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Document Control Desk
U.S. Nuclear Regulatory Commission
Washington, DC 20555

ATTENTION: T. R. QUAY

SUBJECT: REQUESTS FOR ADDITIONAL INFORMATION ON SSAR CHAPTER 2

Attached are responses to five requests for additional information related to site requirements included in your letter dated April 25, 1997. Attachment 1 provides responses for RAIs 231.35, 231.36, 231.37, 231.38 and 231.39.

Subsection 2.5.4.5 has been reorganized in response to RAI 231.37. Attachment 2 provides the changes in Revision 13 of SSAR Section 2.5. For convenience the revisions made in response to these five RAIs are shown in this attachment which is referenced from the responses in attachment 1.

RAI 231.38 requested additional information on the analyses of the AP600 during construction. This is outlined in the proposed SSAR revision provided in the RAI response. A summary of the analyses is also provided in Attachment 3. This summary will be incorporated in the nuclear island basemat summary design report. A draft of this report was reviewed by NRC staff during the audit last December. RAI 231.38 also includes a markup of SSAR Section 3.8.5.4.3 and Figures 3.8.5-3, Sheets 1 and 4.

The NRC April 25, 1997 letter also includes additional questions related to DSER open items 2.5.4.3-2 and 2.5.4.4-1. The additional questions about DSER open item 2.5.4.3-2 are about settlement and construction issues and are addressed in the RAI responses and the construction analysis summary. DSER Open Item 2.5.4.4-1 has been closed previously.

The open items addressed by these letters are as follows:

OITS Number	DSER or Other Item Number	Westinghouse Status
547	DSER 2.5.4.3-2	Action N
5229 - 5233	RAIs 231.35 to 231.39	Action N

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If you have any questions please contact Donald A. Lindgren at (412) 374-4856.

CLHaag for BAM

Brian A. McIntyre, Manager
Advanced Plant Safety and Licensing

jml

Attachment

cc: D. T. Jackson, NRC (w/Attachments)
N. J. Liparulo, Westinghouse (w/o Attachments)

Attachment # 1

RESPONSES TO NRC REQUESTS FOR ADDITIONAL INFORMATION
231.35 - 231.39

RAI # 231.35

Section 2.5.4.5.2.1 (page 2-11, 3rd line from bottom) of the SSAR (revised after Revision 11) states that a series of borings should be drilled on a grid pattern that encompasses the nuclear island footprint and 40 feet beyond the boundary of the footprint. The basis for the proposed 40-foot limit should be explained. The limit should be about one-third to one-half of the length/width of the nuclear island (which measures 256 feet in length and about 160 feet in width). This is RAI #231.35 in SSAR 2.5.4.5.

Westinghouse Response

The average width of the nuclear island footprint is 127 feet. The equivalent rectangular footprint having the same overturning stiffness as the AP600 has a width of 140 feet. The 40-foot extension for the grid of borings was established on the basis of an approximate zone of influence of the foundation mat. The extension is approximately equal to one-third of the equivalent east-west width. The extension to the north and south was taken to be the same on the basis that the stresses induced into the foundation media in the 40-foot wide north-south extension zone will be less than the stresses induced in the extended zone to the east and west.

SSAR Revision

See subsection 2.5.4.5.1 in Revision 13.

RAI #231.36

Section 2.5.4.5.2.1 (page 2-12, first paragraph, 10th line) states that at least one-fourth of the primary borings should penetrate sound rock, or for deep soil sites, to a maximum depth, d_{max} taken as the depth at which the vertical stress during or after construction for the combined foundation loading is less than 10-percent of in situ effective overburden stress. Other borings may terminate at a depth of 160 feet below the foundation (equal to the width of the structure). This SSAR commitment of Westinghouse is not acceptable because the depth at which the borings are stopped should depend on the suspected presence or absence of compressible materials or the suspected presence of voids (i.e. sinkhole, etc.) below the nuclear island footprint. The 160-foot limit should be changed to at least 200 feet (which is approximately equal to the "side" of the equivalent square of the nuclear island footprint). This is RAI #231.36 in SSAR 2.5.4.5.

Westinghouse Response

Consistent with the response for RAI # 231.35, the influence of the nuclear island is expected to extend down to a depth approximately equal to the width of the foundation. The depth of 160 feet for the standard borings is in excess of this depth and accordingly below the zone of influence of the mat.

The presence of compressible materials and voids below this depth is not expected to affect the response of the nuclear island. The geotechnical investigation is preceded by a local and regional geologic investigation. As described in SSAR subsection 2.5.4.5.2, investigation effort would be extended 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.

SSAR Revision

See subsection 2.5.4.5.2 in Revision 13.

RAI #231.37

A review of pages 2-12 and 2-13 in Section 2.5.4.5.2.1 (revised after Revision 11) indicates that, to establish the "uniformity" of a site, there are three criteria that the site must satisfy: (1) the uniformity of the layer thickness (layers must be uniform), (2) the dip angle of the layer (maximum 20 degrees), and (3) uniformity of shear wave velocity within any layer (variation must be less than 10 or 20 percent of the layer average). In addition, there seems to be two other criteria discussed in the third and fourth paragraphs of page 2-12, and in pages 2-14 and 2-15: (4) the depth of a given layer must not deviate by more than 5 percent of the depth of the "best estimate" plane for the layer, and (5) any undulatory bed rock must be at least 40 feet below the bottom of the basemat. Westinghouse should clearly state these five acceptance criteria together in the SSAR. The lengthy discussion of the draft revision is very confusing and is likely to lead to a misinterpretation. The procedure for establishing the acceptability of AP600 design for non-uniform sites should also be established. In addition, for the site to be acceptable as an uniform site, the last paragraph of page 2-12 of the revised SSAR states that the variation of the shear wave velocity in the material below the foundation to a depth of 80 feet below the basemat within the footprint of the plant shall meet the criteria specified on Page 2-13 of the revised SSAR. Westinghouse should justify the basis for the 80-foot limit. This is RAI #231.37 in SSAR 2.5.4.5.

Westinghouse Response

In response to NRC's RAI #231.37, subsection 2.5.4.5 is revised. Subsections 2.5.4.5.1 and 2.5.4.5.2 identify the required site investigations. The procedure for establishing uniformity and acceptability of nonuniform sites is outlined in subsection 2.5.4.5.3 of the SSAR.

SSAR Appendix 2A describes studies on the effect of depth to bedrock. The design profiles assume bedrock at a depth of 120 feet since this case was found to be the most conservative. The depth of 120 feet corresponds to the 80 feet below the foundation mat previously used in the criteria for uniform soils. For consistency the SSAR section on nonuniformity has been revised to express the depth relative to grade rather than below foundation level.

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 farther that a non-uniform layer is located below the foundation, the less influence it has on the bearing pressures at the basemat. The stratigraphy and dynamic characteristics of soil deposits more than 80 feet below the mat are observed by analysis to have negligible effects on the soil structure interaction analyses and on the subgrade modulus. Hence the requirement that the variation in shear wave velocity across the nuclear island footprint need only be demonstrated for the soil layers within 120 feet of grade.

SSAR Revision

See subsection 2.5.4.5 in Revision 13.

RAI #231.38

The statement made in Section 3.8.5.4.3 of the SSAR (revised after Revision 11) concerning the construction-induced stresses is not acceptable. During the previous review meetings, the staff has indicated that the basemat stresses induced by construction settlements can be additive to the basemat stresses induced by other design basis loads. It is not proper to treat these stresses as secondary or self-relieving stresses. The settlement-induced stresses can be additive at some locations depending on the construction sequence remaining, the geometry of the structure, and the sense of the induced moments and shears developed in the basemat. In the December 9 through 13, 1996, meeting, Westinghouse was requested to provide information on those issues typically encountered during construction of large structures (stress relief and expansion due to excavation, effective stress increase and settlements from dewatering effects, and long term consolidation effects on the settlement time history). The staff also requested Westinghouse to provide a possible use of a limitation on the anticipated construction for definition of an adequate site. However, the information has yet to be provided. In addition, the analyses performed by Westinghouse are based on two-dimensional analyses and only considered the effect of immediate settlements on construction-induced stresses. Even then, Westinghouse's calculations indicated that these stresses are sensitive to the particular sequence of construction assumed. The effects of settlement time history were not evaluated. Furthermore, the conversion of two dimensional to three dimensional (real world) effects used an unusually large factor to reduce the predicted bending moments and shears of the basemat without a proper justification. The adequacy of using this reduction factor needs to be demonstrated by Westinghouse. This is RAI #231.38 in SSAR 2.5.4.3.

Westinghouse Response

Following the meetings in December, 1996 analyses during construction have been performed that include the effects of both short and long term settlement during and subsequent to construction. Subsections 2.5.4.3 and 3.8.5.4.3 have been expanded to include a description of the settlement evaluation, and the associated construction-induced stresses, respectively. This information replaces the material in Revision 5 of the SSAR which did not address long term settlement.

Stresses have been determined for critical construction sequences including the effects of short term and long term settlements. The analyses show that the stresses in the reinforcement at each stage of construction are well below yield, therefore limiting the crack widths to acceptable magnitudes. Bending moments and shear forces in the basemat during construction satisfy the ACI 349 strength criteria using a load factor of 1.4.

As stated in the RAI, the settlement-induced stresses can be additive at some locations depending on the construction sequence remaining, the geometry of the structure, and the sense of the induced moments and shears developed in the basemat. These stresses have been considered for construction loads as described above. The basis for treating these stresses as secondary or self-relieving stresses for design basis loads is discussed below for two typical locations in the basemat. If there are stresses locked-in during construction, the later construction that locks in these stresses contributes significant strength to the composite completed structure. As given in Chapter 17 of ACI 349, the structure is designed for design basis loads considering the strength of the composite section.

- Maximum stresses in the flexural reinforcement of the basemat during early construction occur in the bottom reinforcement in the north-south direction adjacent to the shield building on the north side. These stresses may be locked in by subsequent construction activities. This portion of the basemat acts as the bottom flange of the superstructure in the completed structure and may see additional in-plane loads. Membrane strain associated with these loads will relieve the compression stress in the top face while slightly increasing the stress in the bottom reinforcement. The strength of the section for membrane loads is established based on the strength of the top and bottom reinforcement with the reinforcement at a strain of 0.003 in accordance with ACI 349. At this strain in the reinforcement both the top and bottom reinforcement are at yield and the strength of the section is not affected by the initial locked in stresses.

- The primary reinforcement in the basemat on the north side of the auxiliary building for design basis loads is in the east-west direction where the basemat is designed to span between the shear walls. The primary reinforcement in the basemat on the south end of the auxiliary building for design basis loads is in the north-south direction where the basemat is designed to span between the shear walls. The stresses during early stages of construction are small in these locations and are generally not locked in by the construction of the shear walls perpendicular to the direction of span.

SSAR Revisions

See subsection 2.5.4.3 in Revision 13 for additional information on the settlement during construction. Revisions to subsection 3.8.5.4.3 and Figure 3.8.5-3 are shown below.

3.8.5.4.3 Analysis for Loads 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 basemat 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. Locked-in ~~stresses~~ strains during construction become secondary after completion of the auxiliary building walls. They do not reduce the strength of the section and need not be included in the design load combinations for the completed structure.

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 against the vertical sides of the excavation 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, the subsurface rebounds, and the moments in the 6-foot basemat decrease.

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 loading due to construction above this elevation will not result in 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 AP600 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.

~~Alternate construction scenarios and schedules are analyzed to confirm the adequacy of the basemat for unexpected changes in the construction plan.~~ These 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 ~~100~~82′ 6″ for the shield building or containment internal structure.
- Concrete may not be placed above elevation 117′6″ in the auxiliary building.

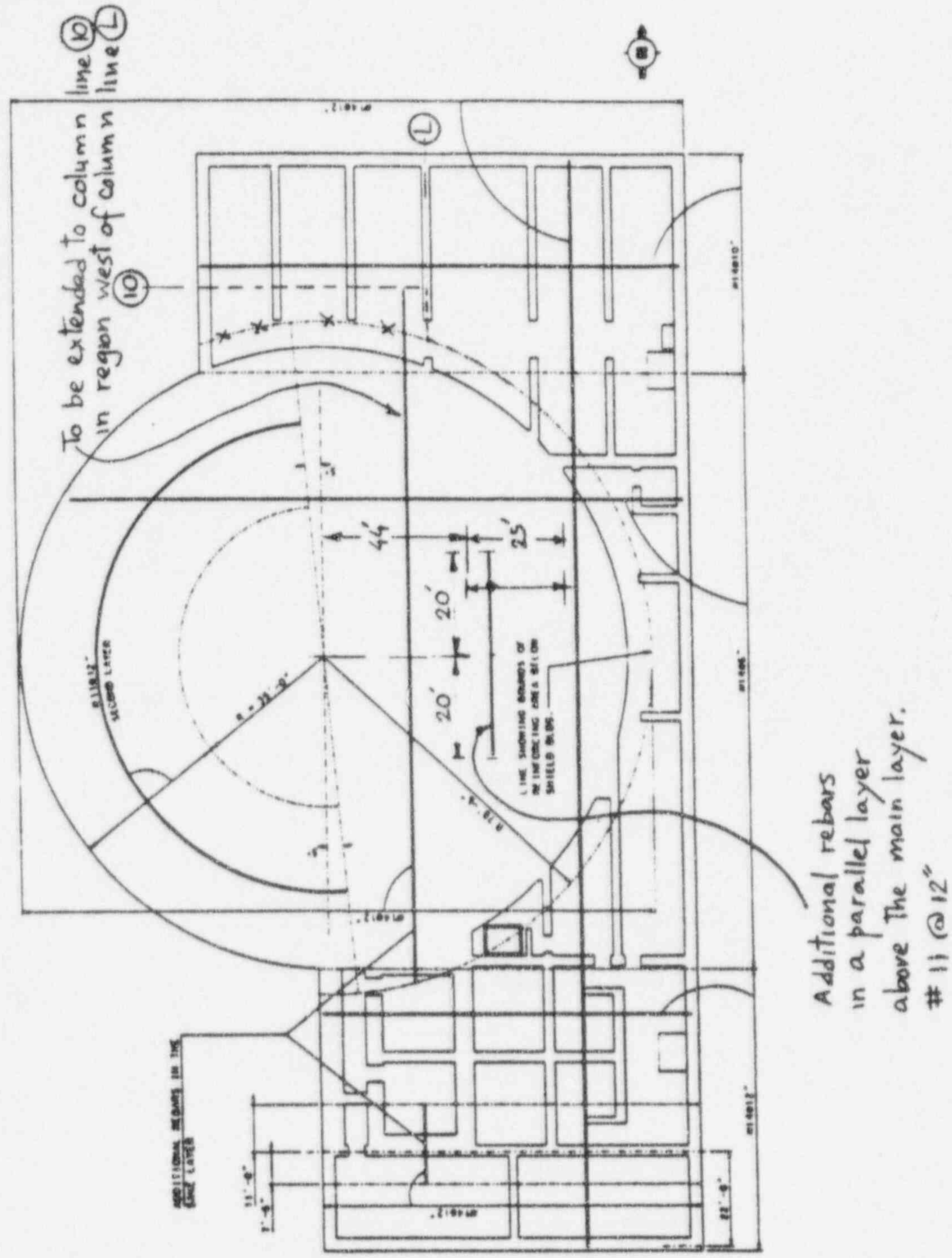

Figure 3.8.5-3 (Sheet 1 of 5)

Basemat Reinforcement – Bottom Face

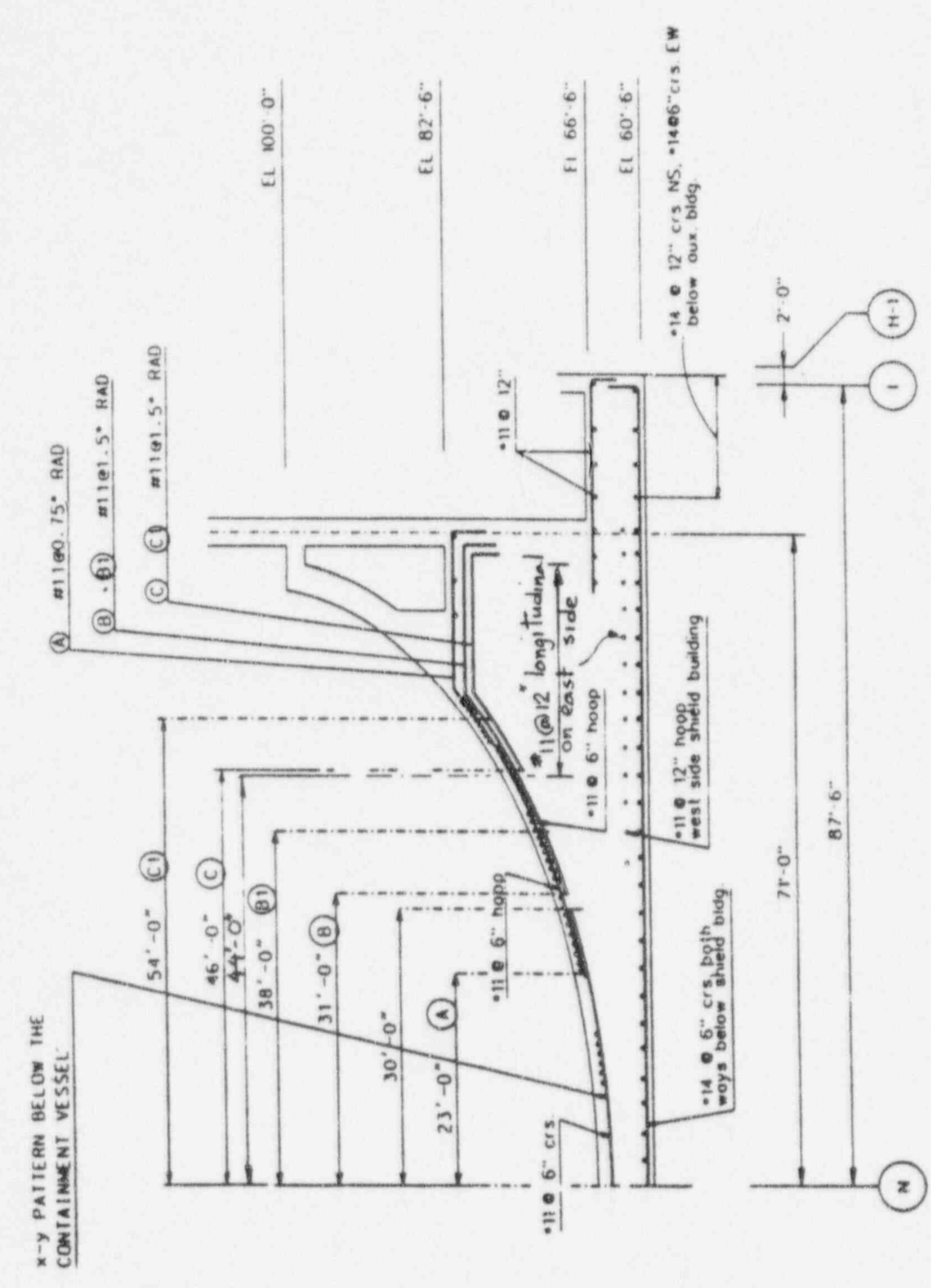

Figure 3.8.5-3 (Sheet 4 of 5)

Basemat Reinforcement – Cross Section

RAI #231.39

In Section 2.5.4.5.2.2 of the SSAR (revised after Revision 11), Westinghouse indicates that if a site is classified as non-uniform based on the criteria listed on the top of Page 2-13, the investigative effort should be extended in such a way that the site may be demonstrated to be acceptable for AP600 by showing that the in-structure response spectra are enveloped by the design instructure response spectrum envelopes. However, it should be clearly stated in the SSAR that the demonstration must specifically include a complete reevaluation of the soil-structure interaction effects for this non-uniform site, because all soil-structure interaction analyses (2D or 3D) performed by Westinghouse were based on uniformly bedded site profiles. The staff, in several review meetings, has raised the concern regarding how the effect of local hills and valleys of the bed rock (or competent material) need to be included in the evaluation. The staff's concern is that these non-uniform conditions would serve to change the input free-field ground motions coming into the site (e.g., local amplification effects). This is RAI #231.39 in SSAR 2.5.4.5.

Westinghouse Response

Subsection 2.5.2.1 of the SSAR (see Revision 13 in response to RAI 231.37) has been revised to show that the site be such that it is adequately represented by the standard horizontal layering used in soil structure interaction analyses.

Topographic features such as mountains and valleys may affect the input free field motion at the plant site by a focusing or divergence of the seismic waves transmitted from the source to the site. These effects generally depend on the size and geometry of the surface feature in relation to the wave type, angle of incidence and wave length. Based on comparison with elastic half-space solutions, NUREG/CR 0693 concludes that "....Potential influence of surface features such as mountains and canyons, on the seismic input motion appeared to be of much less significance compared to other assumptions used in soil structure interaction analysis." In certain frequency ranges the changes in bedrock input motion are on the order of 5 percent. Although for limited frequencies these changes may be 50 percent, uncertainties in source parameters, travel paths and wave types affecting the entire frequency range of interest overshadows the variabilities at limited frequencies.

If topographic features such as mountains and valleys are sufficiently close, their influence would be considered in the development of the site specific free field motion at the plant site. The acceptability for the AP600 at such a site is demonstrated by comparison of the site specific free field spectra to the AP600 design spectra (see SSAR Table 2-1).

SSAR Revision: None

ATTACHMENT #2

MARKUP OF SSAR SECTION 2.5

2.4.1.3 Cooling Water Supply

Combined License applicants will address the water supply sources to provide makeup water to the service water system cooling tower.

2.4.1.4 Groundwater

Combined License applicants referencing the AP600 certified design will address site-specific information on groundwater. No further action is required for sites within the bounds of the site interface criteria.

2.4.1.5 Accidental Release of Liquid Effluents in Ground and Surface Water

Combined License applicants referencing the AP600 certified design will address site-specific information on the ability of the ground and surface water to disperse, dilute, or concentrate accidental releases of liquid effluents. Effects of these releases on existing and known future use of surface water resources will also be addressed.

2.4.1.6 Emergency Operation Requirement

Combined License applicants referencing the AP600 certified design will address any flood protection emergency procedures required to meet the site flood level interface.

2.5 Geology, Seismology, and Geotechnical Engineering

Combined License applicants referencing the AP600 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. ~~Figure 2.5-1 provides a flow chart for site qualification.~~

2.5.1 Basic Geological and Seismic Combined License Information

Combined License applicants referencing the AP600 certified design will address the following site-specific geologic and seismic information:

- regional and site physiography,
- geomorphology,
- stratigraphy,
- lithography,
- structural geology,
- tectonics, and
- seismicity.

2.5.2 Vibratory Ground Motion

The AP600 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. The AP600 design response spectra are developed using the Regulatory Guide 1.60 response spectra as the base and modified to address high frequency amplification effects observed in east coast earthquakes. The maximum 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 AP600 certified design will address the following site-specific information related to seismic and tectonic characteristics of the site and region:

- correlation of earthquake activity with geologic structure or tectonic provinces,
- maximum earthquake potential,
- seismic wave transmission characteristics of the site,
- safe shutdown earthquake (SSE) ground response spectra.

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

- The free field peak ground acceleration at the finished grade level is less than or equal to a 0.30g safe shutdown earthquake, and,

- The site design response spectra at the 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.

- 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 Alternate Site-Specific Seismic Response Design Basis

The AP600 ~~certified~~ design may be located on sites that are outside the bounds of the site parameters for seismic and soil conditions in Table 2-1. The evaluation for the suitability of these sites is based on the design basis outlined below and would be submitted AND REVIEWED BY THE NRC as part of the Combined License application. Figure 2.5-1 provides a flow chart for alternate site qualification.

Site-specific soil structure interaction analyses may be performed by the Combined License applicant to demonstrate acceptability. These analyses would use the site specific soil conditions (including variation in soil properties in accordance with Standard Review Plan 3.7.2) and site specific safe shutdown earthquake. The three components of the site specific ground motion time history must satisfy the 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 and lateral earth pressures determined from the site specific analyses should be compared against the design basis of the AP600 as described below.

The floor response spectra at 5 percent damping at the following four locations should be compared. The site is acceptable if the peaks of the floor response spectra from the site-specific analyses do not exceed the AP600 spectra by more than 10 percent at any frequency.

- Reactor vessel support — Figure 3.7.2-17, Sheets 1-3
- Containment operating floor — Figure 3.7.2-17, Sheets 4-6
- Shield building roof — Figure 3.7.2-15, Sheets 7-9
- Control room floor — Figure 3.7.2-15, Sheets 1-3

Lateral earth pressures from the site specific analyses should be compared against the design values given in Table 2C-1 through 2C-4. The site is acceptable if the lateral earth pressures from the site-specific analyses do not exceed the AP600 design values at any location by more than 10 percent.

2.5.3 Surface Faulting Combined License Information

Combined License applicants referencing the AP600 certified design will address surface and subsurface geological and geophysical information including the potential for surface or near-surface faulting affecting the site.

2.5.4 Stability and Uniformity of Subsurface Materials and Foundations

2.5.4.1 Excavation

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 are 60% to 70% 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' deep on projects in the U.S. Soils have been retained for up to 90' 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 of the AP600 is about 8,000 pounds per square foot. The minimum average allowable static soil bearing capacity is 8,000 pounds per square foot over the footprint of the nuclear island at its excavation depth (see Table 2-1). Net allowable static bearing capacities have been computed for the design soil profiles as shown in Table 2-2. Capacities are calculated using bearing capacity equations in Terzaghi and Peck (Reference 1), for both cohesive and cohesionless soils (both dry and saturated cases).

For cohesive soils, an estimate for undrained shear strength (S_u) was made by using the relationship between low strain shear modulus (G_{max}) and undrained shear strengths. The shear modulus was obtained from the shear wave velocity profiles at a depth of approximately 90 feet. This corresponds to a depth of D+B/2 (Depth, D = 40 feet; Width, B = 104 feet, average) which accounts for the zone of influence under the nuclear island basemat. The water table has been shown to have no effect on the bearing capacity of mats on cohesive soils. For cohesionless soils, relative density and friction angle were calculated from their relationships with shear wave velocity and low strain shear modulus. Location of the ground water table significantly influences the bearing strength of cohesionless soils. In determining the bearing strengths, the ground water table was assumed to be at grade. For the rock profiles, the bearing strengths shown are based on the rock quality designation in accordance with Peck et al. (Reference 2).

In general, higher bearing capacities are associated with more competent soil profiles. For selected soft soil profiles in cohesive soils, soil improvement techniques may be employed to improve the bearing strength. The bearing capacities provided in Table 2-2 are preliminary estimates for static loading conditions only. The Combined License applicant will perform field and laboratory investigations to establish the material type and the associated strength parameters in order to determine the site-specific bearing capacity value.

Generally, once the static bearing capacity at a given site is adequate, the dynamic bearing demand will also be satisfied. For soft sites, site-specific SSI analysis may provide a more reasonable dynamic bearing demand as compared to the enveloping bearing demand.

2.5.4.3 Settlement

Short-term (elastic) and long-term (heave and consolidation) settlement for limiting cases of deep soft soil sites are evaluated 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. The basemat and structure are analyzed at various stages of construction as described in subsection 3.8.5.

The settlement analysis utilizes the one-dimensional consolidation theory in which excess pore pressure is dissipated consistent with the site consolidation parameters such as the initial void ratio, compression and recompression index and the coefficient of consolidation. The limiting cases of deep soft soil sites comprised of compressible soils are represented by subsurface profiles consisting of compressible clay deposits extending down to a depth of 360 feet underlying a 40-foot layer of sand at the surface. The evaluation considers two profiles. One profile has alternate layers of sand and clay and the second profile consists of only clay. Profile 1 maximizes settlements in the early stages of construction while profile 2 maximizes settlement during the later stages of construction and during the operational period of the plant. The elastic properties for the soils are consistent with the minimum shear wave velocity of Table 2-1 and the expected soil strains due to construction loads. The clay is assumed to be normally consolidated and the water table is assumed to be at grade.

The analysis considers the effects of dewatering and excavation, the history of construction loading, elastic deformation and consolidation of the subsurface soils, and the effect of the progressive stiffness of the structure. For the limiting deep soft soil sites examined, the maximum estimated settlement after placement of first concrete for the basemat is 4.5 inches for the postulated alternating sand and clay site and 14 inches for the all clay site.

~~Short term and long term heave and settlement of the nuclear island foundation have been evaluated during construction and operation for two deep soil profiles with soil properties satisfying the lower end of the soil shear wave velocity design parameter. One site has alternating sand and normally consolidated clay layers with a high water table. The other has only the normally consolidated clay with a high water table. The selected soil profiles maximize the rate of settlement in one case and the total settlement in the other case. These analyses are described in subsection 3.8.5 and evaluate forces in the basemat and superstructure during construction. The maximum settlement after placement of first concrete for the basemat is 4 inches for the alternating sand and clay site and 12 inches for the all clay site.~~

The AP600 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 potential for liquefaction was evaluated for the soft soil and the soft-to-medium parabolic soil profiles. In this evaluation, the profiles were assumed to be of clean sand deposits with the water table at ground level. The cyclic shear stresses generated by the safe shutdown earthquake were evaluated against the cyclic shear strengths calculated in accordance with Seed's liquefaction chart (Reference 4). These strengths were estimated using normalized blow count values representative of the shear wave velocities. The evaluation indicated that the soft profile with clean sand deposits may be susceptible to liquefaction under the generic safe shutdown earthquake. However, other factors, such as the age of the deposit or the silt and clay content, can significantly increase the resistance to liquefaction. Such sites would require detailed site-specific investigation. The soft-to-medium parabolic soil profile and any firmer soil profiles are not susceptible to liquefaction.

2.5.4.5 Subsurface uniformity

Soil structure interaction and foundation design are a function of the uniformity of the soil or rock below foundation. Although the AP600 design and analysis of the AP600 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 AP600.

- A "non-uniform" site is acceptable to locate the AP600 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.

Appendix 2A presents a survey of 22 commercial nuclear power plant sites in the United States. This survey 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 AP600 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 geologic investigation, Appendix C to Regulatory Guide 1.132 provides guidance on the spacing and depth of borings 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 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 on the basis of an approximate zone of influence of the foundation mat. The extension is approximately equal to one-third of the equivalent east-west width. 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 spacing of the borings on the grid should be on the order of 50 to 60 feet. For example, an acceptable grid could have 5 borings in the short direction and 7 borings in the long direction, resulting in 35 borings to cover the nuclear island footprint and 40 feet beyond. 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, d_{max}, taken as the depth at which 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 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, the investigation effort is extended to determine if the site is acceptable for an AP600. The following paragraphs identify the site investigations required to demonstrate that the site may be acceptable.

As the AP600 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 borings or soundings at a spacing small enough to detect such features. 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.

To provide guidance for the site investigation of non-uniform sites, three non-uniform cases are described that might occur for nuclear plants. For each of these cases, the type of site investigation is described.

Sloping Bedrock Site

The sloping bedrock site as shown on Figure 2.5-2 is typical for a river front site where in the geologic past the bedrock has been eroded to a valley slope and then the valley was subsequently filled with alluvium. The bedding in the rock is nearly horizontal, but the surface of the rock is sloping on a strike parallel to the direction of the river. The shear wave velocity of the uniform soil layer overlying rock may vary between 1,000 and 2,500 feet per second. The shear wave velocity of 3,500 feet per second for the bedrock is representative of sites with a sloping rock surface. Sites where the bedrock has much higher shear wave velocities are not likely to exhibit such conditions.

Investigations for a site with a sloping bedrock surface must define the depth to bedrock as a function of plan location and the shear wave velocity of the overlying soil and bedrock. More borings may be necessary than required for a uniform site in order to establish the variation in depth to bedrock within the nuclear island footprint.

Undulatory Bedrock Site

An undulatory bedrock site as shown in Figure 2.5-3 is one where the bedding planes in the bedrock are (or nearly) horizontal but the surface is undulatory. Such a situation may occur if the bedrock surface is an erosion surface in a marine or lake environment. Another example might be a limestone site overlain by saprolite as in the southeast United States. The undulations could be the result of differential weathering or by soft zones associated with solution activity in the limestone.

Investigations for a site with an undulatory bedrock surface associated with weathering or karst condition must define the depth to bedrock as a function of plan location and the shear wave velocity of the overlying soil and bedrock. For cases with the overlying soil layer between the foundation level and the bedrock less than 40 feet, the pattern dimensions of the undulations must be defined with borings, specifically the width and depth of the undulations. Boring spacing on the order of 10 feet may be required for undulations having dimensions on the order of 20 feet in order to establish the variation in depth to bedrock within the nuclear island footprint.

Geologically Impacted Site

A geologically impacted site as shown on Figure 2.5-4 is one where the bedrock has abrupt facies change or has been interrupted either by a fault (shear zone) or by an intrusive such as a dike. This leads to the possibility of lateral variation in the bedrock properties affecting soil structure interaction and bearing pressure. Three subcases are identified. The first type includes an abrupt facies change. The second type has a shear zone of varying width and position. The third case is an intrusive dike of very competent rock compared to the surrounding rock.

Investigations for a geologically impacted site must define the width of the zone of the higher (or lower) shear wave velocity. The location of the zone of higher (or lower) shear wave velocity must be determined in relation to the center of containment. The azimuths of the bounding postulated vertical planes of the higher (or lower) shear wave velocity must be determined.

The zone of the higher (or lower) shear wave velocity is shown in Figure 2.5-4 bounded by non-curvilinear vertical parallel planes. It is recognized that such a situation is highly unlikely in nature. In order to define the width and location of the zone of higher (or lower) shear wave velocity, the spacing of the borings will have to be on the order of 10 feet for a zone with a width of 20 feet. It may be more practical to trench the site to locate and define the dimensions and locations of the intrusive or shear zone, thus eliminating many of the borings that would otherwise be required.

2.5.4.5.3 Site Evaluation Criteria

The AP600 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 28). 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 AP600 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 rock site having consolidated natural material with an average zero strain shear wave velocity greater than or equal to 2500 feet per second at the ground surface, the layers should be approximately equal 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 soil site having consolidated natural material with an average zero strain shear wave velocity less than 2500 feet per second at the ground surface, the layers should be approximately equal 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.

- For a site consisting of soil layers on top of rock, the rock and soil layers should meet the criteria for rock and soil sites respectively as described above.

Many sites that do not meet the above criteria for a uniform site are acceptable for the AP600. The key attribute for acceptability of the site for an AP600 is the bearing pressure on the underside of the basemat. This is a function of the subgrade modulus at the elevation of the foundation. The lateral variability of this subgrade modulus is acceptable if the layers satisfy the criteria for uniform soils given above. A site having local soft or hard spots within a layer or layers does not meet the criteria for a uniform site. The subgrade modulus is a function of the properties of the layers below the foundation and failure of one layer to meet the uniform criterion may not make the overall foundation unacceptable.

The design of the nuclear island foundation outlined in subsection 3.8.5 includes sufficient margin specifically to include bearing pressures of 120 percent of the uniform soil properties case. Some postulated types of non-uniform conditions are evaluated as part of the design certification analyses. These evaluations support criteria for some cases based on depth of the non-uniformity below grade to determine the acceptability of the site. The depth criteria are provided below for the three non-uniform cases described in subsection 2.5.4.5.2.2.

- Sloping Bedrock Site

  Sites where the surface of the sloping bedrock surface is greater than 50 feet below finished grade within the nuclear island footprint are acceptable for the certified design without additional analysis.

- Undulatory Bedrock Site

  Sites where the undulatory rock surface is greater than 80 feet below finished grade within the nuclear island footprint are acceptable for the certified design without additional analysis.

- Geologically Impacted Site

  Sites where the hard rock surface is greater than 120 feet below finished grade within the nuclear island footprint are acceptable for the certified design without additional analysis.

2.5.4.5.3.1 Alternate Site-Specific Subsurface Uniformity Design Basis

In addition to the cases provided above, other non-uniform sites are acceptable for the AP600. An alternate evaluation criterion is therefore defined to evaluate sites that do not satisfy the interface criteria directly. This evaluation is 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 AP600 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 AP600 design ground motion at a site having uniform soil properties.

This evaluation method shows acceptability for geologically impacted sites where there is a sufficient soil layer between the foundation level and the abrupt stiffness change of the bedrock.

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 AP600 basemat model and methodology described in subsection 3.8.5. The safe shutdown earthquake loads are those from the AP600 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 need to be less than the capacity of each portion of the basemat.

2.5.4.56 Combined License Information

Combined License applicants referencing the AP600 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.~~5~~6.1 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.~~5~~6.2 The Combined License applicant will demonstrate that the foundation soils are within the range considered for design of the nuclear island basemat. The design basis for sites that require a site specific analysis is defined in subsection 2.5.2.2.

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.

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.

~~Key considerations with respect to the materials underlying the nuclear island are to define the type of site, such as rock or soil, and to determine whether the site can be considered uniform; or, if the site is nonuniform, to define the nonuniform soil characteristics such as the location and profiles of soft and hard spots. These key considerations can be assessed with the information developed in response to Regulatory Guides 1.132 and 1.138. The geological investigations of subsection 2.5.1 and 2.5.4.5.1 provide information on the potential for the site to be non-uniform, whether it may be geologically impacted, and whether the bedrock may be sloping or undulatory. This information should be considered in planning geotechnical investigations.~~

~~2.5.4.5.2.1 Site Evaluation for Uniform Sites~~

~~At uniform sites, a site evaluation, as outlined below, will be performed to justify the determination that the site may be considered uniform for the purposes of foundation design and analysis.~~

~~Appendix C to Regulatory Guide 1.132 provides guidance on the spacing and depth of borings 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 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 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 spacing of the borings on the grid should be on the order of 50 to 60 feet. For example, an acceptable grid could have 5 borings in the short direction and 7 borings in the long direction, resulting in 35 borings to cover the nuclear island footprint and 40 feet beyond. 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, d_{max}, taken as the depth at which 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 borings may terminate at a depth of 160 feet below the foundation (equal to the width of the structure).~~

~~The subsurface may consist of layers and these layers may dip with respect to the horizontal. The physical properties of the foundation medium may or may not vary systematically across a horizontal plane. If the dip is less than 20 degrees, the generic analysis using horizontal layers is applicable as described in NUREG CR-0693 (Reference 28).~~

~~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 depth to a given layer indicated on each boring log may not fall precisely on the postulated plane. There will be some deviation between the elevations indicated on the boring logs for the top of a layer and that suggested by a "best estimate" plane. A deviation of 5 percent of the depth from the ground surface to the plane is considered acceptable. If the deviation is greater than 5 percent, additional planes may be appropriate or additional borings may be required, thereby diminishing the spacing. After the calculated planes are established, the uniformity and dip of the layers are compared against the criteria defined below.~~

~~The AP600 is designed for application at a site where the foundation conditions do not have extreme variation within the nuclear island footprint. The Combined License applicant shall demonstrate that the foundation subgrade modulus is within the range considered for design of the nuclear island basemat or a site specific analysis shall be performed. The subgrade modulus is a function of the soil layers below the nuclear island footprint. The variation of shear wave velocity in the material below the foundation to a depth of 80 feet below the basemat within the nuclear island footprint shall meet the following criteria for the site to be considered acceptable as a uniform site:~~

- ~~For a rock site having consolidated natural material with an average zero strain shear wave velocity greater than or equal to 2500 feet per second at the ground surface, the layers should be approximately equal 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 soil site having consolidated natural material with an average zero strain shear wave velocity less than 2500 feet per second at the ground surface, the layers should be approximately equal 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.~~

- ~~For a site consisting of soil layers on top of rock, the rock and soil layers should meet the criteria for rock and soil sites respectively as described above.~~

~~2.5.4.5.2.2 Site Evaluation for Non-Uniform Sites~~

~~At sites that are determined to be non-uniform or potentially non-uniform during the course of the geological investigations, the investigation effort is extended to determine if the site is acceptable for an AP600. The investigations required and acceptance criteria to determine the site conformance for expected non-uniform sites are outlined below.~~

~~As the AP600 foundation/structural system design 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 borings or soundings at a spacing small enough to detect such features.~~

~~Appendix 2A presents a survey of 22 commercial nuclear power plant sites in the United States. This survey 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 AP600 sites exhibit "uniform" soil properties within the nuclear island footprint. These "uniform" sites would be evaluated as described in subsection 2.5.4.5.2.1.~~

~~To provide guidance for the site investigation of non-uniform sites, three non-uniform cases are described that might occur for nuclear plants. For each of these cases, the type of site investigation is described.~~

Sloping Bedrock Site

~~The sloping bedrock site as shown on Figure 2.5-2 is typical for a river front site where in the geologic past the bedrock has been eroded to a valley slope and then the valley was subsequently filled with alluvium. The bedding in the rock is nearly horizontal, but the surface of the rock is sloping on a strike parallel to the direction of the river. The shear wave velocity of the uniform soil layer overlying rock may vary between 1,000 and 2,500 feet per second. The shear wave velocity of 3,500 feet per second for the bedrock is representative of sites with a sloping rock surface. Sites where the bedrock has much higher shear wave velocities are not likely to exhibit such conditions.~~

~~Investigations for a site with a sloping bedrock surface must define the depth to bedrock as a function of plan location and the shear wave velocity of the overlying soil and bedrock. The bedrock profile can be identified in the same manner as described in subsection 2.5.4.5.2.1 for the various layers for a uniform site. The same criterion, i.e. deviations up to 5 percent of the depth are acceptable. More borings may be necessary than required for a uniform site in order to establish the variation in depth to bedrock within the nuclear island footprint.~~

~~Undulatory Bedrock Site~~

~~An undulatory bedrock site as shown in Figure 2.5-3 is one where the bedding planes in the bedrock are (or nearly) horizontal but the surface is undulatory. Such a situation may occur if the bedrock surface is an erosion surface in a marine or lake environment. Another example might be a limestone site overlain by saprolite as in the southeast United States. The undulations could be the result of differential weathering or by soft zones associated with solution activity in the limestone.~~

~~Investigations for a site with an undulatory bedrock surface associated with weathering or karst condition must define the depth to bedrock as a function of plan location and the shear wave velocity of the overlying soil and bedrock. For cases with the overlying soil layer between the foundation level and the bedrock less than 40 feet, the pattern dimensions of the undulations must be defined with borings, specifically the width and depth of the undulations. Boring spacings on the order of 10 feet may be required for undulations having dimensions on the order of 20 feet in order to establish the variation in depth to bedrock within the nuclear island footprint.~~

~~**Geologically Impacted Site**~~

~~A geologically impacted site as shown on Figure 2.5-4 is one where the bedrock has abrupt facies change or has been interrupted either by a fault (shear zone) or by an intrusive such as a dike. This leads to the possibility of lateral variation in the bedrock properties affecting soil structure interaction and bearing pressure. Three subcases are identified. The first type includes an abrupt facies change. The second type has a shear zone of varying width and position. The third case is an intrusive dike of very competent rock compared to the surrounding rock.~~

~~Investigations for a geologically impacted site must define the width of the zone of the higher (or lower) shear wave velocity. The location of the zone of higher (or lower) shear wave velocity must be determined in relation to the center of containment. The azimuths of the bounding postulated vertical planes of the higher (or lower) shear wave velocity must be determined.~~

~~The zone of the higher (or lower) shear wave velocity has been shown in Figure 2.5-4 bounded by non-curvilinear vertical parallel planes. It is recognized that such a situation is highly unlikely in nature. A procedure similar to that described in subsection 2.5.4.5.2.1 for identifying the horizontal layers of uniform sites can be used to define the nearly vertical planes of a geologically impacted site. Deviations between the site condition and the idealized postulated situations on the order 5 percent are considered acceptable.~~

~~In order to define the width and location of the zone of higher (or lower) shear wave velocity, the spacing of the borings will have to be on the order of 10 feet for a zone with a width of 20 feet. It may be more practical to trench the site to locate and define the dimensions and locations of the intrusive or shear zone, thus eliminating many of the borings that would otherwise be required.~~

~~**Acceptance criteria for non-uniform sites**~~

~~The key attribute for acceptability of the site for an AP600 is the bearing pressure on the underside of the basemat. This is a function of the subgrade modulus at the elevation of the foundation. The lateral variability of this subgrade modulus is acceptable if the layers satisfy the criteria for uniform soils given in subsection 2.5.4.5.2.1. A site having local soft or hard spots within a layer or layers does not meet the criteria for a uniform site. The subgrade modulus is a function of the properties of the layers below the foundation and failure of one layer to meet the uniform criterion may not make the overall foundation unacceptable. An alternative evaluation criterion is therefore defined to evaluate sites that do not satisfy the requirement for all layers to be uniform.~~

~~A site with nonuniform soil properties may be demonstrated to be acceptable by site specific analyses 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 are those from the AP600 design soil case representative of the site specific soil. Alternatively, the safe shutdown earthquake loads may~~

~~be determined from a site-specific seismic analysis of the nuclear island. 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 similar analyses for a site having uniform soil properties. This evaluation method is expected to demonstrate that sloping and undulating bedrock sites are acceptable when the soil layer over the bedrock is sufficient. It will also show acceptability for geologically impacted sites where there is a soil layer between the foundation level and the abrupt stiffness change of the bedrock.~~

~~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 AP600 basemat model and methodology described in subsection 3.8.5. The safe shutdown earthquake loads are those from the AP600 design soil case representative of the site-specific soil. Alternatively, bearing pressures may be determined from a site-specific soil structure interaction analysis. For the site to be acceptable the bearing pressures from the site-specific analyses need to be demonstrated to be less than the capacity of each portion of the basemat.~~

2.5.4.~~5~~6.3 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.

2.5.4.~~5~~6.4 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.~~5~~6.5 Response of Soil and Rock to Dynamic Loading - The dynamic characteristics of the soil and rock will be compared to the assumptions made in the standard design regarding the variation of shear wave velocity and material damping. The parametric analyses described in Appendices 2A and 2B cover a broad range of dynamic characteristics appropriate for most soil types (sand, silts, clays, gravels, and various combinations). The shear wave velocity (based on low strain best estimate soil properties) must be greater than or equal to 1000 feet per second.

2.5.4.~~5~~6.6 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.

2.5.4.56.7 Bearing Capacity - The Combined License applicant will verify that the site-specific soil bearing capacity is equal to or greater than the value documented in Table 2-1 of the SSAR.

2.5.4.56.8 Earth Pressures - The AP600 is designed for static and dynamic lateral earth pressures and hydrostatic groundwater pressures acting on plant safety-related facilities using soil parameters as evaluated in previous subsections. No additional information is required on earth pressures.

2.5.4.56.9 Soil Properties for Seismic Analysis of Buried Pipes - The AP600 does not utilize safety related buried piping. No additional information is required on soil properties.

2.5.4.56.10 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.

2.5.4.56.11 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.

2.5.5 Combined License Information for Stability of Slopes

Combined License applicants referencing the AP600 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 AP600 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.

2.5.7 References

1. Terzaghi, K. and Peck, R.B., "Soil Mechanics in Engineering Practice," 2nd Edition, John Wiley & Sons, New York, 1967.

2. Peck, R.B., Hanson, W.E., and Thornburn, T.H., "Foundation Engineering," John Wiley & Sons, New York, 1974.

3. Harr, M.E.,"Foundations of Theoretical Soil Mechanics," McGraw-Hill Book Co., New York, 1966.

4. Seed, H.B., "Soil Liquefaction and Cyclic Mobility Evaluation for Level Ground During Earthquakes," Journal of Geotechnical Engineering Division, ASCE, Vol. 105, GT2, February 1979.

Table 2-1 (Sheet 1 of 2)

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	110 mph; importance factor 1.11 (safety), 1.0 (nonsafety)
Tornado	300 mph
Seismic	
SSE	0.30g peak ground acceleration [(c)]
Fault Displacement Potential	None
Soil	
~~Bearing Strength~~ Average allowable static soil bearing capacity	~~Soils must support the AP600 under specified conditions. The average static bearing reaction due to the dead weight of the AP600 nuclear island is about 8000 pounds/square foot; the maximum static bearing reaction at a corner is about 12,000 pounds per square foot.~~ Greater than or equal to 8,000 pounds per square foot over the footprint of the nucler island at its excavation depth.
Lateral variability	Soils supporting the nuclear island should not have extreme variations in subgrade stiffness (see subsection 2.5.4.5~~.2~~)
Shear Wave Velocity	Greater than or equal to 1000 ft/sec based on low strain best estimate soil properties
Liquefaction Potential	None

Table 2-1 (Sheet 2 of 2)

SITE PARAMETERS

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 ~~100~~98′
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)
Dispersion Values - X/Q	See subsections 2.3.4 and 2.3.5
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 response spectra as given in Figures 3.7.1-1 and 3.7.1-2.

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

Table 2-2

NET ALLOWABLE STATIC BEARING CAPACITIES (KIPS PER SQUARE FOOT)

Soil Shear Wave Velocity Profile	Cohesive Soil		Cohesionless Soil			
	40 feet below grade	At grade	40 feet below grade		At grade	
			Dry	Submerged	Dry	Submerged
Soft Soil	7	6.8	70.3	32.2	35.1	16.1
Soft to Medium - Linear	18.9	12	102	46.6	55.8	25.6
Soft to Medium - Parabolic	32	24	139	63.8	79.7	36.5
Upper Bound, Soft to Medium - Parabolic	60	50	265	121.3	159.3	73
Soft Rock	>220					
Hard Rock	>450					

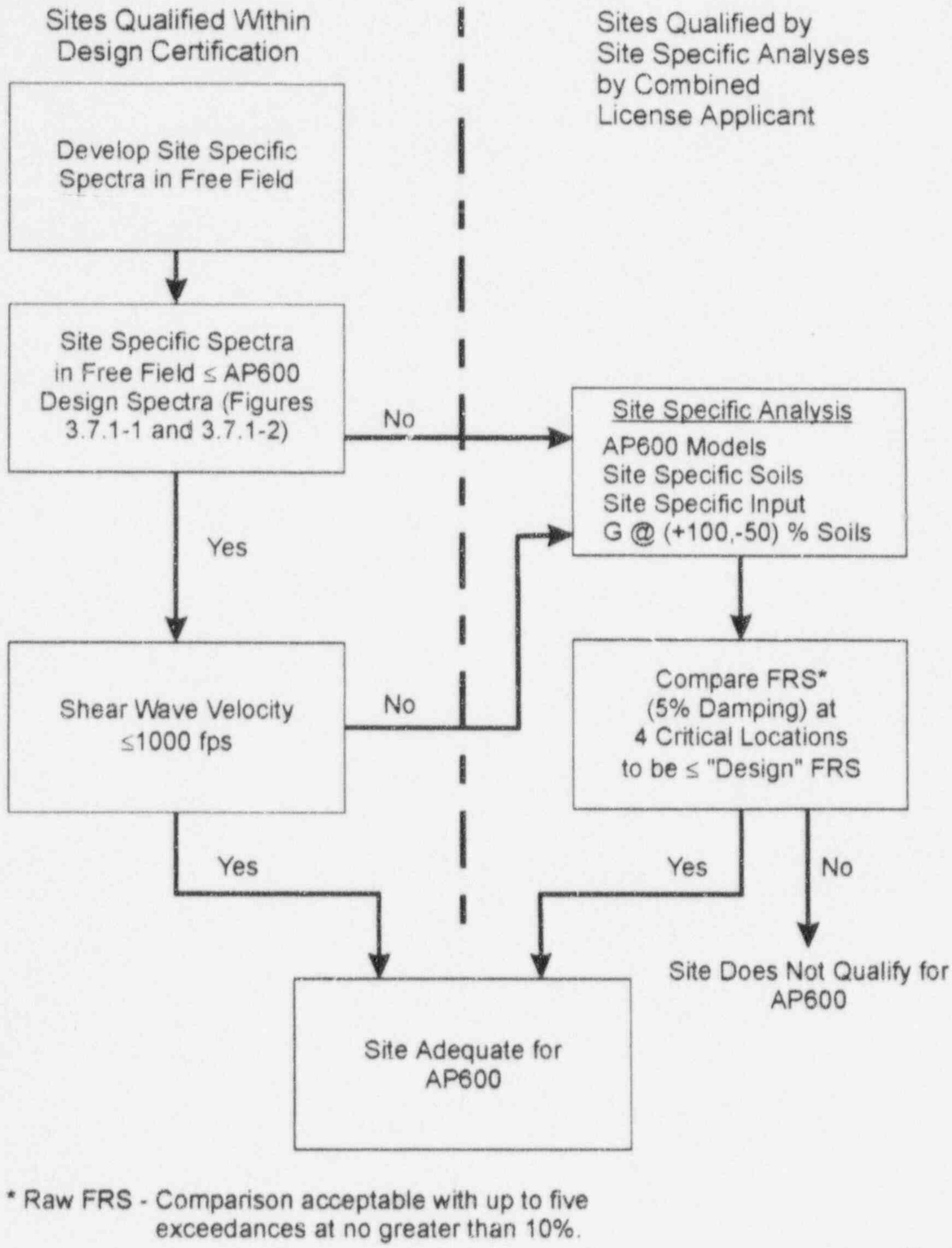

Figure 2.5-1

Alternate Site Specific Seismic Response Qualification Flow Chart

Attachment 3

SUMMARY REPORT
EFFECTS OF SETTLEMENT AND CONSTRUCTION SCHEDULE ON THE AP600 BASEMAT

1.0 INTRODUCTION

This report summarizes a study undertaken to determine the stresses in the AP600 Nuclear Island (NI) basemat during plant construction. The study considered the construction sequence, the associated time varying load and stiffness of the NI basemat, and the resulting settlement time history. Based on the results, the study provides an assurance that the AP600 standard design can accommodate short term and long term settlement for limiting case soft soil sites, and a flexible construction schedule, with certain specified limits.

The study considers the settlement time history associated with deep soft soil sites for the AP600. It focuses on the response of the basemat in the early stages of construction when the basemat could be susceptible to differential loading and deformation. With the subsequent construction of shear walls associated with the Auxiliary Building and the Shield Building, the basemat/superstructure system significantly stiffens, minimizing the impact of differential settlement. The study quantifies the basemat vertical displacements, and the moments and shear forces induced in the basemat at different stages of construction. When the construction is complete, and all of the nuclear island is in place, the moments and shears represent the stress state of the structural elements due to normal operation deadweight of the Nuclear Island, accounting for the effects of settlements. This report demonstrates that these moments and shears are within acceptable limits.

2.0 SITE CONDITIONS

Consistent with the foundation conditions contemplated in the standard design, the settlement evaluation addresses subsurface profiles consisting of compressible clay deposits underlying a 40 foot layer of sand at grade. To conservatively maximize the potential settlement, the evaluation considers a deep site with soil extending to a depth of about 360 feet. This depth is approximately 1.5 times the largest dimension of the basemat, and the analysis incorporates the influence within the stress bulb of the foundation footprint.

The evaluation considers two conservative bounding profiles to accommodate both cohesive and cohesionless site conditions and combinations thereof.

Profile1: A deep soft soil site underlain by alternate layers of sand and clay. The clay is assumed to be normally consolidated and the water table is assumed to be at existing grade. The assumptions of (1) normal consolidation (2) high water table and (3) alternating sand and clay layers will tend to maximize the settlement in the early stages of construction. It is also noted that the assumption of a water table at grade will maximize the impact of dewatering during the early weeks of construction.

Profile 2: It is similar to profile1, except that there are no intervening sand layers. Because of the greater thickness of clay the settlement will occur over a longer period of time. All other conditions are the same as profile 1. This profile tends to maximize settlement during the later weeks of construction and during the operational period of the plant.

These profiles represent limiting soil conditions and provide bounding settlement potential. Shallow depth and intermediate depth soft and soft to medium sites will result in smaller total and differential settlements.

The soil properties used in the evaluation are based on empirical correlations and past experience and reflect the foundation interface criterion that the shear wave velocity of the subsurface soils is greater than 1000 feet per second. Accordingly, the shear wave velocity is assumed to increase linearly from 1000 feet per second at site grade to 1200 feet per second at a depth of 240 feet. The elastic and consolidation characteristics of the soils are consistent with large strains expected to occur under static loads imposed during the construction process. The parameters required for settlement analysis such as void ratio, compression and recompression indices, and elastic modulus are assigned based on an approximate distribution of the stresses imposed on the subsurface soils due to the final construction loads. These parameters are presented in Table 4.

3.0 CONSTRUCTION SEQUENCE AND LOADING

The base construction schedule assumes no unscheduled delays, and is represented in a simplified time line on Figure 1. With this schedule, all the concrete in the Auxiliary Building is completed to El. 100 feet and all the concrete in the Shield Building is completed to El. 98 feet in week 25. The activities include dewatering, excavation, and placement of the concrete for the basemat, exterior and interior walls in the Auxiliary Building, and the Shield Building. Figure 2 presents the time history of construction loads represented as uniform pressure on the basemat. The uniform pressures in Figure 2 represent the net change in the average vertical stress at the base of the foundation and shows the relative magnitudes of the loads at various stages. For example, it illustrates the importance of the load removed due to the excavation down to El. 60 feet.

The study also evaluates two extreme variations to the base construction schedule. These variations assume an arbitrary delay at the "worst" time in either the construction of the Auxiliary Building or in the construction of the Shield Building, after the basemat and the pedestal for the containment vessel (CV) head are in place. The purpose of considering these variations is to validate construction flexibility within these broad limits.

Variation 1 (Delayed Shield Building Case): It postulates a delay in the placement of concrete in the Shield Building while construction continues on the Auxiliary Building. After the pedestal for the CV head is constructed, no additional concrete is placed in the Shield Building. The analysis incorporates the construction of the Auxiliary Building to El. 117 feet and thereafter assumes that all construction is suspended at this stage with no constraints on when the construction must be resumed. This is considered a "worst" case from the perspective that it tends to maximize tension stresses on the top of the basemat by causing it to "bow" upward at the center.

Variation II (Delayed Auxiliary Building Case): It assumes an arbitrary delay in the construction of theAuxiliary Building while concrete placement for the Shield Building continues. The analysis examines the case in which, after the Shield Building concrete is up to El. 84 feet, all subsequent construction is suspended. This variation is a "worst" case scenario in the sense that it causes the basemat to "bow" downward, thus tending to maximize tension stresses in the bottom of the basemat.

4.0 METHODOLOGY

For sites comprised of compressible soils, the elastic deformation and consolidation characteristics of the soils define the site's short-term and long-term settlement potential. The construction loading and the basemat stiffness determine the actual pattern of vertical displacements. This study evaluates the basemat displacements and the resulting moments and shears at discrete time steps in the construction sequence. Table 1 identifies the construction elements contributing to the load and stiffness of the basemat at the time steps evaluated herein.

At each time step, incremental loads result in incremental displacements, moments and shears consistent with the pattern of loading, stiffness of the structure at this time step, and the distribution of the effective subgrade stiffness (K). The effective K reflects the elastic settlement potential due to the incremental bearing pressures, as well as consolidation potential due to the cumulative bearing pressures going forward from this specific time step. The relatively high stiffness of the basemat/superstructure system affects the distribution of bearing pressures due to the applied construction loads as the settlement progresses, which in turn, modifies the settlement pattern and consequently, the distribution of the effective subgrade stiffness. This is addressed analytically in an iterative procedure at each time step.

The first step in the iteration estimates elastic and consolidation settlements at the base of the basemat, and for each layer comprising the subsurface material, assuming that the loaded area is flexible, i.e., the basemat has no stiffness. Settlements are estimated at eleven (11) profile points covering the plant footprint. The settlement calculation dissipates the excess pore pressure using the one-dimensional consolidation theory consistent with the given consolidation parameters such as the initial void ratio, compression index and recompression index and the coefficient of consolidation. The methodology follows the general industry practice as outlined in a text book by Lamb and Whitman (1969) and has been used at several nuclear power plant sites where soft soil sites result in the potential for long-term consolidation.

In Step 2, the settlements at the base of the foundation from Step 1 are used to estimate equivalent Winkler springs representing the deformation characteristics of the subsurface which include the effect of time-dependent consolidation settlements. This is equivalent to decreasing the modulus of subgrade reaction to account for consolidation, following the method described in Bowles (1988). These springs are incorporated into a three-dimensional structural finite element model which appropriately represents the stiffness of the basemat, Auxiliary Building walls and the Shield Building.

Utilizing the applied loads, Step 3 of the analysis estimates deformations, bearing reactions, and moments and shears in the basemat. The finite element model, developed for the analysis and illustrated on Figure 3, comprises of 457 nodes, 406 plate elements and 137 boundary elements. The elements are grouped to allow the representation of variable stiffness as the construction progresses

At each discrete time step (i), the total elastic and consolidation settlements occurring during the period from the placement of the basemat until the next time step in the construction sequence, (i+1), due to the cumulative bearing pressures at (i) are used to compute the effective subgrade modulus and its distribution under the Nuclear Island. The subgrade modulus is then used to calculate effective soil springs under the basemat. At each time step, the analysis of the soil-structure system results in reactions in the effective soil springs and a distribution of bearing pressures which is somewhat different than the distribution used in the initial settlement analysis. The new bearing pressures are used in a subsequent iteration of the settlement analysis to determine new subgrade modulus and spring constants and the analysis repeated until satisfactory convergence. In the analysis reported herein, the bearing pressures converged in two or three iterations.

The analysis thus accounts for changes in the distribution of bearing pressures due to the time varying loads and stiffness of the basemat and results in a deformation pattern of the basemat which is consistent with the settlement potential. The moments and shears in the basemat, calculated for each time step in the construction schedule, are evaluated and checked for acceptability.

5.0 RESULTS

In general, the alternating sand and clay layer site maximizes the rate of settlement and the short term settlements, e.g., during the early stages of construction. The clay-only site maximizes the long term settlement. Consequently, the alternating sand and clay layer site maximizes moments and shears in the basemat during the construction period. Because this case governs, only the results pertaining to the sand/clay site are presented herein.

For the base construction schedule, the basemat deformations, moments and shears are evaluated at 5 time steps, namely Weeks 18, 22, 25, 48 and 93 of the construction schedule. Although the analysis at Weeks 48 and 93 includes the respective loads, it does not reflect the stiffness of the structural elements above El. 100 feet because the structural model is limited to elements below grade level. Consequently, the differential displacements predicted at these two load steps are overestimated and the moments and shears are carried by a substantially larger section than that included in the analysis.

Figure 4 presents the vertical displacements of the basemat at the centers of the Shield Building and the North and South Auxiliary Buildings for the Base Construction Schedule.

The vertical displacements presented in the above Figures reflect the settlement occurring after the basemat cures. Prior to placing the basemat (Week 13), dewatering and excavation result in settlements of approximately 7 inches and 5 inches at the center of the Shield Building and the centers of the Auxiliary Buildings, respectively. Surface preparation prior to placing the 6-foot basemat at Week 13 will level out the differences in the settlements prior to placement of concrete. Additionally, immediate elastic settlement due to the weight of the 6-foot basemat will not induce moments in the basemat as the basemat will naturally conform to the displaced shape of the subgrade.

Figure 4 illustrates the magnitude of the potential differential displacements in the basemat between the Shield Building and the Auxiliary Building areas. For the Base Construction Schedule, the analysis at the final load step at Week 93 includes the total cumulative load on the foundation and reflects substantially all of the time-dependent consolidation. This analysis results in a vertical deflection at the center of the Shield Building of 4.5 inches and a differential displacement of about 1.6 inches.

Compared to the Base Construction Schedule, the Delayed Shield Building results in smaller displacements reflecting the smaller imposed loads. The analysis at Week 36, which includes construction in the Auxiliary Buildings to El. 117 feet and all of the consolidation due to the total cumulative load at this time step, results in a vertical deflection at the center of the Shield Building of 0.8 inches. The associated relative displacement between the center of the Auxiliary Building and the Shield Building is about 0.3 inches. Similarly, the analysis at Week 25 for the Delayed Auxiliary Building case results in a vertical deflection of 1.4 inches. The associated relative displacement between the Shield Building and the Auxiliary Buildings is about 0.6 inches.

Table 2 presents the maximum moments in the basemat at various stages of construction for the base construction schedule. This table presents the maximum moments in the 6-foot basemat in the Auxiliary Building and three concentric rings of the Shield Building, each of uniform thickness. The center ring of the Shield Building represents the area within a radius of 30 feet, the intermediate ring is the area between 30 and 44 feet radius, and the outer ring is the area from 44 feet to 70 feet radius.

For the Base Construction Schedule, the largest moments in the 6-foot basemat of the Auxiliary Buildings occur due to loads and conditions at Week 22 when no connections between the Auxiliary Building and the Shield Building are credited. The maximum moment is 739 kip feet per foot and occurs at the interface between the North Auxiliary Building and the Shield Building. As dewatering is discontinued at Week 25 and the subsurface rebounds, the moments in the 6-foot basemat decrease. Indeed, this is in part also attributed to the presence of the struts between the Auxiliary and the Shield Buildings which are now effective. The largest moments in the Shield Building concrete occur near the Building's east-west centerline. At Week 25 for example, the maximum moments in the center, intermediate and outer rings are respectively, 434, 2583 and 4229 k-ft/ft. The moments increase at Week 48 as more load is placed in the Shield Building. However, the larger moments are offset by the increased capacity of the sections at Week 48.

For the Delayed Shield Building case the maximum negative moments occur in the region along the interface of the Auxiliary and the Shield Buildings. As additional loads are imposed in the Auxiliary Building areas, the maximum negative moments along the interface increase. The maximum negative moment occurs at Week 36 when the Auxiliary Building is up to El. 117 feet and is 109 kip feet per foot.

The maximum acceptable moments for the Delayed Auxiliary Building case occur when the Shield Building is constructed to El. 84 feet, at Week 25, and also occur at the interface with the Shield Building concrete. Of the three construction scenarios analyzed, this case results in the largest moments in the 6-foot basemat and is 844 kip feet per foot.

Table 3 presents the bearing pressures under the Shield Building and the North and South Auxiliary Buildings and the resulting maximum shears in the 6-foot basemat at the interface with the Shield Building. The maximum vertical shear in the 6-foot basemat shown in the table is within the acceptable limit.

Based on conservatively ignoring the contribution to the stiffness from elements above El. 100 feet, the analysis at Week 48 also results in acceptable moments and shears in the 6-foot basemat. Indeed, the structural connections between the Auxiliary Building walls and the Shield Building concrete at El. 100 as well as the floors at Elevations 84 feet and 100 feet are expected to result in a significantly more robust structure than assumed in the analysis. Similarly, the concrete in the Shield Building above the Containment Vessel, which is presently ignored except for its weight, is expected to contribute significant stiffness and capacity.

For the Delayed Shield Building construction, when construction is complete to El. 117 feet - 6 inches, the applied load and the forces in the basemat are small, relative to the loads and forces associated with the Base Construction Schedule. The delayed Shield Building case results in moments and shears that are within allowable capacity as long as the construction in the Auxiliary Building area is suspended at El. 117 feet. Eventual resumption in construction should place the elements in the Shield Building until construction in the Shield Building area is completed to El. 100 feet, the ties between the Shield and North Auxiliary Building have been established and dewatering has also been suspended. Following this, the construction in both the Auxiliary Buildings and the Shield Building can proceed as planned.

The maximum acceptable moments for the Delayed Auxiliary Building case occur when the Shield Building is constructed to El. 84 feet. These moments and shears are also within allowable limits as long as the construction in the Shield Building is suspended at El. 84 feet. Resumption of construction in this case can occur when the Auxiliary Building walls reach El. 100 feet, the ties are established at El. 84 feet and the floor placed at El. 82 feet 6 inches. This construction as well as suspension of dewatering after concrete has hardened to El. 100 feet returns the Delayed Auxiliary Building condition to correspond to the Base Construction Schedule at Week 25. Thereafter, the construction can proceed as planned.

6.0 CONCLUSIONS

On the basis of the evaluation of the construction loads and the settlement potential at various stages of construction, we conclude the following:

- The design of the basemat and the Shield Building concrete can accommodate 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 El. 117 feet. Prior to continuing construction in the Auxiliary Buildings, the Shield Building must reach El. 100 feet.

- The design can accommodate delays in the Auxiliary Building so long as the Shield Building construction is suspended at El. 84 feet. The Auxiliary Building must reach El. 100 feet prior to continuing construction of the Shield Building.

- After the structure is in place and cured to El. 100, the loading due to construction above this elevation will not result in significant additional flexural demand with respect to the basemat and the Shield Building concrete below the CV. Accordingly, we do not see the need to place particular constraints on the construction above El. 100 feet.

In considering deep Soft Sites, the study reported herein bounds the effects of settlements on the basemat. Shallow depth and intermediate depth Soft and Soft to Medium Sites are expected to result in smaller total settlements and smaller potential for differential settlement. Accordingly, the construction related basemat moments and shears for these site conditions are enveloped by the bounding analysis.

The analysis also assumes uniform horizontal soil layers extending to a depth about 1.5 times the largest dimension of the Nuclear Island (a deep soft site). The conclusions of this study may be applied to any site however, as long as the soil layers comprising the site are nearly horizontal, and variations in key characteristics in any horizon is less than about 10 percent. The elastic characteristics of the sand and clay layers are tied to the site requirements related to the shear wave velocity. Therefore, a site may be considered uniform as long as the shear wave velocity at any location in a horizontal plane does not vary from the average shear wave velocity in that plane by more than 10 percent. Implicit in this claim is the assumption that for deep sites, the total thickness of compressible deposits is nearly uniform under the entire footprint.

Sites that exhibit non-uniformity with respect to the above definition could still be suitable from the point of view of settlement induced stresses in the basemat. "Sloping Bedrock" sites and the "Undulatory Bedrock" sites could be considered in this subset. Sites with anomalous hard and soft spots would not be suitable from the point of view of settlement induced stresses. On the basis of estimates of the differential settlement potential and the associated equivalent springs, "Sloping Bedrock" sites and "Undulatory Bedrock" sites could be shown to be acceptable. If the maximum variation of the equivalent soil springs is less than about 10 percent of the average, the resulting settlement induced stresses should be enveloped by the bounding analysis.

The analysis is based on the assumption that plant grade is the same as the original grade. However, some sites may need to change plant grade due to flood or other considerations. The addition of fill reduces the extent of excavation and dewatering and generally influences the change in effective stress in the subsurface soils. Consequently, raising the grade could affect the settlement potential and the settlement induced stresses in the basemat. When all the construction load is in place, a 4-foot surface fill is expected to increase the effective stress immediately below the foundation by about 5 percent and a 8-foot surface fill is expected to increase the effective stress by about 15 percent. Accordingly, raising the plant grade by a nominal amount of about 4 to 5 feet is acceptable.

In conclusion, the AP600 basemat design is adequate for all soil sites that are potential candidates to place AP600 plant, within the construction limits defined. It can accomodate major variations in the construction sequence without causing excessive deformations, moments and shears due to settlements over the plant life.

TABLE 1

FOUNDATION MAT LOAD AND STIFFNESS
AP600 SETTLEMENT EFFECTS

AP600 FOUNDATION MAT

Item	Week 13	Week 18	Week 22	Week 25
Load	6'-0" Mat	6' Mat + AB Walls to 84'	AB Walls to 100' SB Conc. to 84'	AB to 100' SB Conc. to 100'
Structural Stiffness	None	6' Mat	6' Mat +AB to 84' SB Conc. to 76' t=(8/16/16) (1)	6' Mat+AB to 100' SB Conc. to 84' t= (8/16/22) (2)

Notes:

1. Thickness of center ring = 8'
   Thickness of intermediate ring = 16'
   Thickness of outer ring = 16'

2. Strut connection between Auxiliary Building Walls and Shield Building concrete is effective.

TABLE 2

MAXIMUM POSITIVE MOMENT [1]
ALTERNATING SAND AND CLAY SITE
BASE CONSTRUCTION SCHEDULE

AP600 FOUNDATION MAT

MAXIMUM POSITIVE LONGITUDINAL MOMENT (KIP FT PER FT)

TIME (WEEKS)	18	22	25	48 [2]	LOCATION [3]
SHIELD BUILDING					
Outer Ring	77	1772	4229	6994	Centerline of SB @ column line K
Intermediate Ring	94	2600	2583	4149	Centerline of SB @ column line L
Inner Ring	82	417	434	697	Centerline of SB @ column line M
AUXILIARY BUILDINGS					
South Aux.	73	503	123	217	Interface of SB @ column line K-2
North Aux.	50	739	227	409	Interface of SB @ column line N

MAXIMUM POSITIVE TRANSVERSE MOMENT (KIP FT PER FT)

TIME (WEEKS)	18	22	25	48 [2]	LOCATION
SHIELD BUILDING					
Outer Ring	103	1527	3750	5893	Centerline of SB @ column line 4
Intermediate Ring	106	2107	2296	3650	Centerline of SB mid way between column line 4 and 5
Inner Ring	82	360	397	631	Centerline of SB @ column line 5
AUXILIARY BUILDINGS					
South Aux.	120	437	217	390	Interface of SB @ column line J-1
North Aux.	89	273	217	352	Interface of SB @ column line J

NOTES:

1. Moments are maximum element moments at the respective nodes output from ALGOR.
2. Moments at week 48 may be overestimated because the structural model does not include the stiffness of the walls and floor system above elevation 100 feet.

TABLE 3

DISTRIBUTION OF BEARING PRESSURE AND MAXIMUM SHEAR IN THE BASEMAT ALTERNATING SAND AND CLAY SITE BASE CONSTRUCTION SCHEDULE

AP600 FOUNDATION MAT

WEEK	BEARING PRESSURE (KSF)			MAXIMUM SHEAR
	SHIELD BUILDING	SOUTH AUXILIARY BUILDING	NORTH AUXILIARY BUILDING	IN BASEMAT (K/LF)
18	1.129	1.289	1.126	5
22	3.236	2.220	2.038	26
25	5.229	2.827	2.729	33
48	7.938	5.568	5.489	43

Notes:

1. The bearing pressure is averaged over each area presented.
2. The North Auxiliary Building is represented by two disks and the bearing pressure is a weighted average of both disks.
3. Bearing pressures are calculated from the foundations reactions of the finite element model.
4. Maximum shear in six foot basemat occurs at the interface of the shield building and the north auxiliary building.

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TABLE 4

BASIC SAND AND CLAY PROPERTIES FOR ALTERNATING SAND AND CLAY SITE

AP600 FOUNDATION INTERFACE

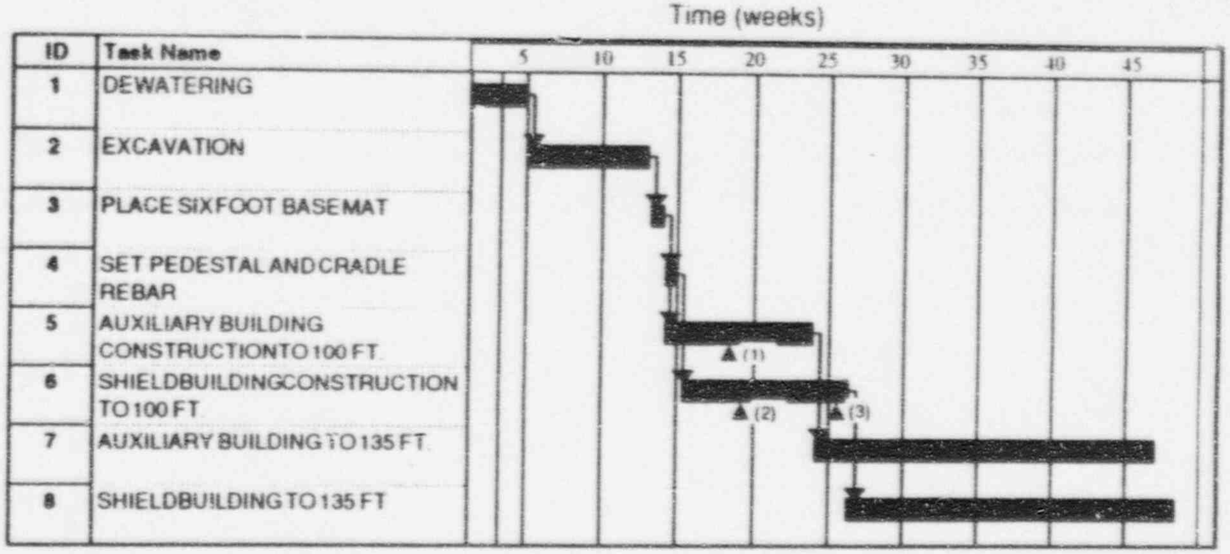
**Sand and Clay Properties with Vs varying from 1000 fps to 1200 fps according to Vs = 1000 + (depth/240)*200

Layer No.	Top Elevation	Bottom Elevation	Vs ** @ Midlayer ft/sec	Type	G @10^{-4} strain #/ft²	E @ 10^{-4} strain #/ft²	Initial σ_{vo} at mid layer #/ft²	e_o	Cc	Ccr	Cα	m_v ft²/#	c_v ft²/yr
0	100	60	1016.7	sand	4.01E+06	1.08E+07	1,252						
1	60	57	1034.6	sand	4.16E+06	1.12E+07	2,598						
2	**57**	**30**	**1047.1**	**clay**	**4.26E+06**	**1.15E+07**	**3,537**	**0.43**	**0.049**	**0.00492**	**0.002953**	**0.000003**	**61.24**
3	30	0	1070.8	sand	4.45E+06	1.20E+07	5,321						
4	**0**	**-30**	**1095.8**	**clay**	**4.66E+06**	**1.26E+07**	**7,199**	**0.75**	**0.143**	**0.01435**	**0.008609**	**0.000004**	**38.90**
5	-30	-60	1120.8	sand	4.88E+06	1.32E+07	9,077						
6	**-60**	**-90**	**1145.8**	**clay**	**5.10E+06**	**1.38E+07**	**10,955**	**0.88**	**0.183**	**0.018**	**0.0110**	**0.000004**	**46.27**
7	-90	-120	1170.8	sand	5.32E+06	1.44E+07	12,833						
8	**-120**	**-150**	**1195.8**	**clay**	**5.55E+06**	**1.50E+07**	**14,711**	**0.94**	**0.202**	**0.020**	**0.0121**	**0.000003**	**56.53**
9	-150	-180	1220.8	sand	5.79E+06	1.56E+07	16,589						
10	**-180**	**-210**	**1245.8**	**clay**	**6.03E+06**	**1.63E+07**	**18,467**	**0.97**	**0.211**	**0.021**	**0.0127**	**0.000002**	**67.98**
11	210	-240	1270.8	sand	6.27E+06	1.69E+07	20,345						
12	**-240**	**-270**	**1295.8**	**clay**	**6.52E+06**	**1.76E+07**	**22,223**	**0.99**	**0.215**	**0.022**	**0.0129**	**0.000002**	**80.26**
13	-270	-300	1320.8	sand	6.77E+06	1.83E+07	24,101						

Layer No.	Estimated Shear Strain	Strain-Softened Moduli for Settlement in Sand		Strain-Softened Moduli for Settlement in Clay	
		G @ γ #/ft²	E @ γ #/ft²	G @ γ #/ft²	E @ γ #/ft²
0					
1	0.34%	3.80E+05	1.03E+06		
2	**0.34%**			**1.28E+06**	**3.45E+06**
3	0.32%	5.42E+05	1.46E+06		
4	**0.30%**			**1.49E+06**	**4.03E+06**
5	0.23%	1.01E+06	2.72E+06		
6	**0.16%**			**2.12E+06**	**5.73E+06**
7	0.12%	1.98E+06	5.34E+06		
8	**0.09%**			**3.07E+06**	**8.29E+06**
9	0.07%	3.14E+06	8.47E+06		
10	**0.05%**			**4.45E+06**	**1.20E+07**
11	0.05%	4.26E+06	1.15E+07		
12	**0.0357%**			**5.86E+06**	**1.58E+07**
13	0.02%	5.79E+06	1.56E+07		

PLC7		DRAWN BY	LNH	CHECKED BY			FILE NUMBER	r3-f1-1652/97
1=1			2/13/97	APPROVED BY				

Not to Scale



NOTES:

▲ 1 - Aux. Building Walls to 84'

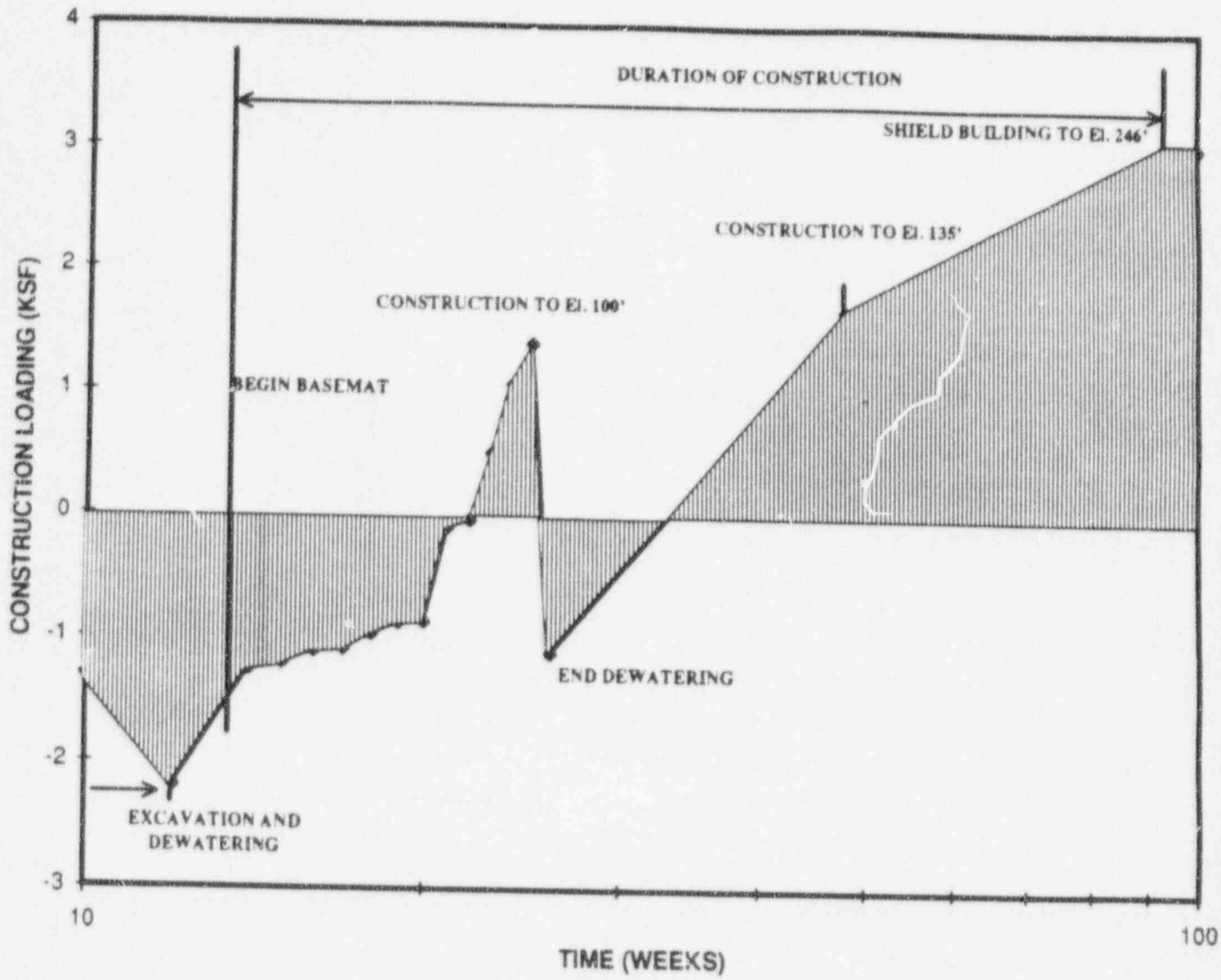
▲ 2 - Shield Building conc. to 84'

▲ 3 - Stop Dewatering

FIGURE 1
SIMPLIFIED TIME LINE OF
CONSTRUCTION ACTIVITIES
BASE CONSTRUCTION SCHEDULE

13

PLOT	DRAWN BY	DVZ	CHECKED BY	FILE NUMBER
1=1		2/13/97	APPROVED BY	r3-f2-1652/97



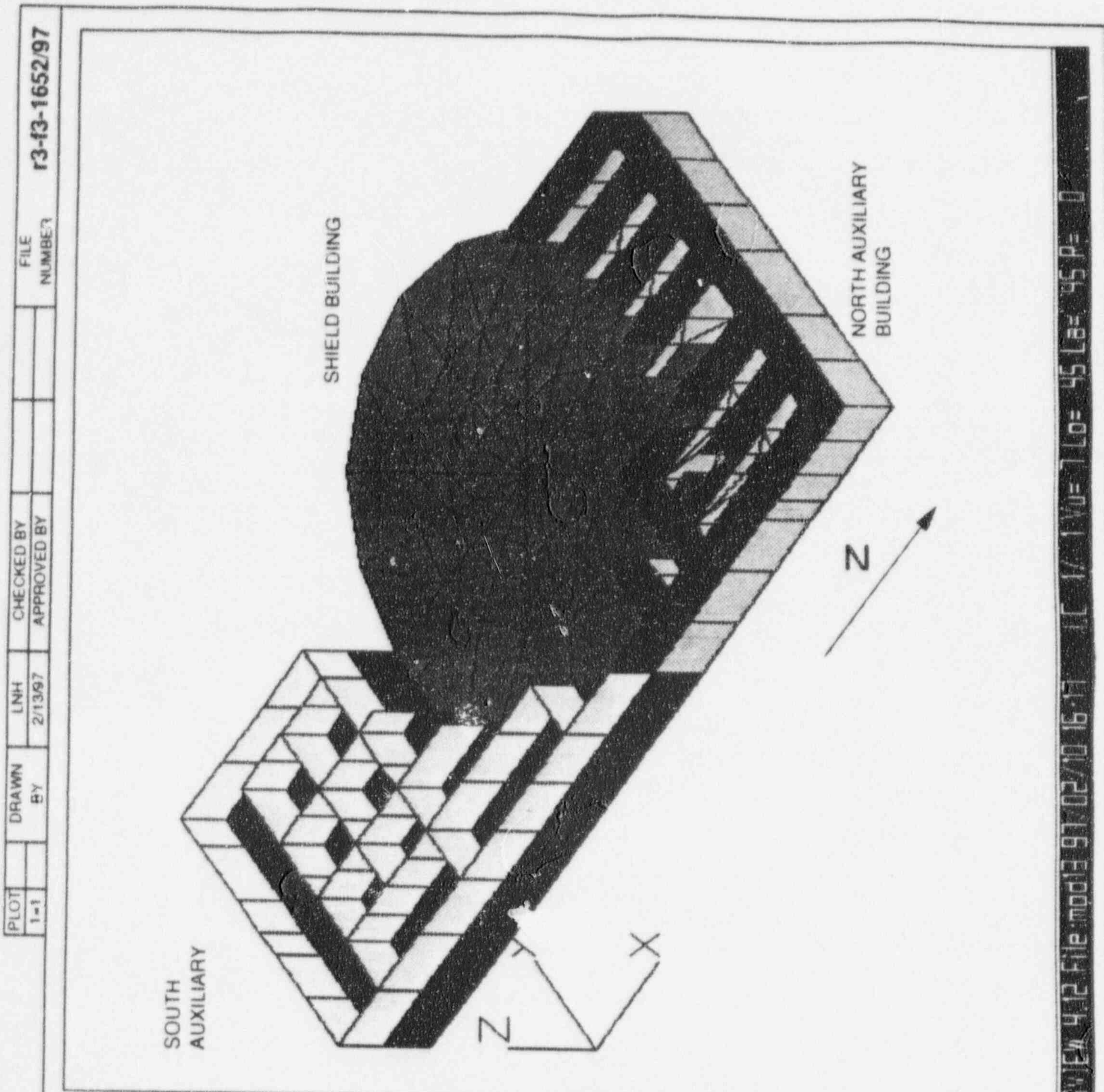
NOTES:

1. Construction Load is expressed on the net change in the average vertical stress at the base of the foundation.

FIGURE 2

IMPOSED LOADING
FROM CONSTRUCTION ACTIVITIES
BASE CONSTRUCTION SCHEDULE

Not to Scale

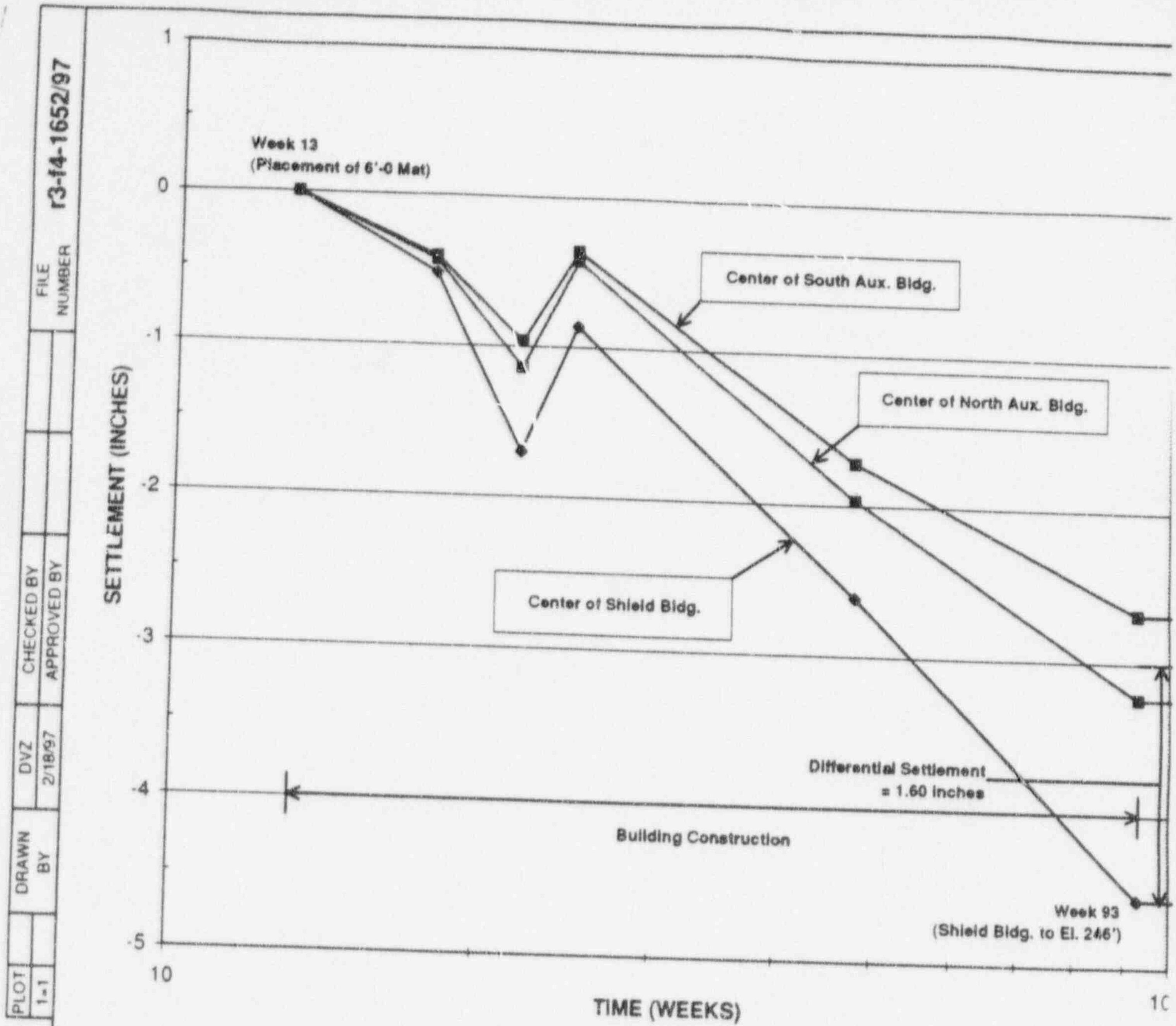


NOTE: Colors represent different material types used in the finite element analysis. Material stiffness was set to zero or Ec to model different stages of construction and variations on the base construction schedule.

FIGURE 3

FINITE ELEMENT MODEL OF AP600 BELOW-GRADE STRUCTURE

Not to Scale



NOTES:

1. Foundation Mat Displacement at week 93 reflects all time-dependent consolidation due to the entire construction load.
2. The differential settlement is found by taking the difference between the Center of the Shield Building and the average of the North and South Auxiliary Buildings.

FIGURE 4

FOUNDATION MAT VERTICAL DISPLACEMEN
ALTERNATING SAND AND CLAY SITE
BASE CONSTRUCTION SCHEDULE

Not to Scale