2.4 HYDROLOGIC ENGINEERING

2.4.1 Hydrologic Description

2.4.1.1 Site and Facilities

The site is located on an irregularly shaped prominence in the Delaware Estuary. It is believed that hydraulic fill, dredged from the Delaware River or Bay was placed on and between two small bars. The preconstruction configuration of the area is shown on Figure 2.4-1, Map of Area.

The area was and is quite flat, previously having an average elevation of about 9 feet above sea level. This was raised slightly in the plant area, to Elevation +10.5 Mean Sea Level (MSL) or 99.5 Public Service Datum (PSD). A levee, about 10 feet high, had been constructed around most of the westerly bar. As subsequently discussed, this levee became the basis for a protective sea wall. The predominant form of vegetation is Phragmites, a rather tall reed-like grass which is characteristically found in low-lying wetlands in the region.

Aside from the access roads and bridges, the only modification to the island and the adjacent river and marsh area is within the station construction area in this area. The site grade has been raised about 1 1/2 feet except for the protective structures at the shoreline. There is a slight gradient toward the Delaware Estuary. The present configuration of the site is shown on Plant Drawing 232091.

There was no established systematic surface-drainage system on the site prior to construction. Precipitation either ran off to the Delaware Estuary in a random pattern or collected in puddles where it infiltrated into the ground or evaporated. All surface drainage at the site flowed directly into the Delaware Estuary.

The island upon which the site is located is separated from the New Jersey mainland by Hope Creek, a tidal stream which connects Alloways Creek with the Delaware Estuary. Hope Creek drains a rather large marsh, and has undergone some channel dredging and straightening. It is a brackish water stream and is used to a small extent for fishing and hunting.

Studies of historical high and low water elevations indicated a maximum high water mark of 8.5 feet MSL datum, Sandy Hook (+97.5 PSD), and minimum water level of -5.9 feet MSL datum (83.1 PSD).

Station structures have been designed to not only withstand extreme recorded water levels, but also postulated extreme conditions, as subsequently discussed. Safety-related structures have been designed as follows:

1. The service water pumps can operate to a low water level of 76 feet PSD.

2. The service water structure is shown on Plant Drawing 211612. The portion of the service water intake enclosing the pumps, motors, and vital switchgear is watertight up to Elevation 126 feet PSD with wave runup protection to elevation 128 feet PSD. The service water intake can also withstand the static and dynamic effects of the storm. Each vertical, turbine type service water pump column bowl and suction bell is installed in an individual chamber which is open to the river. The chamber is isolated from the watertight compartments where the pump discharge heads and motors are located. The pump discharge heads are bolted down to pads at Elevation 92 feet 6 inches. The joint between the pump discharge head and the pad at Elevation 92 feet 6 inches is watertight to prevent leakage of water into the compartments. Provisions have also been made to prevent leakage from the discharge head glands and leakoff connections into the watertight compartments. A sump pump is provided in each

compartment to remove any accumulated water in the event a minor leak should occur.

3. All safety-related structures are watertight.

4. The Containment is, by nature, watertight and can withstand the static and dynamic loads associated with a storm producing a stillwater level of 113.8 feet PSD and the corresponding wave runup to 120.4 feet PSD (See Section 2.4.5 for the design storm water levels.)

5. The Auxiliary Building is watertight up to Elevation 115 feet PSD. All doors in the outer Auxiliary Building walls below Elevation 120.4 feet are watertight. All watertight doors and structural walls can withstand the static and dynamic effects associated with a storm that produces a stillwater level of 113.8 feet PSD with wave runup to Elevation 120.4 feet. Conduit penetrations above Elevation 115 feet and below Elevation 120.4 feet are packed to eliminate gross inleakage during the design storm.

   Each residual heat removal pump room, the lowest point in the Auxiliary Building, contains two sump pumps, each adequate to provide the minimum capacity of 50 gpm.

6. The main steam and feedwater pipe penetration area is watertight below Elevation 120.4 feet. The structural walls and watertight doors are also capable of withstanding the static and dynamic effects of the storm

which produces a stillwater level of 113.8 feet PSD and wave runup to 120.4 feet PSD.

2.4.1.2 Hydrosphere

The station is located on the east shore of the estuarian zone of the Delaware River - Delaware Bay system. Delaware River flow enters the head of Delaware Bay 2 miles downstream of the site. The largest tributaries of the Delaware River are the Schuylkill River in Pennsylvania; the Christina River in Delaware; the Assunpink, Crosswicks, Rancocas, and Salem Rivers; and Big Timbers, Hope, and Alloways Creeks in New Jersey.

The head of the Delaware Estuary is at Trenton, New Jersey, about 83 miles upstream of the site. The Chesapeake and Delaware Canal, which connects the Delaware River with Chesapeake Bay, is located about 7 miles north of the Salem site. Figure 2.4-4 presents the site location in relation to the surrounding area.

The Delaware River has a drainage area of 12,765 square miles and its average freshwater discharge into the head of the estuary at Trenton is about 12,000 cfs (16,000 cfs at the site). The average tidal flow at Wilmington, Delaware, about 20 miles above the site, has measured at 400,000 cfs. Hence the tidal flow, which greatly exceeds the runoff flow, dominates the flow velocity at the site. The normal daily range in the height of the tide at the site is 5.8 feet. Larger fluctuations have been caused by hurricanes which bring heavy precipitation and may cause storm surges and severe wave action, and by strong northerly winds which push the Delaware River water into Delaware Bay. The highest tide ever recorded in the vicinity of the site (+8.5 feet MSL) occurred in November 1950. The lowest tide likely experienced, based on projections of data recorded at Reedy Point, Delaware, would have occurred on January 25, 1939 (-5.9 feet MSL). Hence, the maximum estimated historical tidal range is about 14.4 feet.

The net tidal flow has been estimated at 400,000 cfs, which produces a relatively high current velocity in the station vicinity.

Some small dams are in existence well upstream of the site (in New York State). Currently no major dams are planned for the river. As subsequently discussed (Section 2.4.2) the existence of dams upon the Delaware River does not influence the site safety analysis.

The nearest public water supply is located about 8 miles northeast of the site. It utilizes both surface water and groundwater. There are five other public water supplies in New Jersey within 25 miles of the site and five in Delaware within 15 miles of the site. All are located upgradient from the site.

Private water supplies in the area utilize groundwater as a source of water. The nearest producing well is located more than 2 miles from the site. There are 20 known wells in New Jersey within 4 miles of the site. All are located upgradient from the site. For a more detailed discussion of groundwater supplies, see Section 2.4.13.

2.4.2 Floods

The water body to the west of the site is considered to be a tidally affected estuary by the U. S. Geologic Survey. As such, water levels are recorded by tidal gauges and no "flood record" is kept. The tidal flow in the site area is estimated to be more than an order of magnitude greater than the average fresh water flow in the site vicinity. Thus, maximum and minimum water levels that may be of concern to plant safety were derived through considerations of coastal environmental conditions rather than riverine conditions.

2.4.3 Probable Maximum Flood

Not applicable, see Sections 2.4.2 and 2.4.5.

2.4.3.1 Probable Maximum Precipitation

The maximum probable rainfall is of consideration only in design of yard drainage facilities and as a possible loading on critical structures, not as it may pertain to river flooding.

The Yard Drainage System is designed to pass the drainage associated with a rainfall rate of 4 inches per hour for a period of 20 minutes (based on 90 percent runoff from paved areas and 50 percent runoff from graded areas). This rainfall intensity has a return frequency of 15 years (see Figure 2.4-5) and therefore, an unusually severe storm producing a rainfall rate in excess of 4 inches per hour for time periods of less than 20 minutes can be handled by the system.

In the unlikely event that the Yard Drainage System were to be loaded beyond its capacity, the excess water would accumulate and run off as the storm subsided. All doors and penetrations in the Class I (seismic) buildings are watertight up to Elevation 115 feet (PSD). The interior drains in the Auxiliary and Fuel Handling Buildings are independently piped to the Liquid Waste Disposal System and are not connected to the Yard Drainage System.

Roof drains are designed to dispose of a maximum rainfall rate of 4 inches per hour for a period of 20 minutes through the Yard Drainage System. Roof slabs are watertight to prevent building interiors from being damaged by severe rainstorms. The slabs are designed to withstand a loading equivalent to a depth of water up to the full height of the building's parapet or roof curb. In the unlikely event that some of the roof drains become plugged, the backed up water will spill down the outside of the building. Wall penetrations above Elevation 115 feet (PSD) on Class I (seismic) buildings are designed to prevent roof spillage or heavy rain from seeping inside the building.

In the event the capacity of the Yard Drainage System were to be exceeded as a result of an unusually severe rainstorm, the excess water would accumulate in puddles in the vicinity of the catch

basins and run off. This water would not enter any safety-related structure, since these structures are watertight up to Elevation 115 feet (PSD). Therefore, safety-related equipment would not be adversely affected as a result of a severe rainstorm.

2.4.4 Potential Dam Failures

Not applicable, see Sections 2.4.2 and 2.4.5.

2.4.5 Probable Maximum Surge and Seiche Flooding

2.4.5.1 Probable Maximum Winds and Associated Meteorological Parameters

Probable Maximum Hurricane (PMH) storm surges have been calculated for the site using the bathystropic storm tide theory described by Marinos and Woodward (1968) (1). The hurricane surge was computed at the mouth of Delaware Bay and routed up the bay in accordance with a method described by Bretschneider (1959) (2).

Components of the stillwater level are 1) the mean low water depth, 2) the astronomical tide, 3) the rise in water level resulting from the hurricane's atmospheric pressure reduction, 4) the wind stress component perpendicular to the bottom contours (onshore wind components), 5) the wind stress component parallel to the bottom contours which produces a longshore flow that is deflected to the right (in the northern hemisphere) by the Coriolis forces, and 6) the initial surge (a slow general rise in sea level existing before the actual hurricane winds arrive).

The PMH is defined by the U. S. Department of Commerce Report HUR 7-97 (3) as: "A hypothetical hurricane having that combination of characteristics which will make it the most severe that can probably occur in the particular region involved. The hurricane should approach the point under study along a critical path and at an optimum rate of movement." Indices used to calculate maximum storm surge are taken in part from HUR 7-97 where values are grouped according to defined coastal zones and by latitude within each zone. The following parameters and characteristics are based on empirical observations, assumptions, and experience. PMH indices and parameters include:

1. CPI (P_o) - The maximum surface pressure in the center of a particular hurricane, in inches of mercury.

2. Asymptotic Pressure (P_n) - The surface pressure at the outer limits of the hurricane, in inches of mercury.

3. Radius of Maximum Winds (R) - The distance from the storm center to the point of maximum wind velocity in nautical miles.

4. Forward Speed (V_t) - Rate of forward movement of the center of the storm, in knots.

5. Maximum Wind Speed (^{U}Max) - The absolute highest surface wind speed in the belt of maximum winds (measured as a maximum average 10-minute wind at a height of 30 feet above the water) calculated using equations from HUR 7-97.

6. PMH Path - The path selected for the PMH's approach is a critical factor, which in combination with other indices will determine the duration and magnitude of the storm winds over the critical fetch and the resulting peak hurricane surge elevation at the site. The path which produces peak hurricane surge will approach the area of

interest normal to the general bottom contours. The hurricane's center will pass to the left (when facing shoreward) of the profile through the bay by a distance that allows the hurricane's maximum winds to pass directly over this profile.

7. Astronomical Tide (Ha) - Data for the predicted high astronomical tides are taken from the National Oceanic and Atmospheric Administration Tide Tables.

8. Initial Surge (Hi) - The initial surge is attributed to a tidal anomaly evaluated on the basis of variations between the observed and predicted tide. Data for initial surge as determined by the Coastal Engineering Research Center (CERC) were used.

9. Bottom Friction Coefficient (k) - The bottom friction coefficient is a function of several variables, among them the slope and width of the Continental Shelf in the area of study.

10. Wind Speed Adjustment Near Shore - The computed overwater wind must be adjusted when moving onshore. The overwater wind field was reduced, from its full value 2 miles offshore to 0.89 of its full value at the shoreline.

11. Wind Stress Factor - The wind stress factor is generally given as a function of wind speed, although other variables enter into its determination. The wind stress factor relationship suggested by CERC was used for the surge computations in this report.

Analyses were undertaken to predict the surge heights at the mouth of Delaware Bay generated by a PMH at latitude 39°N. Maximum surge elevation was calculated by moving the hurricanes across the continental shelf on a track normal to the bathymetric contours.

The track of the postulated hurricane is shown on Figure 2.4-6. Two different forward speeds of translation were used to determine the effect that the rate of forward movement of the hurricane would have on the surge elevation.

The PMH utilized in the analyses was a large radius, moderate forward speed hurricane which generated the maximum surge on the open coast. The quantitative meteorological parameters describing the PMH are:

1. CPI: 27.09 inches Hg

2. Peripheral Pressure: 30.72 inches Hg

3. Radius of Maximum Winds: 39 nautical miles

4. Maximum Wind Speed: 132 miles per hour

5. Forward Speed: 27 knots

A computer program was developed by Dames and Moore using previous work by the Galveston District Corps of Engineers.

The program is described by Marinos and Woodward (1968) (1). Input data to the computer program describing the storm and the bathymetric conditions included the basic parameters of the hurricane, an initial surge of 1 foot, wind friction factor, bottom friction factor (0.008), wind speed at various radial distances and angles of wind direction relative to the translational velocity vector of the hurricane, bathymetric traverse data and astronomical tide (5.6 feet).

Winds which approach the site from a direction off the axis of the bay produce a component which is perpendicular to the axis of the bay. This cross-wind component causes the water surface to be raised on the upwind side of the bay and depressed an equal amount on the downwind side of the bay.

As the PMH is moved along its postulated track, wind speed and direction at the site change because of the effects of friction and filling over land and also because of the position of the storm center with respect to the site. The cross-wind effects were calculated for the six wind directions chosen for analysis. The six wind directions or fetches radiate downbay from the site at 15-degree intervals from the east bank of Delaware Bay.

The calculations consist of determining the corrected wind speed along the fetch, the cross-wind component of the wind speed, and the resulting cross-wind setup or drawdown. A summary of the calculations for each of the fetches is presented in Table 2.4-1.

The wind speed was corrected to include the effect of the fetch distance from the storm center and also for friction and filling overland.

The computer maximum surge elevation at the mouth of Delaware Bay was 21.9 feet above mean low water. This surge included the effects of the astronomical high spring tide.

The maximum surge of 21.9 feet above mean low water at the mouth of Delaware Bay was routed to the site using the procedure of Bretschneider (1959) (2). The model surge hydrographs for Delaware Bay computed by Bretschneider were then used to determine hurricane surge values at the Salem site (which is within Bretschneider's Section 4) as a function of time.

The maximum stillwater elevation at the site is a combination of the storm surge and the crosswind setup or drawdown. Storm surge elevations have been calculated for the six fetches chosen and are presented in Table 2.4-1 with the computed crosswind setup and the maximum stillwater elevation at the site. The six wind fetches radiate downbay from the site at 15-degree intervals from the east bank of the Delaware Bay. Subsequently, site hydrologic design parameters were developed using a maximum surge elevation of

113.8 feet PSD, as recommended by the Nuclear Regulatory Commission consultants.

Table 2.4-2 contains a list of agencies and individuals contacted relative to this section.

2.4.5.2 Surge and Seiche History

A review of local tidal gage history indicates that the maximum recorded water level was +8.5 feet MSL. It was recorded in November 1950. The lowest recorded level reached -5.9 feet MSL on January 25, 1939. The lowest "historic" water levels at the site that could be postulated from projections of data recorded in Philadelphia (December 31, 1962) (4) is -8.0 feet MSL.

2.4.5.3 Surge and Seiche Sources

The most severe storm postulated for the site is the PMH. The PMH indices developed by the U. S. Department of Commerce studies (Memorandum HUR 7-97) (3) and utilized by CERC were described in Section 2.4.5.1.

2.4.5.4 Wave Action

The primary factors influencing the generation of waves will be the maximum wind speed over the water, the effective fetch length, and the average depth of water along the fetch. The values of these parameters used in the computations of wave heights and periods were determined for the fetches analyzed by:

1. Determining the location of the center of the storm required to produce winds along the fetch,

2. Calculating corrected wind speeds to account for friction and filling over the land and distance from the storm center to the fetch center,

3. Calculating the still water elevation at the center of the fetch due to storm surge at the time the storm center is located to produce the maximum wind speed along the pre-selected fetch,

4. Computing the average depth along the fetch.

The basic assumptions used in the analyses were:

1. Storm generated waves from the open sea are dissipated at the mouth of Delaware Bay.

2. Steady state waves are generated along each fetch (these waves are independent of time).

3. Only the area northwest of Ben Davis Point generates significant wave energy at the site.

The PMH was located so as to produce maximum waves. In the vicinity of the site, the PMH winds had a maximum sustained wind velocity of 85 miles per hour from the southeast. With the surge level at 113.8 feet PSD, wave runup elevations on safety-related structures inside the sea wall were calculated to be a maximum of 120.4 feet PSD. Maximum wave run up elevation on the service water intake structure was calculated to be 127.3 feet PSD.

2.4.5.5 Resonance

As a result of the nature of the estuary upon which the site is located, resonance was not a necessary consideration.

2.4.5.6 Runup

As noted in Section 2.4.5.4, maximum wave runup elevation was calculated to be +120.4 feet PSD on critical structures inside the sea wall and 127.3 feet PSD on the service water intake structure. The Sainflou method was used, assuming a minimum sea wall height of Elevation 108 feet PSD in the most critical area. As discussed in Section 2.4.1, all safety-related structures are protected for water levels to equal or greater elevations.

2.4.5.7 Protective Structures

[redacted]

The stability of the dike was checked by Dames and Moore, using a computer program based on the Fellinius method of slices under the effect of the assumed wave forces. Some of the softer soils in the previously existing dike area were replaced with granular fill.

2.4.6 Probable Maximum Tsunami Flooding

The occurrence of tsunamis is infrequent in the Atlantic Ocean. Other than the tidal fluctuation recorded on the New Jersey Coast during the Grand Banks earthquake of 1929, there has been no record of tsunamis on the northeastern United States coast.

The earthquake of November 18, 1929, on the Grand Banks about 170 miles south of Newfoundland, resulted in a tsunami which struck the south end of Newfoundland about 750 miles northeast of the Massachusetts coast. The tsunami occurred at a time of abnormally high tide and resulted in some loss of life and destruction of property. The effect of this tsunami was recorded on tide gages along the United States east coast, as far south as Charleston, South Carolina. A tidal fluctuation of approximately nine-tenths of one foot was noted at Atlantic City, New Jersey and Ocean City, Maryland.

The Lisbon earthquake of November 1, 1755, produced great waves, which contributed heavily to the destruction on the coast of Portugal. These waves were noticeable in the West Indies. It had been reported that the Cape Ann, Massachusetts, earthquake of November 18, 1755, caused a tsunami in Saint Martin's Harbor in the West Indies; however, there is no record of a tsunami occurrence along the east coast of the United States at this time and it has since been determined that the Saint Martin's Harbor report actually refers to the tsunami caused by the Lisbon earthquake, which occurred within three weeks of the Cape Ann shock. Some tsunami activity has occasionally followed earthquakes in the Caribbean, but none of these was reported in the United States.

There is no evidence of surface rupture in East Coast earthquakes and no history of significant tsunami activity in the region. Hence, we do not believe that the plant site would be subjected to any significant tsunami effect. The maximum expected tsunami would result in only minor wave action, and the maximum expected storm wave effect is the critical factor in design.

2.4.7 Ice Flooding

Ice barriers are provided for the service water intake structure. Surface ice jams will not exert direct structural loading. The barrier will also enable the intake components to operate normally without the effect of ice.

2.4.8 Cooling Water Canals and Reservoirs

The Delaware Estuary is the cooling water reservoir for the plant. For discussions of the design parameters intended to provide a

secure source of water, see Sections 2.4.1.1, 2.4.2, 2.4.3, 2.4.5, 2.4.10, and 2.4.11.

2.4.9 Channel Diversions

As the source of cooling water is the Delaware Estuary, no channel diversions need be considered.

2.4.10 Flood Protection Requirements

The relationship of hurricane induced surge and wave flooding and the site design parameters are discussed in Sections 2.4.1.1 and 2.4.5. No other possible sources of flooding are as critical; hence, station design was predicated upon the worst possible meteorological event as previously described (Section 2.4.5).

2.4.11 Low Water Considerations

2.4.11.1 Low Flow in Rivers and Streams

Not applicable, see Sections 2.4.2 and 2.4.5.

2.4.11.2 Low Water Resulting from Surges, Seiches, and Tsunamis

The anticipated minimum stillwater elevation for the Delaware River Estuary in the vicinity of the Salem Nuclear Generating Station is -10.6 feet MSL. This extreme water level was developed from critically locating a postulated PMH (HUR 7-97) (3).

The PMH was located in its more severe position as follows:

Latitude of storm center: 39 degrees north.

1. CPI: 27.09 inches Hg

2. Peripheral pressure: 30.72 inches Hg

3. Radius of Maximum Winds: 39 nautical miles

4. Forward Speed: 0 knot

5. Maximum Wind Speed: 124 miles per hour

The location of the storm center was chosen so that the radius of maximum winds from the northwest would coincide with the axis of the bay between the Salem site and the mouth of the bay. The location of the storm is shown on Figure 2.4-8.

The maximum winds associated with the PMH would be from the northwest (N45°W) along the axis of Delaware Bay when the stillwater level is at the postulated minimum. In the vicinity of the site, the maximum wind velocity would be 85 miles per hour. With the stillwater level at -10.6 feet MSL, the winds would generate waves having a significant wave height and period of

5.0 feet and 4.8 seconds, respectively. This would correspond to a maximum wave height of 8.3 feet. The waves would travel along the axis of Delaware Bay in the most critical condition.

Routing these waves to the service water screen well structure, the waves will undergo the effects of refraction, diffraction, and breaking. With the maximum winds of 85 miles per hour from the northwest, local waves trying to refract into this wind would become unstable and break; therefore, the effects of refraction have been ignored.

The offshore topography from the service water screen well indicates that during the PMH low water level, there would be exposed shoreline with a northwest alignment, adjacent and to the northwest of the service water screenwell, projecting about

150 feet into the Delaware River from the entrance to the service water screen well. Waves coming from the northwest would diffract around this exposed point of land in reaching the screen well entrance. The significant wave height would diffract to 1.5 feet in height while the maximum wave height would first be subjected to breaking due to depth restrictions. A maximum nonbreaking wave of 6.5 feet would diffract to a height of 2.0 feet in reaching the screen well.

As the diffracted waves pass the screen well entrance, they will undergo several severe effects causing the wave to become unstable and deformed in shape. Some of these effects are: further diffraction of the waves as they strike the protruding ice barriers and enter the individual service water pump channels, and the reflection of waves in several directions causing a confused sea state at the screen well entrance. To be conservative, the pump channel walls and the ice barriers were treated as a pile array. Using this assumption, the 1.5 feet and 2.0 feet wave heights would be reduced to 1.1 feet and 1.5 feet, respectively, as they entered the individual pump chambers.

These waves then must travel 50 to 60 feet in reaching the service water pumps, passing through a trash rack, curtain wall, stop log guide, ladders, etc. Therefore, there essentially would be no wave action at the pumps, but only a choppy water level. Water level amplification due to resonance is negligible because the fundamental period of the pump channels is approximately 13 to 16 seconds and the only possible wave excitation would come from a high order harmonic, resulting only in ripples.

It is concluded that the highest possible wave at the service water pumps is 0.8 feet to 1.0 feet in height resulting in a water level change of approximately plus or minus 0.5 feet. Therefore, the lowest instantaneous water elevation at the service water pumps is -11.1 feet MSL.

2.4.11.3 Historical Low Water

See Sections 2.4.2 and 2.4.5.

2.4.11.4 Future Control

There are no provisions required for control of the flow in the Delaware Estuary area.

2.4.11.5 Plant Requirements

Plant water requirements are predominantly determined by the need for heat dissipation within the plant. The primary heat removal system is the Circulating Water System. The monthly flow is about 9.6×10^{10} gallons, total for both units. The Service Water System averages approximately 4.3×10^{7} gallons per month (both units). Requirements in a safe shutdown mode are much less. However, even using operating flow as a criterion, the daily average plant requirement is only about one-eighth of the tidal flow.

2.4.11.6 Heat Sink Dependability Requirements

Essentially, the ultimate heat sink is the Atlantic Ocean. The Water Intake System is designed to operate at the lowest postulated water level in the estuary (Elevation -13.1 feet MSL). Also see Sections 2.4.1, 2.4.11.2, and 2.4.11.5.

2.4.12 Environmental Acceptance of Effluents

The significance of onsite release of effluent is also discussed in Section 2.4.13.3. Basically, the Delaware River Estuary will be the final recipient of onsite spills or operating discharge. As the water is brackish, there are no public water supplies affected by estuary flows.

The Delaware Estuary behaves as a mixed estuary. It is essentially homogeneous vertically; salinity averages 10 to 15 ppt with vertical variations at a given point limited generally to less than 1 ppt. Some variation in salinity is observed across the estuary due to Coriolis Forces which tend to concentrate less-than-average salinities on the west (Delaware shoreline and slightly greater than average salinities on the east (New Jersey shoreline). As a well-mixed estuary, the tidal mixing is sufficiently vigorous to keep the vertical salinity stratification to a low value; thus the dynamic and kinematic processes, which govern salinity, act to produce a relatively one-dimensional salinity distribution until a point is reached in the lower Delaware Bay where the tidal velocities are low enough to permit a degree of vertical stratification to develop. In the lower bay, below the Salem Station, there is an extensive amount of nontidal circulation brought about by the combination of salinity gradients and meteorological conditions. However, above the site the classic salinity profile for the vertically homogeneous estuary is prevalent.

The Pritchard-Carpenter Consultants have estimated secondary, or nontidal flow as it can relate to the dispersion of effluent below the Salem Station. Their information indicates that as the observer travels seaward from the upstream freshwater end of the Estuary, there is an increasing amount of nontidal circulation. The relationship of this nontidal circulation to the transport of materials seaward has not been quantitatively established for the Salem Station and is of interest only in a qualitative overview. Based on computations using the vertical salinity measurements taken in conjunction with biological assessments, the net nontidal circulation in the station vicinity due to Coriolis Forces, wind stress, and gravity-induced circulation, produces salinities on the order of one-third of those in the lower bay. Other estimates of nontidal flow as high as six times the net freshwater supply are suggested, but insufficient data are available to assess either the numerical accuracy or the significance of this phenomenon in relation to the dispersion and advection of effluents from the Salem Station. However, it is clear that surface flow at the site is to the Estuary and the Estuary is a well mixed body of water in direct connection with the Atlantic Ocean.

2.4.13 Groundwater

2.4.13.1 Description and Onsite Use

On a regional basis, the site is located on the Atlantic Coastal Plain about 18 miles south of the Fall Zone. The aquifers of the Coastal Plain are almost entirely unconsolidated sand and gravel, and water is stored in and transmitted through the primary pore spaces between the sand grains. The most productive aquifers in the region are the Cohansey Sand and the Raritan and the Magothy Formations. Other aquifers include all or portions of the Wenonah and Mount Laurel Sands, the Englishtown Formation and the Vincetown Formation. Sands and gravels of Pleistocene and Recent Age are irregularly distributed throughout the Coastal Plain, but are used as aquifers only in a few areas adjacent to the Delaware River.

A summary of the hydrologic characteristics of geologic formations in the regions is presented in Table 2.4-3, Hydrologic Characteristics of Geologic Formations. They are discussed in order of the youngest formation to the oldest. Additional geologic information is given in Section 2.5.1, Geology and Seismology.

A total of six production wells have been drilled at the site. They are screened in Wenonah - Mount Laurel and in the Upper and Middle Raritan Formations. Average flow of the wells is 1000 gallons per minute (gpm) with a maximum anticipated requirement of 1400 gpm. The location of these wells is shown on Figure 2.4-9.

2.4.13.2 Sources

At the time of the preparation of the original Safety Analysis Report (late 1960s), nearly all water used for consumptive purposes within 25 miles of the site was groundwater. With the exception of the highly industrialized Wilmington, Delaware area, the major use of water is for domestic and agricultural purposes. This situation has not changed significantly in recent times.

Public Water Supplies

There are six towns in New Jersey within 25 miles of the proposed site that have public water supplies. There are five public water supplies within 15 miles of the site. Data concerning these public water supplies are shown in Table 2.4-4, Public Water Supplies in the Vicinity of the Site. The locations of these supplies are shown on Figure 2.4-10, Public Water Supplies in the Vicinity of the Site.

Private Wells

Nearly all domestic water supplies in this region are obtained from private wells. Most wells are 2 inches in diameter and greater than 75 feet in depth. The aquifer commonly utilized in the vicinity of the site is the Mount Laurel-Wenonah Formation. Information pertaining to these wells is presented in Table 2.4-5, Private Water Wells in Vicinity of Site. The locations of wells in the vicinity of the site are shown on Figure 2.4-11, Known Water Wells in New Jersey in Vicinity of Site.

There are no known productive water wells within 2 miles of the site other than those installed by Public Service Electric & Gas (PSE&G) (see Section 2.4.13.1). There are three abandoned wells near the site. The wells are reported to be several hundred feet deep. The location of the offsite wells are shown on Figure 2.4-11; the onsite wells are shown on Figure 2.4-9.

The nearest residences to the site are about 3 miles distant. Their water supply is obtained from shallow driven wells, or, in some cases, is carried in along with other provisions.

Most water wells inventoried were located 3 to 4 miles from the site. The nearest wells in Delaware are more than 3 miles from the site and were not canvassed since it is believed that they would not be affected by a change in the groundwater regimen at the site because of the intervening Delaware Estuary.

Site Groundwater

The subsurface soils and groundwater conditions at the site are consistent with the regional picture. The upper soils at the site are dredged fills which were placed there by the United States Army Corps of Engineers around the turn of the century. The fill material apparently came from the channel of the Delaware River. Information obtained from test borings drilled on the site

indicates the thickness of the hydraulic fill is generally less than 10 feet. Dames and Moore's report on Foundation Studies for Hope Creek Generating Station states:

> "At the surface, the hydraulic fill extends to a depth of about 30 feet below the present ground surface. The fill deposit is of man-made origin, having been deposited on the site as a result of channel maintenance in nearby areas..."

We have been calling the 30 foot upper layer as hydraulic fill all through the project work, including the correspondence with Nuclear Regulatory Commission.

Dames and Moore's site subsurface section designated the upper 30 feet as hydraulic fill also. It is is of the same designation in "Engineering Seismology" (page 2-9).

The fill material is composed of a heterogeneous mixture of silt, silty clay, fine sand, and organic material. Four soil percolation tests were conducted on these materials to measure the absorption rate of the surficial soil. These tests were conducted in accordance with the U. S. Army Corps of Engineers' procedures. The absorption rate ranged from 1 to 4 gallons per day per square foot. The average rate was 2.7 gallons per day per square foot. Water levels are approximately at the level of the adjacent estuary waters.

Below the hydraulic fill, a grey sandy and gravelly material, which formally comprised the bed of the Delaware River, was found. This layer varies in thickness from 2 to 5 feet and is composed of fine-to-coarse sand, a little fine-to-coarse gravel, and a trace of silt. The permeability of the sand, based upon particle size analyses, ranges from about 50 to 150 gallons per day per square foot. The clay facies is essentially impermeable. The lateral extent of this sand member is unknown, but it appears to exist in most of the site area. It is hydraulically connected with the Delaware Estuary, and water levels in this formation change in response to tidal variations. Water levels in this formation are essentially horizontal and although changes in response to tides do occur, the horizontal component of groundwater movement is small.

The Kirkwood Formation of Miocene Age underlies the Quaternary soil and extends to about 70 feet in depth. It consists of gray silty clay and is an aquitard. Permeability values are less than 50 gallons per day per square foot.

The Vincetown Formation is about 45 to 75 feet thick and is encountered at a depth of about 70 feet. It consists of a fine-to-medium-grained sand with occasional gravel and is separated from the Quaternary soils by about 35 feet of impermeable silty clay of the Kirkwood Formation. Grain size analyses of this sand indicate a permeability of about 200 gallons per day per square foot. Water levels in this formation are essentially horizontal with an artesian pressure head just slightly lower than the surficial groundwater table. The horizontal component of groundwater movement in this formation is probably negligible, except for tidal oscillations.

Two piezometers were installed about 75 feet from the Delaware Estuary to determine the tidal efficiency of the Vincetown Formation. Water level measurements were made in the estuary from high to low tide and corresponding measurements were made in the piezometers. Total tidal fluctuation amounted to 6.3 feet, and the maximum variation in the piezometers was 3.9 feet. The time lag between peaks in the estuary and in the piezometers was about 20 minutes.

The Vincetown Formation is underlain by the Hornerstown Sand which, according to published information, and information from the borings at the site, is an aquitard. Underlying the Hornerstown is the Navesink and Wenonah-Mount Laurel Sands.

The Raritan-Magothy Formation is encountered at a depth of approximately 450 feet at the site. It consists of interbedded clays, gravel, and sands. The sand layers are generally 20 to 30 feet thick and the clay layers on the order of 100 feet in thickness. Fresh water was encountered in the sand layers to a depth of 900 feet at the site. At greater depths, the sands probably contain salt water.

Although the site is underlain by sand and gravel formations which are utilized as a source of water supply in the region, these aquifers are separated from the surficial soils by one or more impermeable silty clay beds. Since the hydraulic gradient of these aquifers at the site is too small to measure, it is probable that the only groundwater movement at the site is a result of tidal influences. Except for production wells recently constructed at the site by PSE&G, there are no water wells within

2 miles of the site, and the possiblity of offsite wells being affected by changes in the groundwater regimen at the site is remote.

2.4.13.3 Accident Effects

In summary, the hydrological conditions at the site are well suited for the operation of the proposed power station. Fluid spills at the surface would be contained within the station drainage system or be drained toward the Delaware Estuary. All public water supplies in the Delaware are upstream of the site. Because of salt water intrusions, industrial use of the river water below Marcus Hook, some 25 miles upstream of the site is limited to cooling water applications. Thus radioactive wastes discharged to the river will remain well downstream of any industrial or domestic usage of river water.

Any accidental spills that reached the subsurface would tend to move slowly to the southwest, although short-term reversals occur as a result of tidal fluctuations in the estuary. All water wells in the vicinity of the site are located upgradient. The closest domestic well is a shallow well located about 3 miles from the site.

Movement of groundwater through the site is quite low as a result of the comparatively low coefficients of permeability and the low hydraulic gradients.

Fluid infiltration in the area surrounding the actual construction site is low as many of the strata are relatively impermeable. Even in the station area, where the Pleistocene-aged and Miocene-aged Kirkwood Formation was removed, infiltration of fluids will be quite slow as the plant structures are founded on a lean concrete fill placed upon the Vincetown soils (which also have low permeabilities as a result of their cemented nature).

The Vincetown is a fine to medium-grained calcareous sand, containing variable amounts of cementing material. The groundwater in the Vincetown is artesian and contains chloride concentrations of several thousand parts per million, thus, not suitable for drinking water.

Below the Vincetown are the underlying Hornerstown and Navesink Formations which act as confining beds.

A groundwater protection program was designed and implemented to provide reasonable assurance that a groundwater leak or spill of radioactive materials should be detected early and effectively remediated well before any potential impact to the offsite public health and safety or onsite workers.

2.4.13.4 Monitoring or Safeguard Requirements

Surface and subsurface flow is toward the estuary. In general, infiltration and surface flow are slow. No public water supplies are down-gradient or downstream of the station. Thus, special monitoring or safeguard requirements are not necessary,

2.4.13.5 Technical Specifications and Emergency Operation Requirements

Consistent with Section 2.4.13.4, no technical specifications have been prepared. No emergency plans, other than those presented in Section 13.3 are contemplated.

2.4.14 References for Section 2.4

1. Marinos, G. and Woodward, J. W., "Estimation of Hurricane Surge Hydrographs," American Society of Civil Engineers, Journal of Waterways and Harbors Division, Vol. 94, No. WW2, pp. 189-216, 1968.

2. Bretschneider, C.L., "Hurricane Surge Predictions for Delaware Bay and River," Beach Erosion Board, U.S. Army Coastal Engineering Research Center, Misc. Paper No. 4-59, November 1959.

3. U.S. Dept. of Commerce, Interim Report Meteorological Characteristics of the Probable Maximum Hurricane, Atlantic and Gulf Coast of the United States: Environmental Science Services Administration, Memorandum HUR 7-97, May 7, 1968.

4. Lendo, A.C., "Record Low Tide of December 31, 1962 on the Delaware River," U.S. Geological Survey Water Supply Paper 1586-E, 1966.

5. U.S. Army Coastal Engineering Research Center, Shore Protection Planning and Design, Technical Report No. 4, 3rd Edition, 1966.

2.4.15 Bibliography for Section 2.4

Anon, Geology of Northeast Corridor, USGS Misc. Map 1-S14-C.

Back, J.L., Hydrochemical Facies and Ground-Water Flow Patterns in the Northern Part of the Atlantic Coastal Plain, U.S.G.S. Professional Paper 498-4, 1966.

Barksdale, H.C.; Greenman, D.W.; Land, S.W.; Hilton, G.S.; and Outlaw, D.E., Ground-Water Resources in the Tri-State Region Adjacent to the Lower Delaware River. New Jersey Division of Water Supply and Policy Special Report 13, 1958.

Bretschneider, C.L., Storm Surges, Advances in Hydrosciences, Academic Press. Vol. 4, pp. 341-418, 1967.

Marine, I. W. and Rasmussen W.C., Preliminary Report on the Geology and Ground-Water Resources of Delaware, Delaware Geological Survey Bulletin No. 4, 1955.

Richards, H.G.; Olmstead, F.H.; and Ruhl, J.L., Generalized Structure Contour Maps of the New Jersey Coastal Plain, State of New Jersey, Department of Conservation and Economic Development, 1962.

Rima, D.R.; Coskery, O.J.; and Anderson, P.W., Ground-Water Resources of Southern New Castle County, Delaware, Delaware Geological Survey Bulletin No. 11, 1964.

Sundstrom, R.W., et al, The Availability of Ground-Water from the Potomac Formation in the Chesapeake and Delaware Canal Area, Delaware, University of Delaware Water Resources Center, 1967.

U.S. Army Corps of Engineers, Sewage Treatment Plants, Engineering Manual 345-243, 1959.

U.S. Army Corps of Engineers, Report on Hurricane Study, Delaware River and Bay, Pennsylvania, New Jersey and Delaware, Unpublished Report, North Atlantic Division, Philadelphia District Corps of Engineers, September 13, 1963.

2.4.16 Agencies and Individuals Contacted

Agency	Location	Individual
U.S. Geological Survey Water Resources Division	Trenton, New Jersey	Mr. H. Gill Mr. H. Meisler
New Jersey Division of Water Policy and Supply	Trenton, New Jersey	Mr. J. C. Mearill
Coleman Well Drilling Co.	Hancocks Bridge, New Jersey	Mr. P. Coleman
	Vicinity of site	Numerous local residents