2.7 SUBSURFACE AND FOUNDATIONS

2.7.1 INTRODUCTION

This summarizes the results, analyses, and evaluation of the subsurface and foundation investigations. The field exploration and the laboratory testing were done under the supervision and direction of Bechtel Associates. These studies included site and area reconnaissance, field supervision of the boring operations, a review of pertinent literature, and the foundation analysis and evaluation. The initial graphic boring logs and laboratory test data cited were presented in the Preliminary Safety Analysis Report (PSAR). Subsequent data and subsurface profiles are presented here.

The Civil Engineering Design Report for the Emergency Diesel Generator Project describes: (1) exploration, (2) the properties of subsurface materials, (3) groundwater conditions, (4) response of soil and rock to dynamic loading, (5) liquefaction potential, and (6) static and dynamic stability at the site of the Diesel Generator Buildings.

2.7.2 EXPLORATION

2.7.2.1 Field Reconnaissance

The geologic field work for this site was started concurrently with a drilling program. The site reconnaissance was a continuation of the field work done in the early part of 1967. Local soil outcrops were examined on the Chesapeake Bay bluffs and the soils, types, orientation, and variations noted.

In addition to the site reconnaissance, the area was examined as the drilling progressed. Also, geological information was gained from the Maryland Geological Survey and various publications (References 1, 2, 3, 4, and 6). The geologic information gained from the site reconnaissance and literature review was used in conjunction with the borings to prepare the site geology portion of this section. Also, Section 2.4, Geology, gives a comprehensive presentation of the site and regional geology.

2.7.2.2 Boring and Sampling Investigations

In June and July 1967, a preliminary foundation exploration for the proposed nuclear power plant was conducted at the site by BGE. The field program included five borings, B1 through B5, and numerous split-spoon soil samples.

The field exploration for the PSAR and plant design began with the initial reconnaissance of the site by personnel from BGE and Bechtel Associates. Subsurface exploration started on August 17, 1967, and was completed on September 22, 1967. This exploration included drilling 22 soil test borings in the vicinity of the plant site and one geologic boring on a bluff approximately 2,000' north of the site for the Maryland Academy of Science.

During November and December 1968, a supplementary subsurface investigation was undertaken. A series of 18 borings were drilled in the plant area. The primary purpose of the supplementary program was to provide additional information for the plant foundation analyses.

In order to obtain additional information necessary for the design of the Intake Structure and switchyard foundations, a final subsurface investigation was undertaken during spring 1969. Five test borings were drilled in switchyard area and borings WB-1 through WB-35 were drilled from a barge offshore of the Intake Structure. The location plan of the borings is shown on Figures 2.7-1 to 2.7-3.

During all field investigations, a geologist or soil engineer from Bechtel Associates continuously supervised and inspected drilling operations and modifications in the boring programs as deemed necessary.

Split-spoon samples and undisturbed Shelby tube samples were obtained at desired sampling intervals. All samples were initially visually inspected and classified in the field. Samples were then either forwarded to Bechtel Associates' Washington Area Engineering Office for further examination, or to a soils laboratory for testing. The boring logs subsequent to the PSAR are shown on Figures 2.7-4 to 2.7-26.

2.7.2.3 Geotechnical Investigations to Support Diesel Generator Building Siting

Two field investigations were conducted, one in 1980/1981 and another in 1992, which assessed the geotechnical conditions at the proposed site of the Diesel Generator Buildings. In 1980/1981, a field investigation was performed to assess the North Parking Area's suitability as a location for a generic Category I structure. The investigation consisted of sample borings, cone penetration soundings, and observation wells. In 1992, a second field investigation was performed which consisted of sample borings, dilatometer soundings, a crosshole seismic survey, and field soil resistivity tests. In both investigations, standard penetration test samples were obtained in accordance with ASTM D 1586. Thin-walled tube samples were obtained in general accordance with ASTM D 1587. The crosshole seismic survey was performed in accordance with the requirements of ASTM D 4428/4428M-84 by using the "preferred method." Field soil resistivity testing was conducted using the "Wenner four-electrode method" in accordance with ASTM G 57. Additional details about the geotechnical investigations can be found in the Civil Engineering Design Report for the Diesel Generator Project (Reference 29).

2.7.3 SITE CONDITIONS

2.7.3.1 Area Geology

A geology summary for the purpose of understanding the foundation evaluation is presented herein. The main geologic presentation is in Section 2.4.

The CCNPP site lies in the Atlantic Coastal Plain physiographic region of Maryland. It is in an area of sedimentary deposits formed by the ancient rivers which carried large quantities of solids from the northern and western uplands of the Piedmont and Appalachian physiographic provinces into the once larger Atlantic Ocean. These deposits were formed in both a freshwater (fluvial) and a saltwater (marine) environment.

The upper deposits in this coastal plain area are the Recent and Pleistocene deposits of tan and brown silts, sands and clays with some inclusions of seashell fossils. Below the Pleistocene deposits lie the older Miocene sediments. The Miocene deposits represent the soils of significant interest for the foundations of this power plant. The foundation properties of both the Pleistocene and Miocene are more than adequate for the plant loads. The crystalline basement rocks are approximately 2500' below the present ground surface.

2.7.3.2 Soil Conditions

For foundation engineering purposes, the soils at the site can be divided into an upper zone and a lower zone. These soils are a mixture of marine and fluxial deposits. Each zone has both continuous and discontinuous strata with isolated pockets or lenses of slightly different materials. The soils are non-uniform sedimentary deposits of silty sands, sandy silts, clayey sands and sandy clays with layers of shell fossils.

The upper zone is generally yellowish tan, brownish tan and light brown in color. This color is caused by oxidation of the mineral constituents. The soil is firm to dense in consistency. The predominant soil types are sandy silts and silty sands. The average thickness is 18' and varies in elevation dependent upon the topography. This zone is primarily the Pleistocene deposit.

The lower zone, the Miocene deposit, is greenish gray in color with several occurrences of medium to light gray soil at the top of the zone. The soil below +5' MSL is very dense to extremely dense with a few lenses, which are isolated but dense in consistency. The major soil types are sandy silts, silty sands, and slightly clayey sands. The lower part of this zone is classified as fine sands and silts.

The upper zone of soil can support light loads, on the order of 2000 to 3000 psf, with a small amount of anticipated consolidation, while the lower zone can support heavy loads on the order of 15,000 to 20,000 psf with slight consolidation.

The original groundwater surface was between +15' and +20' MSL in the plant area; however, a permanent pipe drain system, subsurface drain system, surrounding the plant will maintain the ground water below Elevation +16'. Additional information concerning groundwater appears in Section 2.5. Subsurface profiles are shown on Figures 2.7-27 and 2.7-28.

2.7.4 LABORATORY TESTING

The laboratory testing program provided the soils' physical characteristics for foundation design. The testing was conducted in accordance with currently accepted procedures (References 7 through 12).

The testing program was divided into three parts, to determine the soil parameters under static, dynamic, and remolded (fill) conditions. The laboratory program included: grain size and specific gravity tests to determine particle size and distribution; Atterberg limit tests to determine soil plasticity characteristics; consolidation tests, to determine the soil settlement characteristics; unconfined compression and static triaxial shear tests to aid in the evaluation of foundation bearing capacity and slope stability analysis; dynamic triaxial shear tests to determine the dynamic properties used in the evaluation of liquefaction potential of foundation materials; compaction tests; and numerous moisture-density, void ratio and relative density determinations.

2.7.5 STRUCTURAL DATA

The foundations for the Turbine Building, Auxiliary Building, Containments, Turbine-Generators, and Circulating Water System are mat foundations on the Miocene soils. Individual bearing capacities were required because such a value depends on load, elevation of foundation, settlement tolerance, foundation size, and proximity to other loads. The final design bearing loads are as follows:

Structure	Contact Pressure
Containment Structure Mat	8000 psf
Auxiliary Building Mat	8000 psf
Turbine Pedestal Mat	5000 psf
Turbine Building Column Footings	5000 psf
Intake & Discharge Structure Mat	2500 psf

In all the above cases, the allowable soil bearing capacity exceeds the contact pressure. Two groups of structures, the circulating water structures (i.e., intake and discharge structures), and the switchyard structures have been studied since the PSAR and are discussed below.

2.7.5.1 Circulating Water Structures

The intake structure is located between the Turbine Building and shoreline, and is approximately 90 x 385’ in size. The total effective load due to the structure is approximately 42,000 tons. As a result, net soil pressures due to the structure will be approximately 2500 psf. The size and total loads were increased due to design changes necessitated by the structure being changed from a partial to a total Category I Structure. A 300’ segment of anchored sheet piling extends from the intake structure to the inside intake channel at the shoreline. The invert inlet elevation of the intake structure is Elevation -26’ MSL and the elevation at the junction of the anchored sheet piling and the intake channel is -51’ MSL. The approximate slope of the excavation from the intake inlet to the channel junction is approximately 10 horizontal to 1 vertical. The channel extends 4500’ offshore from the shoreline with 5 horizontal to 1 vertical side slopes excavated in the dense, silty slightly, cemented sand.

The discharge facility is located north of the plant. It consists of four conduits extending from the Turbine Building to a point 850’ offshore. The top of the conduit is Elevation -6’ MSL and the invert is at Elevation -19.5’ MSL at the point of discharge. The conduit will be constructed and buried by cut and cover methods. The plans of the intake and discharge scheme are shown on Figure 2.7-29.

2.7.5.2 Switchyard Structures

The switchyard area is located approximately 500’ west of the plant area. The north half of the yard is in a cut area, the maximum cut being approximately 30’and the added extension bay in on the hill cut area. The south half of the yard is a fill area with a maximum thickness of about 20’.

Analyses show an allowable soil bearing capacity of 2000 psf in the fill area and 3000 psf in the cut excavated area. Drilled piers were used where higher loads and/or uplift conditions required deep foundations.

2.7.6 FOUNDATION EVALUATION

The soils at this site are suited for the construction of the plant. The upper zone, Pleistocene soils, will support light loads of 2000 to 3000 psf without adverse settlements. The lower zone, the Miocene, is exceptionally dense and will support heavy foundation loads on the order of 15,000 to 20,000 psf.

2.7.6.1 Site Excavation and Earthwork

The general site grading and excavation was done with conventional earth moving equipment. Soil compaction requirements were prepared and based on the proposed utilization of a filled area. In general, the bearing capacity of the fill was not the controlling engineering parameter; but rather, it was found that settlement controls.

A maximum settlement of 1" was the limit set in the computations to determine the contact pressure for 10’x10’ and smaller footings to be placed on the compacted fills.

The following criteria were formulated for the various loading conditions based on the soil test data for the proposed fill materials.

Fill Compaction (minimum)	Areas Where Criterion Is Used
85% Standard Proctor (ASTM D698; AASHO T-99)	Shore protection landfill except within 100’ of bulkhead or anchor sheet piling and within 25’ of culverts.
90% Standard Proctor	General, nonstructure supporting, fill areas and plant parking lot lower than 5’ below finished grade.
95% Standard Proctor	Shore protection landfill within 100’ of bulkhead or anchor sheet piling and within 25’ of culverts and switchyard fill.
97% Standard Proctor	Structural backfill areas supporting facilities with footings 10’x10’ or smaller with contact pressure of 4000 psf or less.
95% Modified Proctor (ASTM D1557: AASHO T-180)	Roadway embankments and subgrades.
95% Modified Protector (ASTM D155; AASHO T-180)	Structural fill for the Diesel Generator Buildings (consisting of well graded, sound, dense and durable crushed stone).
100% Modified Proctor	Structural backfill areas supporting facilities with footings 10’x10’ or smaller with contact pressures of 5000 psf or less.

The above criteria covered the majority of the backfill conditions. Unique conditions of footing size and load were evaluated on an individual basis.

The minor plant excavation and embankment slopes were constructed according to the following tabulation:

Slope Height	Temporary Slope	Permanent Slope
0-30’	1:1	1 1/2:1
30-50’	1 1/4:1	2:1

These slopes have a factor of safety in excess of 1.5. In addition to the minor slopes, five other slopes adjacent to the plant were evaluated. Attached, at the end of this section, are the five cross-sections of the slopes around the plant. The locations of these slopes are indicated on Figure 2.7-30, "Slope Cross-Sections at Plant."

These cross-sections show the range of topographic conditions that exist. The backfill on the north, west, and south sides of the plant is to Elevation 45’, and on the east side to approximately Elevation 45’ but sloping to the Chesapeake Bay.

Stability analyses (Reference 24) were made for the design slopes shown on the cross-sections. The slopes shown on Section DD and EE are the maximum in height within the immediate plant area. These slopes have a safety factor in excess of 1.5. The other slopes around plant area are flatter or of less height; therefore, by inspection, safety factors greater than 1.5 can be assigned to these slopes. All of the slopes are acceptable for permanent slopes (Reference 22).

Two dynamic slope stability analysis methods were used to evaluate the safety of the slopes for conditions resulting from the SSE.

In the conventional method of dynamic slope stability analysis (Reference 24), the severity of the earthquake is expressed by relating the ground acceleration to the acceleration of gravity as a percentage. The horizontal severity is 8% g and the vertical severity is taken as two-thirds of this, or 5.3% for the OBE. For the SSE these values are 15% g for the horizontal severity and 10% g for the vertical severity. During the earthquake, all parts of the mass of soil are assumed to be acted on by a steady vertical and horizontal force, equal to unit weight times acceleration, in addition to all other forces to which the slope is subjected. These forces act in the direction of instability. With these two forces determined, the analysis was completed similarly to the conventional static analysis.

The other analysis method used was the procedure proposed by N.M. Newmark (Reference 23). This is fully explained in the cited reference; therefore, it is not discussed here.

The factor of safety for the dynamic conditions is approximately equal for both methods of analysis. These design slopes have a factor of safety of 1.3 or more, which is acceptable (References 13 and 22).

Also, an extensive slope stability analysis of the intake channel and structure was undertaken with the assistance of a computer program. The factor of safety was computed using the Swedish slip circle method of analysis. Both static and the SSE dynamic conditions were analyzed. The intake channel slopes away from the intake structure at a gradual slope of approximately 10 horizontal to 1 vertical. The minimum factors of safety for the intake structure were computed to be 2.7 and 1.6 for the static and dynamic conditions, respectively. The minimum factors of safety for the intake channel side slopes, which are 5 horizontal to 1 vertical, were computed to be 6.5 for static conditions and 2.0 for dynamic conditions. The minimum factors of safety for the critical circle perpendicular to the anchored sheet pile section of the intake were computed to be 3.0 for the static conditions and 1.6 for the dynamic conditions.

During investigations done to support siting the Diesel Generator Building, the stability of the western slope of the site was evaluated to determine a factor of safety under both static and dynamic conditions. For the static condition, an analysis was made of both the total and effective stress cases. Actual conditions will fall somewhere in between the two. Since dynamic conditions are developed under undrained conditions, only the total stress case was evaluated for the dynamic conditions. The Simplified Bishop method of computing the factor of safety was used to perform the slope analysis. The results of the evaluation are summarized as follows:

Condition	Factor of Safety
Static, Total Stress	1.72
Static, Effective Stress	1.74
Dynamic, Total Stress	1.22

Results of an earlier analysis performed on the western slope yielded similar results. The results of both analyses demonstrate that an adequate factor of safety exists against mass failure of the western slope of the site under static and dynamic conditions.

Since the crib wall adjacent to the Diesel Generator Buildings was not seismically designed, it is possible that some localized failure may occur during a seismic event. The postulated worst case is a complete failure of the crib wall, which would result in fill material sloughing against the wall of the Diesel Generator Building. The resulting wedge of fill material would reach a maximum height of 7.5’ above grade. The static lateral loadings which result from this failure were found to be enveloped by the loading imposed by the design basis tornado. The west wall of the Safety-Related Diesel Generator Building is, therefore, designed for the fill loading under dynamic conditions.

2.7.6.2 Plant Foundations

The foundation elevations, original ground surface and amount of stress unloading by excavation are shown below:

Structure		Average Ground Elevation	Foundation Elevation	Average Excavation Unloading
North Containment Structure		+75’ MSL	-1’ MSL	8400 psf
South Containment Structure		60	-1	6600
Auxiliary Building	West End	70	-14	8300
	East End	70	-19	8850
North Turbine Building		60	-11	7300
South Turbine Building		40	-11	4900
Intake Structure		80	-30	10800
Discharge Structure		20	-27	4050

Generally, the weight of soil removed by site grading and pit excavation for the structures is greater than the loads imposed by plant construction. This verified the results of the analyses made using the triaxial shear data, i.e., that bearing capacity is no problem. The ultimate bearing capacity of the foundation strata is in excess of 80,000 psf. The allowable bearing capacity is in excess of 15,000 psf.

In addition to bearing capacity, settlement of the proposed structures was also considered. The settlement of the foundations can be divided into two categories: (1) elastic settlement; and (2) time-dependent or hydrodynamic settlement.

Elastic expansion of the confined soil occurred as a result of excavation unloading. This resulted in a slight upward movement. During construction, the soil moved downward as load was applied. This elastic movement is small and was complete when construction was completed. It had no effect on the structures or function of the plant. The excavation unloading and structural loading caused a small change in void ratio. This change allowed a very small amount of hydrodynamic settlement to occur. The time-dependent or hydrodynamic settlement will be very small or negligible because the structural load is either less than the overburden removed, or only slightly greater than the removed overburden weight. Considering the types of soils present, contact pressures of 1500 to 2000 psf greater than the overburden removed would not result in large consolidation settlements. The magnitude of maximum possible post-construction settlement is 1/2".

The excavation for the power plant structures was below the water table. Conventional dewatering was done to maintain a dry and stable condition during construction.

2.7.6.3 Liquefaction Potential

If a loose, saturated sandy soil (a soil with less than 10 to 15% silt and clay fines and less than 200 to 300 psf cohesion) is subjected to ground vibrations, as during an earthquake, it tends to compact and decrease in volume. If the soil cannot drain during the rapid load fluctuations imposed by an earthquake, there is a buildup in pore pressure until it is equal to the overburden pressure. The effective stress then becomes zero, the soil loses its strength, and develops a "quick" or liquefied condition. If this condition is of a general areal extent and the pressure not otherwise relieved, it can cause a flow or bearing capacity failure (References 14, 16, 17, 18, 19, 20, and 21).

For the evaluation of liquefaction potential at this site, data were used from the dynamic triaxial testing, standard penetration resistances from the borings, in-place density determinations and geologic origin of the sedimentary soils at the site. All of these data showed that the soil at the site was not of a liquefaction potential. The dynamic tests showed exceptional strength under constant cyclic stress.

Other characteristics also support that there is no liquefaction potential at this site. The amount of material passing the No. 200 sieve in the gradation analysis (the silt and clay fines) and the amount of cohesion observed in the static triaxial shear tests supports the conclusion that there is no potential to liquefy. This is concluded because the soil is not truly cohesionless or reasonably suited to be susceptible to the liquefaction phenomenon. The last significant indicator that a liquefaction potential does not exist is the geologic origin of the site soils. The areas where liquefaction is believed to have happened are areas of flacial outwash, recent alluvium, or loose artificial fills (Reference 15). The Calvert Cliffs site soils are preconsolidated deposits several million years old.

During the siting of the Diesel Generator Buildings, the liquefaction potential of various strata of the North Parking area were evaluated using the standard penetration test blow counts obtained during subsurface explorations. Factors of safety against liquefaction were computed for all standard test blow counts performed for the loose-to-medium dense granular sand strata (considered to be the most susceptible to liquefaction). These computed factors of safety ranged from 1.3 to 2.4 with a median value of 1.8. Previous analyses performed in 1981 indicated a minimum factor of safety of 1.37 against liquefaction based upon standard penetration test blows, and between 1.6 and 2.0 based upon laboratory cyclic triaxial tests data. The results indicated that the site of the Diesel Generator Buildings has an adequate factor of safety against liquefaction under design earthquake conditions.

2.7.6.4 Lateral Earth Pressure

The lateral earth pressures for the walls of this plant were evaluated for both the static and dynamic conditions.

The rigidity of the walls and the fact that backfilling was done after the walls were framed at the top do not allow sufficient movement for developing the active earth pressure case.

Therefore, the at-rest condition was developed. For convenience of design, the earth pressure has been converted to an equivalent fluid pressure method. This utilizes the Ranking approach which is a conservative estimate of lateral earth pressures. The earth pressure has been determined based on the characteristics of the material stockpiled for use as backfill. This backfill was sand, silty sand, and gravely silty sand.

The equivalent fluid unit weight above the water table is 47 lbs/ft^3. Below the water table, the equivalent fluid unit weight is 85 lbs/ft^3. The pressure distribution will be hydrostatic.

The dynamic earth pressures were considered for this plant. The analysis was based on work by N. M. Newmark, Y. Ishii, et al., K. Terzaghi, and the Corps of Engineers (References 25, 26, 27, and 28, respectively). These references provided at-rest coefficients of earth pressure which are dependent on the magnitude of the earth shock acceleration. Based on this information, an increase of the equivalent fluid unit weight should be 10% and 17% for the OBE and SSE, respectively.

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