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Your ref: Project Number 740
Our ref: DCP/NRC1791

October 19, 2006

Subject: AP1000 COL Standard Technical Report Submittal

In support of Combined License application pre-application activities, Westinghouse is submitting Revision 0 of AP1000 Standard Combined License Technical Report Number 85. The purpose of this report is to provide information to support closure of COL information item 3.7-4 by documenting the Basement and Foundation Design and Analysis, making it available for audit. Technical Report 85 provides Design Control Document mark-ups in support of Technical Report 85 and support of Technical Report 03 (APP-GW-S2R-010) "Extension of Nuclear Island Seismic Analysis to Soil Sites", which was transmitted with letter DCP/NRC1751, dated June 14, 2006. This report is submitted as part of the NuStart Bellefonte COL Project (NRC Project Number 740). The information included in this report is generic and is expected to apply to all COL applications referencing the AP1000 Design Certification.

The purpose for submittal of this report was explained in a March 8, 2006 letter from NuStart to the U.S. Nuclear Regulatory Commission.

Pursuant to 10 CFR 50.30(b), APP-GW-GLR-044, Revision 0, "Nuclear Island Basemat and Foundation," Technical Report Number 85, is submitted as Enclosure 1 under the attached Oath of Affirmation.

It is expected that when the NRC review of Technical Report 85 is complete, the NRC should consider that the design of the critical sections represented in Technical Report 85 is acceptable and satisfies the analysis and design requirements in the Design Control Document Section 3.7 and 3.8.

D079
Addl copy sent to PM
Steven Bloom

Questions or requests for additional information related to the content and preparation of this report 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,

A. Sterdis, Manager
Licensing & Customer Interface
Regulatory Affairs and Standardization

/Attachment

1. "Oath of Affirmation," dated October 19, 2006

/Enclosure

1. APP-GW-GLR-044, Revision 0, "Nuclear Island Basemat and Foundation," Technical Report Number 85, dated October 2006.

cc:	S. Bloom	-	U.S. NRC	1E	1A
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ATTACHMENT 1

"Oath of Affirmation"

ATTACHMENT 1

UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

In the Matter of:)

NuStart Bellefonte COL Project)

NRC Project Number 740)

APPLICATION FOR REVIEW OF
"AP1000 GENERAL COMBINED LICENSE INFORMATION"
FOR COL APPLICATION PRE-APPLICATION REVIEW

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

W E Cummins

W. E. Cummins
Vice President
Regulatory Affairs & Standardization

Subscribed and sworn to
before me this 19th day
of October 2006.

COMMONWEALTH OF PENNSYLVANIA
Notarial Seal
Debra McCarthy, Notary Public
Monroeville Boro, Allegheny County
My Commission Expires Aug. 31, 2009
Member, Pennsylvania Association of Notaries

Notary

ENCLOSURE 1

APP-GW-GLR-044, Revision 0

"Nuclear Island Basemat and Foundation"

Technical Report Number 85

AP1000 DOCUMENT COVER SHEET

TDC: | Permanent File: APY

RFS#: | RFS ITEM #:

AP1000 DOCUMENT NO.	REVISION NO.		ASSIGNED TO
APP-GW-GLR-044	0	Page 1 of 85	W - A. Sterdis

ALTERNATE DOCUMENT NUMBER: TR 85 | WORK BREAKDOWN #:

ORIGINATING ORGANIZATION: Westinghouse Electric Company

TITLE: **Nuclear Island Basemat and Foundation**

ATTACHMENTS:	DCP #/REV. INCORPORATED IN THIS DOCUMENT REVISION:
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ORIGINATOR R. S. Orr	SIGNATURE/DATE Richard Orr 10/6/06		
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VERIFIER L. Tunon-Sanjur	SIGNATURE/DATE Leonardo Tunon-Sanjur 10/6/06	VERIFICATION METHOD DETAILED REVIEW	
AP1000 RESPONSIBLE MANAGER L. Tunon-Sanjur	SIGNATURE* Leonardo Tunon-Sanjur 10/6/06	APPROVAL DATE	

* Approval of the responsible manager signifies that document is complete, all required reviews are complete, electronic file is attached and document is released for use.

APP-GW-GLR-044
Revision 0

October 2006

AP1000 Standard Combined License Technical Report

Nuclear Island Basemat and Foundation

Revision 0

Westinghouse Electric Company LLC
Nuclear Power Plants
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TABLE OF CONTENTS

1.0 INTRODUCTION

The AP1000 design has been certified for application at a hard rock site. Seismic analyses of the AP1000 at soil sites are described in Reference 3. This report summarizes the final design of the nuclear island basemat and exterior walls below grade for both hard rock and soil sites. It describes interface demands to be satisfied at a site.

The AP1000 Design Certification Document (DCD) includes COL information items and ITAAC that require reconciliation of the as-built structure to information and criteria included in the DCD and to analyses supporting the DCD. This report provides an updated baseline for the as-designed configuration and validates the basemat and foundation design against the updated seismic spectra and foundation conditions.

COL Information Item 3.7-4 and ITAAC 2. a) i) in Tier 1 Section 3.3 apply to the design and analysis of the structures addressed in this report.

COL Information Item 3.7-4 (NRC FSER Combined License Action Item 3.7.5-1) is associated with the as-built reconciliation of seismic analyses and is as follows:

> The Combined License applicant will reconcile the seismic analyses described in subsection 3.7.2 for detail design changes at rock sites such as those due to as-procured equipment information. Deviations are acceptable based on an evaluation consistent with the methods and procedure of Section 3.7 provided the amplitude of the seismic floor response spectra including the effect due to these deviations, do not exceed the design basis floor response spectra by more than 10 percent

The COL item as written requires as-built information and can not be satisfied at the Time of COL Application. This timing issue is addressed in a separate technical report. The information in this report validates that the design of the subject structures is acceptable for the updated seismic spectra and foundation conditions at COL application.

ITAAC 2.a)i) in Tier 1 Section 3.3 provides for verification of critical sections of the nuclear island structure. This ITAAC is included in DCD Tier 1, Table 3.3-6 and provided below. These critical sections include the basemat covered by this report. The information in this report validates that the design of the subject structures can satisfy the acceptance criteria. This technical report does not require that the ITAAC be revised.

Design Commitment	Inspections, Tests, Analyses	Acceptance Criteria
2.a) The nuclear island structures, including the critical sections listed in Table 3.3-7, are seismic Category I and are designed and constructed to withstand design basis loads as specified in the Design Description, without loss of structural integrity and the safety-related functions.	i) An inspection of the nuclear island structures will be performed. Deviations from the design due to as-built conditions will be analyzed for the design basis loads.	i) A report exists which reconciles deviations during construction and concludes that the as-built nuclear island structures, including the critical sections, conform to the approved design and will withstand the design basis loads specified in the Design Description without loss of structural integrity or the safety-related functions.

2.0 TECHNICAL BACKGROUND

2.1 Description of Nuclear Island Basemat and Embedded Portion

The nuclear island structures, consisting of the containment building, shield building, and auxiliary building, are founded on a common, cast-in-place, reinforced concrete basemat. Figure 2.1-1 shows a plan view of the AP1000 basemat and Figures 2.1-2 and 2.1-3 show cross section views at the containment center line. The basemat below the auxiliary building is 6 feet thick. Below the shield and containment building, the thickness of the basemat varies from 6 feet at the center to 22 feet under the annular tunnel to 39′-6″ on the west side where there is no tunnel. The nuclear island is embedded to a depth of 39′-6″ below nominal plant grade at elevation 100′. The bottom of the foundation is at elevation 60′-6″.

The plan view footprint is the same as the AP600. The section views are also similar to the AP600. The height of the AP1000 shield building and containment vessel is increased by 25′-6″. The shield walls around the reactor coolant loop in the containment internal structures are a few feet higher. The annular tunnel is full circumference in the AP600 and is eliminated on the west side for the AP1000.

The auxiliary building is a concrete shear-wall structure consisting of vertical shear/bearing walls and horizontal floor slabs. It wraps around approximately 50 percent of the circumference of the shield building. Walls are spaced 18 to 25 feet apart. The floor slabs and the structural walls of the auxiliary building are structurally connected to the cylindrical section of the shield building. The walls carry the vertical loads from the structure to the basemat. Lateral loads are transferred to the walls by the roof and floor slabs. The walls then transmit the loads to the basemat. The walls also provide stiffness to the basemat and distribute the foundation loads between them. This configuration of the structures above the basemat, in combination with the basemat, provides an efficient overall structure.

Adjoining buildings, such as the radwaste building, turbine building, and annex building are structurally separated from the nuclear island structures by a 2-inch gap at and below the grade. A 4-inch minimum gap is provided above grade. This provides space to prevent interaction between the nuclear island structures and the adjacent structures during a seismic event.

Resistance to sliding of the concrete basemat foundation is provided by passive soil pressure and soil friction. This provides the required factor of safety against lateral movement under the most stringent loading conditions.

Plant north is defined toward the turbine building so that the wall on line number 11 (see Figure 2.1-1) is the north wall. The plant coordinate system is defined with X north, Y west, and Z vertical.

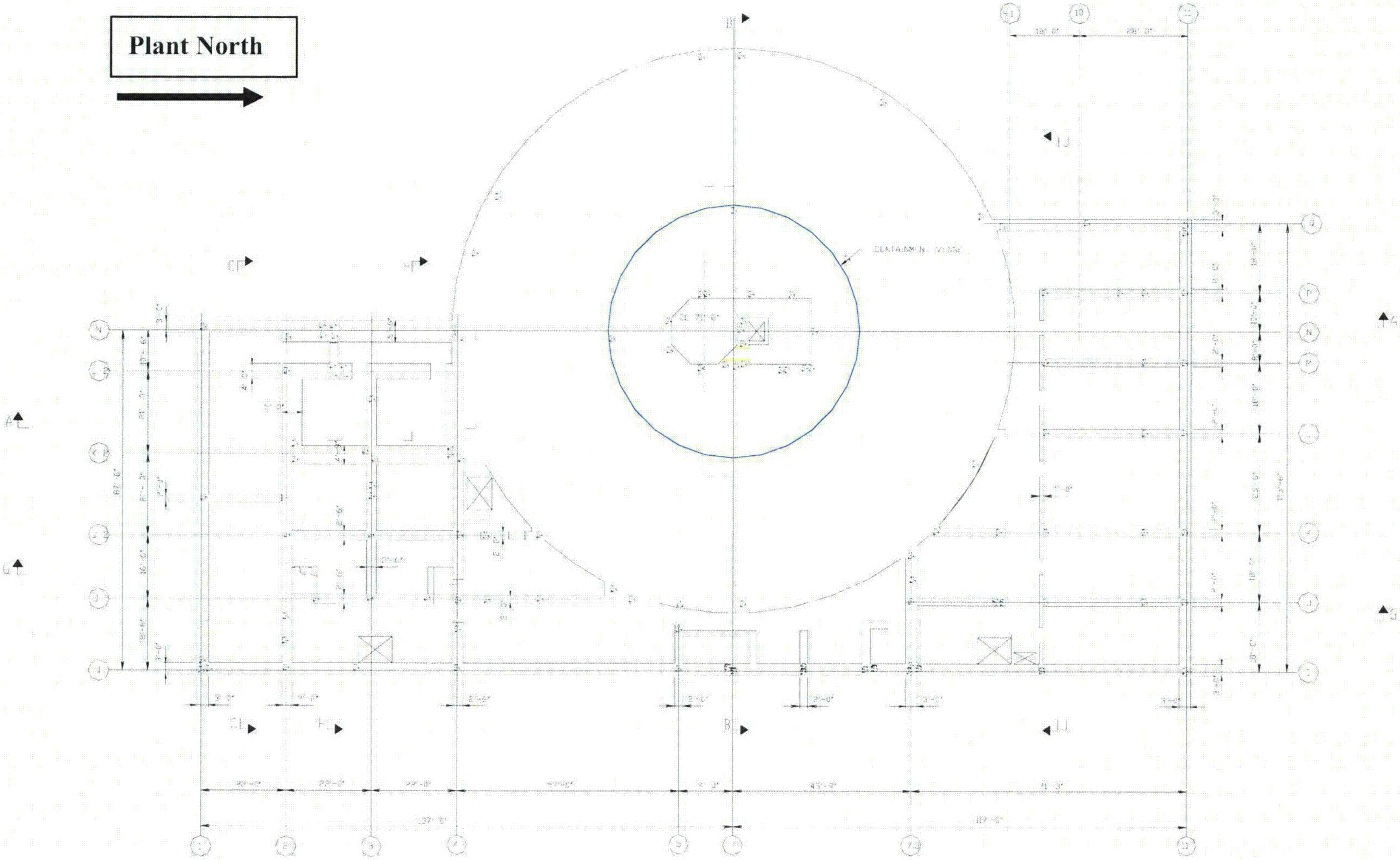


Figure 2.1-1

**Nuclear Island Key Structural Dimensions
Plan at El. 66′-6″**

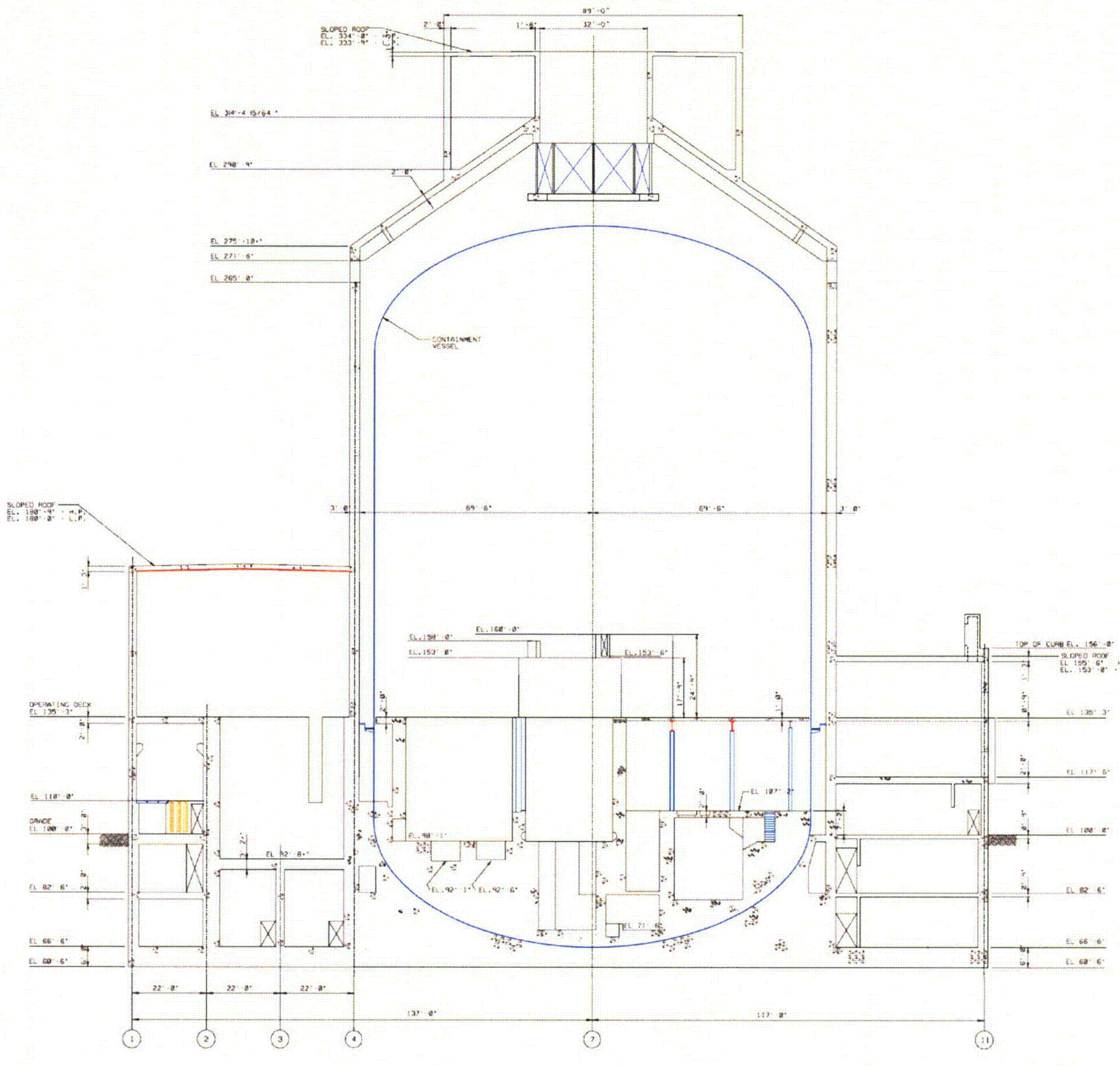


Figure 2.1-2

**Nuclear Island Key Structural Dimensions
Section A - A**

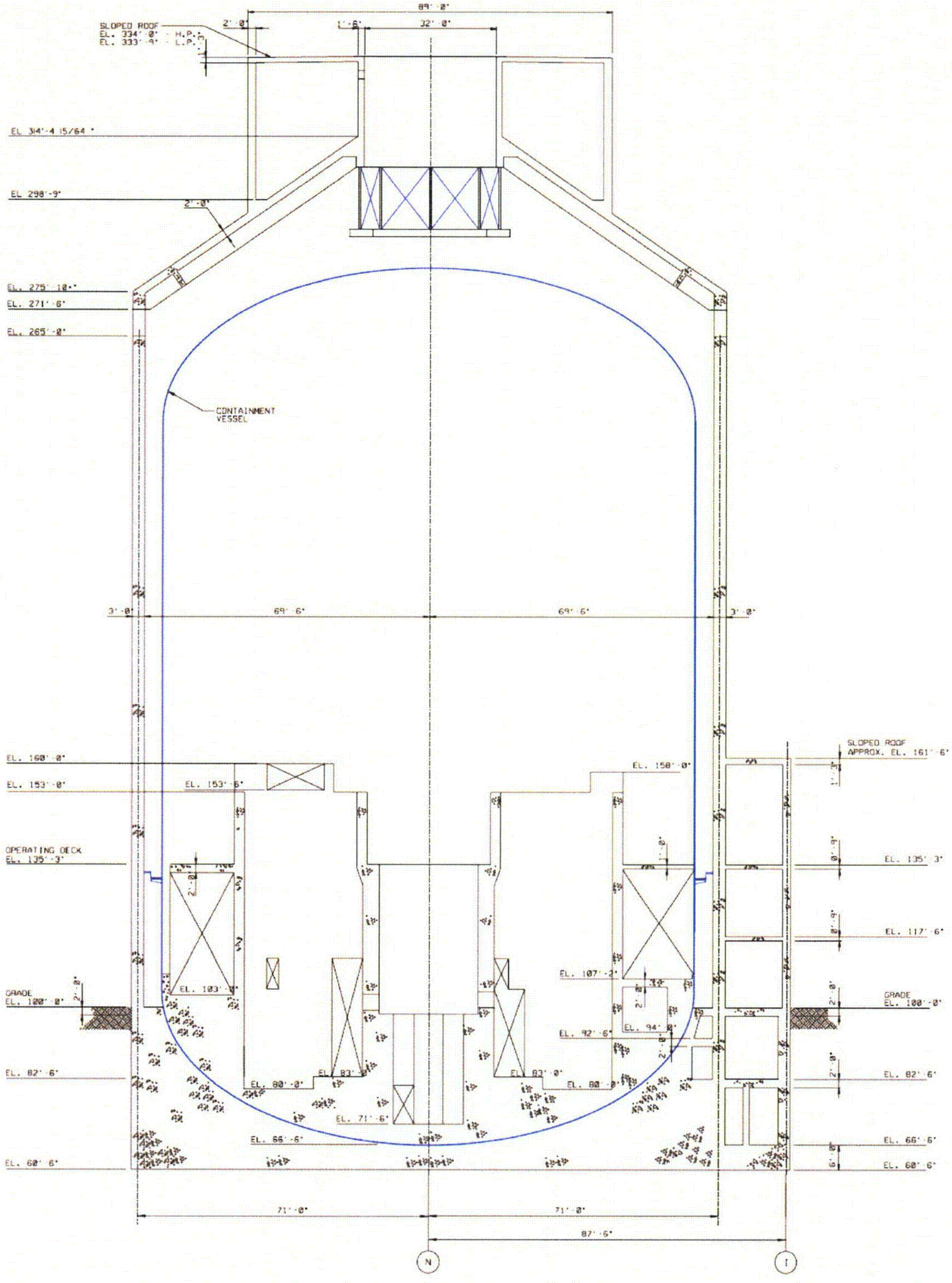


Figure 2.1-3

Nuclear Island Key Structural Dimensions
Section B - B

2.2 AP600 certified design for hard rock and soil sites

This section summarizes the design of the AP600 basemat and foundation. The methodology approved for the design of the AP600 forms the basis for the methodology applied in the design of the AP1000 basemat for soil sites.

The design of the AP600 basemat is described in subsection 3.8.5 of the AP600 DCD (Reference 4). This design is applicable for sites with the nuclear island founded on soil having a shear wave velocity greater than 1000 feet per second.

2.2.1 AP600 basemat analyses and design

The basemat was analyzed using a three-dimensional finite element ANSYS model of the basemat and attached superstructure. The model extended to elevation 100′ for the auxiliary building and to elevation 236′-0″ for the shield building. Some of the shear walls on the north side were modeled to their full height. The model considered the interaction of the basemat with the overlying structures and with the soil. Two possible uplifts were considered - uplift of the containment internal structures from the lower basemat and uplift of the basemat from the soil.

The vertical stiffness of the soil was represented by a subgrade modulus representative of the soft-to-medium soil case of 520 kips per cubic foot. The horizontal stiffness was represented by horizontal springs attached to some of the basemat nodes. Reactions on the side walls below grade were conservatively neglected. The containment internal structures were simulated with tetrahedral elements connected to the basemat with spring elements normal to the theoretical surface of the containment vessel.

The analyses considered dead loads, live loads, safe shutdown earthquake and containment pressure loads. Safe shutdown earthquake loads were conservatively applied as equivalent static loads for the soft rock case, in combination with the properties of soft-to-medium soil, since the soft rock case produces higher applied seismic forces to the structure than the soft-to-medium soil case. The safe shutdown earthquake loads were applied as static loads using the assumption that while maximum response occurs from one direction, the responses from the other two directions are 40 percent of the maximum.

Linear analyses were performed for the specified load combinations assuming that the soil springs can take tension. Critical load cases were then selected for non-linear analyses with basemat liftoff based on the results of the linear cases. The results from the analysis included forces, shears, and moments in the basemat, bearing pressures under the basemat, and the area of the basemat that is uplifted. Reinforcing steel areas calculated from the member forces for each load combination case were used for design of the DISH portion below the containment and shield building.

The refinement of the finite element model in some areas of the 6’ thick basemat below the auxiliary building was not considered adequate for design. Hence, the required reinforcing steel for the portion of the basemat under the auxiliary building was calculated from shears and bending moments in the slab obtained from separate calculations using the bearing reactions from the finite element analyses. Beam strip models of the slab segments were loaded with the bearing pressures under the basemat from the three-dimensional finite element analyses. The reinforcement required by these analyses on uniform soil springs was increased such that the basemat can resist loads 20 percent greater. This increase accommodated lateral variability of the soil investigated separately in a series of parametric studies described below.

The design of two critical bays of the basemat was described in subsection 3.8.5.4.3:

- Basemat between column lines 9.1 and 11 and column lines K and L
- Basemat between column lines 1 and 2 and column lines K-2 and N

A series of parametric analyses were performed to investigate the assumptions of a uniform subgrade modulus used as the design basis for the nuclear island basemat as described in the previous subsection.

- The three-dimensional finite element model had a subgrade modulus (520 kips per cubic foot) corresponding to a soft-to-medium soil. A parametric study was performed that indicated soft-to-medium soil resulted in higher shears and bending moments in the basemat than stiffer soils or rock.

- The three-dimensional finite element model used a uniform soil stiffness (520 kips per cubic foot) over the entire nuclear island foundation. Parametric studies were performed using a simplified model for two other soil stiffness variations. One variation considered the subgrade modulus equal to 1200 kips per cubic foot at the exterior walls and varied linearly to 400 kips per cubic foot at the center of the basemat. The other global variation considered 400 kips per cubic foot at the edges and varied linearly to 1200 kips per cubic foot at the center. Shear forces and bending moments in the exterior bay of the basemat were compared against the design shear forces and bending moments which were calculated by applying the maximum bearing pressure from the uniform soil case to a slab spanning in one direction. Neither of these cases resulted in higher shears nor bending moments than those from the uniform stiffness of soft-to-medium soil.

- Local variation of soil stiffness was considered. A buried rock pinnacle was considered at a soft-to-medium soil site and the increase in reactive soil pressure was estimated using linear elastic models. The analysis indicated that the increase in soil pressure was less than 15 percent for 15 feet of cover and less than 5 percent with 20 feet.

- Lateral variation of soil stiffness was evaluated using a rigid basemat model on soil springs. The AP600 was represented by an equivalent rectangular basemat. Bearing reactions for cases with lateral variation of the subgrade modulus were compared against the bearing reactions at the same locations for the same loading on a uniform subgrade modulus. These investigations showed that lateral soil variability which would be identified during the site investigation does not affect the bearing reactions by more than 20 percent unless the lateral variability is fairly extreme.

2.2.2 AP600 analyses of settlement during construction

AP600 DCD subsection 3.8.5.4.3 describes the analyses of settlement during construction. Construction loads were evaluated in the design of the nuclear island basemat. This evaluation was performed for soil sites meeting the site interface requirements at which settlement is predicted to be maximum. In the expected basemat construction sequence, concrete for the mat is placed in a single placement. Construction continues with a portion of the shield building foundation and containment internal structure and the walls of the auxiliary building. The critical location for shear and moment in the basemat is around the perimeter of the shield building. Once the shield building and auxiliary building walls are completed to elevation 82′ 6″, the load path changes and loads are resisted by the basemat stiffened by the shear walls.

The analyses of settlement for the AP600 were similar to those described in section 2.5 for the AP1000.

2.2.3 AP600 design for lateral earth pressure

AP600 DCD Appendix 2C describes the seismic lateral earth pressures used to design the exterior walls of the AP600. The loads were based on 2D SASSI analyses that considered interaction between the nuclear island and the adjacent buildings. The lateral earth pressures obtained from SASSI were adjusted to consider the effect of torsional motion of the nuclear island and to consider the local distribution at the corners of the nuclear island.

2.2.4 AP600 nuclear island stability

AP600 DCD subsection 3.8.5.5 describes the evaluation of the minimum factors of safety against sliding, overturning, and flotation for the AP600 nuclear island structures.

The sliding resistance is based on the maximum soil passive pressure resistance and the friction force developed between the basemat and the foundation using a coefficient of friction of 0.55.

The factor of safety against overturning of the nuclear island during a safe shutdown earthquake is evaluated using the static moment balance approach assuming overturning about the edge of the nuclear island at the bottom of the basemat. The resisting moment is equal to the nuclear island dead weight, minus maximum safe shutdown earthquake vertical force and buoyant force from ground water table, multiplied by the distance from the edge of the nuclear island to its center of gravity.

2.3 AP1000 certified design for hard rock sites

2.3.1 AP1000 basemat analyses and design

The analysis and design of the AP1000 nuclear island basemat for a hard rock site is described in subsection 3.8.5 of the AP1000 DCD (Reference 1). It generally followed the methodology previously described in section 2.2.1 for the AP600. Only differences from the AP600 analyses are described below.

The three-dimensional finite element model of the basemat included all of the nuclear island structures. The finite element model of the basemat is more refined than that used for the AP600 and had sufficient refinement that the member forces from the ANSYS analyses were used directly for the design of the reinforcement, thus eliminating the separate hand calculations using bearing pressure.

The subgrade modulus used in the analyses for hard rock was 6263 kips per cubic foot instead the 520 kips per cubic foot used for the AP600 soft to medium soil case.

In the AP600 design certification, soil bearing requirements were specified only under static loads. For the AP1000 the static demand increased 7.5% due primarily to the increase in height of the shield building. For AP1000 a requirement was added for dynamic loads. The value of 120,000 lb/ft^2 was based on the maximum bearing reaction from the equivalent static non-linear nuclear island basemat analyses described in subsection 3.8.5. This maximum bearing reaction occurs below the west edge of the thick concrete basemat below the shield building. This value was included in DCD Table 2-1 since it was expected that a hard rock site would provide adequate bearing.

2.3.2 AP1000 analyses of settlement during construction

Settlement at a hard rock site is small and is not significant to the design of the AP1000. No analyses were performed for the hard rock site.

2.3.3 AP1000 design for lateral earth pressure

The exterior walls of the seismic Category I structures below the grade are designed to resist the worst case lateral earth pressure loads (static and dynamic), soil surcharge loads, and loads due to external flooding. The lateral earth pressure loads are evaluated for two cases:

- Lateral earth pressure equal to the sum of the static earth pressure plus the dynamic earth pressure calculated in accordance with ASCE 4-98 (Reference 5), Section 3.5.3, Figure 3.5-1, "Variation of Normal Dynamic Soil Pressures for the Elastic Solution"

- Lateral earth pressure equal to the passive earth pressure

2.3.4 AP1000 nuclear island stability

AP1000 DCD subsection 3.8.5.5 describes the evaluation of the minimum factors of safety against sliding, overturning, and flotation for the AP1000 nuclear island structures. The methodology is similar to that described in subsection 2.2.4 for the AP600. Maximum base shear and overturning moments were taken from the time history analyses of the nuclear island lumped mass stick model.

2.4 Analyses of AP1000 foundation response on hard rock and soil sites

This section describes dynamic analyses of the nuclear island and the bearing reactions on the underside of the basemat. The assumptions in the analyses are described and the soil bearing reactions are discussed. The requirement for site bearing is determined from these analyses.

2.4.1 2D SASSI analyses

Parametric 2D SASSI linear elastic analyses were performed for a variety of soil conditions as described in Section 4.4.1.2 of Reference 3. These analyses used AP1000 building models comprising 3 sticks (ASB, CIS and SCV). Six soil cases with shear wave velocity profiles shown in Figure 2.4-1 were analyzed in each horizontal direction. Bedrock with shear wave velocity of 8000 fps was assumed at a depth of 120' below grade. Thus the depth of soil below the foundation mat is 80.5'.

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

Table 2.4-2 shows the reactions at the underside of the basemat for each soil case. These are conservative estimates using the results of the 2D SASSI horizontal analyses also used for the member forces in Table 2.4-1. Horizontal loads on the portion below grade are added absolutely to the sum of the member forces above grade. The reactions in this table are used in the evaluation of nuclear island stability described in section 2.9.

Soil pressures in all elements surrounding the nuclear island were calculated in 2D SASSI analyses for dead load and for the SSE in the east west direction. Figure 2.4-2 shows the maximum bearing pressures below the basemat. The dotted lines show compression based on dead load minus the |SSE|; the solid lines show compression or tension for dead load plus |SSE|. The tension may result in small lift off. The maximum bearing pressures are associated with the soft to medium soil case.

The soft-to-medium soil case and the upper bound soft to medium soil case result in the largest overturning moments for seismic input in the east west direction. In the north south direction the firm rock, soft rock and upper bound cases give larger overturning moments than the soft to medium case. These moments are lower than those for east west input. The AP1000 footprint is shorter along the east west axis than along the north south axis. Softer sites typically have lower soil strength than the firmer sites. From review of the member forces in Table 2.4-1, the bearing reactions in Table 2.4-2 and the maximum bearing pressures in Figure 2.4-2, the soft to medium soil case is selected as the basis for the bearing demand. The effect of lift off is investigated for this case as described in the following section.

2.4.2 2D ANSYS non-linear dynamic analyses

The SASSI analyses described in section 2.4.1 are linear elastic analyses. They permit tension to be carried across the interface between the soil and the basemat. Dead and live load bearing pressures from the ANSYS analyses on soil springs are shown in Figure 2.6-3. The bearing pressures vary from about 6 ksf on the east side to 14 ksf below the edge of the shield building on the west side. The absolute value of some of the seismic bearing pressures calculated by SASSI exceed the dead load bearing pressures giving a resultant tension uplift as shown in Figure 2.4-2. The effect of lift off was analyzed in ANSYS. Linear seismic analyses were performed on the ANSYS models to confirm similar behavior to the SASSI analyses. Non-linear analyses were then performed for dead plus seismic loads with compression only contact elements.

Section 7.0 of Reference 3 describes analyses to investigate the effect of liftoff during the safe shutdown earthquake of 0.3g on a hard rock and a soft to medium soil site using an East-West lumped-mass stick model of the nuclear island structures supported on a rigid basemat with nonlinear springs. Analyses for the hard rock site were performed on a model with an equivalent rectangular basemat of 140.0' × 234.5'. Analyses for the soft to medium soil site were performed on a model with the actual footprint of the basemat. The overall width is 161' whereas the equivalent rectangle only had a width of 140'. Both have the same overturning resistance in linear analyses where soil springs take tension. Both models have the same eccentricity between the center of mass of the nuclear island and the centroid of the basemat.

The rock and soil were modeled as horizontal and vertical spring elements with viscous damping at each node of the rigid beam. The NI combined stick is attached to the rigid basemat at the NI gravity center, which is about 9 feet from the center of the rigid basemat. In the north-south direction, the stick is fixed at the bottom (EL. 60.5'). The stiffness properties of the ASB and CIS in the NI combined stick model are reduced by a factor of 0.8 to consider the effect of cracking as recommended in Table 6-5 of FEMA 356.

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

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

Damping was included as mass and stiffness proportional damping matching the modal damping specified for each structure at frequencies of 3 and 25 Hertz.

Comparison of floor response spectra and the maximum member forces and moments for these two cases show that the liftoff has insignificant effect on the SSE response.

Figure 2.4-3 shows the maximum dynamic subgrade pressure during the analysis on hard rock. Figure 2.4-4 shows the time history of the pressure at the west and east edge around the time that the peak pressure occurs at the west edge. Lift off increases the subgrade pressure close to the west edge from 25.5 ksf to 27.8 ksf with insignificant effect beneath most of the equivalent rectangular basemat. The effect on the pressure at the west edge is significantly less than that calculated in the non-linear basemat analyses using equivalent static accelerations.

Figure 2.4-5 shows the time history of uplift displacements at the basemat edges for the soft to medium soil site. Maximum uplift at the east edge occurs at the time around 5 seconds for both linear and non-linear (liftoff) analyses. The figure also shows the time history of bearing pressures at the basemat edges. Lift off increases the subgrade pressure close to the west edge from 29.7 ksf to 34.5 ksf. The maximum bearing pressure at the west edge occurs at the time around 5 seconds due to the maximum uplift at the east edge. The maximum pressure of 29.7 ksf for the linear analyses is about 20% higher than the maximum reaction from the 2D SASSI analyses shown in Figure 2.4-2. The differences in results are attributed to:

- The 2D SASSI analyses include the effect of side soil on the lower part of the embedded nuclear island. This soil is not considered in the 2D ANSYS analyses
- The Boussinesq effect of higher reactions close to the edge shows up in the 2D SASSI analyses which use a continuum representation of the soil layers but not in the 2D ANSYS analyses which use Winkler soil springs.

2.4.3 Site interface for soil

The AP1000 requirements to be included in DCD Table 2-1 are as follows:

Soil	
Average Allowable Static Bearing Capacity	Greater than or equal to 8,600 lb/ft^2 over the footprint of the nuclear island at its excavation depth
Maximum Allowable Dynamic Bearing Capacity for Normal Plus SSE	Greater than or equal to 35,000 lb/ft^2 at the edge of the nuclear island at its excavation depth

In the AP600 design certification, soil bearing requirements were specified only under static loads. For the AP1000 the static demand increased 7.5% due primarily to the increase in height of the shield building. The AP1000 DCD for hard rock added a requirement of 120,000 lb/ft^2 for dynamic loads. This was based conservatively on the maximum bearing reaction from the equivalent static non-linear nuclear island basemat analyses described in section 2.3. This maximum bearing reaction occurs below the west edge of the thick concrete basemat below the shield building. This value was included in DCD Table 2-1 since it was expected that a hard rock site would be capable of satisfying this bearing demand. The dynamic non-linear analyses described in section 2.4.2 show much lower bearing reactions (27.8 ksf for hard rock) than those from the equivalent static design analyses for the basemat. The 2D ANSYS non-linear analyses show that the soft-to-medium soil case gives higher bearing pressures (34.5 ksf) than the hard rock case. This establishes the soil bearing interface of 35,000 lb/ft^2. The bearing pressures from the ANSYS analyses are conservative because the effect of the side soil is conservatively neglected.

Limitations on soil variability were included in the revisions to Table 2-1 in Reference 3 related to seismic response. These limitations are also applicable to foundation design.

Table 2.4-1

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

Units: 1000 kips & 1000 ft-kip

	North-South model		East-West model	
Soil case	**North-South Shear**	**Moment about E-W axis**	**East-West Shear**	**Moment about N-S axis**
	F_X	M_{YY}	F_Y	M_{XX}
Hard rock (HR)	49.75	6923	52.55	6122
Firm rock (FR)	50.07	7330	52.40	6731
Soft rock (SR)	52.08	7466	56.06	7477
Upper bound soft to medium (UB)	54.18	7357	62.46	7985
Soft to medium (SM)	51.87	6528	62.05	7983
Soft (SS)	27.79	2489	33.00	4218

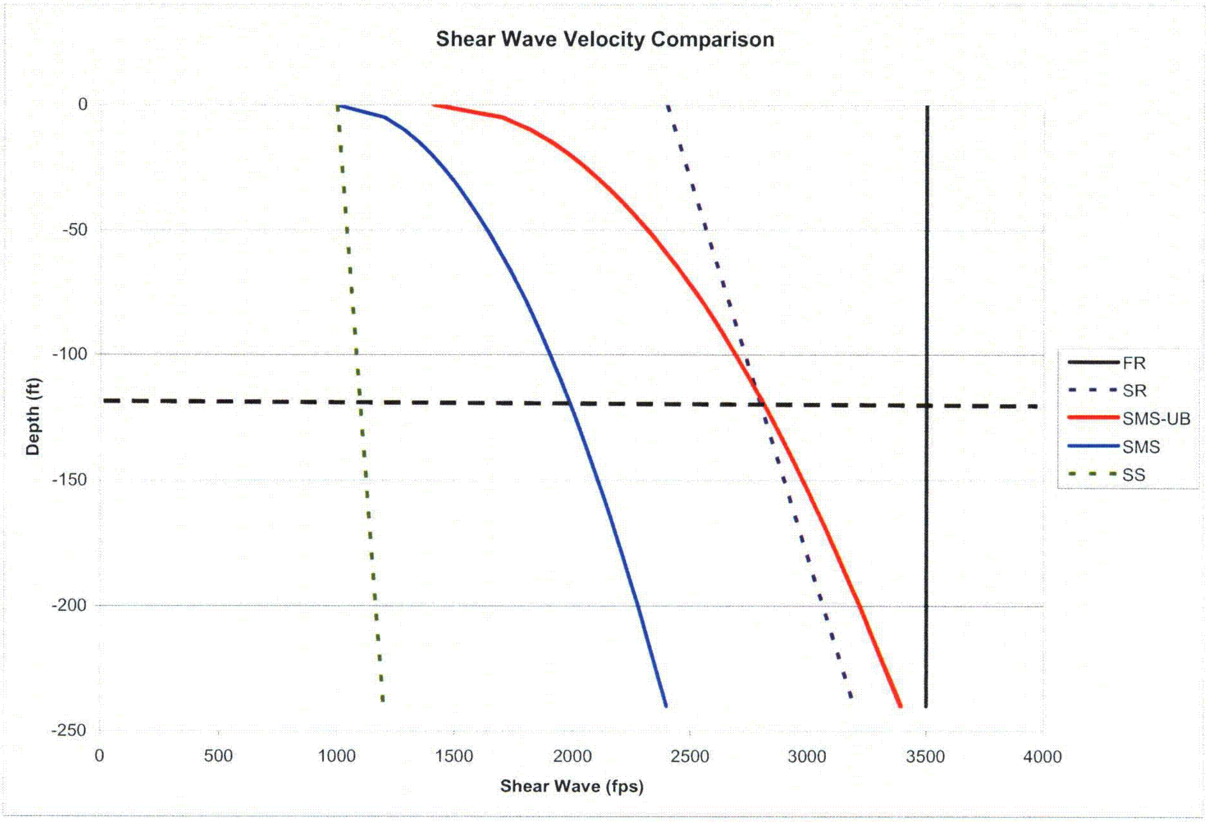
Table 2.4-2 – Maximum Seismic Reactions at Center Line of Containment

Units: 1000 kips & 1000 ft-kip

Seismic Reactions	**HR**	**FR**	**SR**	**UBSM**	**SM**	**SS**
Shear NS, F_X	112.70	113.04	114.79	117.86	106.73	71.42
Shear EW, F_Y	111.89	112.24	122.02	128.43	117.84	75.92
Vertical, F_Z	111.97	109.23	113.19	109.27	116.22	99.29
Moments Relative to Centerline of Containment						
M_{XX} EW Excitation	10,641	11,273	12,127	12,624	12,363	7,034
M_{XX} Vertical Excitation	1,879	1,821	1,364	2,009	1,906	1,196
M_{XX} SRSS	10,806	11,419	12,203	12,783	12,509	7,135
M_{YY} NS Excitation	11,521	11,957	11,804	11,577	10,474	4,871
M_{YY} Vertical Excitation	1,040	1,011	843	991	1,065	740
M_{YY} SRSS	11,568	12,000	11,834	11,619	10,528	4,927

Notes:

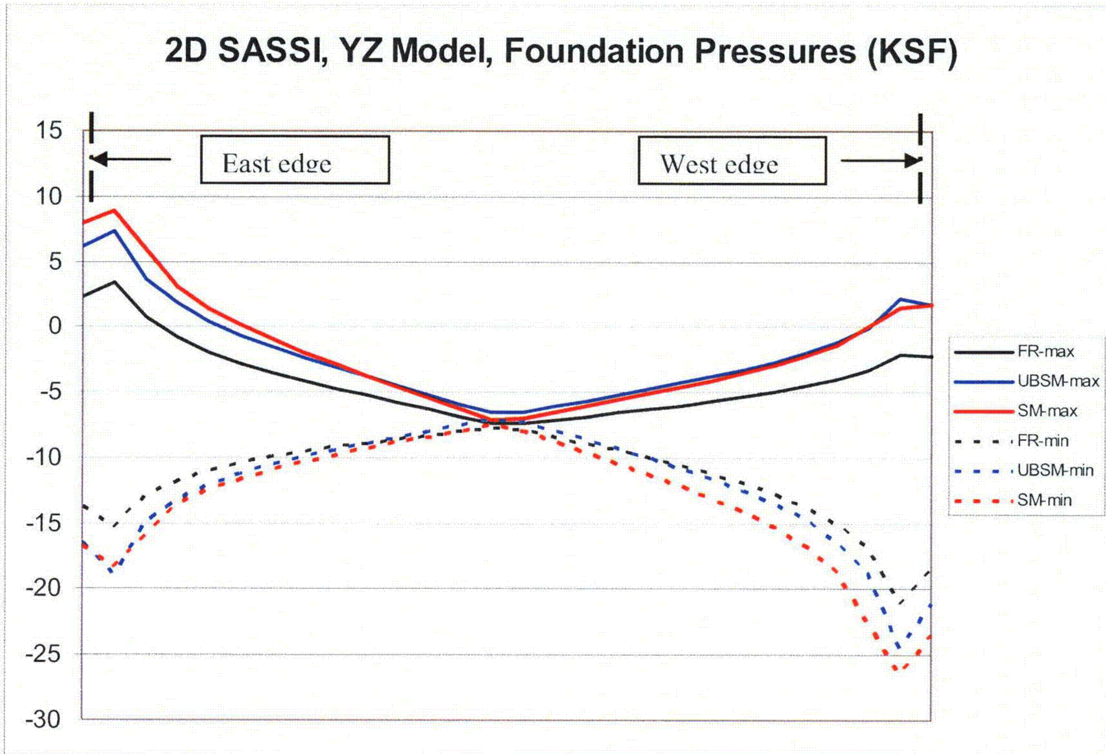
1. HR = Hard Rock, FR = Firm Rock, SR = Soft Rock, UBSM = Upper Bound Soft to Medium Soil, SM = Soft to Medium Soil, SS = Soft Soil.
2. Reactions for horizontal input are calculated from member forces at grade in 2D SASSI analyses plus maximum acceleration times mass below grade. Reactions due to vertical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



Notes:

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

Figure **2.4-1** Generic Soil Profiles



Notes:

1. Uplift tension positive; bearing compression negative
2. Dotted lines show dead load minus |SSE|
3. Solid lines show dead load plus |SSE|

Figure **2.4-2** Maximum Bearing Pressures on Underside of Basemat

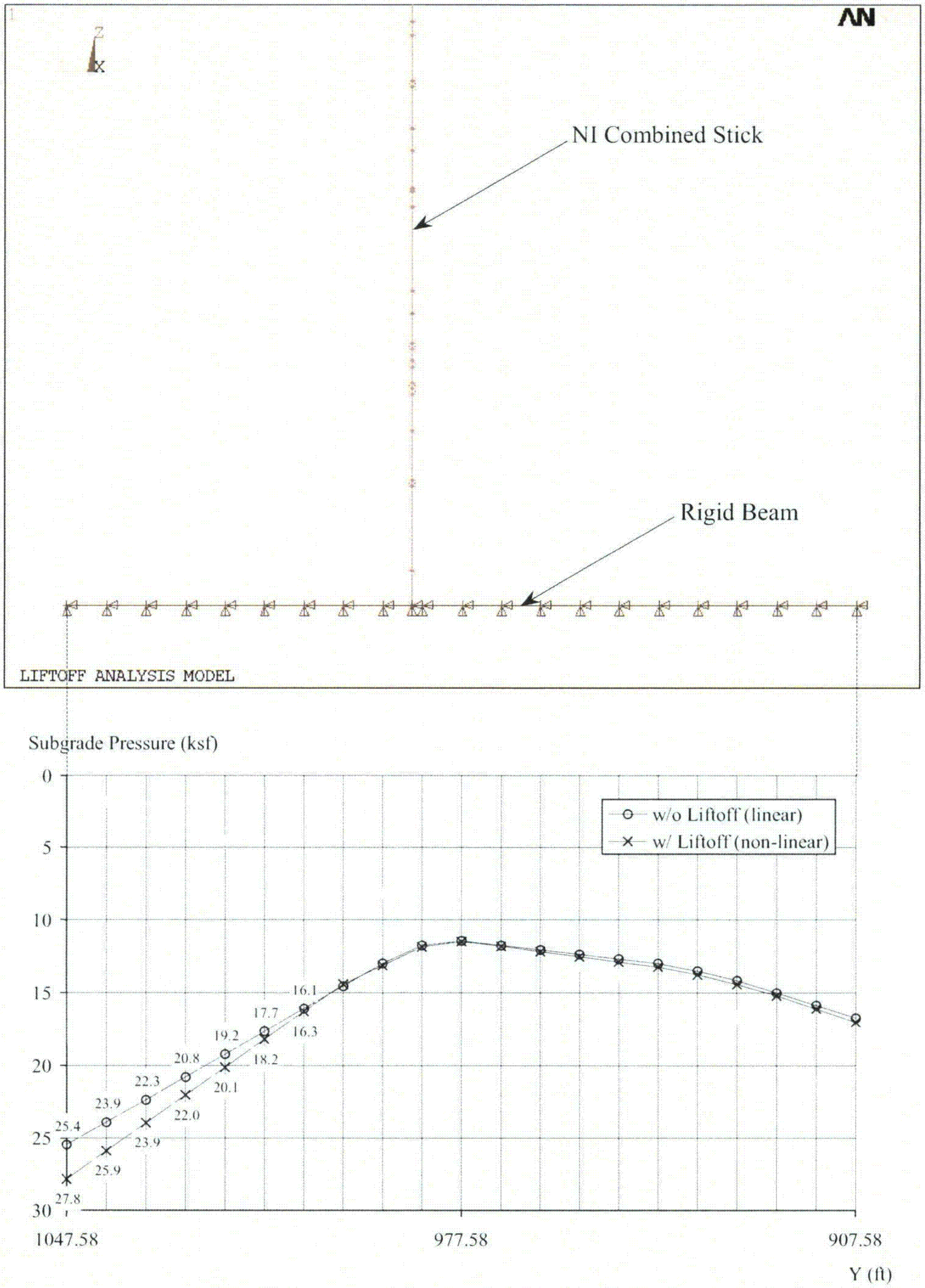


Figure 2.4-3 - Maximum Dynamic Subgrade Pressure Distribution on Hard Rock

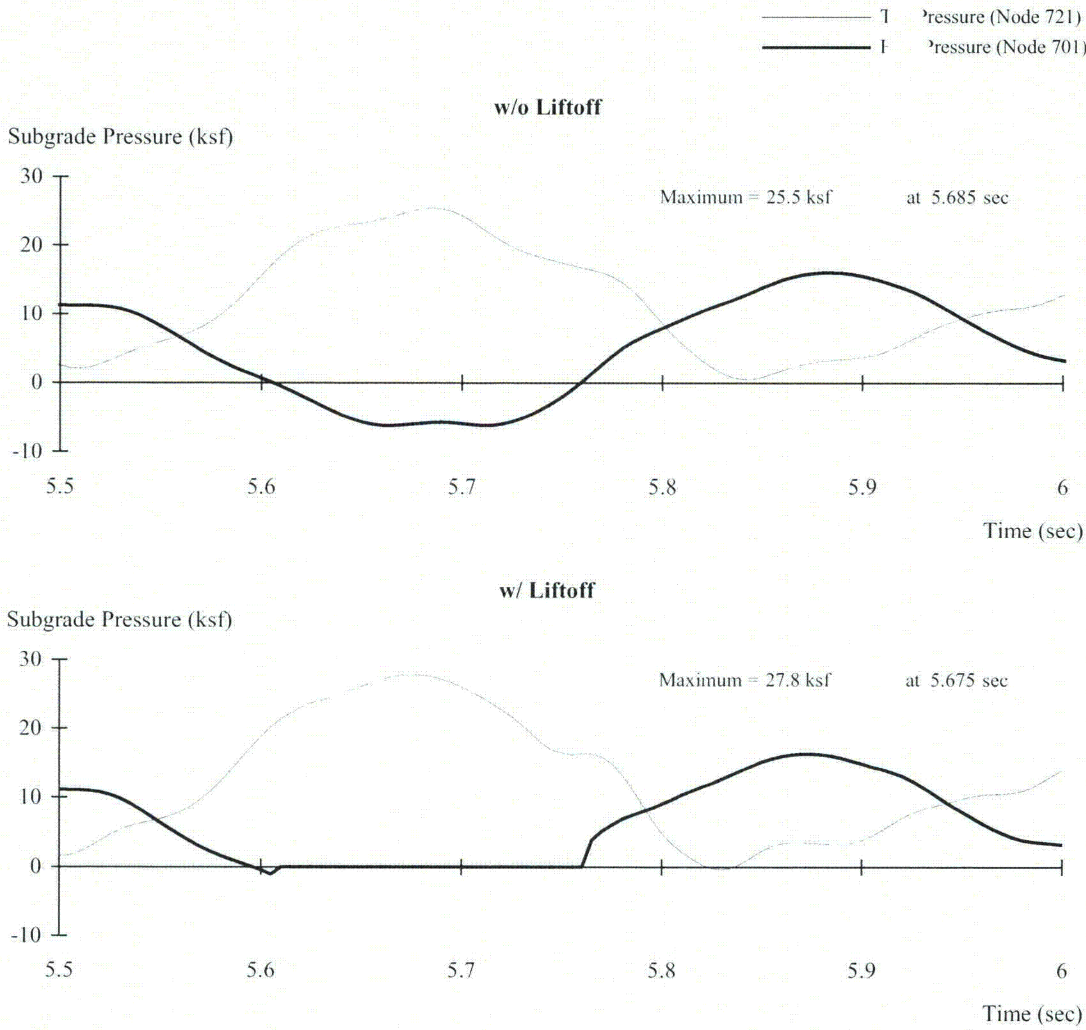


Figure 2.4-4 – 2D ANSYS Time History of Basemat Edge Pressure - Hard Rock

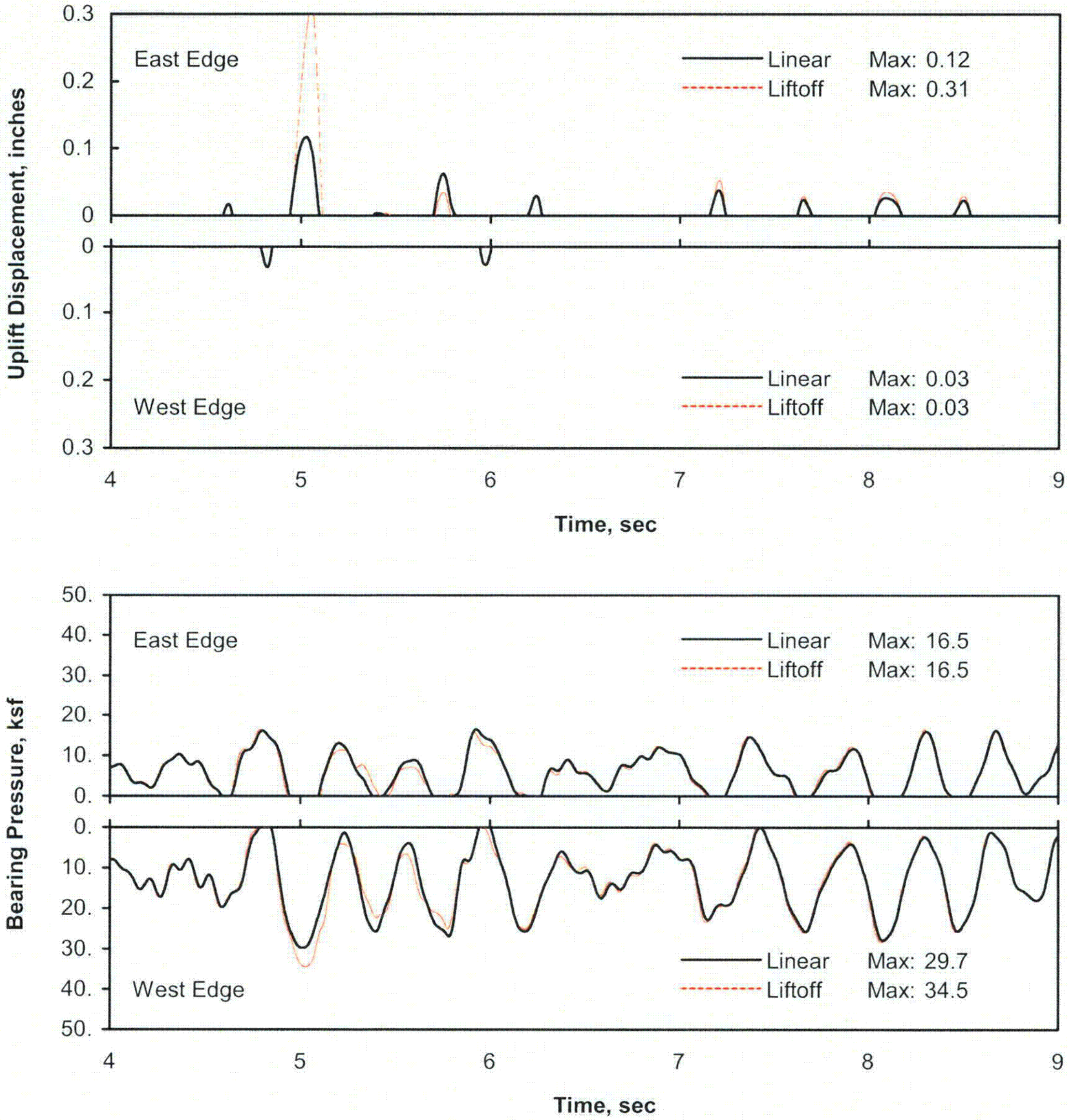


Figure 2.4-5 – 2D ANSYS Time History of Basemat Edges - Soft to medium soil

2.5 *Analyses of settlement during construction*

Construction loads were evaluated in the design of the nuclear island basemat. This evaluation was performed for soil sites meeting the site interface requirements at which settlement is predicted to be maximum. In the expected basemat construction sequence, concrete for the mat is placed in a single placement. Construction continues with a portion of the shield building foundation and containment internal structure and the walls of the auxiliary building. The critical location for shear and moment in the basemat is around the perimeter of the shield building. Once the shield building and auxiliary building walls are completed to elevation 82′ -6″, the load path changes and loads are resisted by the basemat stiffened by the shear walls.

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

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

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

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

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

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

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

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

Of the three construction scenarios analyzed, the delayed auxiliary building case results in the largest demand for the bottom reinforcement in the basemat. The delayed shield building results in the largest demand for the top reinforcement in the basemat.

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

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

- Concrete may not be placed above elevation 117′ -6″ in the auxiliary building.

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

2.6 Nuclear island basemat design

The design of the nuclear island basemat is described in the basemat design summary report prepared in accordance with the guidelines of Standard Review Plan 3.8.4. The design is based on the worst combination of seismic loads and soil properties. Non-linear equivalent static analyses are performed which consider lift off of the basemat from the soil. The analyses use the detailed model of the nuclear island (NI05) shown in Figures 2.6-1 and 2.6-2. The soft-to-medium soil case is considered as described in section 2.4.1. These analyses are similar to those described in section 2.2.1 for the AP600 and in section 2.3.1 for the AP1000 on hard rock. The equivalent static loads are developed from the maximum accelerations given by time history analyses of the nuclear island on hard rock and soil sites. No credit is taken in these analyses for the effect of side soils.

2.6.1 3D ANSYS Equivalent Static Non-Linear Analysis

2.6.1.1 Subgrade modulus

The basemat under the auxiliary building is 6 feet thick and supports a grid work of walls. These walls in turn stiffen the slab by producing relatively short spans, in the range of 3 to 4 times the thickness. The design of the 6' thick portion of the mat is controlled by the maximum bearing pressure under the slab during a seismic event. Maximum bearing pressures occur for the case of maximum overturning moment. Due to the shape of the footprint of the nuclear island seismic loads in the east-west direction give the largest bearing pressures and the greatest potential for lift off.

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

Reinforcement design uses member forces from analyses of the nuclear island on soil springs. The shear and bending moment in the basemat are dependent on the relative stiffness of material supporting the foundation and the global stiffness of the nuclear island buildings and the local bending stiffness of the basemat. The walls of the nuclear island are stiff relative to a soil. The contact pressure is nearly linearly distributed and the actual magnitude of the subgrade modulus has small effect on the member forces in walls of the nuclear island. The local slabs of the basemat, spanning 18 to 25 feet between walls, are flexible relative to the subgrade. For such a case, there will be a decrease in pressure near the center of the slab and an increase in pressure near the walls. This redistribution decreases as the subgrade modulus decreases. It is therefore conservative for the design of the basemat to use a low value of the subgrade modulus. This is discussed further in section 2.7 which describes analyses of a detailed model of portions of the basemat on both soil springs and soil finite elements.

The AP600 basemat analysis used the soft to medium linear profile (this profile was subsequently changed to the parabolic profile thus increasing shear wave velocity below the nuclear island). Soil springs of 520 kcf were established by the Steinbrenner method using undegraded properties and soil up to grade.

Although the subgrade modulus calculated for the AP1000 soil cases in Table 2.6-1 could have justified use of a subgrade modulus of 1000 kcf for the dry soft to medium soil or 1300 kcf with the water table above the foundation level, it was decided to retain the 520 kcf used in the AP600 analyses. As described above this is conservative since it maximizes the bending moments in the slabs. It also permitted a direct comparison of the AP1000 analyses to those for the AP600.

2.6.1.2 Equivalent static accelerations

Seismic loads for the evaluation of the basemat of the Nuclear Island are developed from the results of the global seismic analyses as described in Section 6.2 of Reference 3. They are specified as equivalent static seismic accelerations as shown in Table 6.2-7 of Reference 3. These accelerations envelope the response of all soil conditions.

Table 2.6-2 compares the sum of the soil reactions on the basemat for the equivalent static accelerations applied in the design analyses of the basemat on soil springs to those obtained from linear time history analyses of the nuclear island. The values for the fixed base analyses are from the nuclear island stick model time history analyses documented in the AP1000 DCD for the hard rock analyses. The values for the soft to medium soil are from 2D SASSI analyses described in section 2.4.1. Comparison of the base reactions demonstrate the conservatism of the equivalent static accelerations applied in the basemat analyses.

2.6.1.3 Normal load bearing reactions

The bearing reactions under dead and live load from the 3D ANSYS analyses on soil springs with subgrade modulus of 520 kcf are shown in Figure 2.6-3.

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. 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.

2.6.2 Basemat reinforcement design

The Nuclear Island basemat is a reinforced concrete structure designed in accordance with the following American Concrete Institute (ACI) standard:

ACI 349-01, Code Requirements for Nuclear Safety Related Concrete Structures

Additional reinforcement is provided in the design of the 6' mat for soil sites such that the basemat can resist loads 20 percent greater than the demand calculated by the equivalent static acceleration analyses on uniform soil springs. This increase is based on the AP600 precedent and accommodates lateral variability of the soil investigated separately in a series of parametric studies.

The reinforcement required is calculated for the member forces for each of the following load combinations. These load combinations are selected from the load combinations in Table 3.8.4-2 of the DCD (Reference 1). Other load combinations do not control design of the basemat.

1 1.4 x (D + H) + 1.7 x (L)

3 D + L + Es

9 1.4 x (D + H) + 1.7 x (L) + 1.5 x (Pd)

10 D + L + Pd + Es

The reinforcement is calculated for each shell element in the 6' basemat and for a series of paths through the solid elements of the DISH. Tables 2.6-4 to 2.6-8 show the reinforcement required in both the 6' basemat and the DISH. The tables also show the reinforcement provided. Reinforcement for the 6' thick mat and the DISH is provided in up to 10 layers with layer number 1 being the lowest layer at the bottom of the mat. Layers 1 and 2 are at the bottom of the 6' mat and the DISH. Layer 3 is an additional circumferential layer below the DISH. Layers 4 and 5 are at the top of the 6' mat and below the center portion of the containment vessel. Layers 6 through 10 are below the containment vessel. These layers are shown in Figures 2.6-9 and 10. The reinforcement arrangement for each layer is shown in Figure 2.6-9 to 2.6-15.

2.6.2.1 Comparison of the AP1000 to AP600 in the Two Critical Bays

The reinforcement provided for the AP1000 has been compared to the reinforcement provided for the AP600. In general the reinforcement for the AP1000 has stayed the same or has increased due to the higher bearing demand caused by the increase in height of the shield building. In a few cases the reinforcement has decreased. These cases were reviewed.

Table 2.6-9 summarizes the reinforcement design for the AP1000 and AP600 in the two critical bays of the basemat described in subsection 3.8.5.4.3 of the DCD:

- Basemat between column lines 9.1 and 11 and column lines K and L
- Basemat between column lines 1 and 2 and column lines K-2 and N

The reinforcement provided for the AP1000 between column lines 9.1 and 11 and column lines K and L is equal to or greater than that for the AP600 with the exception of the east west bottom and shear reinforcement The spacing of this reinforcement was increased from 10" to 12". This change in spacing was made to improve constructability since the dowel bars for the walls are also at multiples of 12" spacing. Review of the calculations showed that the required reinforcement in the east west direction could be reduced due to the large conservatism of the hand calculations used for the AP600 design. These hand calculations applied maximum bearing pressures at the edge of the bay to a continuous one way beam representing the middle of the bay. The AP1000 design uses directly the member forces from the finite element analysis, which considers the two way action, and permits redistribution of bearing pressures due to flexibility of the slab.

The reinforcement provided for the AP1000 between column lines 1 and 2 and column lines K-2 and N is equal to or greater than that for the AP600 with the exception of the shear reinforcement. This reinforcement was decreased from #11 to #9 to improve constructibility. The required shear reinforcement decreased due to the change in assumption of one way action to the use of the member forces from the finite element analysis, which considers two way action. The use of two way action required an increase in the east west top reinforcement in this bay.

Table 2.6-1

Subgrade modulus for AP1000 Soil Cases

Soil case	Subgrade modulus
	kcf
Soft rock	3230
Upper bound soft to medium	2334
Soft to medium	963*
Soft	312

*Note: For water table up to grade this increases to 1280 kcf

Table 2.6-2

Nuclear Island Base Reactions

Units: 1000 kips & 1000 ft-kip

Seismic Reactions		Base Reactions		
		Equivalent Static Accelerations applied to NI in Basemat Design Analyses	**Fixed Base Time History Analysis (Hard Rock)**	**2D Time History Analysis (Soft to Medium)**
Shear NS	FX	124.48	100.68	106.73
Shear EW	FY	120.51	94.84	117.84
Vertical	FZ	110.38	98.78	116.22
Moment about NS	MXX	11,520	9,670	12,509
Moment about EW	MYY	11,357	10,323	10,528

Notes:
1. Moment summation point is at the center of the shield building at EL 60'-6" (X=1000, Y=1000, Z=60.5).
2. Equivalent static results for three directions are combined by SRSS
3 See Table 2.4-2 for 2D analysis results for other soils.

Table 2.6-3

Maximum soil pressure at corners from equivalent static non-linear analyses

Location	Maximum bearing pressure (ksf)	Load Case	S_{NS}	S_{EW}	S_{VT}
West side of shield building	52.8	3-13	-0.4	1.0	0.4
NW corner of auxiliary building	28.9	3-2	1.0	0.4	-0.4
NE corner of auxiliary building	29.7	3-11	0.4	-1.0	0.4
SE corner of auxiliary building	26.7	3-15	-0.4	-1.0	0.4
SW corner of auxiliary building	33.1	3-5	-1.0	0.4	0.4

Table 2.6-4

Longitudinal Reinforcement, Top face of DISH in Radial and Hoop Directions (Layers 6 to 10)

Zone			Required (in^2/ft)	Provided (in^2/ft)	Rebar Placement
Layer	**Radius range**	**Direction**			
4	0 – 30'	NS	2.615	3.12	#11@6"
5	0 – 30'	EW		3.12	#11@6"
10	23' – 31'	radial	2.332	5.18 – 3.84	Layer 6: #11@1.5°
8+10	31' – 46'	radial	3.110	5.77 – 3.89	Layer 8: #11@1.5°
6+8+10	46' – outside	radial	2.897	5.18 – 3.45	Layer 10: #11@0. 75°
7+9	31' - outside	hoop	4.666	6.24	Layer 7: #11@6" Layer 9: #11@6"

Note: See Figure 2.6-9 and 2.6-10

Table 2.6-5

Longitudinal Reinforcement, Top Face in NS direction (Layer 4)

Zone		Required (in^2/ft)	Provided (in^2/ft)	Rebar Placement
NS Wall Lines	**EW** Wall Lines			
General Area		Less than 2.25	2.25	#14@12"
Within Wall 1 to Wall 2	Within Wall J-2 to Wall N	2.719	3.25	#14@12" + #9@12" in one layer
East side of DISH, rectangular zone		2.418		
The four Pit Areas		0.911	1.56	#11@12"

Note: See Figure 2.6-11

Table 2.6-6

Longitudinal Reinforcement Top Face in EW direction (Layer 5)

Zone		Required (in^2/ft)	Provided (in^2/ft)	Rebar Placement
NS Wall Lines	**EW** Wall Lines			
General Area		Less than 2.25	2.25	#14@12"
Within Wall 4 to Wall 7.3	Within Wall I to Wall J	2.758	3.25	#14@12" + #9@12" in one layer
North side of DISH		2.684		
The four Pit Areas		0.911	1.56	#11@12"

Note: See Figure 2.6-12

Table 2.6-7

Longitudinal Reinforcement, Bottom Face in NS and EW direction (Layers 1, 2 and 3)

Zone	Direction	Required (in^2/ft)	Provided (in^2/ft)	Rebar Placement
All below SB	NS (layer 1)	4.41	4.5	#14@6"
	EW (layer 2)			
West-half of DISH, radius = 50' and more (See figure 2.6-13)	Circumference (layer 3)	4.67	1.56 Additional	#11@12" additional
All below AB	NS (layer 1)	Less than 2.25	2.25	#14@12"
	EW (layer 2)			
South side of DISH	NS (layer 1)	3.581	4.5	#14@6"
	EW (layer 2)	3.581		
North side of DISH	NS (layer 1)	3.119		
	EW (layer 2)	3.119		
East side of DISH, beneath Wall 7.3	EW	3.187	3.25	#14@12" + #9@12"
The four Pit Areas	NS (layer 1)	Less than 2.25	2.25	Same as the General Area
	EW (layer 2)			

Note: See Figure 2.6-13 and 14

Table 2.6-8

Shear Reinforcement

Zone: NS Wall Lines	Zone: EW Wall Lines	Required (in^2/ft^2)	Provided (in^2/ft^2)	Rebar Placement Intervals are shown as NS x EW direction
All other below AB than listed below		Less than 0.25	0.25	#9@24" x 24"
Wall 1 to Wall 2	Between Wall J-2 to Wall N	0.469	0.50	#9@12" x 24"
Between Wall 2 to Wall 4	Wall I to Wall J-1	0.163		
Between Wall 4 to Wall 7.3	Wall I to SB	0.382		
SB to Wall 10	Between Wall K to Wall P	0.328		
EL62'-0" pit to near Wall I, south side*		0.764	1.24	#5@6 x 6"
EL62'-0" pit to near Wall I, north side*		0.962		
EL63'-6" pit to South near Wall I		0.739		
EL62'-0" Pit to South side of DISH		0.181	0.62	#5@6 x 12"

Note: See Figure 2.6-15

Table 2.6-9

Comparison of the AP1000 to AP600 in the Two Critical Bays

Applicable Column Lines	Elevation Level Range	Concrete Thickness	Reinforcement Type		Rebar Arrangement	Reinforcement Provided
Column line K to L and from Col. Line 11 wall to the intersection with the shield building	From EL 60' 6" to 66' 6"	6'-0"	Top Reinforcement	AP1000	NS: #14@12" EW: #14@12"	NS: 2.25in^2/ft EW: 2.25in^2/ft
				AP600	*NS: #11@12" EW: #14@10"*	*NS: 1.56in^2/ft EW: 2.7in^2/ft*
			Bottom Reinforcement	AP1000	NS: #14@12" EW: #14@12"	NS: 2.25in^2/ft EW: 2.25in^2/ft
				AP600	*NS: #14@12" EW: #14@10"*	*NS: 2.25in^2/ft EW: 2.7in^2/ft*
			Shear Reinforcement	AP1000	#9@24"(NS) x @24"(EW)	0.25in^2/ft^2
				AP600	*#9 @20"(NS) x @24"(EW)*	*0.3in^2/ft^2*
Column line 1 to 2 and from Column Line K-2 to N wall	From EL 60' 6" to 66' 6"	6'-0"	Top Reinforcement	AP1000	NS: #14@12", and Locally #14@12"+#9@12" EW: #14@12"	NS: 2.25in^2/ft Locally 3.25^2/ft EW: 2.25in^2/ft
				AP600	*NS: #11@12" (#11@6" for 17'-0" from Wall_1) EW: #11@12"*	*NS: 1.56in^2/ft (3.12in^2/ft for 17'-0" from Wall_1) EW: 1.56in^2/ft*
			Bottom Reinforcement	AP1000	NS: #14@12" EW: #14@12"	NS: 2.25in^2/ft EW: 2.25in^2/ft
				AP600	*NS: #14@12" (#14@6" for 7'-6" from Wall_2) EW: #14@12"*	*NS: 2.25in^2/ft (4.5in^2/ft for 7'-6" from Wall_2) EW: 2.25in^2/ft*
			Shear Reinforcement	AP1000	#9@12"(NS) x @24"(EW)	0.50in^2/ft^2
				AP600	*#11 @12"(NS) x @24"(EW)*	*0.78in^2/ft^2*

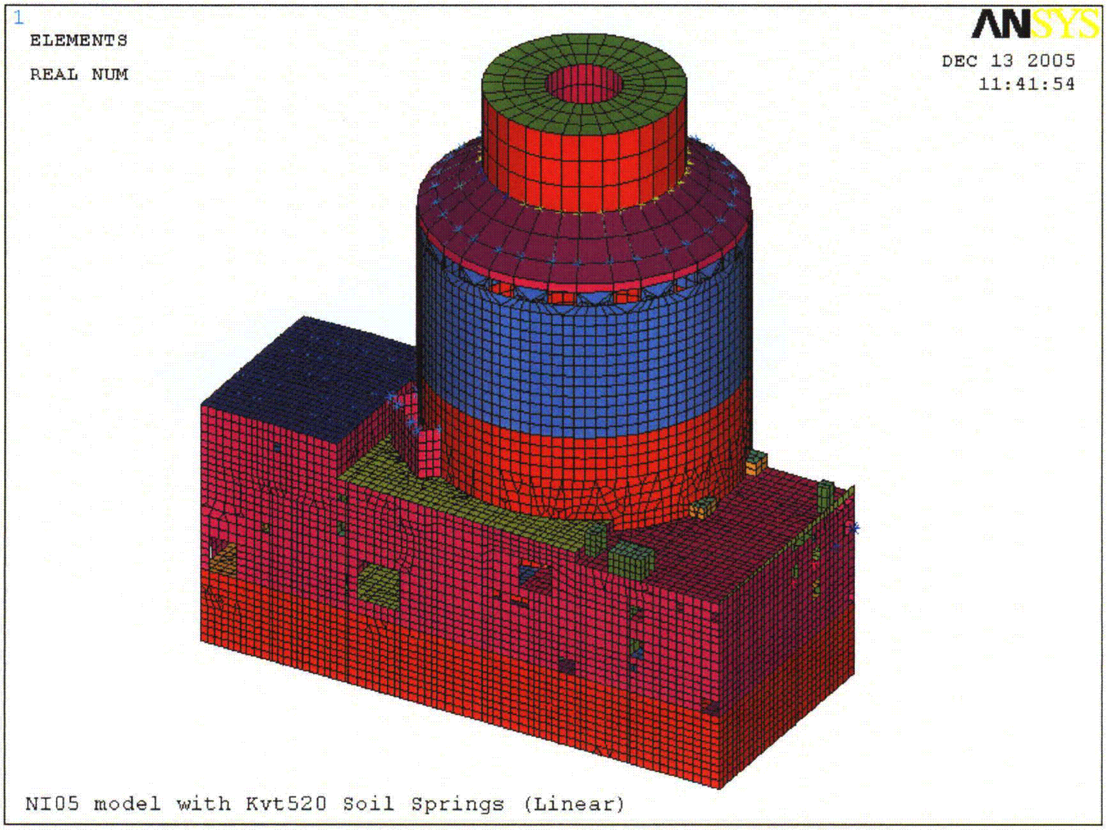


Figure 2.6-1 NI05 Model with Soil Springs

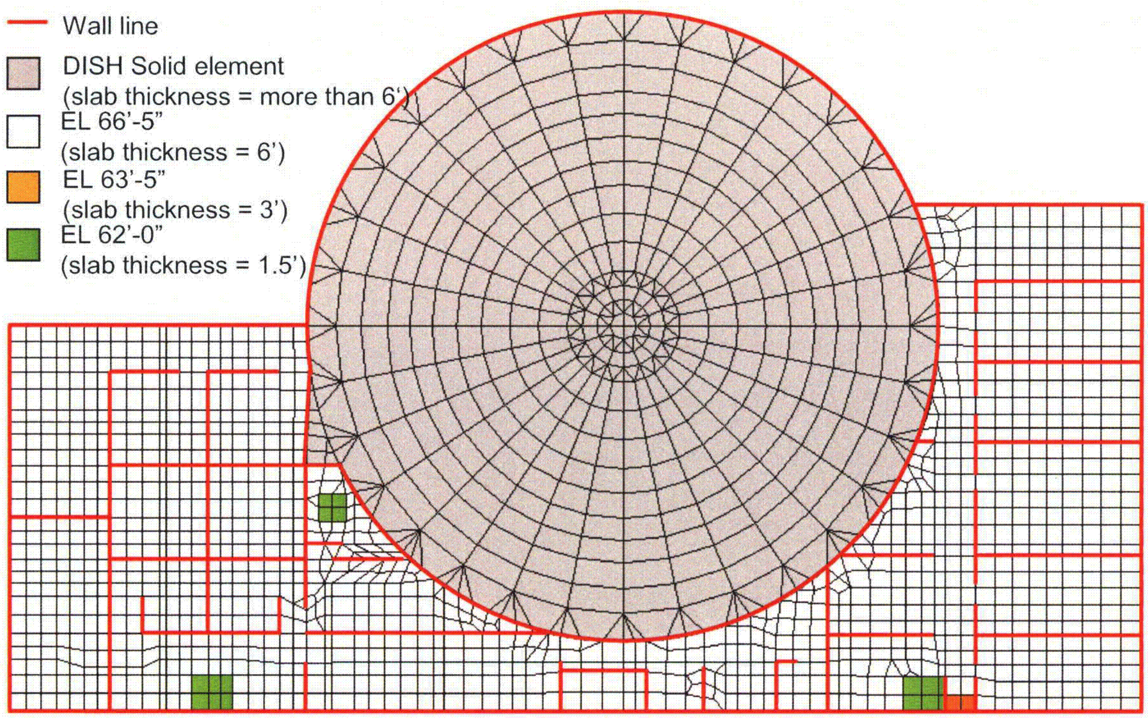
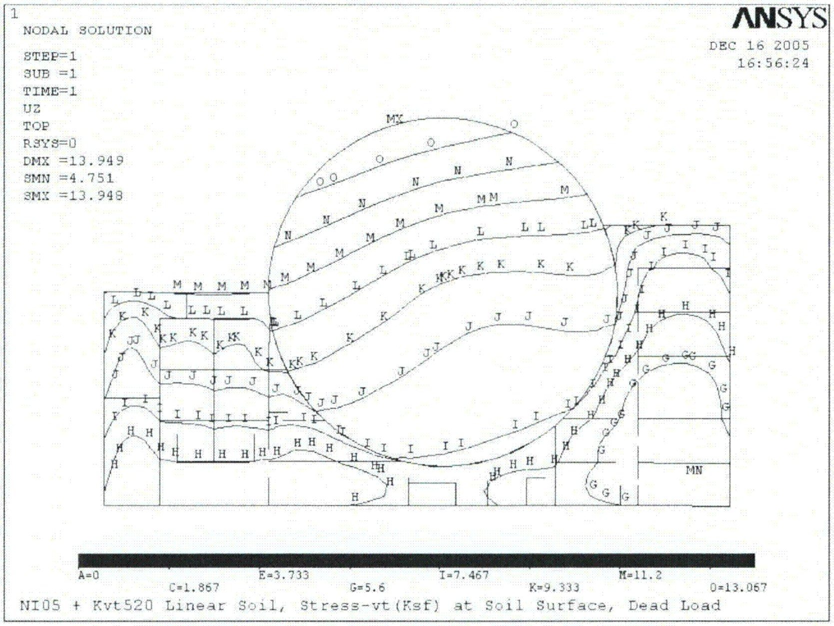
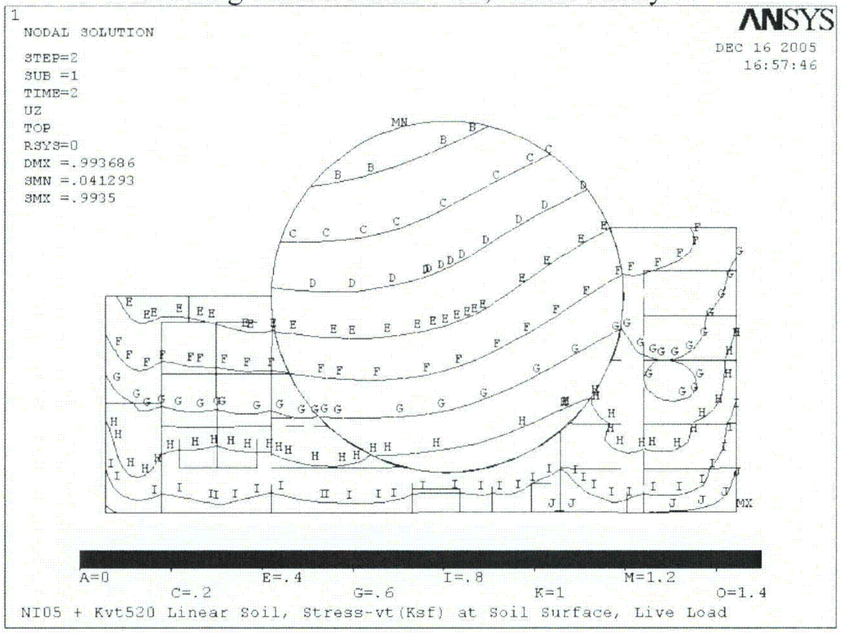


Figure 2.6-2 Basemat Elements along with Wall Lines above the Basemat



Bearing Pressure under DL, Linear Analysis



Bearing Pressure under LL, Linear Analysis

Figure 2.6-3 Soil Bearing Pressure for Normal Operating Loads

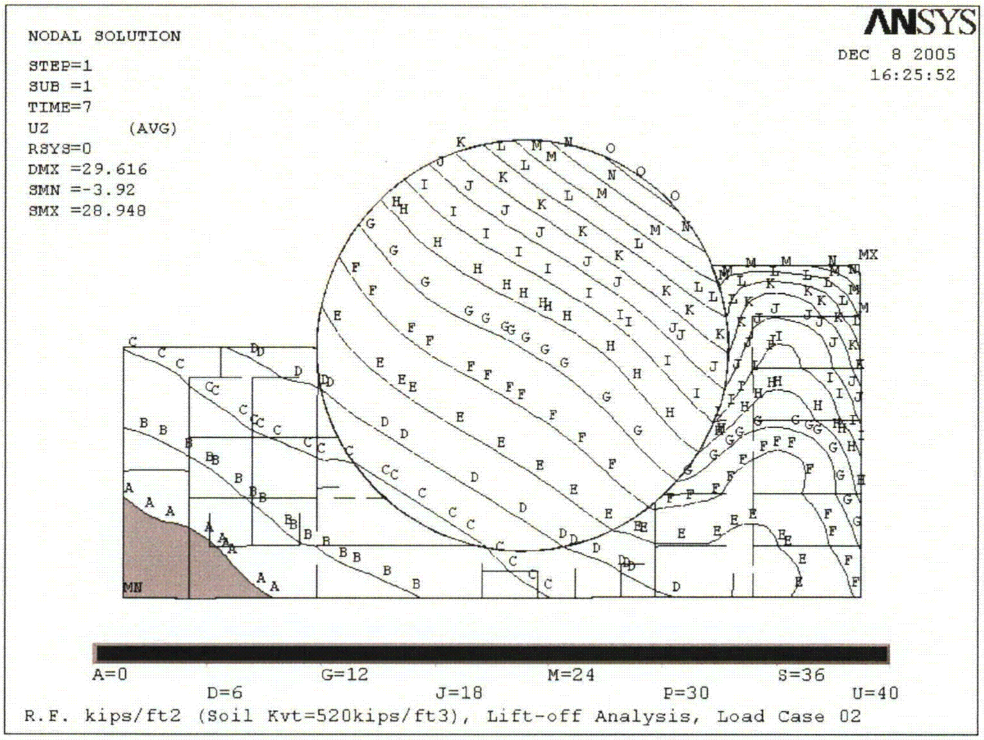


Figure 2.6-4 Soil Bearing Pressure in Load Case 3-2 (Es= 1.0xSns+0.4xSew-0.4xSvt)

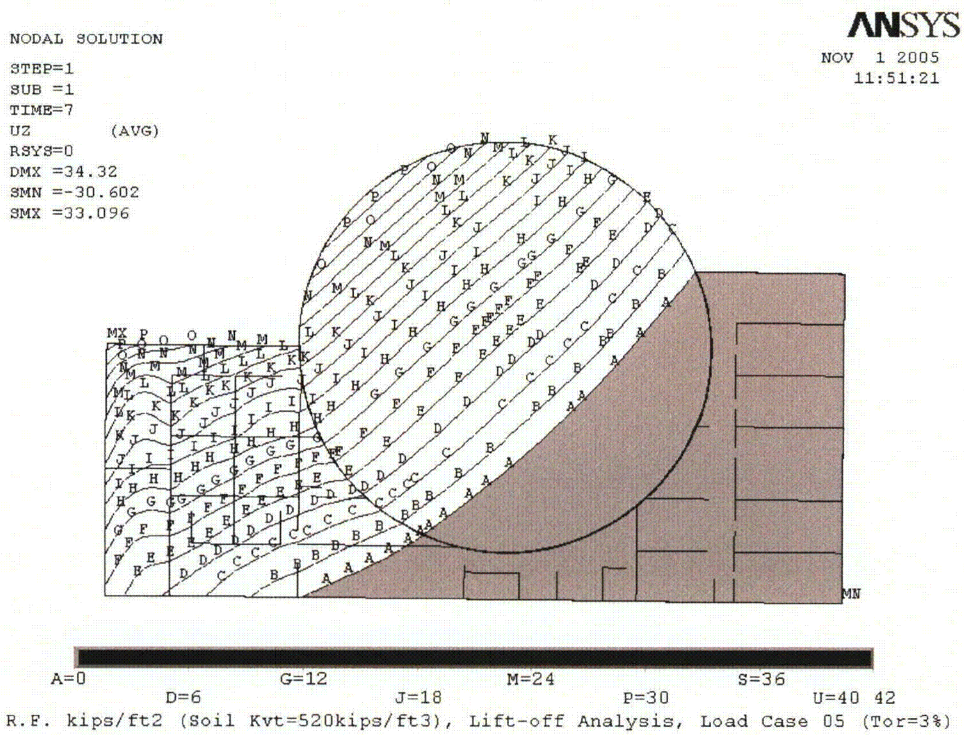


Figure 2.6-5 Soil Bearing Pressure in Load Case 3-5 (Es= -1.0xSns+0.4xSew+0.4xSvt)

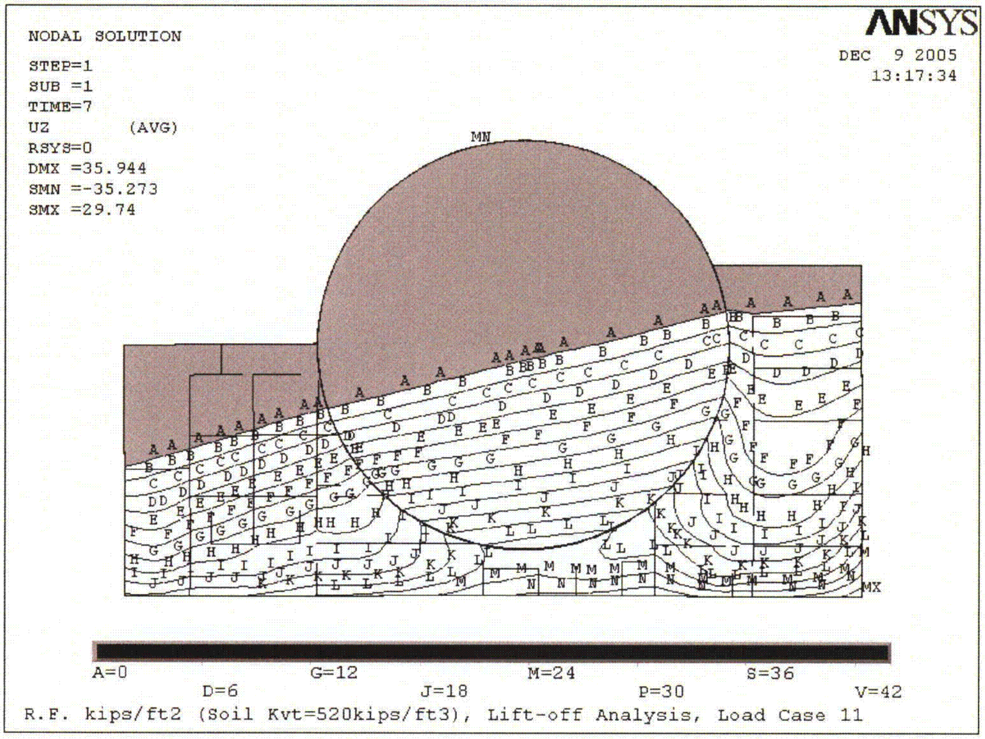


Figure 2.6-6 Soil Bearing Pressure in Load Case 3-11 (0.4xSns-1.0xSew+0.4xSvt)

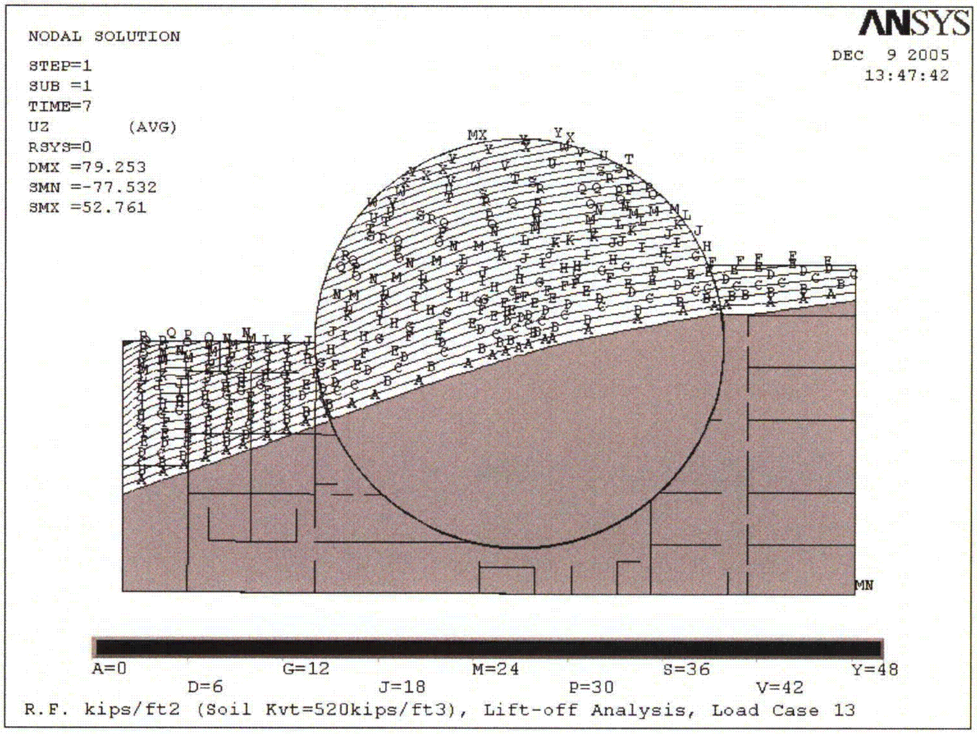


Figure 2.6-7 Soil Bearing Pressure in Load Case 3-13 (-0.4xSns+1.0xSew+0.4xSvt)

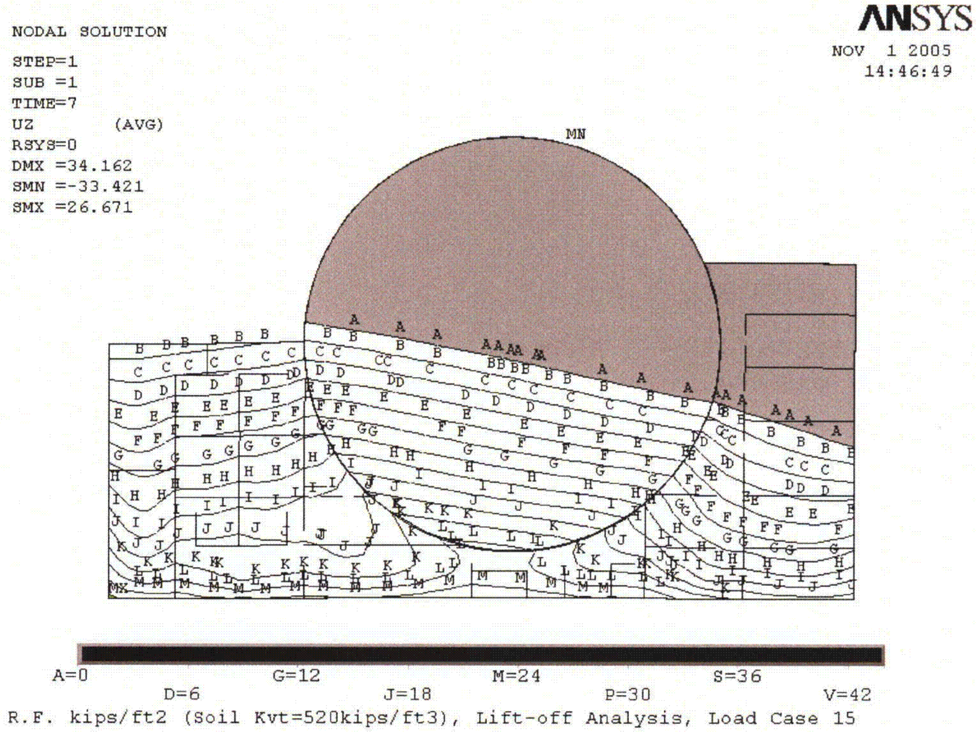


Figure 2.6-8 Soil Bearing Pressure in Load Case 3-15 (-0.4xSns-1.0xSew+0.4xSvt)

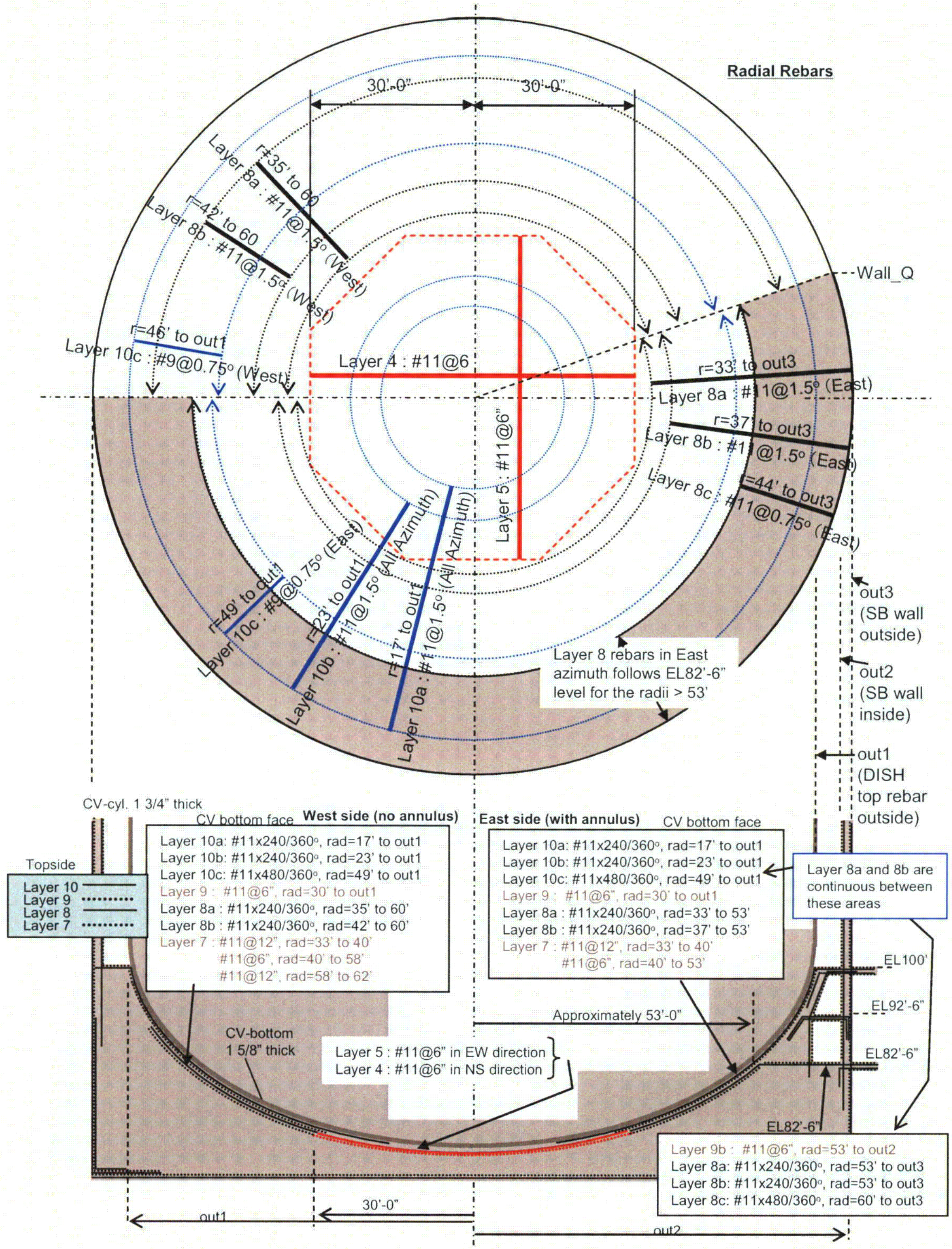


Figure 2.6-9 Radial Reinforcement, Top side of DISH

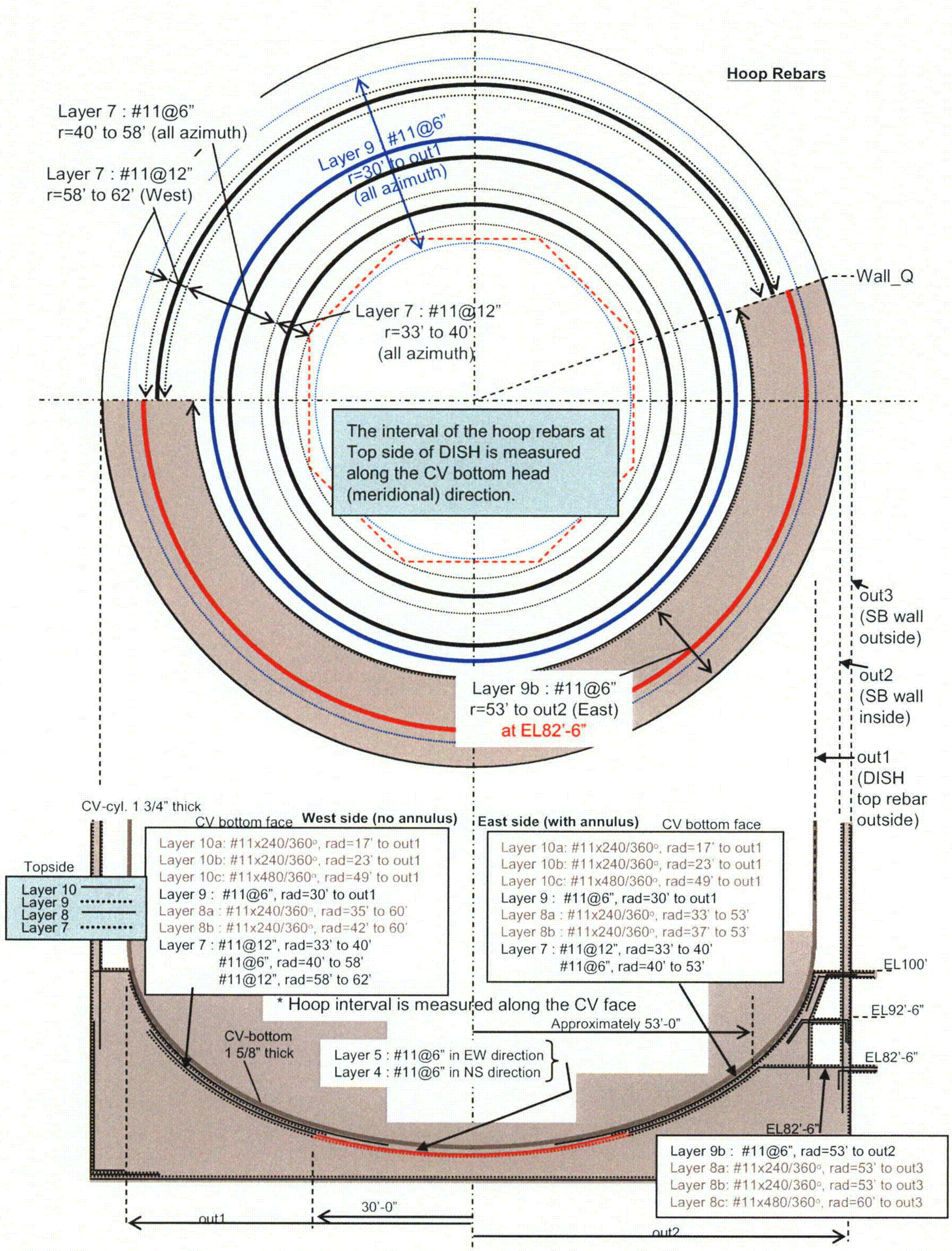
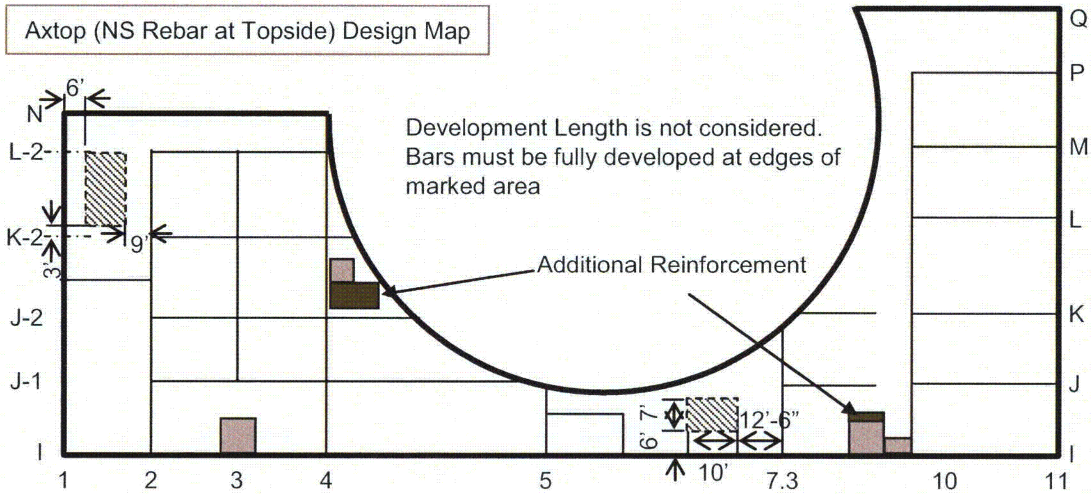
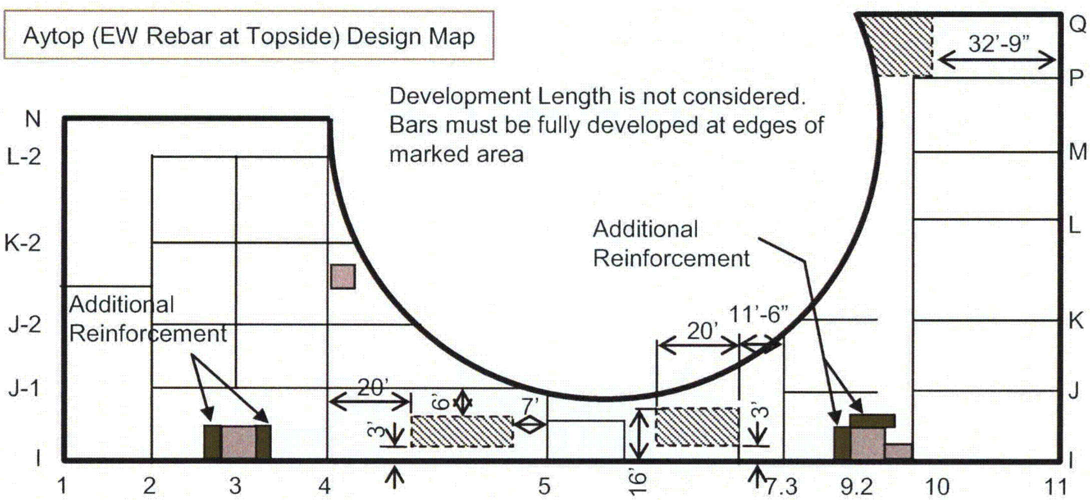


Figure 2.6-10 Circumferential Reinforcement, Top side of DISH



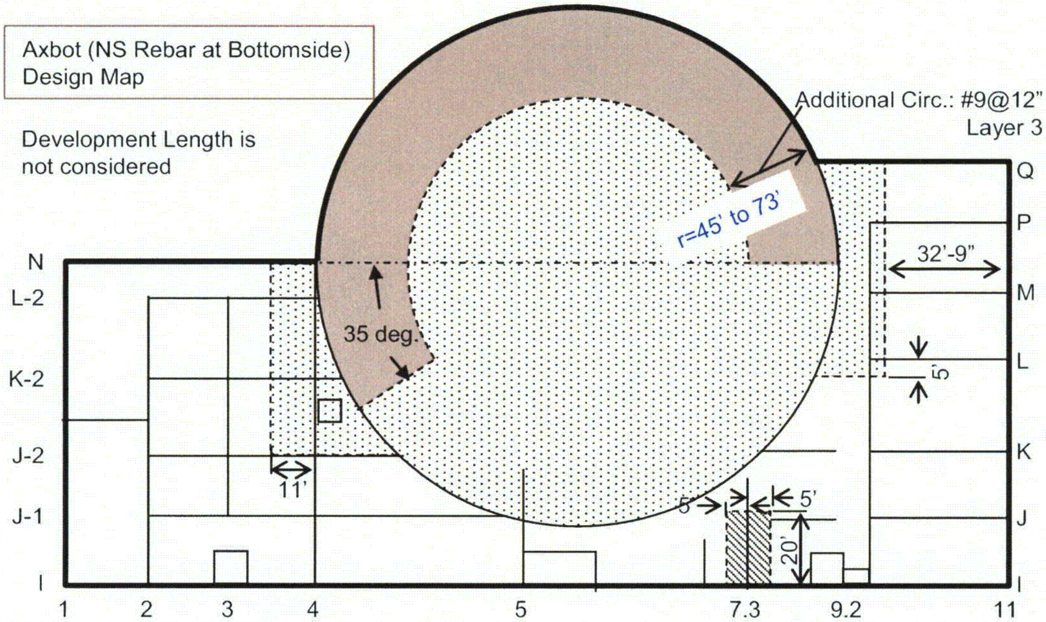
Rebar arrangement	: #14@12"	: #14@12"+#9@12"	: #11@12"
Location	Layer 4		Top side of the 1.5' & 3' thick Pits

Figure 2.6-11 Longitudinal Reinforcement Map, Top side in NS direction



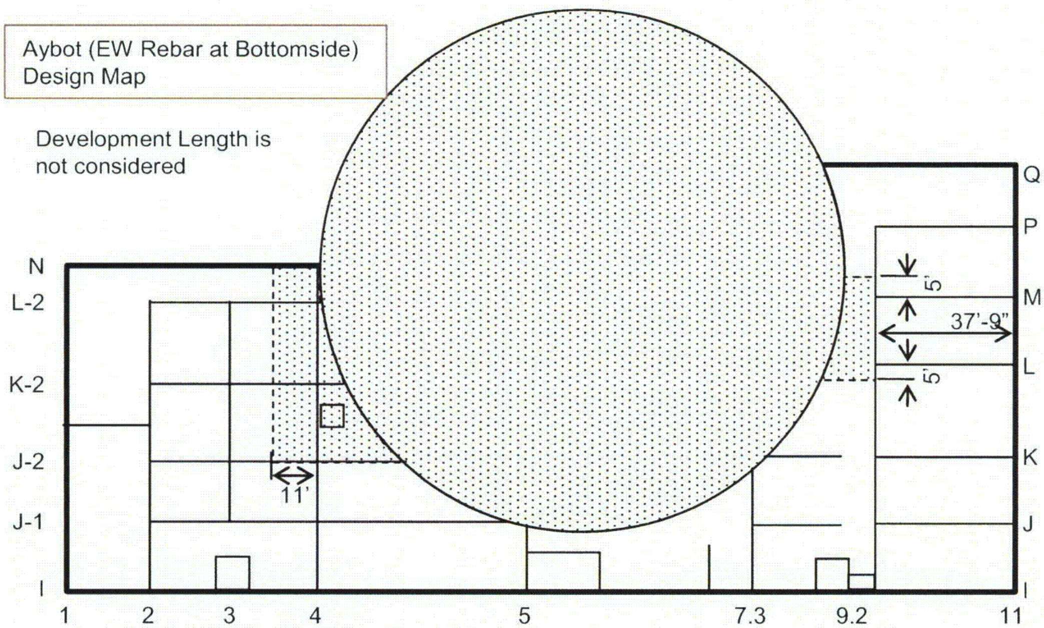
Rebar arrangement	: #14@12"	: #14@12"+#9@12"	: #11@12"
Location	Layer 5		Top side of the 1.5' & 3' thick Pits

Figure 2.6-12 Longitudinal Reinforcement Map, Top side in EW direction



Rebar arrangement	: #14@12"	: #14@12"+#9@12"	: #14@6"	: #14@6"	+#9@12"
Location	Layer 1				Layer 3

Figure 2.6-13 Longitudinal Reinforcement, Bottom side of DISH and 6' basemat (NS)



Rebar arrangement	: #14@12"	: #14@12"+#9@12"	: #14@6"
Location	Layer 2		

Figure 2.6-14 Longitudinal Reinforcement, Bottom side of DISH and 6' basemat (EW)

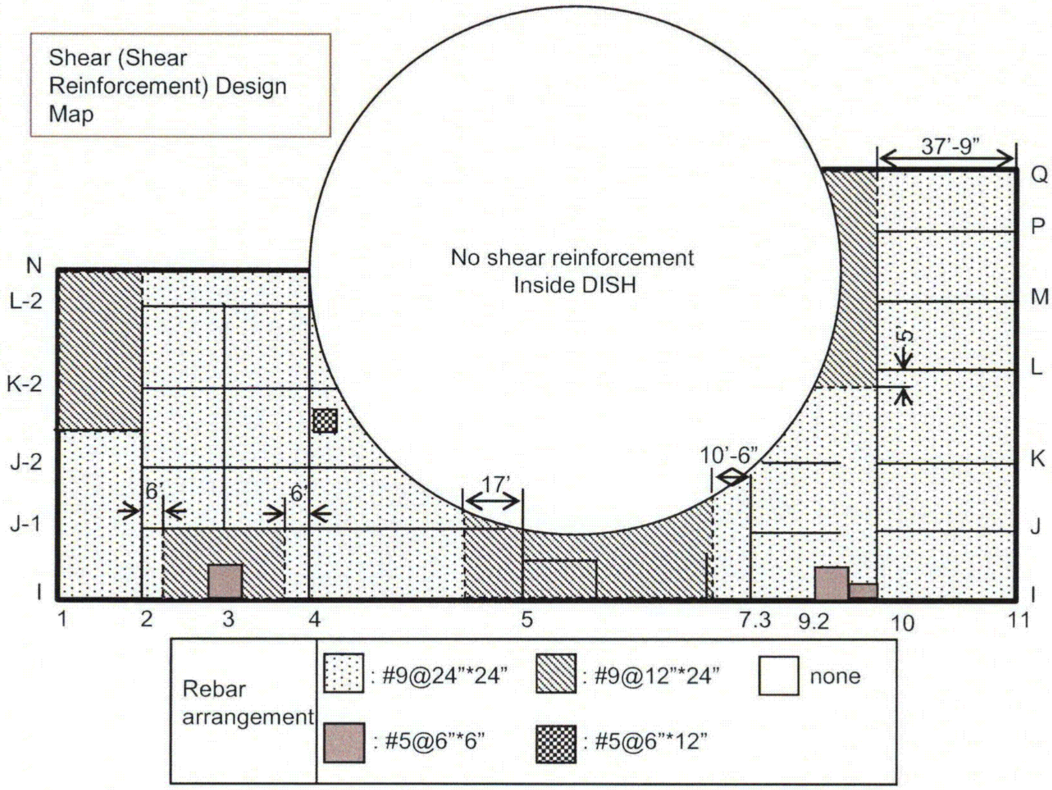


Figure 2.6-15 Shear Reinforcement Map

2.7 Basemat design studies

2.7.1 Soil modeling

2.7.1.1 Effect of Lower Stiffness Soil Springs

A study was performed to investigate the effect of a reduced subgrade modulus on the reinforcement required in the 6' basemat of the AP1000 Nuclear Island on soil sites. The study used the nuclear island finite element model on soil springs with subgrade moduli of 520 and 260 kcf. Lift-off analyses similar to the design analyses were performed on selected critical cases. Reinforcement required for the basemat was calculated for each case. This study concluded that the reinforcement design for the basic design case using a subgrade modulus of 520 kcf with the margin of 20% used in design would envelope the results with the subgrade modulus of 260 kcf.

A subgrade modulus of 260 kcf corresponds to the soft site in the SASSI analyses where the overturning moment is only 53% of the soft to medium soil overturning. This case does not need to be considered further.

2.7.1.2 Comparison of soil finite element ANSYS models versus subgrade springs

A study was performed to investigate the effect of soil modeling. Finite element models of soil were combined with a fine mesh Nuclear Island model (NI10) with a simple CIS portion as shown in Figure 2.7-1. Most cases were performed for dead load only. One case was analyzed for horizontal loads. The cases are summarized in Table 2.7-1. Deflections, soil stresses and member forces in the basemat were compared against those obtained using Winkler springs.

2.7.1.2.1 Effect of soil depth under vertical loads

The first parametric study with these models was performed for dead loads to investigate soil bearing characteristics and basemat member forces for a soil site represented by the subgrade modulus of 520kcf. As the subgrade modulus only defines a vertical stiffness (Kvt) at the soil surface, soil models can take several patterns (soil layer depth with appropriate soil stiffness) of soil structures. In these models, embedment of NI building is not considered.

In Table 2.7-1 Model-W has Winkler springs with subgrade modulus of 520 kcf. Soil model H has soil to a depth of 816' below the foundation of the nuclear island. Soil models named L"nnn" have a surface soil depth of "nnn" feet. The elastic modulus of the soil elements, as shown in the table, is adjusted to have the same vertical stiffness as the Winkler spring of 520 kcf.

Figure 2.7-2 shows a typical comparison of the bearing pressure under dead load. Section (a) is along an east west section through the south end of the auxiliary building. Section (c) is along an east west section through the north end of the auxiliary building.

The solid soil models show higher bearing pressures at the edges (Boussinesq distribution) than the Winkler springs. The ratio of the higher bearing pressures at the edge is influenced by the soil modeling. These higher bearing pressures at the edges reduce the bearing pressures away from the edges.

The figures show the effect of the relative stiffness of the soil versus the 6 foot thick basemat and superstructure. There is significant variation in bearing stress between a location below the walls of the auxiliary building and a location at mid span between the walls. This difference is largest for Model-H with the largest soil modulus. The soil acts to stiffen bending of the mat. The soil properties analyzed range from soft rock (Model-H) to soft soil (Model-L020).

Member forces of the 6' basemat in solid soil models are generally smaller for out-of-plane forces when compared with the Winkler spring model (see also discussion in Section 2.7.2). The solid soil cases with the thinner layers of soil below the basemat respond closer to the Winkler springs.

2.7.1.2.2 Effect of side soils under vertical loads

A second parametric study investigated the effects of embedment modeling. The soil model for this case used the soil properties for the "soft to medium" soil case (Vs = 939 fps at surface to 1675 at 120 feet, Vp = 5000 fps full height assuming water to grade). This has an equivalent subgrade modulus higher than the cases described in subsection 2.7.1.2.1. In these models, the effect of the embedment of the NI building up to the grade level is considered. Three cases of side soil connectivity were considered.

Bearing pressures are shown in Figure 2.7-3 for the same east west sections at the south and north end of the auxiliary building. The distribution is similar to that of Figure 2.7-2. The side soil effect in vertical loading tends to reduce the higher bearing forces at the edge observed in the results in Figure 2.7-2. The weight of the side soil reduces the difference in vertical stress between the area beneath the basemat and the adjacent areas.

2.7.1.2.3 Horizontal loads

A third parametric study investigated characteristics of horizontal loading. One representative finite element soil model in the first study and the conventional Winkler spring model were compared. Vertical bearing reactions at the edge under horizontal loading are similar to those for loading in the vertical direction.

2.7.1.2.4 Conclusion of study

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).

2.7.2 VECTOR analyses

A study was performed to assess the behavior of the basemat and its interaction with the soil. The two critical bays of the basemat in the north and south west corners were modeled as single or multi-span deep beams using the University of Toronto VECTOR2 F/E software. This software is primarily a development tool based on the state-of-the-art of reinforced concrete research. The theoretical bases of VecTor2 are the Modified Compression Field Theory (MCFT) (Vecchio and Collins, 1986) and the Disturbed Stress Field Model (Vecchio, 2000) – analytical models for predicting the response of reinforced concrete elements subject to in-plane normal and shear stresses. VecTor2 models cracked concrete as an orthotropic material with smeared, rotating cracks. Originally, VecTor2 employed the constitutive relationships of the MCFT. Subsequent developments have incorporated alternative constitutive models for a variety of second-order effects including compression softening, tension stiffening, tension softening, and tension splitting. Also, the capabilities of the VecTor2 have been augmented to model concrete expansion and confinement, cyclic loading and hysteretic response, construction and loading chronology for repair applications, bond slip, crack shear slip deformations, reinforcement dowel action, reinforcement buckling, and crack allocation processes.

Over a period of more than twenty years, VECTOR2 constitutive relations for reinforced concrete were corroborated, refined and validated by extensive test programs at the University of Toronto as well as at several other research establishments, involving hundreds of test specimens. The conditions investigated have encompassed a wide range of specimen construction details and loading conditions. In all cases, the MCFT was able to accurately predict behavior in terms of crack patterns, deformations, reinforcement stresses, ultimate strengths and failure modes. Detailed comparisons of experimental versus theoretical response, for each of the test series, are found in literature.

The multi span model for the bay below the north auxiliary building is shown in Figure 2.7-4. Longitudinal reinforcement is #14@12" with 2" cover, top and bottom, and transverse stirrups are #9@24"x24". The reinforcement is modelled as bilinear with an elastic modulus of 30000 ksi up to yield and 1428 ksi beyond yield. Three commonly used analytical models were considered for the soil contact stresses:

a) Uniformly Distributed Load (UDL) as shown in Figure 2.7-5.
b) Winkler springs as shown in Figure 2.7-6 with vertical stiffness of 520 kcf.
c) Half-space granular soil layer coupled with the basemat contact nodes as shown in Figure 2.7-7. Soil properties match the 520 kcf vertical stiffness used in Case (b).

The VECTOR2 program considers cracking of the concrete and non-linear behavior of the reinforcement. Structural response is calculated up to failure for a monotonically applied uniformly distributed load in case (a) and for monotonically applied vertical displacement of the shear walls for cases (b) and (c). The results of the analyses are summarized in Table 2.7-2.

The peak vertical and horizontal deformations of the basemat relative to its supports for the three soil-structure interaction models are plotted in Figure 2.7-8 as a function of the average applied load. All models are linearly elastic up to about 0.53 Mpa (11 Ksf) and have fairly equal stiffness. This initial range represents the behavior of uncracked concrete. Beyond this loading, however, the UDL and springs models exhibit significant stiffness degradation, with clearly distinguishable cracking, yield and strain hardening zones. The half-space model shows far less stiffness degradation since the longitudinal rebar remains within the elastic range due to horizontal restraint from the soil.

Contact stresses are shown along the length of the mat for the spring and half space model in Figures 2.7-9 and 2.7-10 respectively. The contact stresses are shown at various loading steps. As the loading increases and the reinforced concrete mat stiffness reduces, contact stresses redistribute from the mid span of each bay towards the supports. They also redistribute from the long span which is most flexible to the shorter adjacent spans. The contact stresses in the soil are fairly high as the concrete slab approaches failure and could result in soil failure prior to the reinforced concrete failure. The validity of the elastic soil model for sand sites was confirmed in a separate analysis in ANSYS in which the soil was modeled with Drucker-Prager non-linear properties with an internal friction coefficient of 35 degrees. This analysis with Drucker-Prager non-linear properties showed the same failure in the reinforced concrete with no failure in the cohesionless soil.

The study leads to the following conclusions:

- In terms of average contact pressures, the UDL model grossly underestimates the failure loads, being only about 56% of Winkler springs and 35% of half-space
- The higher average load capacity of the Winkler springs model is caused by load re-distribution
- The UDL and Winkler springs models exhibit significant horizontal deformations caused by concrete dilation in the post-cracking regime
- Though not exceeding the frictional capacity, significant contact shear forces develop between the basemat and the soil in the half-space model as a result of resistance to the concrete dilation
- Similar to the Boussinesq stress distribution, contact stresses below the loaded walls within 6' of the edge of the basemat in the half space model are sharply higher than elsewhere
- Contact shear stresses, due to their post-tensioning effect on the underside of the basemat, cause a reduction of the rebar stresses, partial closing of the shear cracks and a significant increase in the basemat stiffness and its failure load

- Consideration of soil structure interaction demonstrates the capacity margin built into the AP1000 reinforced concrete basemat which is designed to ACI 349 using conventional design analysis methods with Winkler springs.

Table 2.7-1
Soil models for dead load analyses

Model name	Soil Model	Soil Property	VS (fps)	Depth of Soil below found.	Embed -ment	Boundary to Side Soil
Model-W	Spring	Kvt=520kcf				
Model-H	Solid	E=59,000ksf, ν=0.35	2529	816'	None	N/A
Model-L240	Solid	E=44,500ksf, ν=0.35	2196	240'		
Model-L120	Solid	E=31,000ksf, ν=0.35	1833	120'		
Model-L080	Solid	E=23,000ksf, ν=0.35	1579	80'		
Model-L040	Solid	E=12,000ksf, ν=0.35	1141	48'		
Model-L020	Solid	E=6,100ksf, ν=0.35	813	24'		
Model-E080a	Solid	Soft to medium soil profile			40'	Free to side soil
Model-E080b	Solid	Soil Depth=120' below grade				Half-fix to side soil
Model-E080c	Solid					No side soil layers
	Solid					

Note: The weight of the soil was not considered in the analyses using models W, H and L.
The weight of the soil was considered in the E models.

Table 2.7-2: Summary of results for VECTOR2 North West Analyses

Case	Elastic limit	Initial strain hardening(*)						
	Average Contact Pressure	Average Contact Pressure	Max. Contact Pressure	Min. Contact Pressure	Support Displ.	Max. Vert. Def.	Max. Horiz. Def.	Max. Crack Size
NWUDL	0.53 Mpa (11 Ksf)	1.15Mpa (24 Ksf)	N/A	N/A	N/A	7.1 mm (0.28in)	3.5 mm (0.14in)	1.8 mm (0.07in)
NWSPR	0.53 Mpa (11 Ksf)	1.44Mpa (30 Ksf)	1.72Mpa (36 Ksf)	1.0 Mpa (21 Ksf)	19.0mm (0.75in)	6.9 mm (0.27in)	5.5 mm (0.22in)	2.2 mm (0.09in)
NWHALF SP	0.72 Mpa (15 Ksf)	Rebar does not reach strain hardening						

		90 % of ultimate(**)						
Case		Average Contact Pressure	Max. Contact Pressure	Min. Contact Pressure	Support Displ.	Max. Vertical Def.	Max. Horiz. Def.	Max. Crack size
NWUDL		1.34Mpa (28 Ksf)	N/A	N/A	N/A	23.0mm (0.9 in)	13.0mm (0.51in)	6.1 mm (0.24in)
NWSPR		2.4 Mpa (50 Ksf)	3.35Mpa (70 Ksf)	1.0 Mpa (21 Ksf)	35.6mm (1.4 in)	23.0mm (0.9 in)	32.0mm (1.26in)	7.2 mm (0.28in)
NWHALF SP		3.83Mpa (80 Ksf)	10.0Mpa (210Ksf)	1.44Mpa (30 Ksf)	53.3mm (2.1 in)	6.0 mm (0.24in)	2.5 mm (0.1 in)	1.7 mm (0.07in)

Note(*): For NWUDL and NWSPR only. Rebar does not reach strain hardening for NWHALFSP

Note(**): For NWUDL and NWSPR, this represents 90% of ultimate rebar stress. For NWHALFSP it represents 90 % of shear failure.

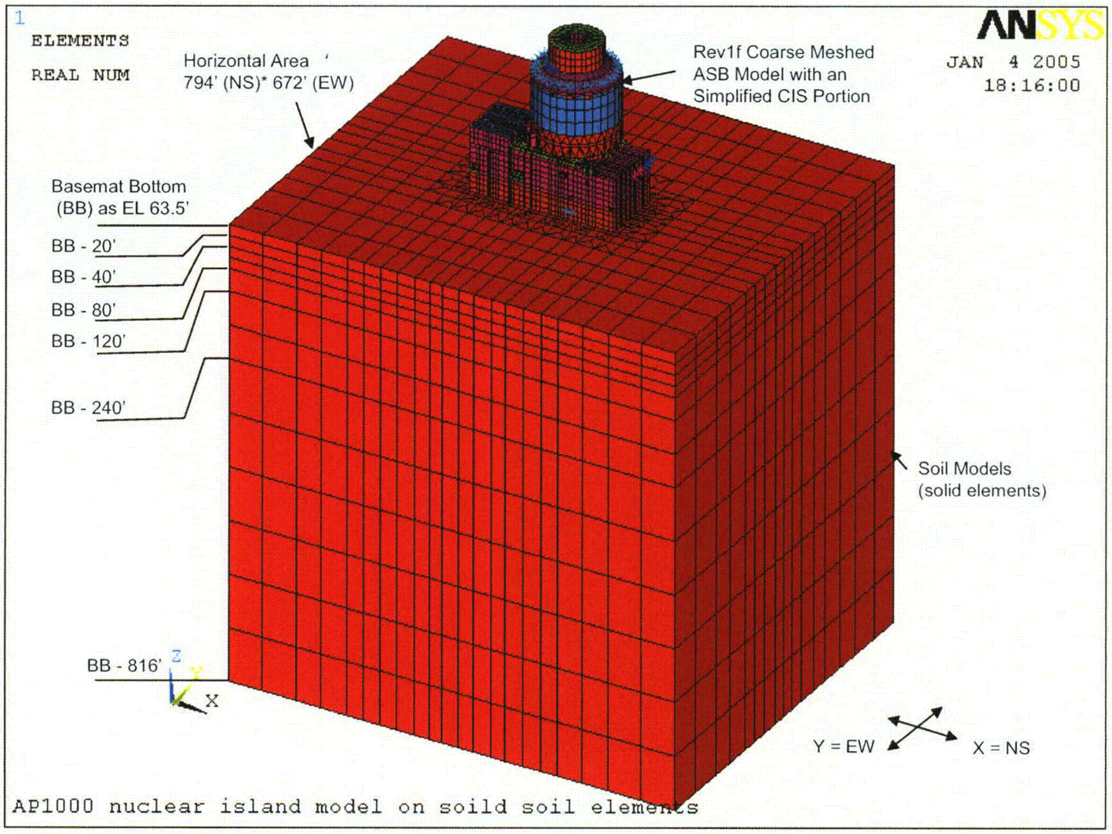
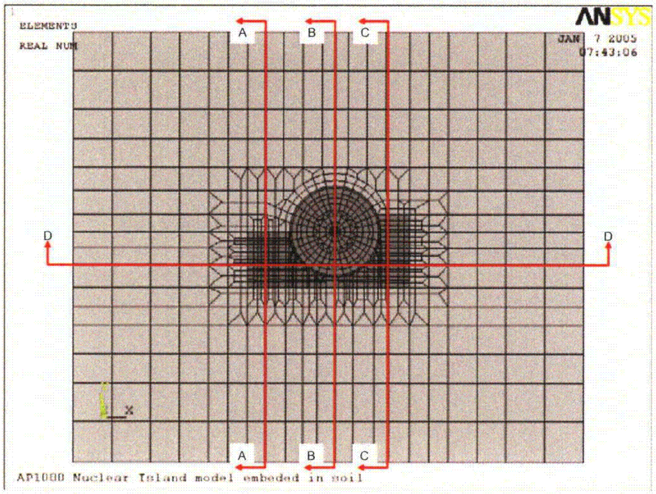
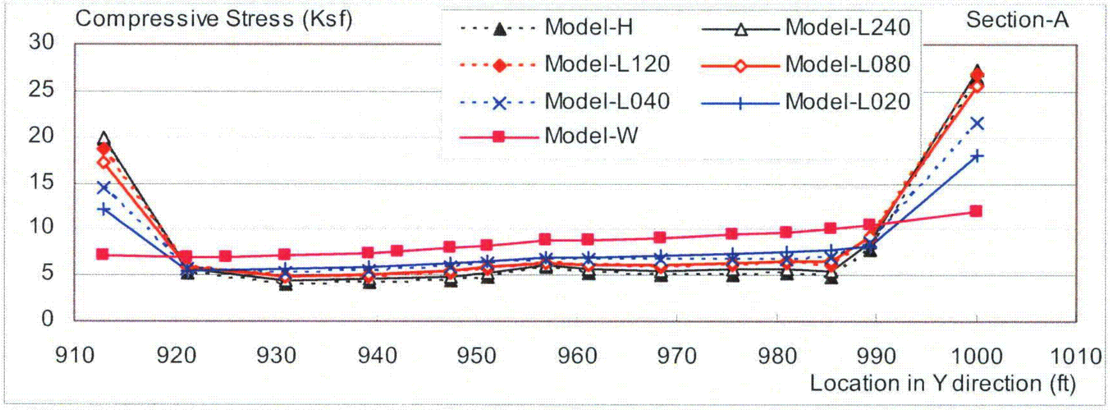


Figure 2.7-1 Analysis Model with Finite Element Models of Soil (No Embedment Case)





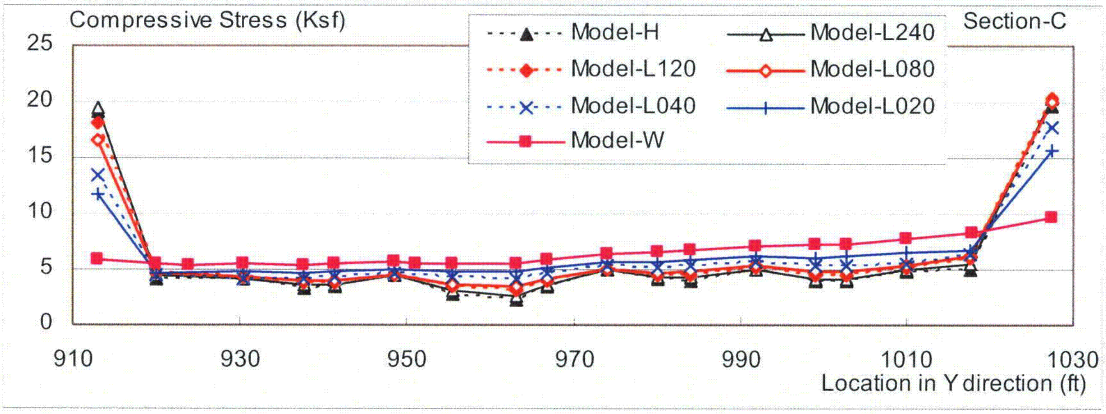
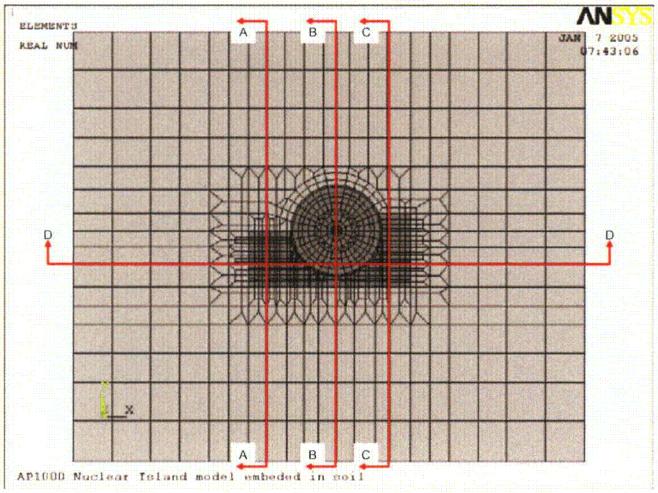
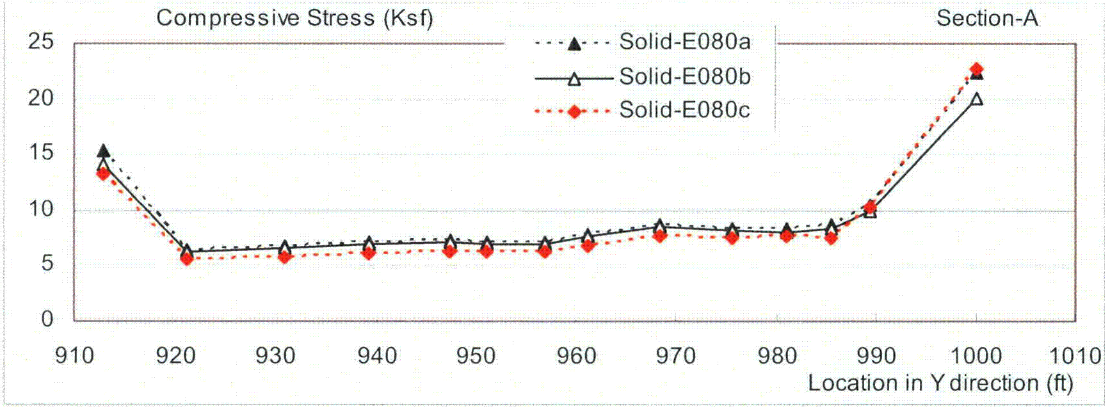


Figure 2.7-2 Comparison of Vertical Stress at Basemat Bottom Node – No embedment





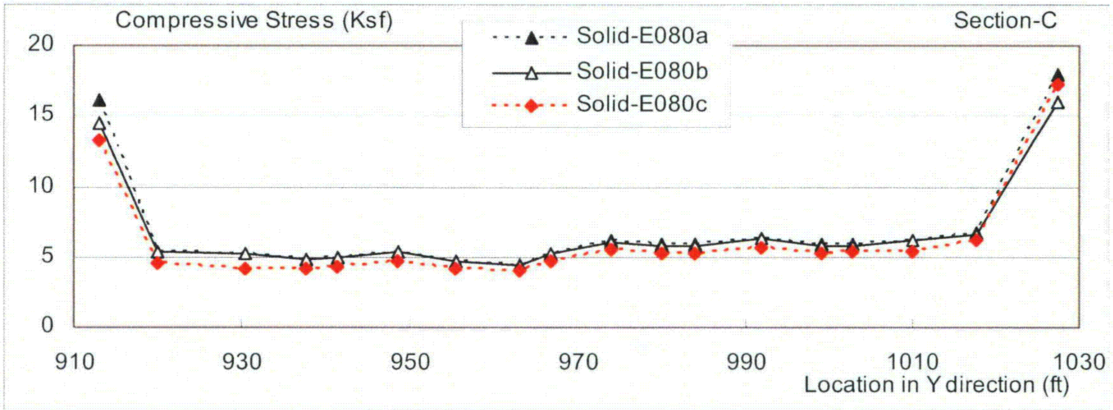


Figure 2.7-3 Comparison of Vertical Stress at Basemat Bottom Node

(Soft to medium soil including embedment)

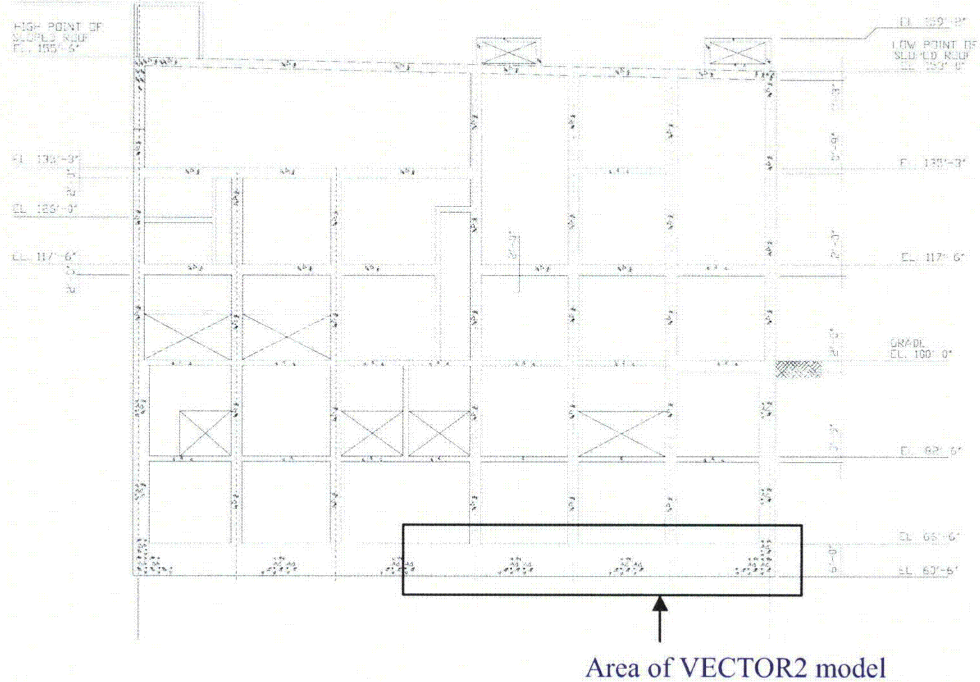


Figure 2.7-4 Cross section through north end of auxiliary building looking south

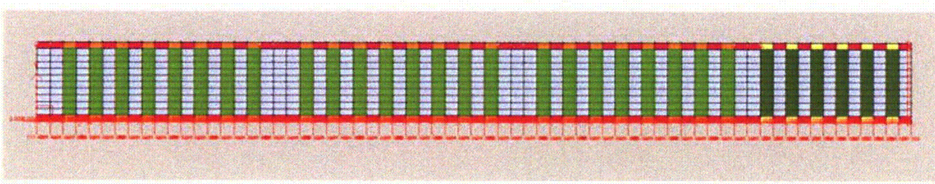


Figure 2.7-5 Vector2 model looking north with Uniform Distributed Load (UDL)

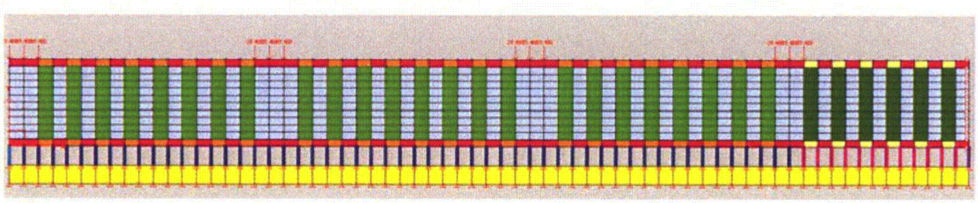


Figure 2.7-6 Vector2 model looking north with Soil Springs

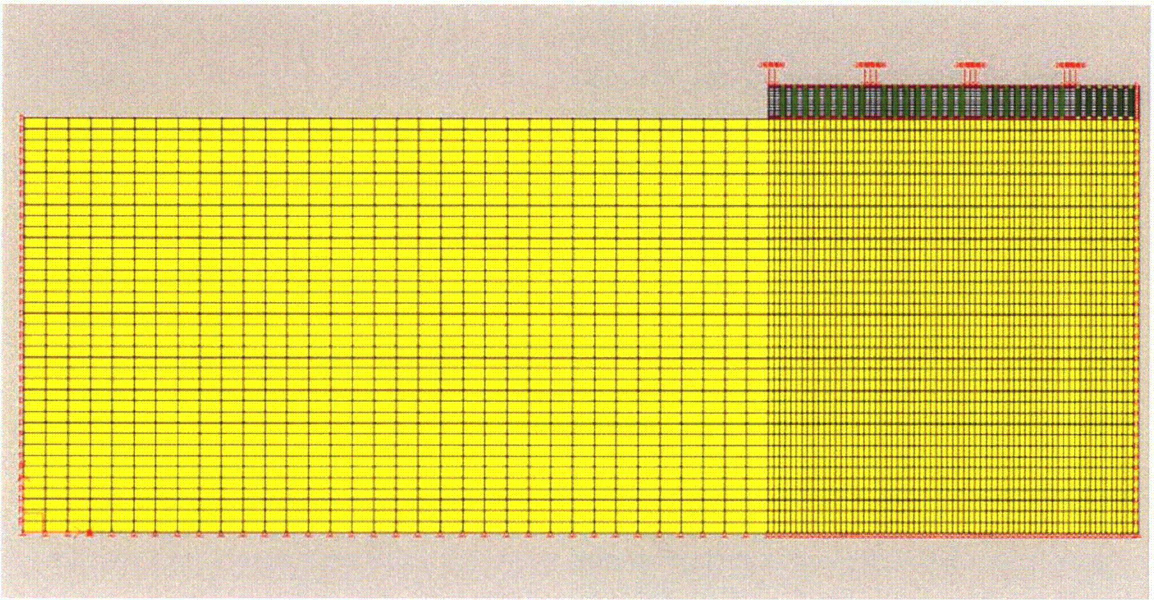
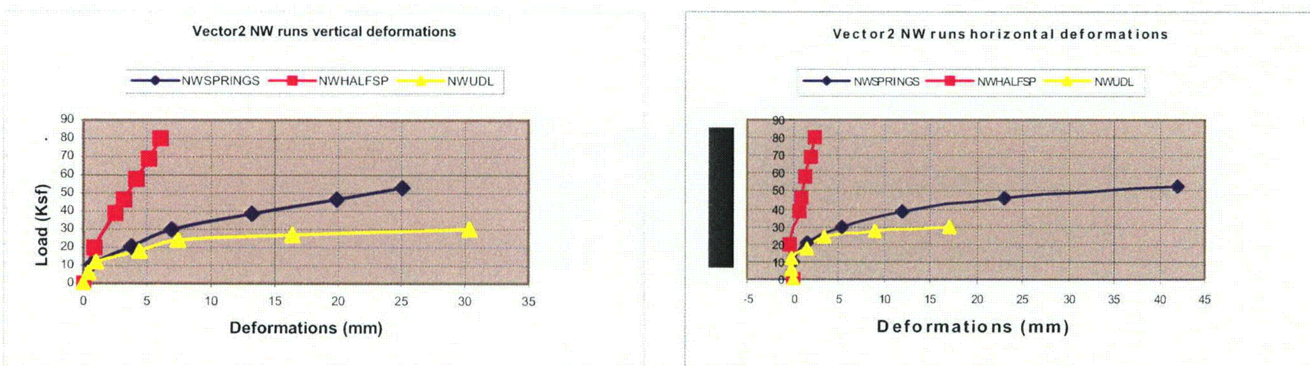


Figure 2.7-7 Vector2 model looking north with Soil Elements



(a) Vertical deformations (b) Horizontal (dilation) deformations

Figure 2.7-8 Maximum basemat deformations versus average contact pressure

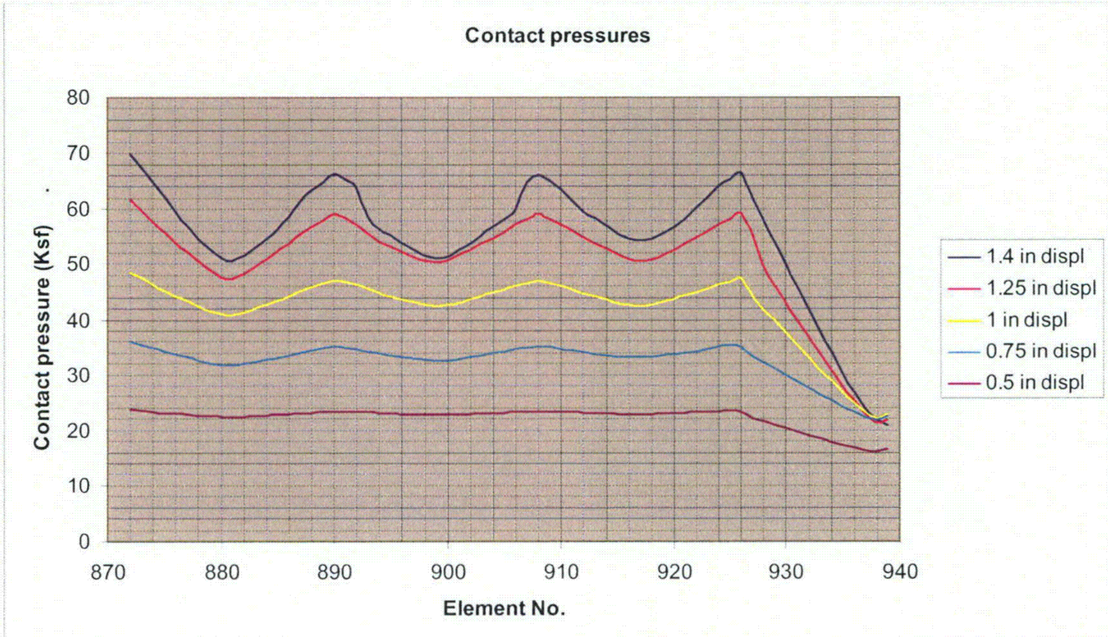


Figure 2.7-9 Contact stresses along mat with Winkler Springs

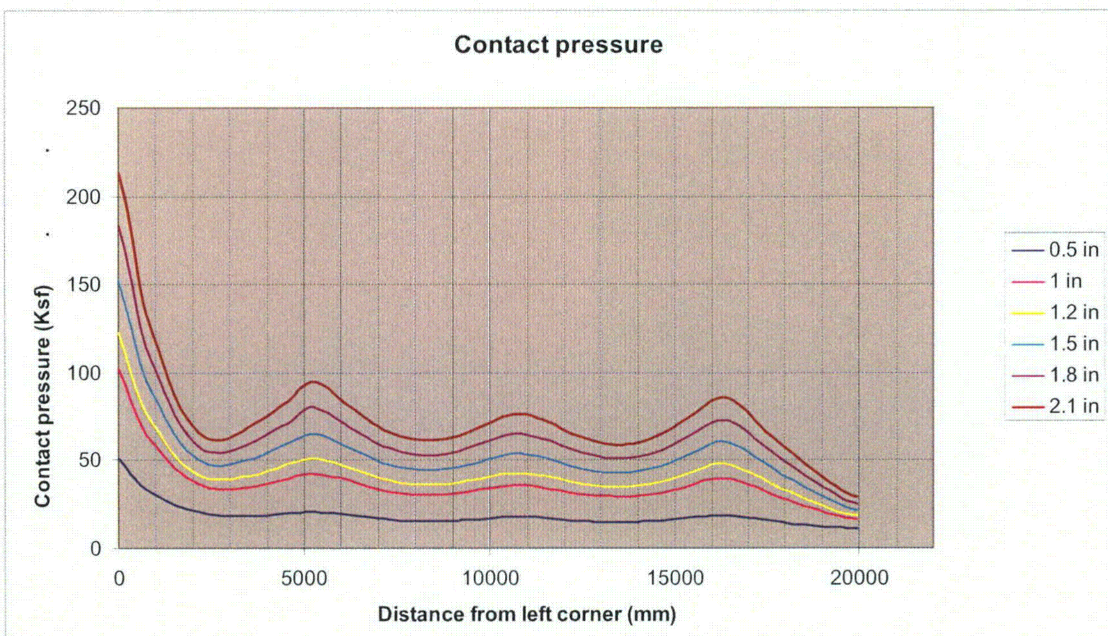


Figure 2.7-10 Contact stresses along mat for Half Space

2.8 Summary of basemat design

The nuclear island basemat has been designed to satisfy the ACI 349 code for the member forces given by conservative analyses. These analyses apply equivalent static loads to a detailed model of the nuclear island on Winkler soil springs. As described in section 2.6.1.2, the loads envelope the seismic response for the worst soil condition. As described in section 2.4.2, the non-linear lift off analyses give very conservative maximum bearing pressures when there is significant lift off. As described in section 2.6.1.1, the soil springs have a stiffness of 520 kcf corresponding to a soft soil site. This spring stiffness is significantly lower than that corresponding to the soft to medium soil case giving the maximum seismic response. Use of the lower stiffness springs is conservative since it maximizes the bending moments in the basemat. The restraint of the side soils is conservatively neglected. Lift off is considered using compression only springs.

The reinforcement in the 6' basemat is sized to have a minimum margin of 20% above that required in the equivalent static analyses on uniform soil springs. This margin was established by studies of the AP600 basemat which has the same configuration as the AP1000. It provides margin to cover variability in the soil properties across the plan of the footprint of the nuclear island.

Studies described in section 2.7.1 demonstrate that the analyses using Winkler soil springs give conservative member forces for design of the basemat reinforcement. Analyses using finite element models of the soil generally showed lower design member forces in the basemat.

Behavior of the basemat for loads beyond the design basis was investigated as described in section 2.7.2. These analyses of the interaction between the basemat and the soil showed ductile behavior of the basemat. As the concrete cracked and subsequently the reinforcement yielded, the deflections of the mat were sufficient to permit significant redistribution of the soil reactions to locations below the walls thus reducing bending moments in the slabs. The final failure mechanism was a shear failure close to the walls at a loading of about three times the SSE design load.

2.9 Nuclear island stability

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

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

The factors of safety for sliding and overturning for the SSE are calculated for each soil case for the base reactions shown in Table 2.4-2. The minimum values are reported in Table 2.9-1. The method of analysis is as described in subsection 3.8.5.5 of the DCD with the exception that the sliding resistance is based on the friction force developed between the basemat and the foundation using a coefficient of friction of 0.70. The governing friction value at the interface zone is a thin soil layer (soil on soil) under the mud mat assumed to have a friction angle of 35 degrees. The Combined License applicant will provide the site specific angle of internal friction for the soil below the foundation. In the case of a rock foundation, the mud mat will interlock with the rock, and therefore, the friction angle will be at least 55 degrees.

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

Load Combination	Sliding		Overturning		Flotation	
	Factor of Safety	Limit	Factor of Safety	Limit	Factor of Safety	Limit
D + H + B + W	Design Wind					
North-South	23.2	1.5	51.5	1.5	–	–
East –West	17.4	1.5	27.9	1.5	–	–
D + H + B + W_t	Tornado Condition					
North-South	12.8	1.1	17.7	1.1	–	–
East –West	10.6	1.1	9.6	1.1	–	–
D + H + B + W_h	Hurricane Condition					
North-South	18.1	1.1	31.0	1.1	–	–
East –West	14.2	1.1	16.7	1.1	–	–
D + H + B + E_S	SSE Event					
North-South	1.28	1.1	–	–	–	–
East-West	1.33	1.1	–	–	–	–
Line 1	–	–	1.39	1.1	–	–
Line 11	–	–	1.42	1.1	–	–
Line I	–	–	1.07[(1)]	1.1	–	–
West Side Shield Bldg	–	–	1.06[(1)]	1.1	–	–
	Flotation					
D + F	–	–	–	–	3.51	1.1
D + B	–	–	–	–	3.70	1.5

Notes:

(1) Considering active and passive soil pressures on the external walls below grade, the minimum factor of safety against overturning (1.07 and 1.06) increases to 1.12 (Line I) & 1.10 (West Side of Shield Building). This factor of safety meets the requirement of 1.1 based on the conservative moment balance formula treating the seismic moment as static loads. ASCE/SEI 43-05, Reference 7, recognizes that there is considerable margin beyond that given by the moment balance formula. Reference 7 permits a nonlinear rocking analysis. A nonlinear (liftoff allowed) time history analysis is described in Section 2.4.2 showing that the nuclear island basemat uplift effect is insignificant. Further, these analyses were performed for free field seismic ZPA input as high as 0.5g without significant uplift. Therefore the factor of safety against overturning is greater than 1.67 (0.5g/0.3g).

3. REGULATORY IMPACT

The design of the nuclear island basemat and evaluation of stability is addressed in subsection 3.8.5 of the NRC Final Safety Analysis Report (FSER, Reference 2) write-ups.

The changes to the DCD presented in this report do not represent an adverse change to the design functions, including the pressure boundary integrity functions and the access function, or to how design functions are performed or controlled. The analysis and design of the nuclear island basemat for soil sites is consistent with the description of the AP600 analysis in 3.8.5 of the AP600 DCD. The changes to the DCD do not involve revising or replacing a DCD-described evaluation methodology. The changes to the DCD do not involve a test or experiment not described in the DCD. The design changes, including the Tier 1 DCD change, will not result in a significant decrease in the level of safety otherwise provided by the design. The Tier 2 DCD changes identified in this report do not require a license amendment per the criteria of VIII. B. 5.b. of Appendix D to 10 CFR Part 52.

The regulations included in 52 Appendix D VIII. A. identify that requests for exemptions from Tier 1 Information by the COL applicants are governed by the requirements in 10 CFR 52.63(b)(1). In addition to requiring that the design change will not result in a significant decrease in the level of safety otherwise provided by the design, the exemption must comply with the requirements of 10 CFR 50.12(a). The criteria of 10 CFR 50.12(a) require that special circumstances are present to grant an exemption. The second of these special circumstances is as follows: "(ii) Application of the regulation in the particular circumstances would not serve the underlying purpose of the rule or is not necessary to achieve the underlying purpose of the rule." Since the underlying purpose of 10 CFR Part 52 Appendix D is to provide for the licensing and construction of standard AP1000 nuclear power plants, an exemption to Tier 1 of the AP1000 DCD to permit application of the Standard AP1000 to a wider range of soils conditions is clearly needed to achieve applicability of the AP1000 to site currently being considered by COL applicants.

The DCD changes do not affect resolution of a severe accident issue and does not require a license amendment based on the criteria of VIII. B. 5.c of Appendix D to 10 CFR Part 52.

The DCD changes will not alter barriers or alarms that control access to protected areas of the plant. The DCD change will not alter requirements for security personnel. Therefore, the DCD change does not have an adverse impact on the security assessment of the AP1000.

4. REFERENCES

1. APP-GW-GL-700, AP1000 Design Control Document, Revision 15.
2. Final Safety Evaluation Report Related to Certification of the AP1000 Standard Design, September 2004.
3. APP-GW-S2R-010, Revision 0, Extension of Nuclear Island Seismic Analyses to Soil Sites, June, 2006.
4. GW-GL-700, AP600 Design Control Document, Revision 2.
5. ASCE Standard 4-98, "Seismic Analysis of Safety-Related Nuclear Structures and Commentary," American Society of Civil Engineers, 1998.
6. APP-GW-GLR-015, Revision 0, DCD seismic and structural changes
7. ASCE Standard ASCE/SEI 43-05, "Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities", American Society of Civil Engineers, 2005.

8. Vecchio, F.J., and Collins, M.P., "The Modified Compression Field Theory for Reinforced Concrete Elements Subjected to Shear", ACI Journal, V. 83, No. 2, 1986, pp 219-231.
9. Vecchio F.J.," Disturbed Stress Field Model for Reinforced Concrete: Formulation", ASCE Journal of Civil Engineering, Vol. 126, No. 9, pp. 1070-1077.

5. DCD MARK UP

A markup of the DCD for changes to the DCD in Chapter 2 and subsection 3.8.5 resulting from the technical changes described in this basemat and foundation report are shown on the following pages. Section 5.1 shows the revisions proposed in Section 2.5 including those based on the seismic analyses in Reference 3. Section 5.2 shows the revisions to Tier 1 Table 5.0-1. Section 5.3 shows the revisions proposed in subsection 3.8.5.

5.1 Proposed Revisions to DCD Section 2.5

2.5 Geology, Seismology, and Geotechnical Engineering

Combined License applicants referencing the AP1000 certified design will address site specific information related to basic geological, seismological, and geotechnical engineering of the site and the region, as discussed in the following subsections.

2.5.1 Basic Geological and Seismic Combined License Information

Combined License applicants referencing the AP1000 certified design will address the following regional and site-specific geological, seismological, and geophysical information as well as conditions caused by human activities:

- Structural geology of the site
- Seismicity of the site
- Geological history
- Evidence of paleoseismicity
- Site stratigraphy and lithology
- Engineering significance of geological features
- Site groundwater conditions
- Dynamic behavior during prior earthquakes
- Zones of alteration, irregular weathering, or structural weakness
- Unrelieved residual stresses in bedrock
- Materials that could be unstable because of mineralogy or unstable physical properties
- Effect of human activities in the area

2.5.2 Vibratory Ground Motion

The AP1000 is designed for a safe shutdown earthquake (SSE) defined by a peak ground acceleration (PGA) of 0.30g and the design response spectra specified in subsection 3.7.1.1, and Figures 3.7.1-1 and 3.7.1-2. The AP1000 design response spectra were developed using the Regulatory Guide 1.60 response spectra as the base and modified to address high frequency amplification effects observed in eastern North America earthquakes. The peak ground accelerations in the two horizontal and the vertical directions are equal.

2.5.2.1 Combined License Seismic and Tectonic Characteristics Information

Combined License applicants referencing the AP1000 certified design will address the following site-specific information related to the vibratory ground motion aspects of the site and region:

- Seismicity
- Geologic and tectonic characteristics of site and region
- Correlation of earthquake activity with seismic sources
- Probabilistic seismic hazard analysis and controlling earthquakes
- Seismic wave transmission characteristics of the site
- SSE ground motion

The Combined License applicant must demonstrate that the proposed site meets the following requirements:

~~1.~~1. The free field peak ground acceleration at the ~~foundation~~ finished grade level is less than or equal to a 0.30g SSE.

~~1.~~2. The site design response spectra at the ~~foundation~~ finished grade level in the free-field are less than or equal to those given in Figures 3.7.1-1 and 3.7.1-2.

3. In lieu of (1) and (2) above, for a site where the nuclear island is founded on competent rock with shear wave velocity greater than 3500 feet per second and there are thin layers of soft material overlying the rock, the site specific peak ground acceleration and spectra may be developed at the top of the competent rock and shown at the foundation level to be less than or equal to those given in Figures 3.7.1-1 and 3.7.1-2.

4. Foundation material layers are approximately horizontal (dip less than 20 degrees) and the shear wave velocity of the soil is greater than or equal to 1000 feet per second.

2.5.2.2 Site-Specific Seismic Structures

The AP1000 includes all seismic Category I structures, systems and components in the scope of the design certification.

2.5.2.3 Sites with Geoscience Parameters Outside the Certified Design

If the site-specific spectra at foundation level exceed the response spectra in Figures 3.7.1-1 and 3.7.1-2 at any frequency, or if soil conditions are outside the range evaluated for AP1000 design certification, a site-specific evaluation can be performed. This evaluation will consist of a site-specific dynamic analysis and generation of in-structure response spectra to be compared with the floor response spectra of the certified design at 5-percent damping. The site design response spectra at the foundation level in the free-field given in Figures 3.7.1-1 and 3.7.1-2 were used to develop the floor response spectra. They were applied at foundation level for the hard rock site and at finished grade level for the soil sites. The site is acceptable for construction of the AP1000 if the floor response spectra from the site-specific evaluation do not exceed the AP1000 spectra for each of the locations identified below:

Containment internal structures at elevation of r~~R~~eactor vessel support	Figures 4.4.3-1 to 4.4.3-3*
Containment operating floor	Figures 4.4.3-4 to 4.4.3-6*
~~Coupled a~~Auxiliary building NE corner at~~and shield building at~~ control room ceiling~~floor~~	Figures 4.4.3-7 to 4.4.3-9*
~~Coupled auxiliary and s~~Shield building at fuel building roof	Figures 4.4.3-10 to 4.4.3-12*
~~Coupled auxiliary and shield building at s~~Shield building roof	Figures 4.4.3-13 to 4.4.3-15*

Steel containment vessel at polar crane support Figures 4.4.3-16 to 4.4.3-18*

** These Figure numbers are Figures in APP-GW-S2R-010, Revision 0, Extension of Nuclear Island Seismic Analyses to Soil Sites, June, 2006. The Figures will be included in a proposed new DCD Appendix 3G. The Figure references in Chapter 2 will be changed to those in Appendix 3G when the DCD is finalized.*

Site-specific soil structure interaction analyses should be performed using the 3D SASSI models described in Appendix 3G for variations in site conditions that can not be adequately represented in two dimensions. Results should be compared to the results of the 3D SASSI analyses described in Appendix 3G.

Site-specific soil structure interaction analyses may be performed using the 2D SASSI models described in Appendix 3G for variations in site conditions that can be adequately represented in these models. Results should be compared to the results of the 2D SASSI analyses described in Appendix 3G.

The s~~S~~ite-specific soil structure interaction analyses ~~must be performed by the Combined License applicant to demonstrate acceptability of sites that have seismic and soil characteristics outside the site parameters in Table 2-1. These analyses~~ would use the site-specific soil conditions (including variation in soil properties in accordance with Standard Review Plan 3.7.2). The three components of the site-specific ground motion time history must satisfy the regulatory requirements for statistical independence and enveloping of the site design spectra at 5% damping~~enveloping criteria of Standard Review Plan 3.7.1 for the response spectrum for damping values of 2, 3, 4, 5, and 7 percent and the enveloping criterion for power spectral density function~~. Floor response spectra determined from the site-specific analyses should be compared against the design basis of the AP1000 described above. ~~Member forces in each of the sticks should be compared against those given in Tables 3.7.2-11 to 3.7.2-13.~~ These evaluations and comparisons will be provided and reviewed as part of the Combined License application.

2.5.3 Surface Faulting Combined License Information

Combined License applicants referencing the AP1000 certified design will address the following surface and subsurface geological, seismological, and geophysical information related to the potential for surface or near-surface faulting affecting the site:

- Geological, seismological, and geophysical investigations
- Geological evidence, or absence of evidence, for surface deformation
- Correlation of earthquakes with capable tectonic sources
- Ages of most recent deformation
- Relationship of tectonic structures in the site area to regional tectonic structures
- Characterization of capable tectonic sources
- Designation of zones of quaternary deformation in the site region
- Potential for surface tectonic deformation at the site

2.5.4 Stability and Uniformity of Subsurface Materials and Foundations

Combined License applicants referencing the AP1000 certified design will address the following site-specific information related to the stability and uniformity of subsurface materials and foundations.

- Excavation
- Bearing capacity

- Settlement
- Liquefaction

~~Seismic analysis and foundation design for rock sites is described in Sections 3.7 and 3.8. The AP1000 certified design is based on the nuclear island being founded on rock. Soils may be present above the foundation level.~~

2.5.4.1 Excavation

Excavation for the nuclear island structures below grade may use either a sloping excavation or a vertical face as described in subsequent paragraphs. If backfill is to be placed adjacent to the exterior walls of the nuclear island, the Combined License applicant will provide information on the properties of backfill and its compaction requirements as described in subsection 2.5.4.6.3 and will evaluate its properties against those used in the seismic analyses described in subsection 3.7.2.

Excavation in soil for the nuclear island structures below grade will establish a vertical face with lateral support of the adjoining undisturbed soil or rock. One alternative is to use a soil nailing method. Soil nailing is a method of retaining earth in-situ. As the nuclear island excavation progresses vertically downward, holes are drilled horizontally into the adjoining undisturbed soil, a metal rod is inserted into the hole, and grout is pumped into each hole to fill the hole and to anchor the "nail" rod.

As each increment of the nuclear island excavation is completed, nominal eight to ten inch diameter holes are drilled horizontally through the vertical face of the excavation into adjacent undisturbed soil. These "nail" holes, spaced horizontally and vertically on five to six feet centers, are drilled slightly downward to the horizontal. A "nail", normally a metal bar/rod, is center located for the full length of the hole. The nominal length of soil nails is 60 percent to 70 percent of the wall height, depending upon soil conditions. The hole is filled with grout to anchor the rod to the soil. A metal face plate is installed on the exposed end of the rod at the excavated wall vertical surface. Welded wire mesh is hung on the wall surface for wall reinforcement and secured to the soil nail face plates for anchorage. A 4,000 psi to 5,000 psi non-expansive pea gravel shotcrete mix is blown onto the wire mesh to form a nominal four to six inch thick soil retaining wall. Installation of the soil retaining wall closely follows the progress of the excavation and is from the top down, with each wire mesh-reinforced, shotcreted wall section being supported by the soil "nails" and the preceding elevations of soil nailed wall placements. The shotcrete contains a crystalline waterproofing material as described in subsection 3.4.1.1.1.

Soil nailing as a method of soil retention has been successfully used on excavations up to 55 feet deep on projects in the U.S. Soils have been retained for up to 90 feet in Europe. The state of California CALTRANS uses soil nailing extensively for excavations and soil retention installations. Soil nailing design and installation has a successful history of application which is evidenced by its excellent safety record.

The soil nailing method produces a vertical surface down to the bottom of the excavation and is used as the outside forms for the exterior walls below grade of the nuclear island. Concrete is placed directly against the vertical concrete surface of the excavation.

For excavation in rock and for methods of soil retention other than soil nailing, four to six inches of shotcrete are blown on to the vertical surface. The concrete for the exterior walls is placed against the shotcrete. The shotcrete contains a crystalline waterproofing material as described in subsection 3.4.1.1.1.

2.5.4.2 Bearing Capacity

The average bearing reaction under static loads is 8,600 lb/ft^2 with a maximum at one edge of 14,250 lb/ft^2. The maximum bearing reaction ~~on the hard rock~~ determined from the analyses described in ~~subsection 3.8.5.1~~ (section 2.4 of this report – *this material will be summarized in Appendix 3G of the DCD and an appropriate reference made herein once this draft Appendix is finalized*) is less than ~~120~~35,000 lb/ft^2 under all combined loads, including the safe shutdown earthquake. The Combined License applicant will verify that the site-specific allowable soil bearing capacities for static and dynamic loads ~~allowable bearing capacity~~ at the ~~a hard rock~~ site will exceed this demand.

The maximum bearing reaction ~~on the hard rock~~ specified in Table 2-1 is determined from the analyses described in subsection 3.8.5.1. The evaluation of the allowable capacity of the ~~bedrock~~ soil should be based on the properties of the underlying materials (see subsection 2.5.4.5.2), including appropriate laboratory test data to evaluate strength, and considering local site effects, such as fracture spacing, variability in properties, and evidence of shear zones. The allowable bearing capacity should provide a factor of safety appropriate for the design load combination, including safe shutdown earthquake loads.

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

2.5.4.3 Settlement

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

~~Settlement at a hard rock site is small and is not significant to the design of the AP1000.~~ The AP1000 does not rely on structures, systems, or components located outside the nuclear island to provide safety-related functions. Differential settlement between the nuclear island foundation and the foundations of adjacent buildings does not have an adverse effect on the safety-related functions of structures, systems, and components. Differential settlement under the nuclear island foundation could cause the basemat and buildings to tilt. Much of this settlement occurs during civil construction prior to final installation of the equipment. Differential settlement of a few inches across the width of the nuclear island would not have an adverse effect on the safety-related functions of structures, systems, and components.

2.5.4.4 Liquefaction

The Combined License applicant will demonstrate that the potential for liquefaction is negligible.

2.5.4.5 Subsurface Uniformity

Soil structure interaction and foundation design are a function of the uniformity of the soil and rock below the foundation. Although the design and analysis of the AP1000 is based on soil or rock conditions with uniform properties within horizontal layers, it includes provisions and design margins to accommodate many non-uniform sites. This subsection identifies the requirements for site investigation that may be used to demonstrate that:

- A site is "uniform" based on the criteria outlined in subsection 2.5.4.5.3. If the site can be demonstrated to be "uniform" no further site specific analysis is required to qualify the site for the AP1000.

- A "non-uniform" site is acceptable to locate the AP1000 based on the criteria for acceptability outlined in subsection 2.5.4.5.3. Some non-uniform sites are acceptable as described in subsection 2.5.4.5.3 based on evaluation performed as part of design certification. Other non-uniform sites may be shown to be acceptable as described in subsection 2.5.4.5.3.1 using site specific evaluation as part of the Combined License application.

Considerations with respect to the materials underlying the nuclear island are the type of site, such as rock or soil, and whether the site can be considered uniform. If the site is nonuniform, the nonuniform soil characteristics such as the location and profiles of soft and hard spots should be considered. These considerations can be assessed with the information developed in response to Regulatory Guides 1.132 and 1.138. The geological investigations of subsections 2.5.1 and 2.5.4.6.1 provide information on the uniformity of the site, whether it may be geologically impacted, and whether the bedrock may be sloping or undulatory.

A survey of 22 commercial nuclear power plant sites in the United States focused on site parameters that affect the seismic response such as the depth to bedrock, type and characteristic of the soil layers, including the variation of shear wave velocities, the depth to the ground water level, and the embedment depth of the plant structures. Of the 22 sites, 11 are rock sites where competent rock exists at relatively shallow depths. At the other sites, the depth to bedrock varies from about 50 feet (Callaway) to well in excess of 4,000 feet (South Texas). A review of these 11 soil sites, all of which are marine, deltaic, or lacustrine deposits, did not reveal any significant variation of soil characteristics below the nuclear island footprint. There was one possible nonuniform site, Monticello, which is underlain by glacial deposits; the geologic description is such that there might be lateral variability in the foundation parameters within the plan dimension of the plant. The review of the 22 commercial nuclear power plant sites in the United States suggests that the majority of AP1000 sites exhibit "uniform" soil properties within the nuclear island footprint.

2.5.4.5.1 Site Investigation for Uniform Sites

For sites that are expected to be uniform, based on the geologic investigation outlined in subsections 2.5.1 and 2.5.4.6.2, Appendix C to Regulatory Guide 1.132 provides guidance on the spacing and depth of borings of the geotechnical investigation for safety-related structures. Specific language in the Regulatory Guide suggests a spacing of 100 feet supplemented with borings on the periphery and at the corners for favorable, uniform geologic conditions.

For foundation engineering purposes, a series of primary borings should be drilled on a grid pattern that encompasses the nuclear island footprint and 40 feet beyond the boundaries of the nuclear island footprint. The 40-foot extension for the grid of borings is established from a Boussinesq analysis of the zone of influence of the foundation mat which shows that the net change in the effective vertical overburden stress is less than seven percent at a distance of 40 feet from the edge of the foundation mat. The grid need not be of equal spacing in the two orthogonal directions, but it should be oriented in accordance with the true dip and strike of the rock in the immediate area of the nuclear island footprint. If geologic conditions are such that true dip and strike are not obvious, or if the dip is practically flat, then the orientation of the grid can be consistent with the major orthogonal lines of the nuclear island. The depth of borings should be determined on the basis of the geologic conditions. Borings should be extended to a depth sufficient to define the site geology and to sample materials that may swell during excavation, may consolidate subsequent to construction, may be unstable under earthquake loading, or whose physical properties would affect foundation behavior or stability. At least one-fourth of the primary borings should penetrate sound rock or, for a deep soil site, to a maximum depth of 250 feet below the foundation mat. At this depth of 250 feet the change in the vertical stress during or after construction for the combined foundation loading is less than 10 percent of the in-situ effective overburden stress. Other primary borings may terminate at a depth of 160 feet below the foundation (equal to the width of the structure).

2.5.4.5.2 Site Investigation for Non-uniform Sites

At sites that are determined to be non-uniform or potentially non-uniform during the course of the geological investigations outlined in subsections 2.5.1 and 2.5.4.6.2, the investigation effort is extended to determine if the site is acceptable for an AP1000.

As the AP1000 foundation/structural system is robust, the probability of being able to show compliance for all but the worst of sites is high, unless liquefaction or faulting is prevalent on the site. As stated in Regulatory Guide 1.132, where variable conditions are found, spacing of boreholes should be smaller, as needed, to obtain a clear picture of soil or rock properties and their variability. Where cavities or other discontinuities of engineering significance may occur, the normal exploratory work should be supplemented by secondary borings or soundings at spacing small enough to detect such features. The depth of the secondary borings is 160 feet below the foundation mat. At this depth, the maximum change in vertical stress during or after construction is about 11 percent of the in-situ effective overburden stress. The depth of borings should be extended beyond 160 feet if the geologic investigation indicates the possible presence of karst conditions, under-consolidated clays, loose sands, intrusive dikes or other forms of geologic impacts at depth greater than 160 feet.

2.5.4.5.3 Site Foundation Material Evaluation Criteria

The AP1000 is designed for application at a site where the foundation conditions do not have extreme variation within the nuclear island footprint. This subsection provides criteria for evaluation of soil variability. The subsurface may consist of layers and these layers may dip with respect to the horizontal. If the dip is less than 20 degrees, the generic analysis using horizontal layers is applicable as described in NUREG CR-0693 (Reference 2). The physical properties of the foundation medium may or may not vary systematically across a horizontal plane. The recommended methodology for checking uniformity is to calculate from the boring logs a series of "best estimate" planes beneath the nuclear island footprint that define the top (and bottom) of each layer. The planes could represent stratigraphic boundaries, lithologic changes, unconformities, but most important, they should represent boundaries between layers having different shear wave velocities. Shear wave velocity is the primary property used for defining uniformity of a site.

The distribution of bearing reactions under the basemat is a function of the subgrade modulus which in turn is a function of the shear wave velocity. The Combined License applicant shall demonstrate that the variation of subgrade modulus or shear wave velocity across the footprint is within the range considered for design of the nuclear island basemat. The farther that the non-uniform layer is located below the foundation, the less influence it has on the bearing pressures at the basemat. Lateral variability of the shear wave velocity at depths greater than 120 feet below grade (80 feet below the foundation) do not significantly affect the subgrade modulus.

If a site can be classified as uniform, it qualifies for the AP1000 based on analyses and evaluations performed to support design certification without additional site specific analyses. For a site to be considered uniform, the variation of shear wave velocity in the material below the foundation to a depth of 120 feet below finished grade within the nuclear island footprint shall meet the criteria outlined below:

- The depth to a given layer indicated on each boring log may not fall precisely on the postulated "best estimate" plane. The deviation of the observed layers from the "best-estimate" planes should not exceed

5 percent of the observed depths from the ground surface to the plane. If the deviation is greater than 5 percent, additional planes may be appropriate or additional borings may be required, thereby diminishing the spacing.

- For a layer with a low strain shear wave velocity greater than or equal to 2500 feet per second, the layer should have approximately uniform thickness, should have a dip no greater than 20 degrees and the shear wave velocity at any location within any layer should not vary from the average velocity within the layer by more than 20 percent.

- For a layer with a low strain shear wave velocity less than 2500 feet per second, the layer should have approximately uniform thickness, should have a dip no greater than 20 degrees and the shear wave velocity at any location within any layer should not vary from the average velocity within the layer by more than 10 percent.

2.5.4.5.3.1 Site-Specific Subsurface Uniformity Design Basis

Many sites that do not meet the above criteria for a uniform site are acceptable for the AP1000. The key attribute for acceptability of the site for an AP1000 is the bearing pressure on the underside of the basemat. A site having local soft or hard spots within a layer or layers does not meet the criteria for a uniform site. Non-uniform soil conditions may also require evaluation of the AP1000 seismic response as described in subsection 2.5.2.2.

As described in subsection 3.8.5 the nuclear island foundation is designed specifically for bearing pressures of 120 percent of those of the uniform soil properties case. Evaluation criteria are defined to evaluate sites that do not satisfy the site parameters directly. The design basis provided below is included to provide a clear specification of the design commitment and evaluation criteria required to demonstrate that a site specific application satisfies AP1000 requirements. Application of the AP1000 to sites using this site-specific evaluation is not approved as part of the AP1000 design certification and the evaluation should be provided and reviewed as part of the Combined License application.

Rigid Basemat Evaluation

A site with nonuniform soil properties may be demonstrated to be acceptable by evaluation of the bearing pressures on the underside of a rigid rectangular basemat equivalent to the nuclear island. Bearing pressures are calculated for dead and safe shutdown earthquake loads. The safe shutdown earthquake loads used for the evaluation are associated with one of the AP1000 design soil cases evaluated for design certification. The soil case representative of the site-specific soil is used. For the site to be acceptable, the bearing pressures from this analysis need to be less than or equal to 120 percent of the bearing pressures calculated in similar analyses for a site having uniform soil properties.

Alternatively, the safe shutdown earthquake loads may be determined from a site-specific seismic analysis of the nuclear island using site specific inputs as described in subsection 2.5.2.2. For the site to be acceptable, the bearing pressures from the site-specific analyses need to be less than or equal to 120 percent of the bearing pressures calculated in rigid basemat analyses using the AP1000 design ground motion at a site having uniform soil properties.

Flexible Basemat Evaluation

For sites having bedrock close to the foundation level the assumption of a rigid basemat may be overly conservative because local deformation of the basemat will reduce the effect of local soil variability. For such sites, a site-specific analysis may be performed using the AP1000 basemat model and methodology described in subsection 3.8.5. The safe shutdown earthquake loads are those from the AP1000 design soil case representative of the site-specific soil. Alternatively, bearing pressures may be determined from a site-specific soil structure interaction analysis using site specific inputs as described in subsection 2.5.2.2. For the site to be acceptable the bearing pressures from the site-specific analyses including static and dynamic loads need to be less than the capacity of each portion of the basemat.

2.5.4.6~~5~~Combined License Information

Combined License applicants referencing the AP1000 design will address the following site specific information related to the geotechnical engineering aspects of the site. No further action is required for sites within the bounds of the site parameters.

2.5.4.6~~5~~.1 Site and Structures – Site-specific information regarding the underlying site conditions and geologic features will be addressed. This information will include site topographical features, as well as the locations of seismic Category I structures.

2.5.4.6~~5~~.2 The Combined License applicant will establish the properties of the foundation soils to be within the range considered for design of the nuclear island basemat.

Properties of Underlying Materials – A determination of the static and dynamic engineering properties of foundation soils and rocks in the site area will be addressed. This information will include a discussion of the type, quantity, extent, and purpose of field explorations, as well as logs of borings and test pits. Results of field plate load tests, field permeability tests, and other special field tests (e.g., bore-hole extensometer or pressuremeter tests) will also be provided. Results of geophysical surveys will be presented in tables and profiles. Data will be provided pertaining to site-specific soil layers (including their thicknesses, densities, moduli, and Poisson's ratios) between the basemat and the underlying rock stratum. Plot plans and profiles of site explorations will be provided.

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

~~:~~

- ~~Submerged soil density of 60 pounds/ft^3~~
- ~~Angle of internal friction of 32 degrees~~

Laboratory Investigations of Underlying Materials – Information about the number and type of laboratory tests and the location of samples used to investigate underlying materials will be provided. Discussion of the results of laboratory tests on disturbed and undisturbed soil and rock samples obtained from field investigations will be provided.

2.5.4.6~~5~~.3 Excavation and Backfill – Information concerning the extent (horizontal and vertical) of seismic Category I excavations, fills, and slopes, if any will be addressed. The sources, quantities, and static and dynamic engineering properties of borrow materials will be described in the site-specific application. The compaction requirements, results of field compaction tests, and fill material properties (such as moisture content, density, permeability, compressibility, and gradation) will also be provided. Information will be provided concerning the specific soil retention system, for example, the soil nailing system, including the length and size of the soil nails, which is based on actual soil conditions and applied construction surcharge loads. If backfill is to be placed adjacent to the exterior walls of the nuclear island, information will be provided concerning compaction of the backfill and any additional loads on the exterior walls of the nuclear island. Information will also be provided on the waterproofing system along the vertical face and the mudmat. Information will be provided on the mudmat to demonstrate its ability to resist the structural bearing and shear loads described in subsection 2.5.4.2. The maximum bearing pressure is 24~~83~~0 psi. The mudmat may be designed as structural plain concrete in accordance with ACI 318-02 (Reference 1). This requires the specified concrete compressive strength to be no less than 2500 psi. The commentary states this requirement is imposed in the code because "lean concrete mixtures may not produce adequately homogeneous material or well formed surfaces." If the Combined License applicant proposes to use a concrete with strength less than 2500 psi, the applicant must demonstrate that the mix will result in an acceptable homogeneous material.

2.5.4.6~~5~~.4 Ground Water Conditions – Groundwater conditions will be described relative to the foundation stability of the safety-related structures at the site. The soil properties of the various layers under possible groundwater conditions during the life of the plant will be compared to the range of values assumed in the standard design in Table 2-1.

2.5.4.6~~5~~.5 Liquefaction Potential – Soils under and around seismic Category I structures will be evaluated for liquefaction potential for the site specific SSE ground motion. This should include justification of the selection of the soil properties, as well as the magnitude, duration, and number of excitation cycles of the earthquake used in the liquefaction potential evaluation (e.g., laboratory tests, field tests, and published data). Liquefaction potential will also be evaluated to address seismic margin.

2.5.4.6~~5~~.6 Bearing Capacity – The Combined License applicant will verify that the site-specific allowable soil bearing capacities for static and dynamic loads are equal to or greater than the values documented in Table 2-1, or will provide a site-specific evaluation as described in subsection 2.5.4.2. The acceptance criteria for this evaluation are those of Standard Review Plan 2.5.4 as follows:

- The static and dynamic loads, and the stresses and strains induced in the soil surrounding and underlying the nuclear island, are conservatively and realistically evaluated.

- The consequences of the induced soil stresses and strains, as they influence the soil surrounding and underlying the nuclear island, have been conservatively assessed.

2.5.4.~~5~~6.7 Earth Pressures – The Combined License applicant will describe the design for static and dynamic lateral earth pressures and hydrostatic groundwater pressures acting on plant safety-related facilities using soil parameters as evaluated in previous subsections.

2.5.4.~~5~~6.8 Soil Properties for Seismic Analysis of Buried Pipes – The AP1000 does not utilize safety related buried piping. No additional information is required on soil properties.

2.5.4.~~5~~6.9 Static and Dynamic Stability of Facilities – Soil characteristics affecting the stability of the nuclear island will be addressed including foundation rebound, settlement, and differential settlement.

2.5.4.~~5~~6.10 Subsurface Instrumentation – Data will be provided on instrumentation, if any, proposed for monitoring the performance of the foundations of the nuclear island. This will specify the type, location, and purpose of each instrument, as well as significant details of installation methods. The location and installation procedures for permanent benchmarks and markers for monitoring the settlement will be addressed.

2.5.4.6.11 – Settlement of Nuclear Island - Data will be provided on short-term (elastic) and long-term (heave and consolidation) settlement for soil sites for the history of loads imposed on the foundation consistent with the construction sequence. The resulting time-history of settlements includes construction activities such as dewatering, excavation, bearing surface preparation, placement of the basemat and construction of the superstructure.

2.5.5 Combined License Information for Stability of Slopes

Combined License applicants referencing the AP1000 design will address site-specific information about the static and dynamic stability of soil and rock slopes, the failure of which could adversely affect the nuclear island.

2.5.6 Combined License Information for Embankments and Dams

Combined License applicants referencing the AP1000 design will address site-specific information about the static and dynamic stability of embankments and dams, the failure of which could adversely affect the nuclear island.

2.6 References

1. American Concrete Institute (ACI), "Building Code Requirements for Structural Concrete," ACI 318-02.

2. NUREG/CR-0693, "Seismic Input and Soil Structure Interaction," February, 1979.

Table 2-1 (Sheet 1 of 3)

SITE PARAMETERS

Air Temperature	
Maximum Safety [(a)]	115°F dry bulb/80°F coincident wet bulb 81°F wet bulb (noncoincident)
Minimum Safety [(a)]	-40°F
Maximum Normal [(b)]	100°F dry bulb/77°F coincident wet bulb 80°F wet bulb (noncoincident) [(d)]
Minimum Normal [(b)]	-10°F
Wind Speed	
Operating Basis	145 mph (3 second gust); importance factor 1.15 (safety), 1.0 (nonsafety); exposure C; topographic factor 1.0
Tornado	300 mph
Seismic	
SSE	0.30g peak ground acceleration [(c)]
Fault Displacement Potential	None
Soil	
Average Allowable Static Bearing Capacity	Greater than or equal to 8,600 lb/ft^2 over the footprint of the nuclear island at its excavation depth
Maximum Allowable Dynamic Bearing Capacity for Normal Plus SSE	Greater than or equal to ~~120~~35,000 lb/ft^2 at the edge of the nuclear island at its excavation depth
Shear Wave Velocity	Greater than or equal to 1~~8~~,000 ft/sec based on low-strain best-estimate soil properties over the footprint of the nuclear island at its excavation depth
Lateral Variability	Soils supporting the nuclear island should not have extreme variations in subgrade stiffness Case 1: For a layer with a low strain shear wave velocity greater than or equal to 2500 feet per second, the layer should have approximately uniform thickness, should have a dip not greater than 20 degrees, and should have less than 20 percent variation in the shear wave velocity from the average velocity in any layer. Case 2: For a layer with a low strain shear wave velocity less than 2500 feet per second, the layer should have approximately uniform thickness, should have a dip not greater than 20 degrees, and should have less than 10 percent variation in the shear wave velocity from the average velocity in any layer. (see subsection 2.5.4.5)
Liquefaction Potential	None

Missiles	
Tornado	4000 - lb automobile at 105 mph horizontal, 74 mph vertical 275 - lb, 8 in. shell at 105 mph horizontal, 74 mph vertical 1 inch diameter steel ball at 105 mph horizontal and vertical
Flood Level	Less than plant elevation 100′
Ground Water Level	Less than plant elevation 98′

Table 2-1 (Sheet 2 of 3)

SITE PARAMETERS

Plant Grade Elevation	Less than plant elevation 100′ except for portion at a higher elevation adjacent to the annex building
Precipitation	
Rain	19.4 in./hr (6.3 in./5 min)
Snow/Ice	75 pounds per square foot on ground with exposure factor of 1.0 and importance factors of 1.2 (safety) and 1.0 (non-safety)
Atmospheric Dispersion Values - $\chi/Q^{(e)}$	
Site boundary (0-2 hr)	$\leq 5.1 \times 10^{-4}$ sec/m^3
Site boundary (annual average)	$\leq 2.0 \times 10^{-5}$ sec/m^3
Low population zone boundary	
0 - 8 hr	$\leq 2.2 \times 10^{-4}$ sec/m^3
8 - 24 hr	$\leq 1.6 \times 10^{-4}$ sec/m^3
24 - 96 hr	$\leq 1.0 \times 10^{-4}$ sec/m^3
96 - 720 hr	$\leq 8.0 \times 10^{-5}$ sec/m^3
Population Distribution	
Exclusion area (site)	0.5 mi

Notes:

(a) Maximum and minimum safety values are based on historical data and exclude peaks of less than 2 hours duration.

(b) Maximum and minimum normal values are the 1 percent exceedance magnitudes.

(c) With ground response spectra ~~(at foundation level of nuclear island)~~ as given in Figures 3.7.1-1 and 3.7.1-2. Seismic input is defined at finished grade except for sites where the nuclear island is founded on rock.

(d) The noncoincident wet bulb temperature is applicable to the cooling tower only.

(e) For AP1000, the terms "site boundary" and "exclusion area boundary" are used interchangeably. Thus, the χ/Q specified for the site boundary applies whenever a discussion refers to the exclusion area boundary.

Table 2-1 (Sheet 3 of 3)

SITE PARAMETERS

Control Room Atmospheric Dispersion Factors (χ /Q) for Accident Dose Analysis

χ/Q (s/m^3) at HVAC Intake for the Identified Release Points[(1)]

	Plant Vent or PCS Air Diffuser[(3)]	Ground Level Containment Release Points[(4)]	PORV and Safety Valve Releases[(5)]	Steam Line Break Releases	Fuel Handling Area[(6)]
0 - 2 hours	2.2E-3	2.2E-3	2.0E-2	2.4E-2	6.0E-3
2 - 8 hours	1.4E-3	1.4E-3	1.8E-2	2.0E-2	4.0E-3
8 - 24 hours	6.0E-4	6.0E-4	7.0E-3	7.5E-3	2.0E-3
1 - 4 days	4.5E-4	4.5E-4	5.0E-3	5.5E-3	1.5E-3
4 - 30 days	3.6E-4	3.6E-4	4.5E-3	5.0E-3	1.0E-3

χ/Q (s/m^3) at Control Room Door for the Identified Release Points[(2)]

	Plant Vent or PCS Air Diffuser[(3)]	Ground Level Containment Release Points[(4)]	PORV and Safety Valve Releases[(5)]	Steam Line Break Releases	Fuel Handling Area[(6)]
0 - 2 hours	6.6E-4	6.6E-4	4.0E-3	4.0E-3	6.0E-3
2 - 8 hours	4.8E-4	4.8E-4	3.2E-3	3.2E-3	4.0E-3
8 - 24 hours	2.1E-4	2.1E-4	1.2E-3	1.2E-3	2.0E-3
1 - 4 days	1.5E-4	1.5E-4	1.0E-3	1.0E-3	1.5E-3
4 - 30 days	1.3E-4	1.3E-4	8.0E-4	8.0E-4	1.0E-3

Notes:

1. These dispersion factors are to be used 1) for the time period preceding the isolation of the main control room and actuation of the emergency habitability system, 2) for the time after 72 hours when the compressed air supply in the emergency habitability system would be exhausted and outside air would be drawn into the main control room, and 3) for the determination of control room doses when the non-safety ventilation system is assumed to remain operable such that the emergency habitability system is not actuated.
2. These dispersion factors are to be used when the emergency habitability system is in operation and the only path for outside air to enter the main control room is that due to ingress/egress.
3. These dispersion factors are used for analysis of the doses due to a postulated small line break outside of containment. The plant vent and PCS air diffuser are potential release paths for other postulated events (loss-of-coolant accident, rod ejection accident, and fuel handling accident inside the containment); however, the values are bounded by the dispersion factors for ground level releases.

4. The listed values represent modeling the containment shell as a diffuse area source, and are used for evaluating the doses in the main control room for a loss-of-coolant accident, for the containment leakage of activity following a rod ejection accident, and for a fuel handling accident occurring inside the containment.
5. The listed values bound the dispersion factors for releases from the steam line safety & power-operated relief valves and the condenser air removal stack. These dispersion factors would be used for evaluating the doses in the main control room for a steam generator tube rupture, a main steam line break, a locked reactor coolant pump rotor, and for the secondary side release from a rod ejection accident. Additionally, these dispersion coefficients are conservative for the small line break outside containment.
6. The listed values bound the dispersion factors for releases from the fuel storage and handling area. The listed values also bound the dispersion factors for releases from the fuel storage area in the event that spent fuel boiling occurs and the fuel building relief panel opens on high temperature. These dispersion factors are used for the fuel handling accident occurring outside containment and for evaluating the impact of releases associated with spent fuel pool boiling.

5.2 Proposed Revisions to Tier 1 Table 5.0-1 Site Parameters

Revisions to Tier 1 Table 5.0-1

Table 5.0-1 (cont.) Site Parameters	
Soil	
Average Allowable Static Soil Bearing Capacity	Greater than or equal to 8,600 lb/ft^2 over the footprint of the nuclear island at its excavation depth
Maximum Allowable Dynamic Bearing Capacity for Normal Plus Safe Shutdown Earthquake (SSE)	Greater than or equal to 35~~120~~,000 lb/ft^2 at the edge of the nuclear island at its excavation depth
Shear Wave Velocity	Greater than or equal to 1~~8~~000 ft/sec based on low-strain, best-estimate soil properties over the footprint of the nuclear island at its excavation depth

5.3 Proposed Revisions to Subsection 3.8.5

Revise 3.8.5.4 as follows:

3.8.5.4 Design and Analysis Procedures

The seismic Category I structures are concrete, shear-wall structures consisting of vertical shear/bearing walls and horizontal floor slabs. The walls carry the vertical loads from the structure to the basemat. Lateral loads are transferred to the walls by the roof and floor slabs. The walls then transmit the loads to the basemat. The walls also provide stiffness to the basemat and distribute the foundation loads between them.

The design of the basemat consists primarily of applying the design loads to the structures, calculating shears and moments in the basemat, and determining the required reinforcement. For a site with hard rock below the underside of the basemat vertical loads are transmitted directly through the basemat into the rock. Horizontal loads due to seismic are distributed on the underside of the basemat resulting primarily in small membrane forces in the mat. The 6-foot-thick basemat is designed for the upward hydrostatic pressure due to groundwater reduced by the downward deadweight of the mat.

3.8.5.4.1 Analyses for Loads during Operation

The analyses of the basemat use the three-dimensional ANSYS finite element models of the auxiliary building and containment internal structures, which are described in subsection 3.7.2.3 and shown in Figures 3.7.2-1 and 3.7.2-2. The model considers the interaction of the basemat with the overlying structures and with the soil. Provisions are made in the model for two possible uplifts. One is the uplift of the containment internal structures from the lower basemat. The other is the uplift of the basemat from the soil.

The three-dimensional finite element model of the basemat includes the structures above the basemat and their effect on the distribution of loads on the basemat. ~~The finite element models of the auxiliary building above elevation 106′ and the containment internal structures inside containment are reduced to substructures (superelements) within ANSYS. These superelements are then included in the detailed finite model of the basemat, which includes the auxiliary building below elevation 106′ and the mat below the containment vessel.~~ The finite element model of the basemat and lower portion of the nuclear island is shown on ~~sheet 1 of~~ Figure 3.8.5-2.

~~The model of the basemat, including the superelements, is shown on sheet 2.~~

The subgrade is modeled with one vertical spring and two horizontal springs at each node of the basemat. The vertical springs act in compression only. The horizontal springs are active when the vertical spring is closed and inactive when the vertical spring lifts off. The vertical and horizontal stiffness of the springs represents a soft soil site and is conservative for firmer sites~~a rock foundation with a shear wave velocity of 8000 feet per second~~. Horizontal bearing reactions on the side walls below grade are conservatively neglected.

The nuclear island basemat below the containment vessel, and the containment internal structures basemat above the containment vessel, are simulated with solid tetrahedral elements. Nodes on the two basemats are connected with spring elements normal to the theoretical surface of the containment vessel.

Normal and extreme environmental loads and containment pressure loads are considered in the analysis. The normal loads include dead loads and live loads. Extreme environmental loads include the safe shutdown earthquake.

Dead loads are applied as inertia loads. Live loads and the safe shutdown earthquake loads are applied as concentrated loads on the nodes. The safe shutdown earthquake loads are applied as equivalent static loads using the assumption that while maximum response from one direction occurs, the responses from the other two directions are 40 percent of the maximum. Combinations of the three directions of the safe shutdown earthquake are considered.

Linear analyses are performed for all specified load combinations assuming that the soil springs can take tension. Critical load cases are then selected for non-linear analyses with basemat liftoff based on the results of the linear cases. The results from the analysis include the forces, shears, and moments in the basemat; the bearing pressures under the basemat; and the area of the basemat that is uplifted. Reinforcing steel areas are calculated from the member forces for each load combination case.

The required reinforcing steel for the portion of the basemat under the auxiliary building and under the shield building is determined by considering ~~both the reinforcement envelope for the linear analyses that do not consider liftoff and~~ the reinforcement envelope for the full non-linear iteration of the most critical load combination cases. Additional reinforcement is provided in the design of the 6' mat for soil sites such that the basemat can resist loads 20 percent greater than the demand calculated by the equivalent static acceleration analyses on uniform soil springs. This increase accommodates potential site specific lateral variability of the soil investigated separately in a series of parametric studies.

~~The required reinforcing steel for the portion of the basemat under the auxiliary building is calculated from shears and bending moments in the slab obtained from separate calculations. Beam strip models of the slab segments are loaded with the bearing pressures under the basemat from the three dimensional finite element analyses.~~ Figure 3.8.5-3 shows the basemat reinforcement.

3.8.5.4.2 Analyses of settlement during construction

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

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

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

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

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

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

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

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

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

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

- The design of the basemat and superstructure accommodates the construction-induced stresses considering the construction sequence and the effects of the settlement time history.
- The design of the basemat can accommodate delays in the shield building so long as the auxiliary building construction is suspended at elevation 117′ -0″. Resumption in construction of the auxiliary building can proceed once the shield building is advanced to elevation 100′ 0″.
- The design of the basemat can accommodate delays in the auxiliary building so long as the shield building construction is suspended at elevation 84′ -0″ feet. Resumption in construction of the shield building can proceed once the auxiliary building is advanced to elevation 100′ 0″.
- After the structure is in place and cured to elevation 100′ -0″, the basemat and structure act as an integral 40 foot deep structure and the loading due to construction above this elevation is not expected to cause significant additional flexural demand with respect to the basemat and the shield building concrete below the containment vessel. Accordingly, there is no need for placing constraints on the construction sequence above elevation 100′ 0″.

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

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

- Concrete may not be placed above elevation 84′ -0″ for the shield building or containment internal structure.
- Concrete may not be placed above elevation 117′ -6″ in the auxiliary building.

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

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

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

Revise 3.8.5.4.3 as follows:

3.8.5.4.3 Design Summary of Critical Sections

The basemat is designed to meet the acceptance criteria specified in subsection 3.8.4.5. Two critical portions of the basemat are identified below together with a summary of their design. The boundaries are defined by the walls and column lines which are shown in Figure 3.7.2-12 (sheet 1 of 12). Table 3.8.5-3 shows the reinforcement required and the reinforcement provided for the critical sections.

[*Basemat between column lines 9.1 and 11 and column lines K and L*

This portion of the basemat is designed as a two way slab with the shorter direction~~as a one way slab~~ 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 ~~bearing pressures and membrane~~member forces from the analyses]* described in subsection 3.8.5.4.1. ~~[*Negative moments are redistributed as permitted by ACI 349.*~~ *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*

This portion of the basemat is designed *as a two way slab with the shorter direction* ~~*as a one way slab*~~ *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* ~~*bearing pressures and membrane*~~*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.*]*

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.

- The structural design meets the acceptance criteria specified in Section 3.8
- The amplitude of the seismic floor response spectra do not exceed the design basis floor response spectra by more than 10 percent

Depending on the extent of the deviations, the evaluation may range from documentation of an engineering judgement to performance of a revised analysis and design.

Revise 3.8.5.5.1 as follows:

3.8.5.5.1 Nuclear Island Maximum Bearing Pressures

The ~~hard rock~~ foundation will be demonstrated to be capable of withstanding the bearing demand from the nuclear island as described in subsection 2.5.4.5.6.

Revise 3.8.5.5.3 as follows:

3.8.5.5.3 Sliding

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

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

where:

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

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

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

where:

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

The sliding resistance is based on the friction force developed between the basemat and the foundation. The governing friction value at the interface zone is a thin soil layer below the mud mat with an angle of internal friction of 35 degrees giv~~us~~ing a coefficient of friction of 0.~~55~~70. The effect of buoyancy due to the water table is included in calculating the sliding resistance.

Add reference 46 as follows:

3.8.7 References

46. ASCE Standard ASCE/SEI 43-05, "Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities, 2005~~Deleted.~~

Revise Tables 3.8.5-2 and 3.8.5-3 as shown on next pages
Revise Figure 3.8.5-2 (Sheet 1 of 2) to Figure 3.8.5-2 and delete Figure 3.8.5-2 (Sheet 2 of 2)
Replace Figures 3.8.5-3 (5 sheets) by the following 7 sheets.

Table 3.8.5-2 **FACTORS OF SAFETY FOR FLOTATION, OVERTURNING AND SLIDING OF NUCLEAR ISLAND STRUCTURES** **HARD ROCK CONDITION**	
Environmental Effect	**Factor of Safety(1)**
Flotation	
High Ground Water Table	3.7
Design Basis Flood	3.5
Sliding	
Design Wind, North-South	~~18.4~~23.2
Design Wind, East-West	~~14.0~~17.4
Design Basis Tornado, North-South	~~10.3~~12.8
Design Basis Tornado, East-West	~~8.6~~10.6
Safe Shutdown Earthquake, North-South	1.28 (2)~~5~~
Safe Shutdown Earthquake, East-West	1.34 (2)~~5~~
Overturning	
Design Wind, North-South	51.5~~7~~
Design Wind, East-West	27.9~~8.0~~
Design Basis Tornado, North-South	17.7
Design Basis Tornado, East-West	9.6
Safe Shutdown Earthquake, North-South	1.39~~75~~
Safe Shutdown Earthquake, East-West	1.07 (3)~~2~~

Note:

1. ~~1.~~ Factor of safety is calculated for the soil and rock ~~a~~ sites described in subsection 3.7.1.4. Minimum value for all sites is shown in this table. ~~with rock below the underside of the base mat (elevation 60′ 6″) and soil adjacent to the exterior walls above this elevation~~.
2. Factor of safety is shown for soils below and adjacent to nuclear island having angle of internal friction of 35 degrees.
3. ~~2.~~3. The factor of safety of 1.07 does not consider active and passive soil pressures on the external walls below grade. When these soil pressures are considered for overturning (as they are in the sliding evaluation), the minimum factor of safety against overturning increases to 1.12. This factor of safety meets the requirement of 1.1 based on the conservative moment balance formula treating the seismic moment as static loads. ASCE/SEI 43-05, Reference 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).

Table 3.8.5-3

[*DEFINITION OF CRITICAL LOCATIONS AND THICKNESSES FOR NUCLEAR ISLAND BASEMAT*[1]]*

Wall or Section Description	*Applicable Column Lines*	*Applicable Elevation Level or Elevation Level Range*	*Concrete Thickness*[2]	*Reinforcement Required Vertical* (in^2/ft^2)[3]	*Reinforcement Required Horizontal* (in^2/ft)[3]	*Reinforcement Provided Vertical* (in^2/ft^2)[4]	*Reinforcement Provided Horizontal* (in^2/ft)[4]
Auxiliary Building Basemat							
Auxiliary Basemat Area	*Column line K to L and from Col. Line 10 to 11 ~~wall to the intersection with the shield building~~*	*From level 0 to 1*	*6'-0"*	***Shear Reinforcement*** *0.23~~9~~*	***Bottom Reinforcement*** *1.6~~6~~ (East-West Direction)* ***Top Reinforcement*** *1.5~~6~~ (East-West Direction)*	***Shear Reinforcement*** *0.25~~30~~*	***Bottom Reinforcement*** *2.25~~7~~ (East-West Direction)* ***Top Reinforcement*** *2.25~~7~~ (East-West Direction)*
Auxiliary Basemat Area	*Column line 1 to 2 and from Column Line K-2 to N wall*	*From level 0 to 1*	*6'-0"*	***Shear Reinforcement*** *0.47~~9~~*	***Bottom Reinforcement at column line 2*** *2.25~~8~~ (North-South Direction)* ***Top Reinforcement at mid-span*** *2.7~~9~~ (North-South Direction)*	***Shear Reinforcement*** *0.50~~78~~*	***Bottom Reinforcement*** *2.25~~4.5~~ (North-South Direction)* ***Top Reinforcement*** *3.25~~12~~ (North-South Direction)*

Notes:

1. *The applicable column lines and elevation levels are identified and included in Figures 1.2-9, 3.7.2-12 (sheets 1 through 12), 3.7.2-19 (sheets 1 through 3) and on Table 1.2-1.*
2. *These thicknesses have a construction tolerance of +1 inch, -3/4 inch.*
3. *These concrete reinforcement values represent the minimum reinforcement required for structural requirements except for designed openings, penetrations, sumps or elevator pits.*
4. *These concrete reinforcement values represent the provided reinforcement for structural requirements except for designed openings, penetrations, sumps or elevator pits.*

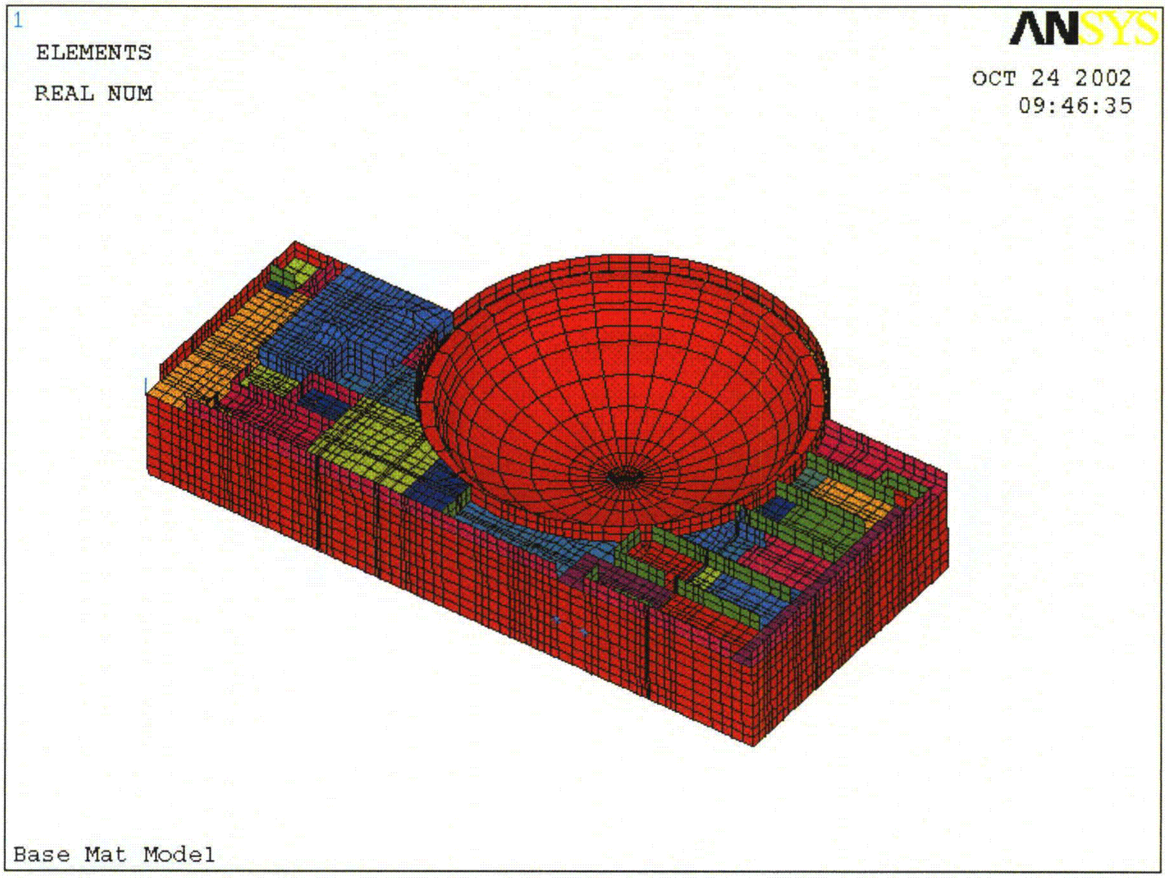


Figure 3.8.5-2 ~~(Sheet 1 of 2)~~

Isometric View of Basemat in Finite Element Model

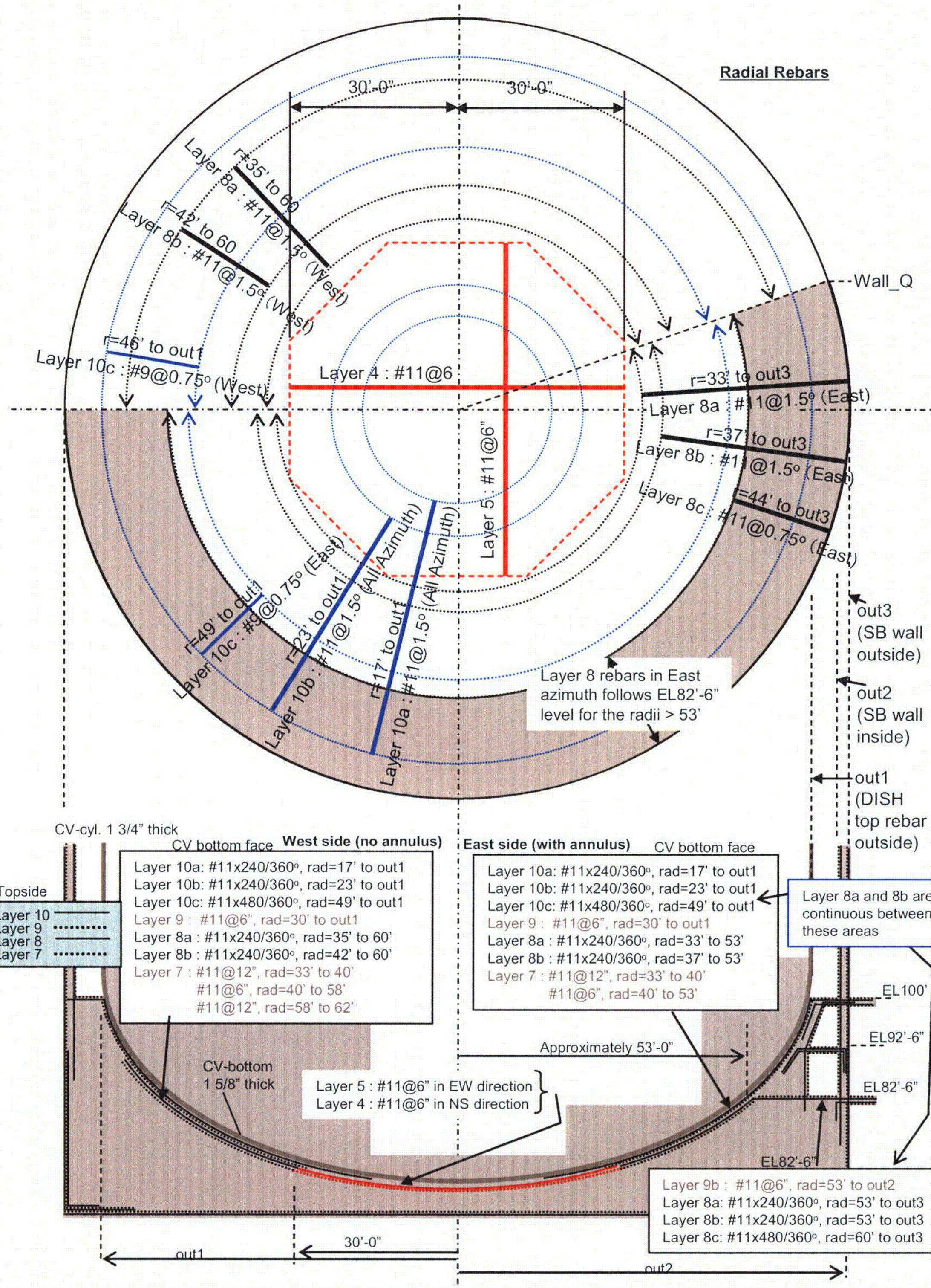


Figure 3.8.5-3 (Sheet 1 of 7)

Radial Reinforcement, Top side of DISH

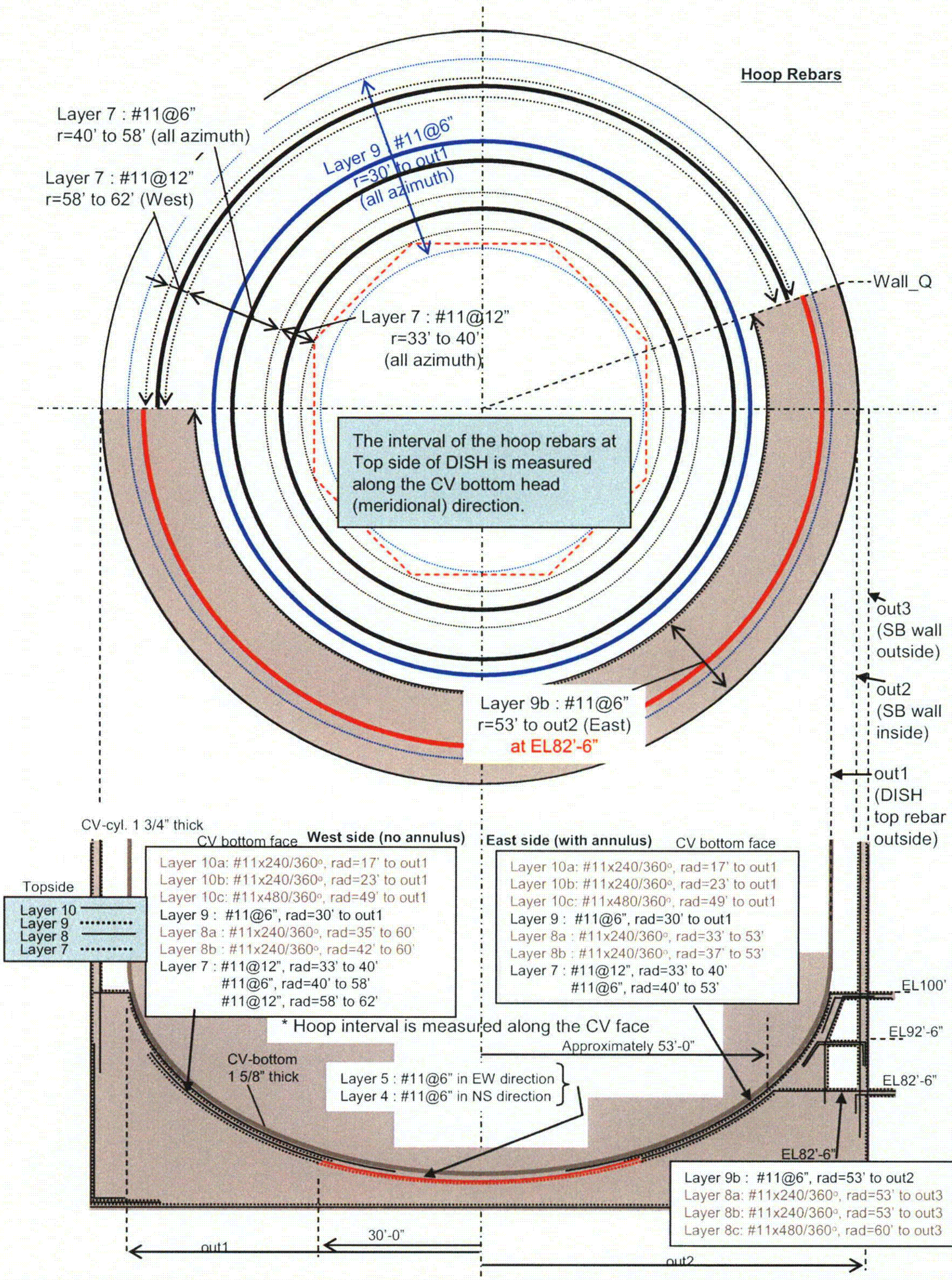
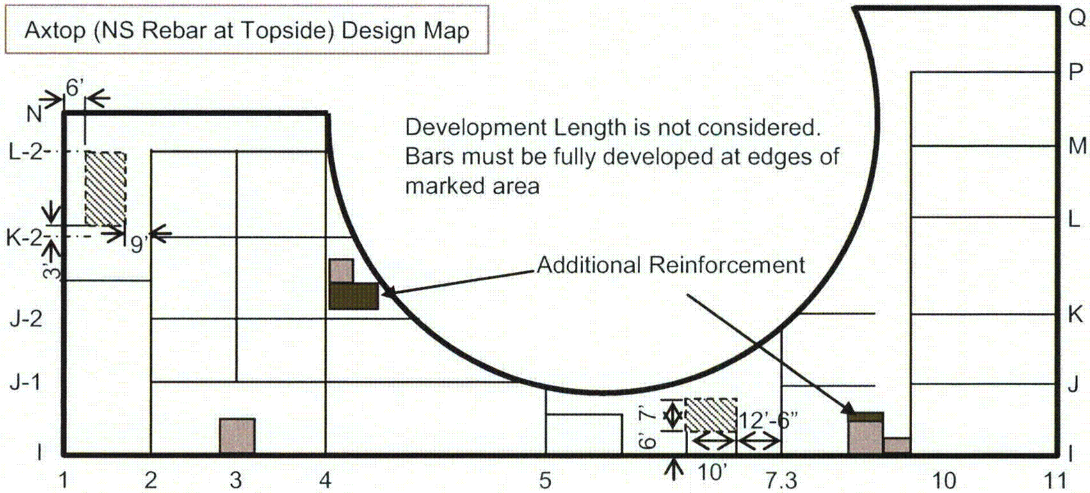


Figure 3.8.5-3 (Sheet 2 of 7)

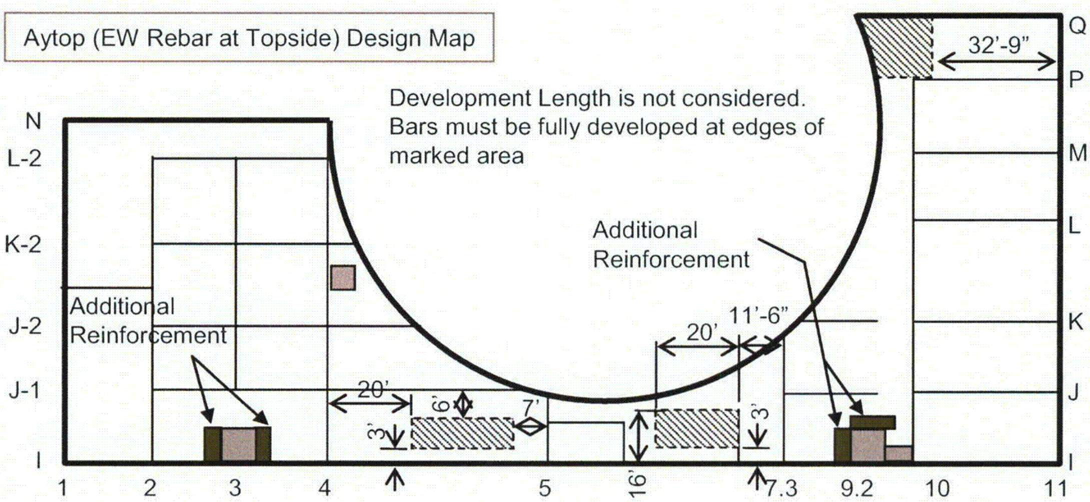
Circumferential Reinforcement, Top side of DISH



Rebar arrangement	: #14@12"	: #14@12"+#9@12"	: #11@12"
Location	Layer 4		Top side of the 1.5' & 3' thick Pits

Figure 3.8.5-3 (Sheet 3 of 7)

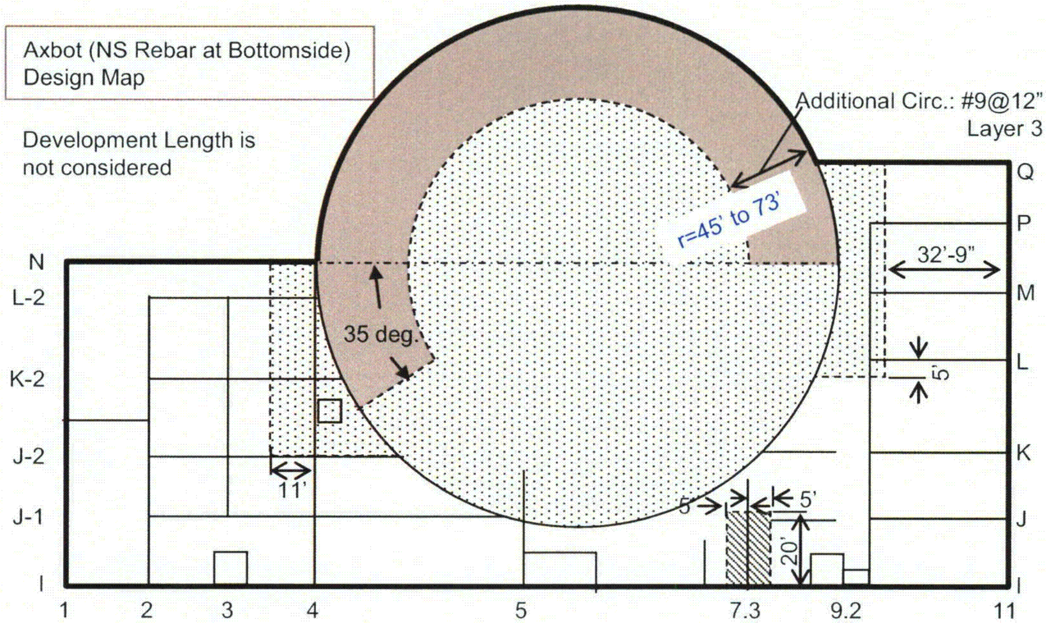
Longitudinal Reinforcement Map, Top side in NS direction



Rebar arrangement	: #14@12"	: #14@12"+#9@12"	: #11@12"
Location	Layer 5		Top side of the 1.5' & 3' thick Pits

Figure 3.8.5-3 (Sheet 4 of 7)

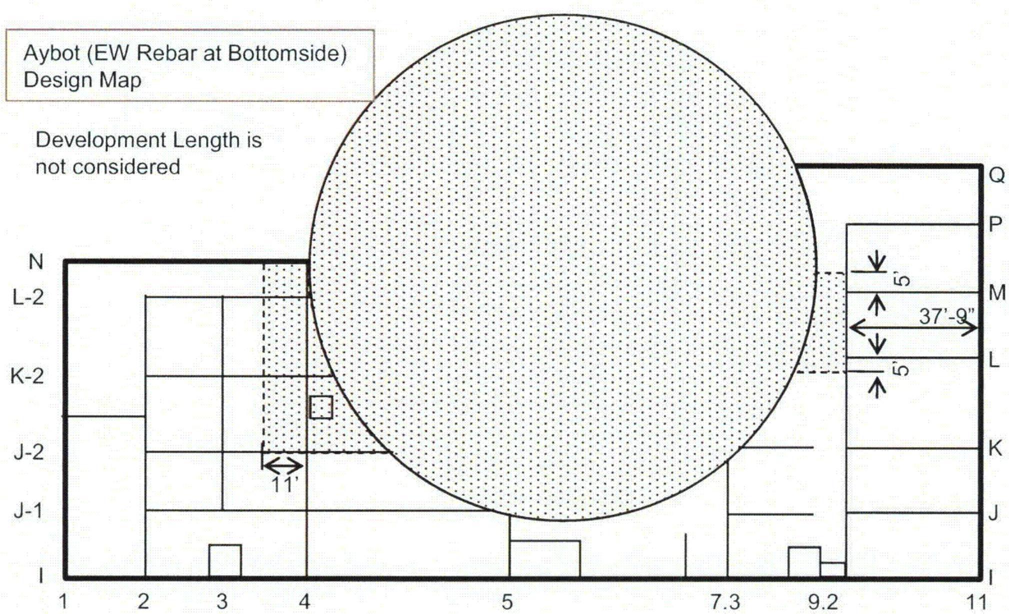
Longitudinal Reinforcement Map, Top side in EW direction



Rebar arrangement	: #14@12"	: #14@12"+#9@12"	: #14@6"	: #14@6"+#9@12"
Location	Layer 1			Layer 3

Figure 3.8.5-3 (Sheet 5 of 7)

Longitudinal Reinforcement, Bottom side of DISH and 6' basemat (NS)



Rebar arrangement	: #14@12"	: #14@12"+#9@12"	: #14@6"
Location	Layer 2		

Figure 3.8.5-3 (Sheet 6 of 7)

Longitudinal Reinforcement, Bottom side of DISH and 6' basemat (EW)

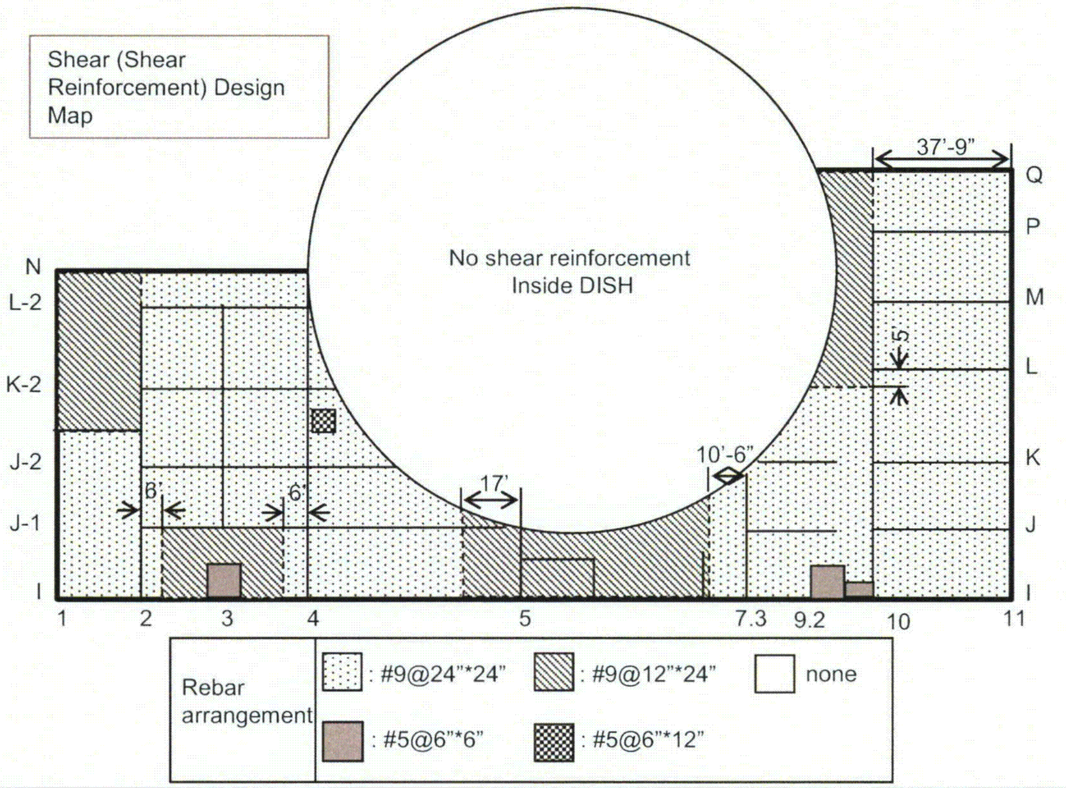


Figure 3.8.5-3 (Sheet 7 of 7)

Shear Reinforcement Map