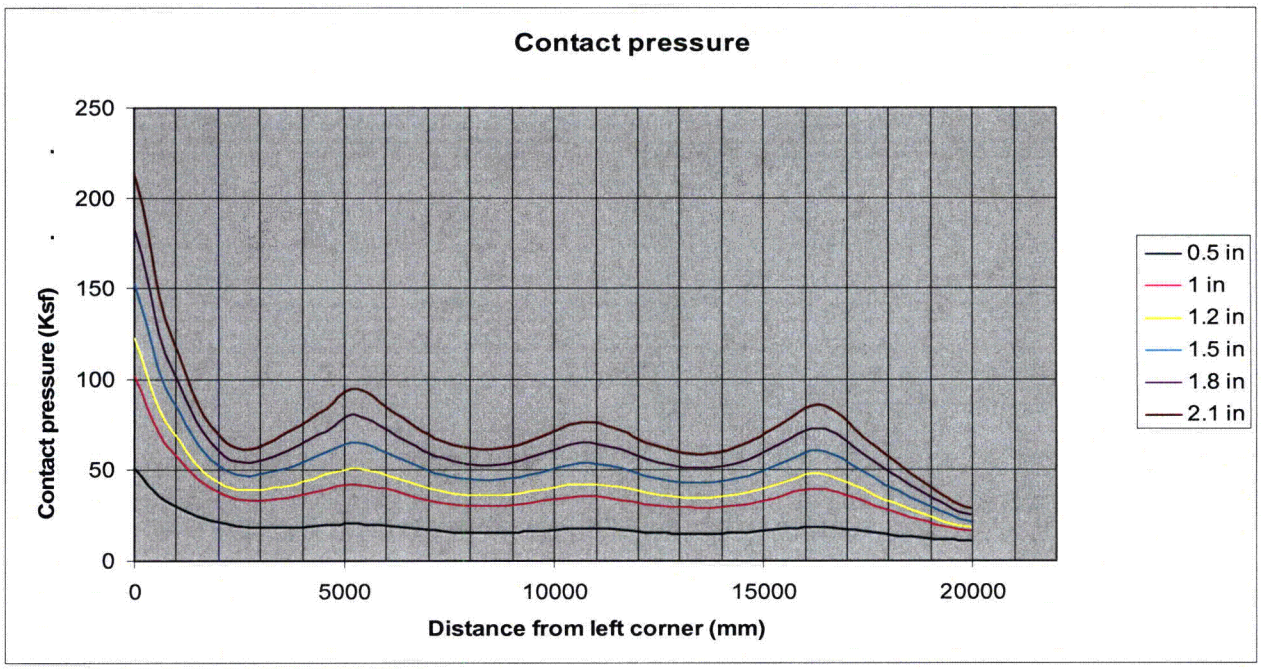
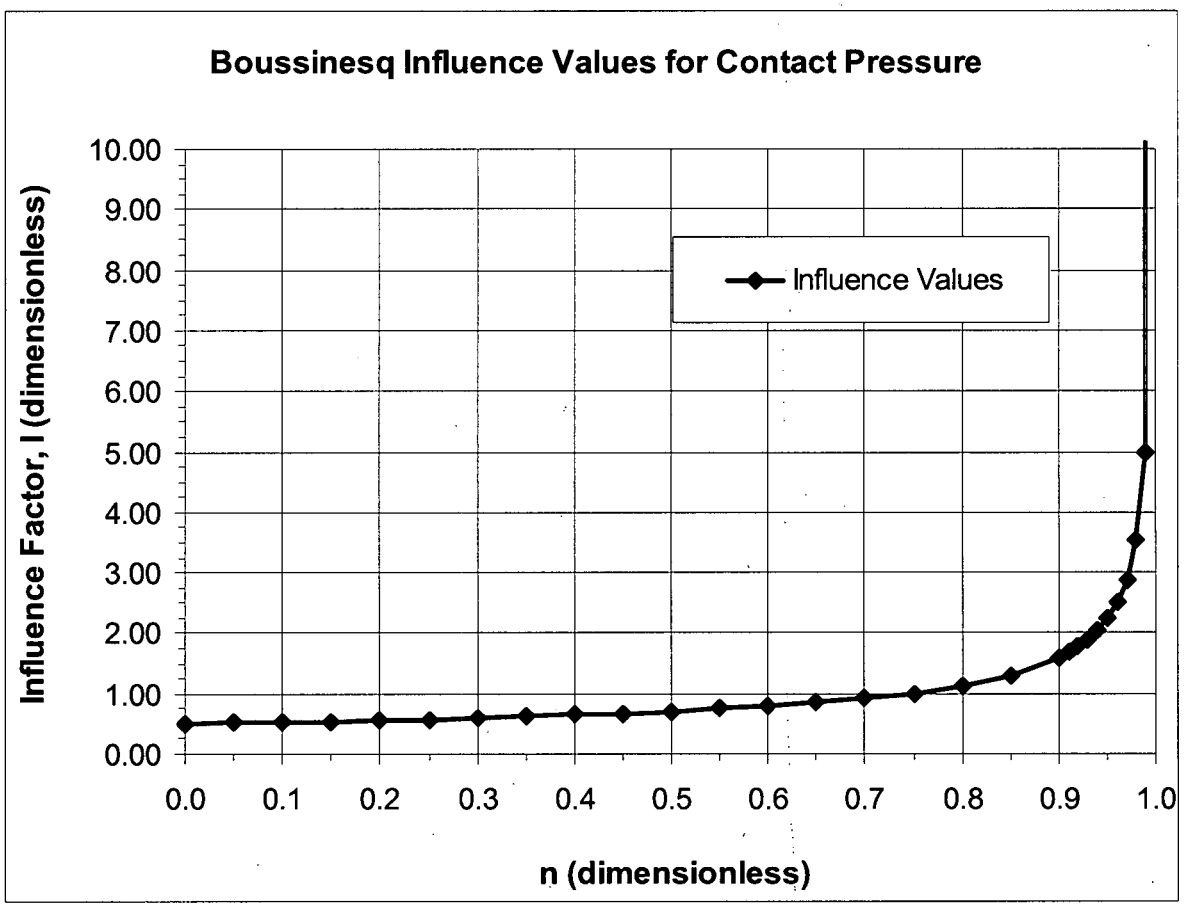


Figure RAI-TR85-SEB1-32-6
Contact Stresses along Mat for Half Space



Notes:
Area (sf) = 32,480 (for NI footprint)
r (ft) = 101.68 (r for equivalent area)
If n = 0.0 is the center of the area (r = 0.0 ft)
then n = 1.0 at the perimeter (r = 101.68 ft),

Boussinesq Method (rigid circular footing)
Qo = Footing Contact Pressure
P = Foundation Load, A = Footing Area
$Qo = (P / A) \times (1 / 2x(1-n)^{0.5})$

with $I = \dfrac{1}{2\,(1-n)^{0.5}}$

Figure RAI-TR85-SEB1-32-7
Boussinesq Influence Values for Footing Contact Pressure
(rigid circular footing at ground surface, for half space)

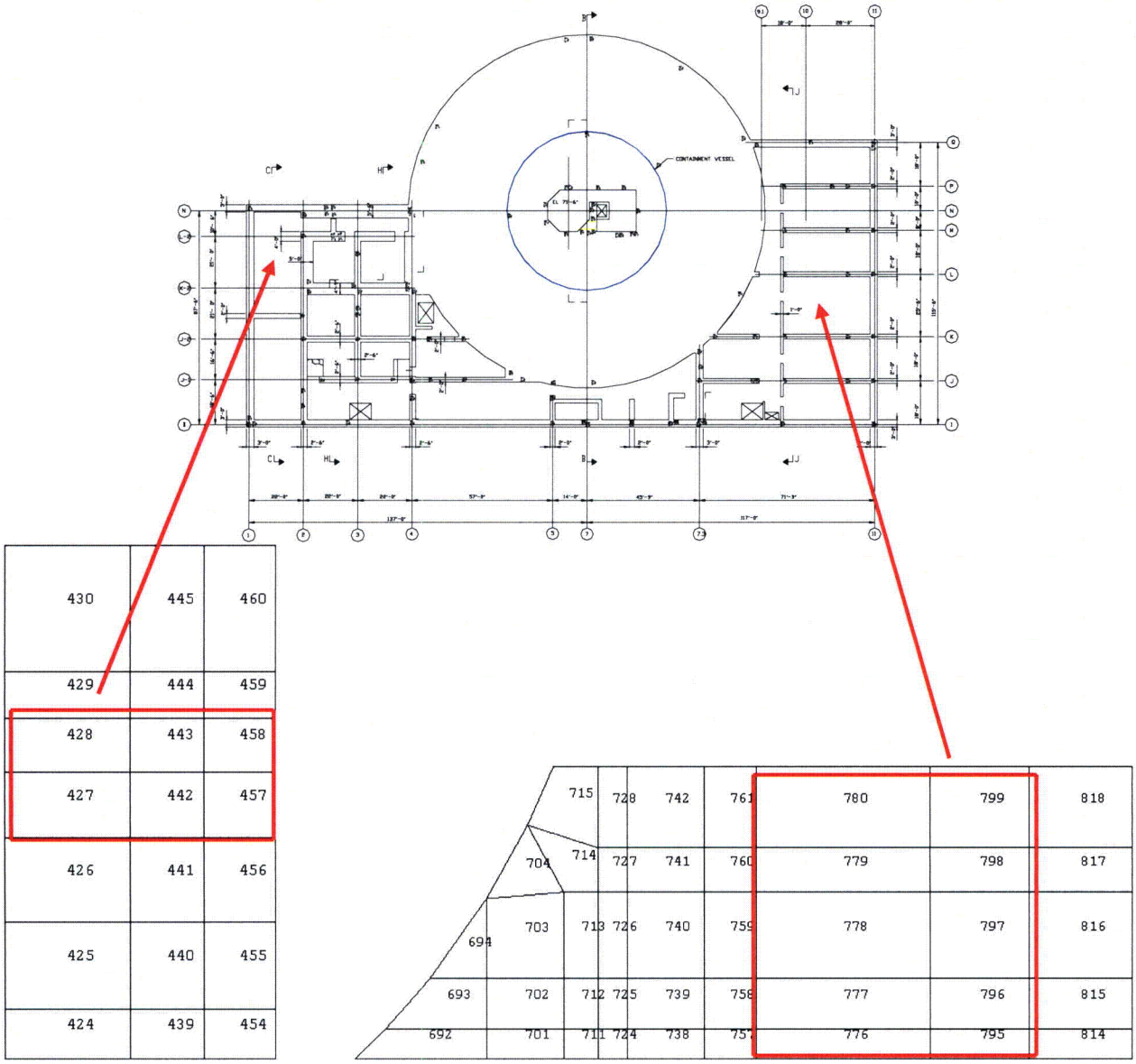


Figure RAI-TR85-SEB1-32-8 Element numbers for Table RAI-TR85-SEB1-32-2

Design Control Document (DCD) Revision:
All changes to the DCD shown in this section are from previous revisions of this RAI. No additional changes have been made to the DCD in Rev 4 of this response.

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

2.5.4.2 Bearing Capacity

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

PRA Revision:

None

Technical Report (TR) Revision:

Revise Tables 2.6-2 (b), 2.6-2 (c) and 2.6-4 as shown below:

Table 2.6-2 (b), revise footnote 2 as follows:

2. Equivalent static results are shown for the response from one direction, (i.e FX and MYY due to X input, FY and MXX due to Y input, and FZ due to Z input. The increase due to combination of three directions is small.

Table 2.6-2(c)

Maximum soil bearing pressures (ksf) at corners from basemat reactions

Location	Equivalent static accelerations Linear analyses	Fixed base time history all soils
West side of shield building	36.8	36.9
NW corner of auxiliary building	27.1	24.8
NE corner of auxiliary building	22.8	25.5
SE corner of auxiliary building	21.1	25.1
SW corner of auxiliary building	29.6	27.1

Table 2.6-4, revise footnotes as follows:

Note 1: See Figures 2.6-9 and 2.6-10 for plan and elevation schematic views of the reinforcement layout.

Note 2: Figures 6-1 and 6-2 in APP-1010-CCC-004, Rev.0 provide graphical presentation of the "Required" (red dash line) and "Provided" (solid black line) areas of radial reinforcement for the top face of the Dish.

RAI Response Number: RAI-SRP3.8.3-SEB1-04
Revision: 2

Question:

Due to design changes, extension of the AP1000 design to soil sites, reanalysis for updated seismic spectra, and updates made to some critical sections, Westinghouse is requested to address a concern with the design details of the structural module connections to the reinforced concrete basemat. Section 3.8.3.5.3 of the DCD indicates that the steel plate modules are anchored to the reinforced concrete basemat by mechanical connections welded to the steel plate or by lap splices where the reinforcement overlays shear studs on the steel plate. Typical details of these two options are shown on DCD Figure 3.8.3-8, sheets 1 and 2. Westinghouse is requested to address the following two items:

1. The left side of Figure 3.8.3-8, sheet 2, shows that the mechanical connectors that are welded to a ¾ inch plate at the base of the module is identified as "CONT" (presumably meaning continuous) on one side of the module and on the other side the term "CONT" is struck out. Explain which detail is correct and revise the figure accordingly. Were the design detail calculations completed for this connection? Explain how the large loads coming from the CIS wall modules can be properly transferred from the module wall plate at a localized point to the embedded connectors.

2. The right side of Figure 3.8.3-8, sheet 2 shows #11 at 10 inch spacing span from the embedded basemat region into the wall module with about 3 inches of concrete cover. Since this type of connection is not addressed in ACI 349, describe how the loads from the module can be properly transferred from the module to the embedded bars in the basemat and how the design will be performed. When this detail was discussed with Westinghouse at an earlier audit this year, Westinghouse indicated that they would consider removing this second option.

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

Revision 1

Provide information on the connection of structural modules to base concrete in the RAI response.

Revision 2

The staff reviewed the response provided in Westinghouse letter dated March 12, 2010. The response provided information to address Item 1 of the original RAI question; however, the response contained in Item 2 did not provide all the needed information. Since the type of connection shown in the right side of DCD Figure 3.8.3-8, sheet 2, is not addressed in ACI 349, describe how the loads from the module can be properly transferred from the module to the embedded bars in the base concrete and how the design is performed. As background

information, this detail was discussed with the applicant at an earlier audit, the applicant indicated that they would consider removing this connection as an option, and instead rely on the other existing option of transferring loads directly from the faceplates to the base concrete using vertical bars and mechanical connectors. In addition, address the items listed below related to the revised typical detail shown in Figure RAI-SRP-3.8.3-SEB1-04-01A (Figure 01A).

1. Explain why the horizontal #11 @ 18" rebars are identified with mechanical connectors, while no mechanical connectors are shown in the sketch.

2. Explain why the enlarged detail in the upper right corner does not show the continuity of the vertical L4 x 3 angle, which is identified in the overall connection of Figure RAI-SRP-3.8.3-SEB1-04-01A.

3. Welds for the connection on the left side of the module should be shown in the figure to demonstrate the load path from the module to the base concrete.

4. Explain why the title of the figure is labeled "...single layer of dowel bars" because the left side of the figure shows two layers of dowel bars.

For the updated DCD Figure 3.8.3-8, Sheet 2, provided in the RAI response, explain why the left hand side connection detail was replaced with the new connection detail shown in Figure RAI-SRP-3.8.3-SEB1-04-01A, rather than replacing the right hand side connection detail. This is not consistent with the text information provided in the RAI response.

Westinghouse Response:

1. The plate at the base of the module does not need to be continuous. The revised typical detail is shown in Figure RAI-SRP-3.8.3-SEB1-04-01 ~~and will be included in the DCD. An alternate version of this detail is used in cases where the trusses are extended into the basemat as shown in RAI-SRP-3.8.3-SEB1-04-02.~~ The vertical dowel bars are placed in two layers. The base plate is stiffened to transfer the loads from the module wall plate to the embedded connectors. The design of the surface plate, base plate, and vertical stiffeners is checked by finite element analysis using the model shown in Figure RAI-SRP-3.8.3-SEB1-04-03. Tension corresponding to yield is applied to each dowel bar. These design calculations have been completed.

2. This connection has been removed as an option, and loads are transferred directly from the faceplates to the base concrete using vertical bars and mechanical connectors. ~~The right side of Figure 3.8.3-8, sheet 2 shows a dowel bar adjacent to the surface plate. The design of this type of connection is based on recommendations and test data given in Reference 1. The reference provides a design equation to calculate the strength of the connection based on key parameters such as concrete strength, dowel bar length and spacing, and concrete cover. This detail may be used when loading on the surface plates is within the range of the test data. It is used at the base of the CA05 module inside containment and the CA20 module outside containment where design loads are smaller.~~

Westinghouse Response to Revision 1:

The revised typical detail is shown in Figure RAI-SRP-3.8.3-SEB1-04-01 and provides information on the connection of structural modules to base concrete. The revised DCD figure identifying the key features of this connection design is provided.

Westinghouse Response to Revision 2:

Westinghouse will remove the "alternate" detail with the lapped splice, and provide mechanical connection of the structural modules to the base concrete using reinforcing bars and mechanical connectors. The mechanical connection detail illustrated in Figure RAI-SRP-3.8.3-SEB1-04-01 is representative of the connection that will be used. This mechanical connection consists of a base plate welded to the CA module liner plate, with gusset plates and vertical reinforcing bars attached to the underside of the base plate. DCD figure 3.8.3-8 is modified to reflect changes to the connection, including deletion of lap splice details. The details presented as typical are intended to provide clarity of the design intent, but not to supersede any changes necessary to accommodate features needed to ensure functionality of the design.

- Based on NRC response to RAI-SRP3.8.3-SEB1-04, Westinghouse has deleted the connection detail that does not have a direct load transfer path from the modules to the base concrete.
- Westinghouse will provide a connection similar to the mechanical connection illustrated in Figure RAI-SRP-3.8.3-SEB1-04-01 (module wall plate welded to base plate with mechanical connectors/reinforcing bar)
- The changes to the design calculations and drawings include consideration of the details required to make the mechanical connection work in all areas and to address fabrication and construction issues

Item 1: Base connections with mechanical connectors

Figure RAI-SRP-3.8.3-SEB1-04-01 in the previous response is replaced by a revised Figure RAI-SRP-3.8.3-SEB1-04-01 which includes revisions to respond to the previous comments. The items listed in the question related to the revised typical detail shown in Figure RAI-SRP-3.8.3-SEB1-04-01 (Figure 01A in Rev 1 response) are addressed below.

1. The horizontal #11 @ 18" rebars are incorrectly identified with mechanical connectors. These bars terminate with hooks as shown in the sketch. This figure has been revised.

2. The enlarged detail in the upper right corner shows a section midway between the vertical L4 x 3 angles. As shown in Figure 3.8.3-SEB1-04-02, the base plate is not continuous across the vertical angles. This design will be clarified in implementation of the changes described in this response.

3. Welds for the mechanical connection have been added to the figure to demonstrate the load path from the module to the base concrete.

4. The title of the figure has been revised to show "...double layer".

Reference:

~~1. Tsuda, K, Nakayama, T., Eto, H., Akiyama, K., Shimizi, A., Tanouchi, K, and Aoyama, H., "Experimental Study on Steel Plate Reinforced Concrete Shear Walls with Joint Bars", SMIRT Paper # 1086, August 2001.~~

Design Control Document (DCD) Revision:

Revise DCD Rev 17 Figure 3.8.3 8, sheet 2 of 3. The updated DCD Figure 3.8.3-8, Sheet 2 is provided on Page 6 of this response.

PRA Revision:

None

Technical Report (TR) Revision:

None

NOTE:
THE TYPICAL DETAILS SHOWN REPRESENT THE FUNDAMENTAL APPROACH FOR THE COMPOSITE WALL MODULES. THE FINAL DESIGN DETAILS MAY DIFFER FROM THOSE SHOWN FOR THE FOLLOWING REASONS.

- ACCESSIBILITY FOR INSPECTION DURING FABRICATION AND CONSTRUCTION
- LESSONS LEARNED FROM IMPLEMENTATION OF THE DESIGN
- EASE OF FABRICATION AND CONSTRUCTION
- RESOLUTION OF CONSTRUCTABILITY ISSUES AND SEQUENCES
- VARIATION IN MODULE SIZE AND CONFIGURATION

CHANGES MADE DURING DETAILED DESIGN ARE IN ACCORDANCE WITH THE SPECIFIC CODES AND STANDARDS INVOKED.

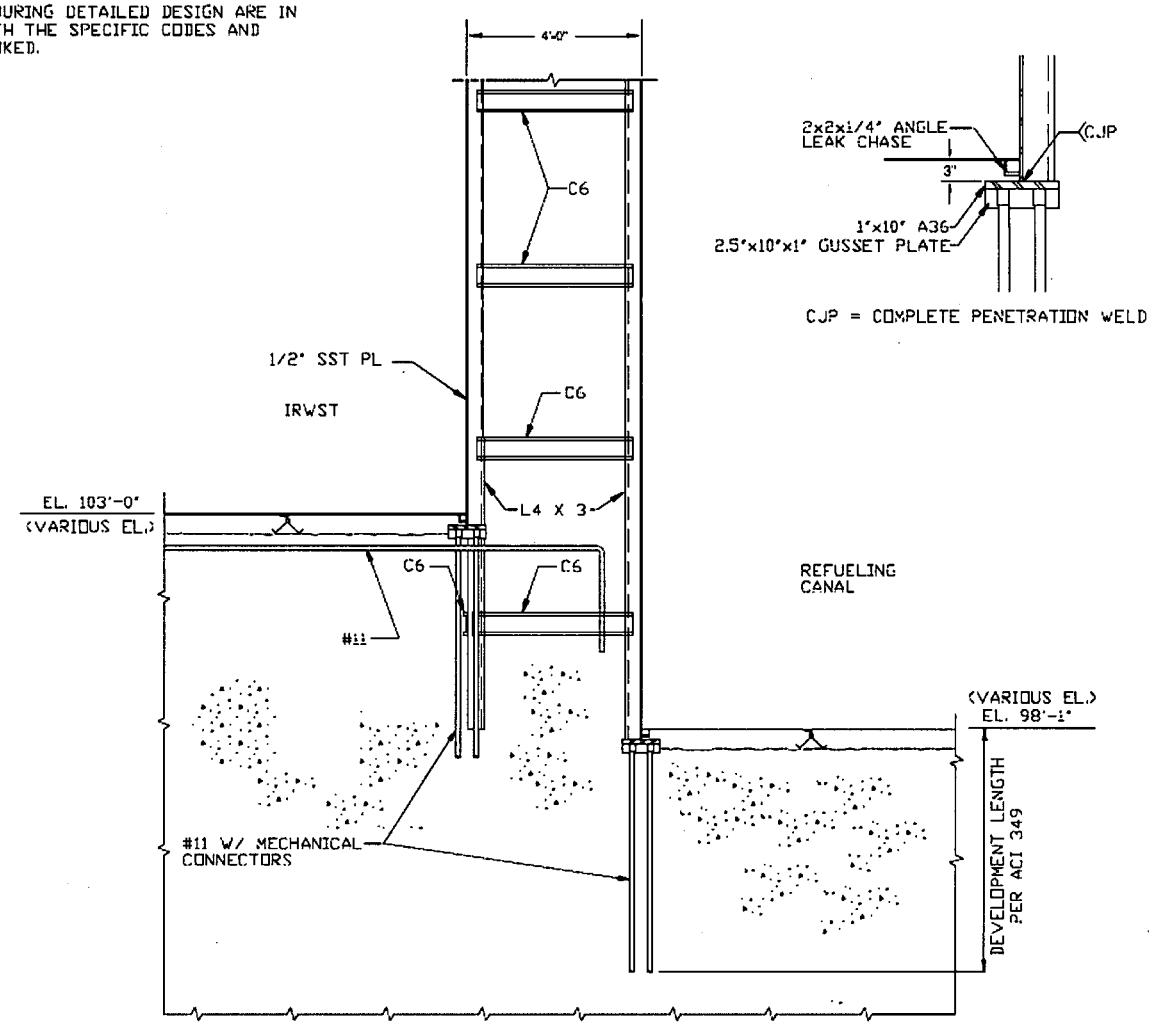


Figure RAI-SRP-3.8.3-SEB1-04-01
Representative details at base of CA01 Module Wall with a Double Layer of Dowel Bars

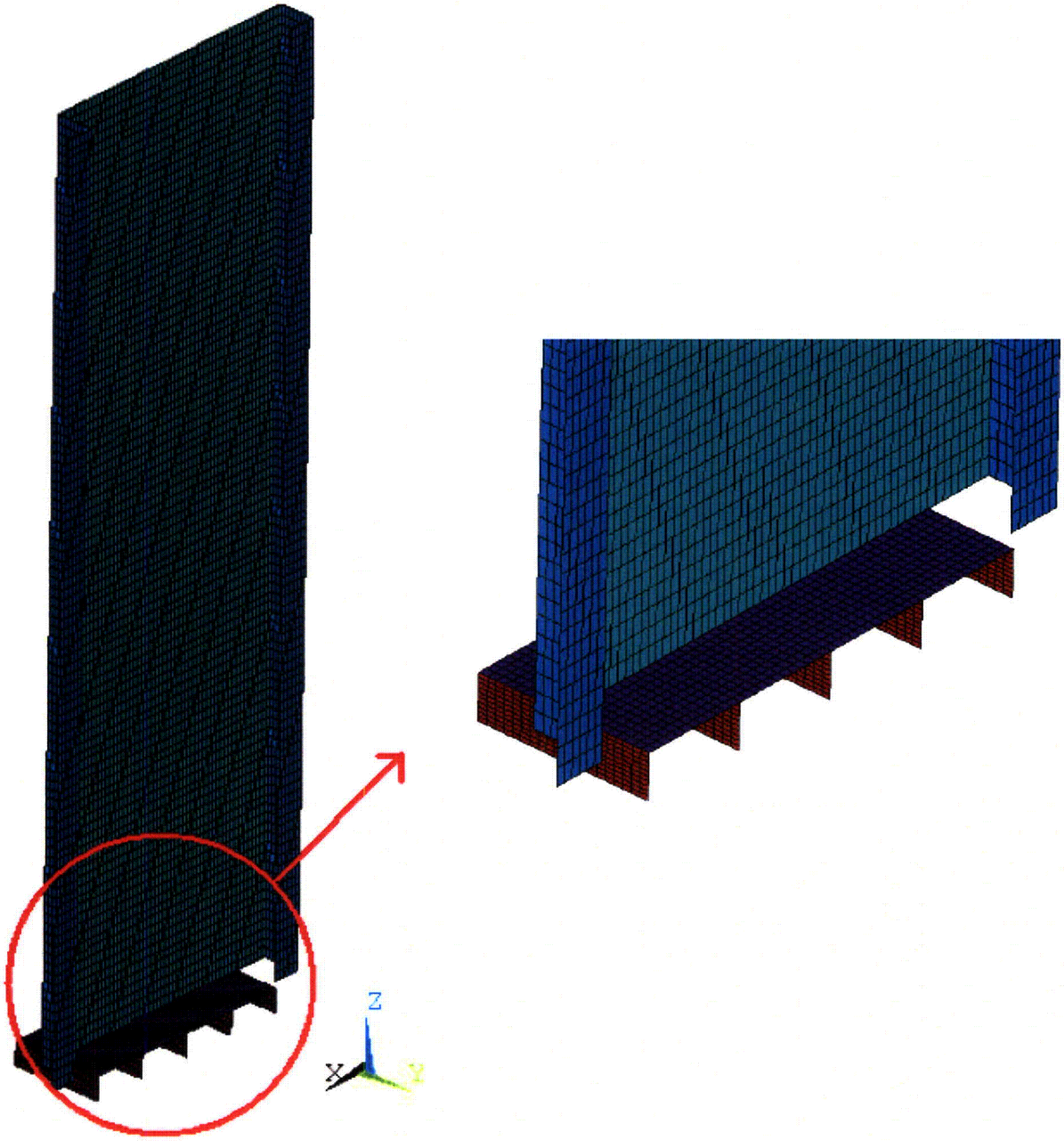
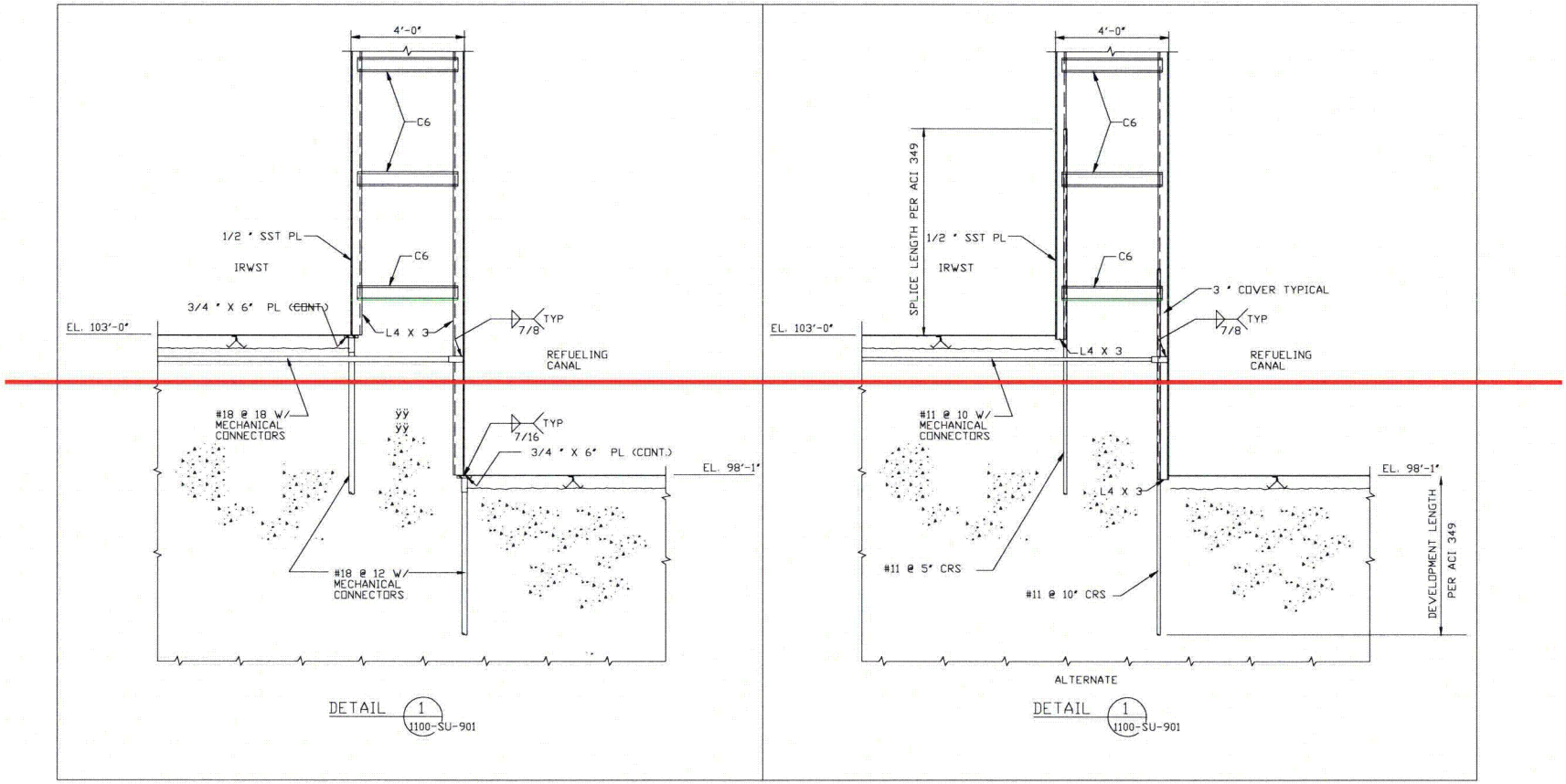


Figure RAI-SRP-3.8.3-SEB1-04-02



Replace Figure 3.8.3-8 (Sh2 of 3) with New Figure RAI-SRP-3.8.3-SEB1-04-01