SECTION 2

SITE CHARACTERISTICS

TABLE OF CONTENTS

TABLE OF CONTENTS (Cont)

TABLE OF CONTENTS (Cont)

TABLE OF CONTENTS (Cont)

TABLE OF CONTENTS (Cont)

TABLE OF CONTENTS (Cont)

LIST OF TABLES

LIST OF TABLES (Cont)

LIST OF TABLES (Cont)

LIST OF TABLES (Cont)

LIST OF TABLES (Cont)

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LIST OF TABLES (Cont)

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SECTION 2

SITE CHARACTERISTICS

2.1 GEOGRAPHY AND DEMOGRAPHY

2.1.1 Site Location and Description

2.1.1.1 Specification of Location

HCGS is located in the southern section of Artificial Island on the east bank of the Delaware River in Lower Alloways Creek Township, Salem County, New Jersey. The site center point is located at latitude 39° 27′ 53″ north and longitude 75° 32′ 12″ west. The Universal Transverse Mercator coordinates of the site center are 4368307mN and 453872mE, Zone 18.

2.1.1.2 Site Area Map

A detailed map of the site area is provided on Figure 2.1-1. This map details plant property lines and site boundaries. Distance to site boundaries may be scaled from this map. The HCGS site dimensions, shape, and area are shown on Figure 2.1-2.

2.1.1.3 Boundaries for Establishing Effluent Release Limits

The land boundary, on which technical specification limits for release of gaseous radioactive effluents are based, is the property line defining land owned by Public Service Electric and Gas Company (PSE&G). Distances from station vents to the property line in any direction may be scaled from Figure 2.1-1.

2.1.2 Exclusion Area Authority and Control

2.1.2.1 Authority

PSE&G has fee simple ownership, including mineral rights, of the 740-acre Artificial Island site.

The site boundary and exclusion area boundary are one and the same except along the Delaware River. The Hope Creek Generating Station property line lies within the site boundary as indicated on Figure 2.1-1.

The minimum distance from the accident release point of the reactor to the nearest exclusion area boundary formed by land is 901 meters. In accordance with 10CFR100, Paragraph 100.3(a), arrangements have been made by a letter of agreement with the United States Coast Guard for the control and evacuation of persons on the adjacent waterway-the Delaware River.

2.1.2.2 Control of Activities Unrelated to Plant Operation

The only activity within the PSE&G property boundary not directly related to power generation is the PSE&G Visitor Information Center. The visitor center is located on the south edge of the property boundary as identified on Figure 2.1-1. Activities at the visitor center are controlled by PSE&G. The location of the two unit Salem Generating Station (SGS) is also identified on Figure 2.1-1.

2.1.2.3 Arrangements for Traffic Control

There are no highways, railways, or waterways crossing the exclusion area.

2.1.2.4 Abandonment or Relocation of Roads

There are no public roads to be abandoned or relocated in the exclusion area.

2.1.3 Population Distribution

Population and population distribution for the 0 to 10 and 10 to 50-mile areas are keyed to sectors and zones. The 0 to 10-mile area is divided into concentric circles around the site center point, at distances of 1, 2, 3, 4, 5, and 10 miles. The same was done for the 10 to 50 miles, but at 10-mile intervals between the 10 and 50-mile radii. The circles are divided into 22-1/2-degree segments with each segment centered on one of the 16 compass points.

2.1.3.1 Population Within 10 Miles

Current residential population within 10 miles of the Artificial Island site by sector and ring is shown on Figure 2.1-3, using References 2.1-1 and 2.1-2. The projected population for the first full year of station operation in 1987 is shown on Figure 2.1-4. The projected populations by sector and ring for each census decade from 1990 through the projected station life in 2030 are shown on Figures 2.1-5 through 2.1-9.

Field surveys were conducted to determine the number and distribution of population by ring and sector within 0 to 5 miles of the site as discussed in Reference 2.1-3. Dwelling units within the 0 to 5-mile area were identified in the field and noted on a map by ring and sector. A dwelling unit vacancy rate was applied to the dwelling units in each sector to determine the number of occupied dwelling units. A separate vacancy rate was established for New Jersey and Delaware sectors, as mentioned in References 2.1-1 and 2.1-2, respectively, based on the 1980 Census of Housing. The following are the 1980 vacancy rates by county:

1. Cumberland County, NJ - Vacancy rate of 6.7 percent

2. Salem County, NJ - Vacancy rate of 6.8 percent

3. New Castle County, DE - Vacancy rate of 5.9 percent

Having identified the 1980 distribution of occupied housing units in the 0 to 5-mile area, an average household size was determined, based on the 1980 census in References 2.1-1 and 2.1-2, as follows:

1. Cumberland County, NJ - Average household size 2.9

2. Salem County, NJ - Average household size 2.9

3. New Castle County, DE - Average household size 3.1.

The number of occupied dwelling units was then multiplied by the average household size to determine the 1980 distribution of population by sector and ring within 0 to 5 miles of the site.

Current 1980 census tract data from Reference 2.1-4 were used to determine the number and distribution of population by ring and sector for the 5 to 10-mile area. Where census tract boundaries did not coincide with sector ring boundaries, it was assumed that population within the census tract was uniformly distributed. A land use comparison with the U.S. Geological Survey (USGS) quadrangles and county master plans was conducted to identify both significant concentrations of population and large vacant undeveloped areas. In this way, the uniform distribution of population within each census tract was adjusted to reflect actual conditions.

The basis for the 0 to 10-mile population projections were field surveys from Reference 2.1-3, the 1980 Census of Population and Housing for New Jersey and Delaware, state and county projections for the years 1987 to 2010 from References 2.1-5 and 2.1-6, and federal projections by state for the years 2020 to 2030 from Reference 2.1-7. The federal projections were consistently lower than the state projections; accordingly, state projections were used in order to be conservative. However, the state projections for the year 2000 were consistently higher than federal projections for the year 2010. To avoid a dip or decline in population, it was assumed that the populations between the years 2000 and 2010 were stable. For the years 2020 and 2030, the federal projections were used. In this way, an artificial decrease in projected population does not occur during the transition period from state to federal projections.

The major difference between state and federal projections appears to be the more optimistic state view of growth compared with the more balanced federal approach, particularly in terms of economic activity.

2.1.3.2 Population Between 10 and 50 Miles

Current residential population within 10 to 50 miles of the Artificial Island site is shown on Figure 2.1-10. The projected population for the first full year of station operation in 1987 is shown on Figure 2.1-11. The projected population for each census decade from 1990 through the projected station life in 2030 are shown on Figures 2.1-12 through 2.1-16.

The bases for the 10 to 50-mile population projections were 1980 Census of Population and Housing for New Jersey, Delaware, Maryland, and Pennsylvania, from References 2.1-1, 2.1-2, 2.1-8 and 2.1-9, state projections by county for the years 1987 to 2010 from References 2.1-5, 2.1-10, 2.1-11, and 2.1-12, and federal projections by state for the years 2020 to 2030 from Reference 2.1-7. The federal projections were consistently lower than the state projections; accordingly, state projections were used in order to be conservative. However, the state projections for the year 2000 were consistently higher than federal projections for the year 2010. To avoid a dip or decline in population, it was assumed that the year 2000 and 2010 populations were stable. For the years 2020 and 2030, the federal projections were used. In this way, an artificial decrease in projected population does not occur during the transition period from state to federal projections.

The major difference between state and federal projections appears to be the more optimistic state view of growth compared with the more balanced federal approach, particularly in terms of economic activity.

2.1.3.3 Transient Population

Transient population is discussed below in terms of 0 to 10 and 10 to 50 miles from the Artificial Island site. It is noted that the transient population includes a substantial double-counting of the resident population.

2.1.3.3.1 Transient Population Within 0 to 10 Miles

The distribution of transient and special facilities population within 0 to 10 miles of the Artificial Island site is shown in Table 2.1-1. This distribution is based on the transient and special facilities populations identified in the Artificial Island site's Evacuation Time Estimates in Reference 2.1-13.

Employment concentration centers within 0 to 10 miles of the Artificial Island site are:

1. Delaware City Industrial Complex, DE - 10 to 10.5 miles north-northwest

2. Middletown, DE - 10.0 miles west

3. Salem, NJ - 8.0 miles north-northeast.

Other than in the city of Salem, there are no major shopping centers within 0 to 10 miles of the site.

The Delaware City Industrial Complex consists of industries predominantly of the petrochemical classification. Approximately 2,000 persons are employed at this complex, about 1,000 of which are within 10 miles of the site, as mentioned in Reference 2.1-14. Employees travel to work from up to 40 miles distant, primarily on the west side of the Delaware River, as mentioned in Reference 2.1-14.

Employment in Salem, New Jersey and Middletown, Deleware, is more localized. Employees generally live within 5 to 15 miles of their place of work, according to Reference 2.1-14.

The locations of special facilities, from Reference 2.1-3, are shown in Table 2.1-2. Special facilities populations, from Reference 2.1-13, are included in Table 2.1-1.

Annual visitations to parks and wildlife areas and other recreational facilities within 0 to 10 miles of the site are shown in Table 2.1-3 and on Figures 2.1-17 and 2.1-18, using References 2.1-15, 2.1-16, and 2.1-17.

Within the 0 to 10-mile area, there are transient populations related to the use of the Delaware River. Ports of landing for both commercial and recreational salt water fishing are shown on Figure 2.1-19, using References 2.1-18 and 2.1-19.

The waters within the Delaware River are locations of both commercial and recreational fish and shellfisheries as shown on Figure 2.1-20, using References 2.1-18 and 2.1-19.

The Delaware River is the major route for barge and freight traffic between the Philadelphia area ports and the Atlantic Ocean. According to the U.S. Army Corps of Engineers Philadelphia District, in Reference 2.1-19, approximately 111,500 vessel trips were made past the Artificial Island site in 1979, carrying 1,443,570 persons.

2.1.3.3.2 Transient Population Within 10 to 50 Miles of the Site

The major employment centers located within the 50-mile radius area are shown in Table 2.1-4. The estimated total employment for these centers is 888,900 persons, as discussed in Reference 2.1-20.

These major employment centers include Philadelphia, which is the core of the Philadelphia Standard Metropolitan Statistical Area, as well as subregional centers such as Camden and Vineland, New Jersey; and Wilmington, Newark and Dover, Delaware. Philadelphia generates employment for a large area that is outside of the 50-mile radius from the Artificial Island site. The other communities, however, attract employees from a relatively narrow area within the 50-mile radius. Philadelphia also generates the largest student population in the area due to a concentration of major colleges and universities. Students at colleges and universities are counted in the U.S. Census as year-round residents in their place of residence in February and March. Therefore, virtually all students are permanent, not transient, persons.

Major shopping areas within the 10 to 50-mile radius area are Philadelphia, Pennsylvania; Wilmington and Newark, Delaware; and Camden (Cherry Hill), New Jersey.

The recreation and tourism area of the site within the 10 to 50-mile area are located along the New Jersey ocean shoreline and also along the Delaware and Chesapeake Bays in New Jersey, Delaware, and Maryland, as mentioned in Reference 2.1-21.

The major seasonal recreation and tourist population concentrations are close to the New Jersey ocean shoreline, 40 to 50 miles, east, east-southeast, and southeast from the Artificial Island site. Table 2.1-5 shows the population concentrations for this area, using Reference 2.1-21.

State parks, forests, and wildlife management areas are used predominantly from the spring through the fall months.

Figures 2.1-17 and 2.1-18 show the locations of these areas as well as annual visitations to them, using References 2.1-17 and 2.1-21.

Within the 10 to 50 mile area, there are transient populations related to the use of Chesapeake Bay and the Delaware Bay and River. Ports of landing for both commercial and recreational salt water fishing are shown on Figure 2.1-19, using References 2.1-18 and 2.1-19.

The waters within the Chesapeake Bay and the Delaware Bay and River are locations of both commercial and recreational fish and shellfisheries as shown on Figure 2.1-20, using References 2.1-18 and 2.1-19.

2.1.3.4 Low Population Zone

The low population zone (LPZ) for the Artificial Island site has a 5-mile radius with a 1980 population of 1190 persons.

The major population clusters within the LPZ are as follows:

1. Hancock's Bridge, NJ - 327 persons, 4.9 miles northeast

2. Port Penn, DE - 236 persons, 4.2 miles north-northwest

3. Bay View, DE - 104 persons, 3.5 miles west-northwest

The location of highways, waterways, beaches, and wildlife areas within the LPZ are shown on Figure 2.1-21, using References 2.1-22, 2.1-23, 2.1-24, and 2.1-25. Seasonal and tourist population distributions within the LPZ are shown in Table 2.1-6, using References 2.1-26 and 2.1-27. Except for Delaware River traffic, virtually all of the seasonal users of recreation facilities in the LPZ come from within 50 miles of the site.

2.1.3.5 Population Center

The nearest population center to the Artificial Island site is Newark, Delaware, with a 1980 population of 25,247 people. The 1980 population of Newark increased by 18.5 percent from the 1970 population of 21,298. Newark is located 17.8 miles northwest of the site. The location and population of Newark and other lesser population centers within 30 miles of the Artificial Island site are shown on Figure 2.1-22. The location of the population center was determined in accordance with 10CFR100.

Bridgeton and Salem, New Jersey, with 1980 populations of 18,795 and 6959, are located 14.9 and 7.9 miles, respectively, from the site, as given in Reference 2.1-1.

Transient populations were not used in determining the population center within the 0 to 30-mile area because they do not significantly alter the general population distribution within the 0 to 30-mile area around the Artificial Island site.

2.1.3.6 Population Density

The 0 to 30-mile cumulative resident population projected for the initial full year of plant operation, 1987, is 1,009,117, shown in Table 2.1-7, using References 2.1-1, 2.1-5, 2.1-6, and 2.1-7.

This population density of 356 persons per square mile is less than the standard uniform population density of 500 people per square mile within 0 to 30 miles of the Artificial Island site.

The 0 to 30-mile projected cumulative resident population for the end of the plant life, the year 2030, is 1,181,153. This population density of 417 persons per square mile is less than the uniform population density standard of 1000 people per square mile.

2.1.4 References

2.1-1 U.S. Bureau of the Census, "1980 Census of Population and Housing, New Jersey," Advance Reports, PHC80-V-32, U.S. Department of Commerce, March 1981.

2.1-2 U.S. Bureau of the Census, "1980 Census of Population and Housing, Delaware," Advance Reports, PHC80-V-9, U.S. Department of Commerce, February 1981.

2.1-3 Field Surveys by Dresdner Associates conducted during May and June 1982.

2.1-4 Unpublished U.S. Bureau of the Census tract data and maps available at county planning departments, and census tract data and maps reviewed at the planning department offices of Cumberland and Salem Counties, New Castle and Kent Counties,. DE.

2.1-5 New Jersey Department of Labor and Industry, "New Jersey Population Projection 1980-2000," February 1982.

2.1-6 New Castle County Planning Board, "Population Projections 1980-2000," March 1982.

2.1-7 U.S. Department of Commerce, "1980 OBERS BEA Regional Projections," February 1982.

2.1-8 U.S. Bureau of the Census, "1980 Census of Population and Housing, Maryland," Advance Reports, PHC80-V-22, March 1981.

2.1-9 U.S. Bureau of the Census, "1980 Census of Population and Housing, Pennsylvania," Advance Reports, PHC80-V-40, March 1981.

2.1-10 Delaware Department of Health and Social Services, "Population Projections by County and City 1975-2000," February 1980.

2.1-11 Maryland Department of State Planning, "Interim Population Projections for Maryland Political Subdivision 1980-2000," June 1981.

2.1-12 Pennsylvania Office of State Planning and Development, "Pennsylvania Projection Series 1980-2000, Summary Report, 78 PPS-1," June 1978.

2.1-13 Parsons, Brinckerhoff, Quade and Douglas, "Evacuation Time Estimates," February 1981.

2.1-14 "Delaware Directory of Commerce and Industry," 1979. Also, personal communications with:

D. Timmens, Technical Supervisor, Diamond Shamrock, May 28, 1982.

L. Fleming, Manager, Schargen Gas Co, May 28, 1982.

D. Press, Public Affairs, Getty Refining & Manufacturing Co, May 25, 1982.

M. Morris, Manager, Stauffer Chemical Co, May 26, 1982.

J. Butler, American Hoechst Corp, May 28, 1982.

L. Evo, Standard Chlorine of Delaware, Inc, May 26, 1982.

H. McFadden, Plant Manager, Chloromone Corp, June 4, 1982.

G. Hobbie, Plant Manager, Cardox, June 2, 1982.

2.1-15 New Jersey Department of Labor and Industry, "Boat Basins in New Jersey," 1978.

2.1-16 Sea Grant Advisory Service, "A Guide to Delaware's Coastline Marinas," 1979.

2.1-17 Personal communications with officials of:

New Jersey Division of Parks and Forestry, June 23, 1982.
Delaware Division of Parks and Recreation, June 23, 1982.
Delaware Division of Fish and Wildlife, June 23, 1982.

2.1-18 Townsend, R., "Guide to New Jersey's Saltwater Fishing, 1974."

2.1-19 Personal communications with:

D. Christian, National Marine Fisheries Service, June 21, 1982.
M. Carson, U.S. Army Corps of Engineers, Philadelphia District, June 21, 1982.

2.1-20 Personal communications with:

R. Alzarski, Economist, Pennsylvania Department of Labor, June 25, 1982.
J. Major, Principal Labor Market Analyst, New Jersey Department of Labor and Industry, June 24, 1982.
E. Simmons, Planning and Research, Pennsylvania Department of Labor, June 25, 1982.

2.1-21 Personal communications with employees of:

Maryland State Park and Recreation Service, June 24, 1982.
Pennsylvania Bureau of State Parks, June 24, 1982.

2.1-22 New Castle County Planning Board, "The Red Lion Planning District Plan, 1995," September 1973.

2.1-23 New Castle County Planning Board, "The Middletown-Odessa-Townsend Planning District Plan 1995," September 1973.

2.1-24 U.S. Department of Agriculture, South Jersey Resource Conservation & Development Area Plan," April 1979.

2.1-25 Personal communications with:

R. Chartawich, New Castle County Department of Planning, May 20, 1982.

C. Warren, Salem Country Department of Planning, May 20, 1982.

2.1-26 Dames & Moore, "New Jersey Shore Area Seasonal Population Study," 1973.

2.1-27 Dresdner Associates, Reevaluations of AGS PSAR Population Projections," 1977.

TABLE 2.1-1

TRANSIENT AND SPECIAL FACILITIES POPULATION
WITHIN 0-10 MILES OF THE ARTIFICIAL ISLAND SITE
1981(1)

	Miles			
Sector	0-2	2-5	5-10	Total
N	0	7	141	148
NNE	0	0	3,569	3,569
NE	0	0	384	384
ENE	0	0	0	0
E	0	0	285	285
ESE	0	0	0	0
SE	2	8	8	18
SSE	2	8	58	68
S	2	2	0	4
SSW	2	0	1,011	1,013
SW	2	0	0	2
WSW	2	0	484	486
W	2	0	2,754	2,756
WNW	2	3	0	5
NW	2	3	1,341	1,346
NNW	2	10	1,391	1,403
Total	20	41	11,426	11,487

(1) Source: Parsons, Brinckerhoff, Quade & Douglas, Evacuation Time Estimates, 1981.

TABLE 2.1-2

LOCATION OF SPECIAL FACILITIES WITHIN
10 MILES OF THE ARTIFICIAL ISLAND SITE
1982(1)

Facilities

Delaware Schools	Location
Silver Lake - Elementary (Middletown)	9.5 mi W
Middletown - High School	9.5 mi W
Redding - Middle (Middletown)	9.5 mi W
Broad Meadow School - Private Middle & Elementary (Middletown)	9.5 mi W
St. Andrews - Private Protestant High School (all boys) (Middletown)	9.5 mi W
Townsend - Elementary (Townsend)	9.8 mi SW
Commodore MacDonough - Elementary (St. Georges)	8.8 mi SW
Delaware City - Elementary (Delaware City)	8.0 mi NNW
Gunning Bedford - Middle (Delaware City)	8.0 mi NNW
Corbit - Elementary (Odessa)	6.5 mi W
Au Clair - Elementary (St. Georges)	9.0 mi NW

TABLE 2.1-2 (Cont)

New Jersey Schools	
Quinton Township - Elementary (Quinton)	8.5 mi NE
John Fenwick - Elementary (Salem)	8.0 mi NE
Salem Day Care Center	8.0 mi NE
Salem - Middle (Salem)	8.0 mi NE
Salem - High School (Salem)	8.0 mi NE
Lower Alloways Creek Elementary (Hancock's Bridge)	4.9 mi NE
Elsinboro - Elementary (Elsinboro)	7.0 mi NNE
Stow Creek - Elementary (Stow Creek)	9.9 mi E
Woodland Country Day School - Pre-School to Elementary (Stow Creek)	97. mi E
St. Mary's Elementary (Salem)	8.0 mi NNE
Hospitals	
Salem County Memorial	8.0 mi NE
Salem County Nursing and Convalescent Center	8.0 mi NE
Governor Bacon Health Center, DE	8.0 mi NNW

Jails

Salem County Jail	8.0 mi NE

(1) Source: Survey by Dresdner Associates, May, 1982.

TABLE 2.1-3

RECREATION AND TOURISM WITHIN 0-10 MILES
OF THE ARTIFICIAL ISLAND SITE
1981

Parks	Location	Annual Visitations
Fort Delaware Park, DE	9.0 mi NNW	12,200[1]
Fort Mott State Park, NJ	10.0 mi N	45,700[2]
Wildlife Refuges & Management Areas		
Mad Horse Creek Wildlife Management Area, NJ	2-8.0 mi SE	1,000[2]
Appoquinimink Wildlife Area, DE	2-5.0 mi WSW	100[3]
Reedy Island Wildlife Refuge, DE	4.0 mi NNW	500[3]
Augustine Creek Wildlife Area, DE	4.5-8.0 mi NNW	500[3]
Woodland Beach Wildlife Area, DE	7-10.0 mi SSE	1,200[3]
Canal National Wildlife Refuge, DE	8-10.0 mi NNW	2,000[3]
Maskells Mill Pond Wildlife Management Area, NJ	7.0 mi ENE	800[2]
Killcohook National Wildlife Refuge, NJ	10.0 mi N	500[2]
Beaches		
Augustine Beach, DE	4.0 mi NNW	Not available
Bay View Beach, DE	3.5 mi WNW	Not available
Woodland Beach, DE	10.0 mi SSE	Not available
Oakwood Beach, NJ	6.5 mi N	Not available
Country Clubs		
Country Club of Salem, NJ	6.5 mi N	11,250[4]
Wild Oaks Country Club, NJ	7.5 mi NE	20,000[5]

TABLE 2.1-3 (Cont)

Parks	Location	Annual Visitations
Boat Marinas and Launches		
Marboro Marina, NJ	8.0 mi NNE	4,000 (6)
Delaware City Marina, DE	8.0 mi NNW	2,600 (7)
Delaware City Launching Ramp, DE	8.0 mi NNW	3,000 (7)
Port Penn Launching Ramp, DE	4.2 mi NNW	2,100 (7)
Woodland Beach Launching Ramp, DE	9.8 mi SSE	2,000 (7)

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(1) Delaware Division of Parks and Recreation, June 23, 1982.

(2) New Jersey Division of Parks and Forestry, June 23, 1982.

(3) Delaware Division of Fish and Wildlife, June 23, 1982.

(4) J. Stradley, President, Country Club of Salem, Salem, New Jersey, June 21, 1982.

(5) J. Hasler, President, Wild Oaks Country Club, Salem, New Jersey, June 21, 1982.

(6) New Jersey Department of Labor and Industry, Division of Travel and Tourism, Boat Basins in New Jersey, 1978.

(7) Sea Grant Advisory Service, A Guide to Delaware's Coastline Marinas, 1979.

TABLE 2.1-4

MAJOR EMPLOYMENT CENTERS LOCATED
WITHIN 10-50 MILES OF THE ARTIFICIAL ISLAND SITE
1981

	Total Employment	Location
Philadelphia, PA	775,700 (1)	35-50 mi NNE
Camden, NJ	40,900 (2)	40 mi NE
Wilmington, DE	32,800 (3)	20 mi N
Vineland, NJ	19,400 (2)	24 mi E
Newark, DE	10,500 (3)	19 mi NW
Dover, DE	9,700 (3)	22 mi S
Total	888,900	

(1) Bob Alzarski, Economist, Pennsylvania Department of Labor, Bureau of Labor Statistics, June 25, 1982.

(2) Jim Major, Principal Labor Market Analyst, New Jersey State Department of Labor and Industry, June 24, 1982.

(3) Ed Simmons, Planning & Research, Delaware Department of Labor, June 25, 1982.

TABLE 2.1-5

SEASONAL AND TOURIST POPULATION

NEW JERSEY SHORELINE,

40-50 MILES

AVERAGE SUMMER DAY POPULATION[1]

	Seasonal
E	17,190
ESE	66,690
SE	191,020

(1) Sources: Dames & Moore, New Jersey Shore Area Seasonal Population Study, 1973.

Dresdner Associates, Reevaluation of AGS PSAR Population Projections, 1977.

TABLE 2.1-6

POPULATION DISTRIBUTION WITHIN THE LOW POPULATION ZONE
1982

	Peak Daily Population	Seasonal Transient Population (annual)
Delaware River Ship Traffic	4,000[1]	1,443,570[1]
Wildlife Areas		
Augustine Creek Wildlife Area, DE	45[2]	500[2]
Reedy Island Wildlife Refuge, DE	15[2]	500[2]
Appoquinimink Wildlife Area, DE	25[2]	100[2]
Mad Horse Creek Wildlife Management Area, NJ	70[3]	1,000[3]

	Peak Daily Population	Seasonal Transient Population (annual)
Beaches		
Augustine Beach, DE	Not available	Not available
Bay View Beach, DE	Not available	Not available

---

(1) U.S. Army Corps of Engineers, Philadelphia District, Waterborne Commerce of the United States, 1979.

(2) Delaware Division of Fish and Wildlife, June 23, 1982.

(3) New Jersey Division of Parks and Forestry, June 23, 1982.

TABLE 2.1-7

CUMULATIVE POPULATION AND DENSITY WITHIN 0-30 MILES
OF THE ARTIFICIAL ISLAND SITE[1]

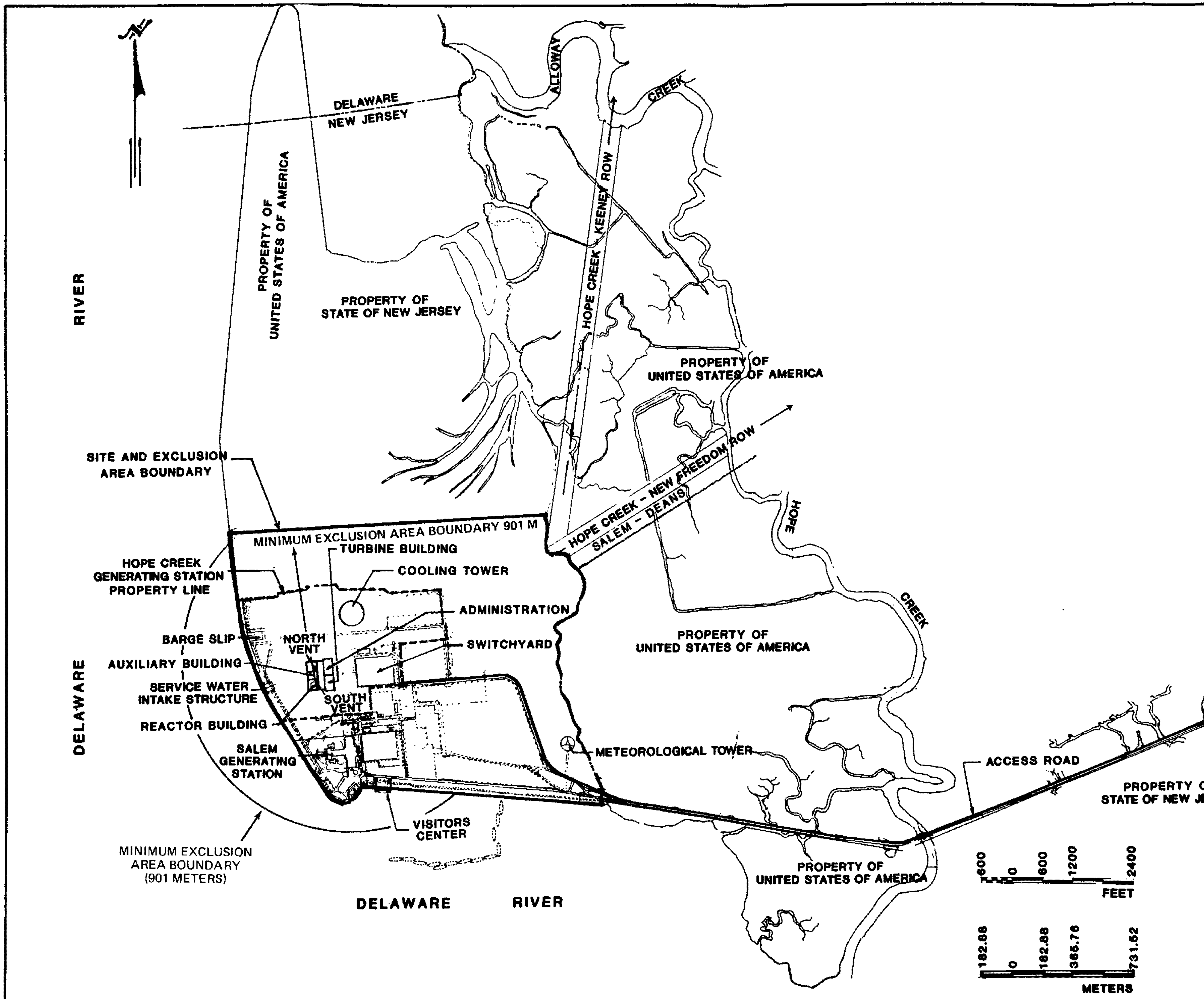
Miles From Site	Cumulative Population 1987	Cumulative Population 2030
0-1	0	0
0-2	0	0
0-3	0	0
0-4	289	357
0-5	1,286	1,565
0-10	25,479	28,921
0-20	395,613	484,340
0-30	1,009,117	1,181,153
Density		
0-30	356	417

(1) Sources: 1980 Census of Population & Housing.

New Jersey Department of Labor, New Jersey Population Projections 1980-2000, February, 1982.

U.S. Department of Commerce, 1980 OBERS BEA Regional Projections (1969-2030), July 1981.

New Castle County Planning Board, Population Projection 1980-2000, March 1982.



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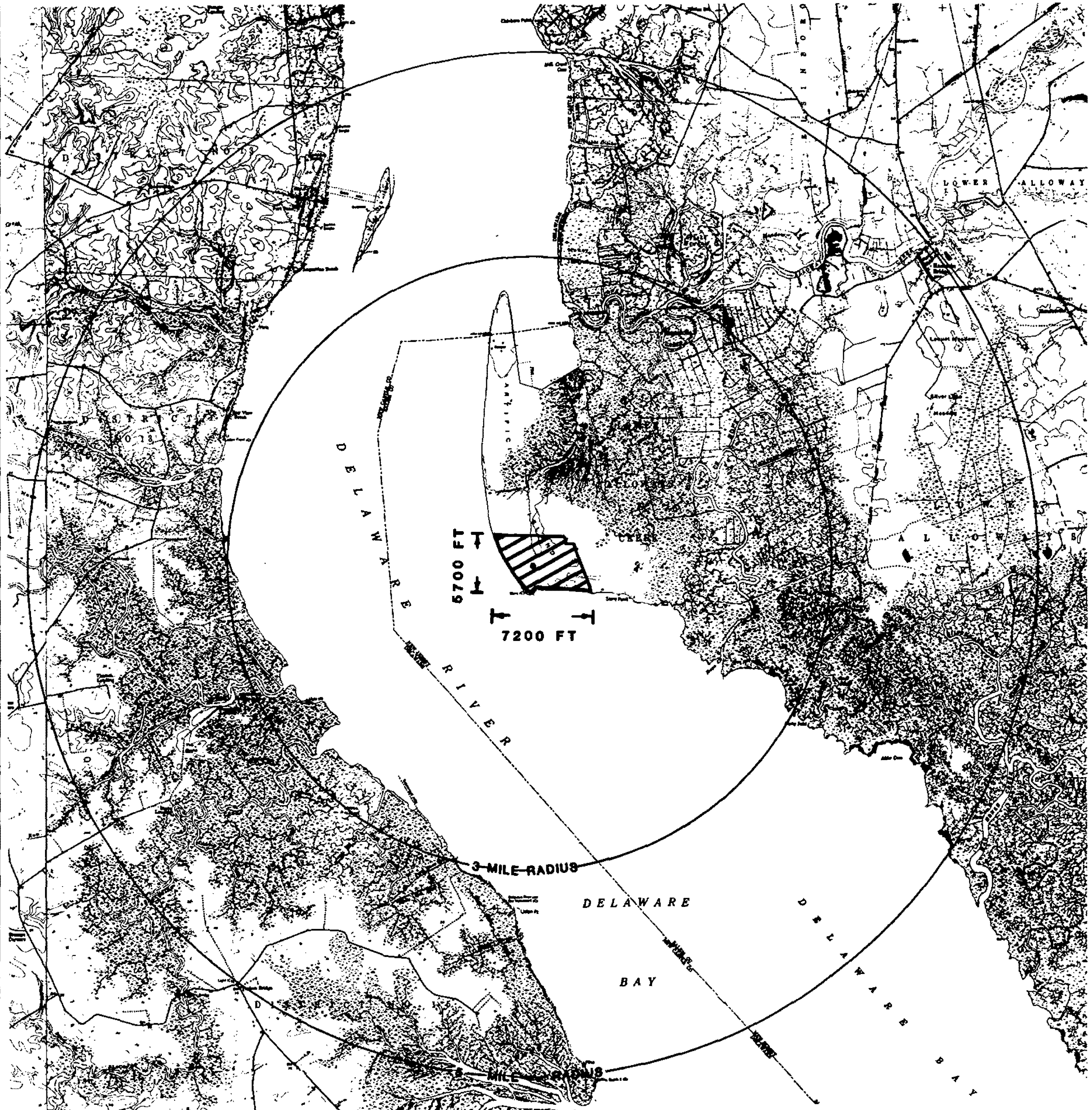
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

SITE PLAN

UPDATED FSAR

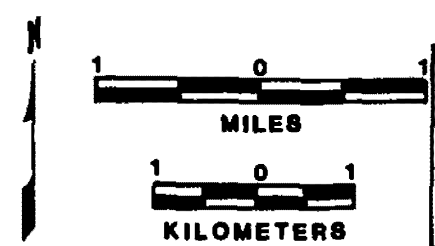
FIGURE 2.1-1



SITE AREA

— PROPERTY LINE/EXCLUSION AREA

740 ACRES, UTILITY PROPERTY



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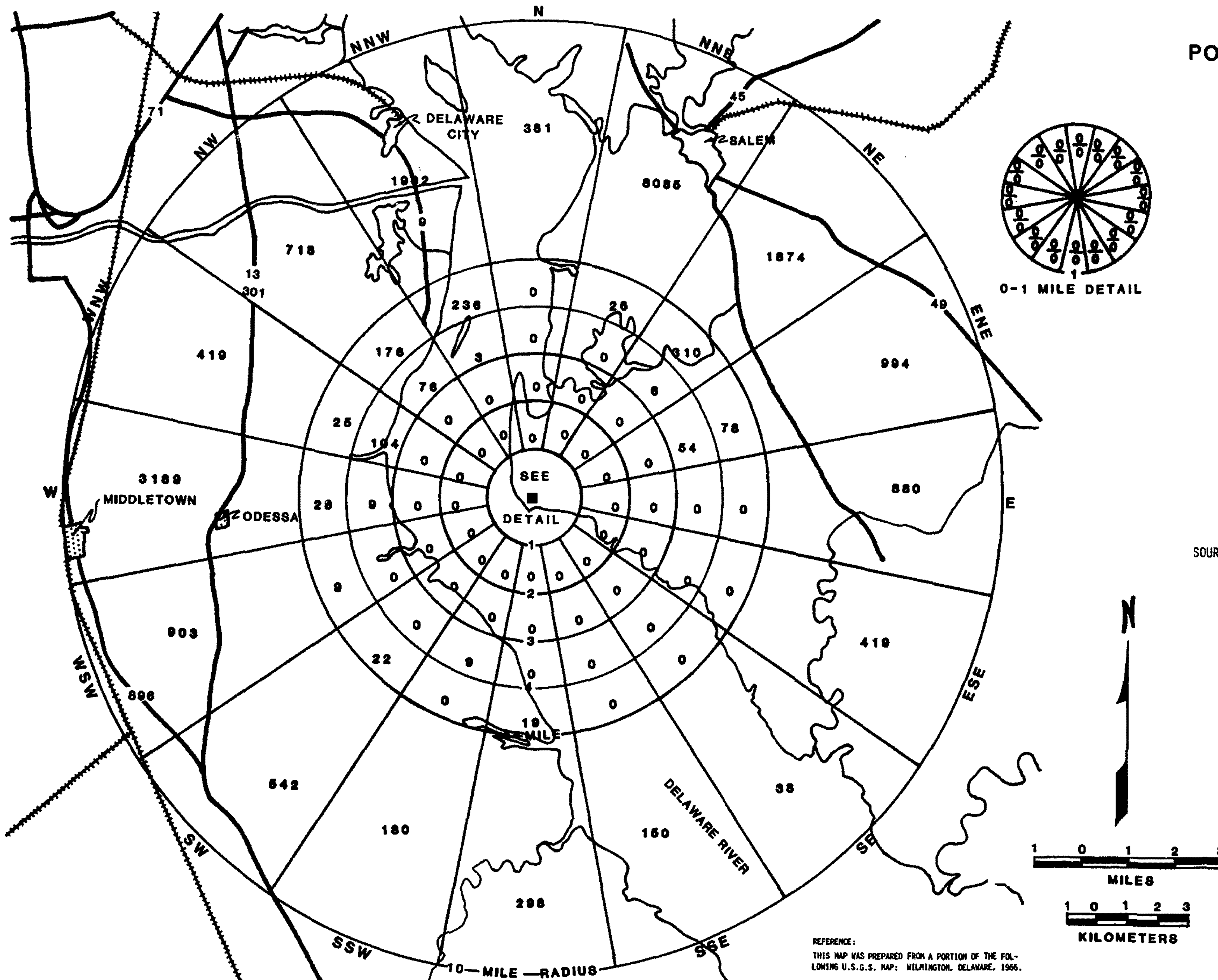
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY HOPE CREEK NUCLEAR GENERATING STATION	
SITE AREA	
UPDATED FSAR	FIGURE 2.1-2

POPULATION DISTRIBUTION
1980



SEE FIGURE 2.1-4
FOR 1980 TABULAR DATA

SOURCES: 1980 CENSUS OF POPULATION & HOUSING
FOR NEW JERSEY AND DELAWARE

0-10 MILES

REFERENCE:
THIS MAP WAS PREPARED FROM A PORTION OF THE FOLLOWING U.S.G.S. MAP: WILMINGTON, DELAWARE, 1966.

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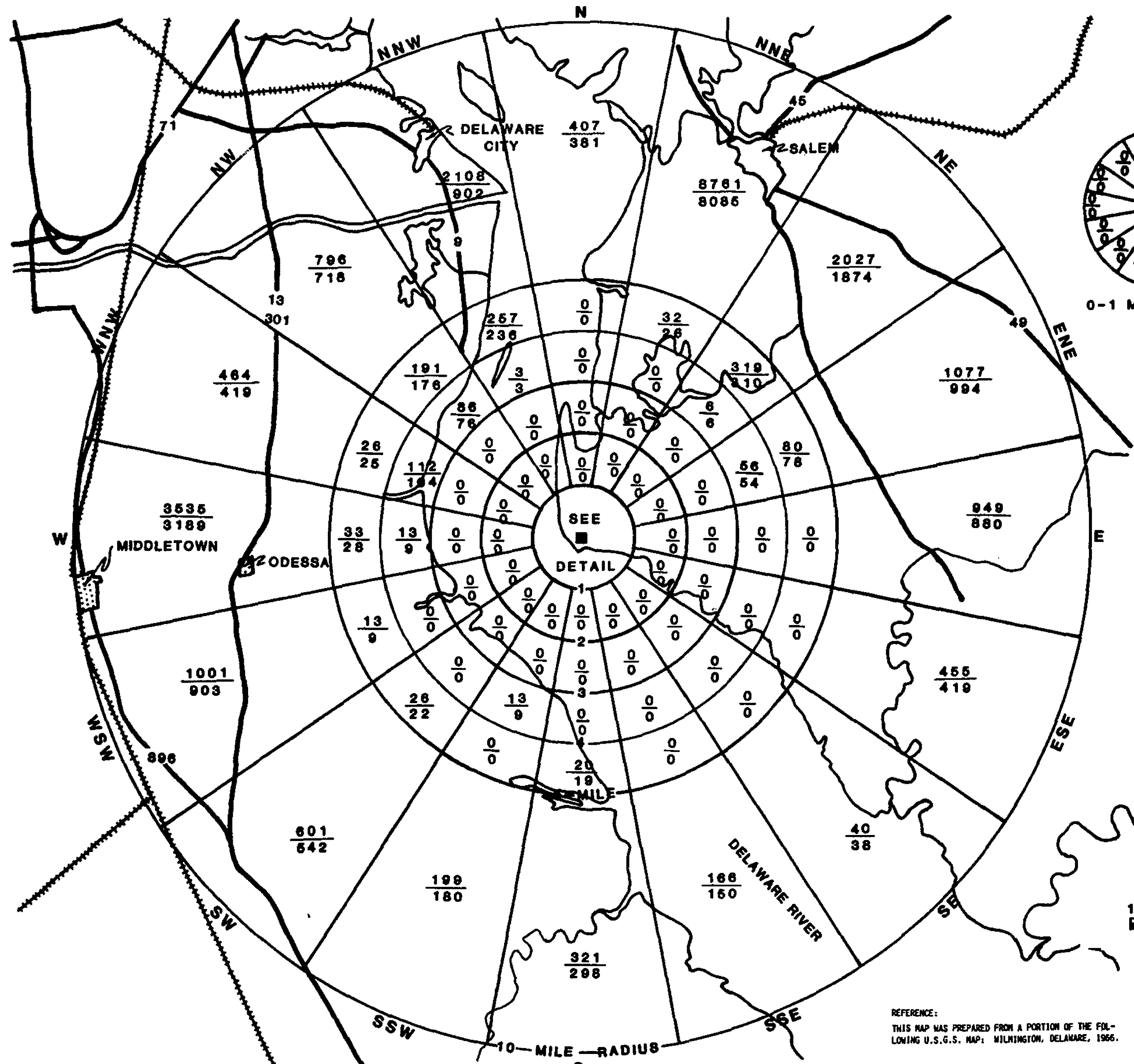
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

POPULATION DISTRIBUTION – YEAR 1980, WITHIN 0 TO 10 MILES

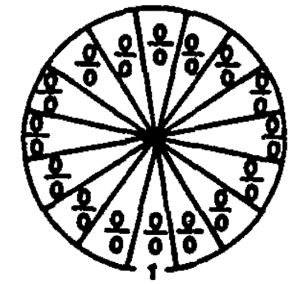
UPDATED FSAR

FIGURE 2.1-3



REFERENCE:
THIS MAP WAS PREPARED FROM A PORTION OF THE FOLLOWING U.S.G.S. MAP: WILMINGTON, DELAWARE, 1966.

POPULATION PROJECTIONS & POPULATION DISTRIBUTION
1987



0-1 MILE DETAIL

MILES	0-1	1-2	2-3	3-4	4-5	5-10	TOTAL
N	–	–	–	–	–	407/381	407/381
NNE	–	–	–	–	32/26	8761/8085	8793/8111
NE	–	–	–	6/6	319/310	2027/1874	2352/2190
ENE	–	–	–	56/54	80/78	1077/994	1213/1126
E	–	–	–	–	–	949/880	949/880
ESE	–	–	–	–	–	455/419	455/419
SE	–	–	–	–	–	40/38	40/38
SSE	–	–	–	–	–	166/150	166/150
S	–	–	–	–	20/19	321/298	341/317
SSW	–	–	–	13/9	–	199/180	212/189
SW	–	–	–	–	26/22	601/542	627/564
WSW	–	–	–	–	13/9	1001/903	1014/912
W	–	–	–	13/9	33/28	3535/3189	3581/3226
WNW	–	–	–	112/104	26/25	464/419	602/548
NW	–	–	–	86/76	191/176	796/718	1073/970
NNW	–	–	–	3/3	257/236	2108/1902	2368/2141
TOTAL	–	–	–	289/261	997/929	22907/20972	24193/22162

1987 POPULATION / 1980 POPULATION — PROJECTED / EXISTING

RADIUS IN MILES	YEAR	0-1	0-2	0-3	0-4	0-5	0-10
ACCUMULATED POPULATION	1987	–	–	–	289	1286	24193
	1980	–	–	–	261	1190	22162

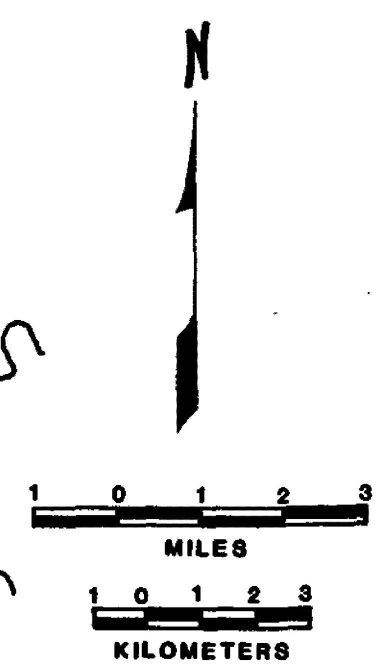
SOURCES: 1980 CENSUS OF POPULATION & HOUSING FOR NEW JERSEY AND DELAWARE

NEW JERSEY DEPT. OF LABOR, NEW JERSEY POPULATION PROJECTIONS 1980-2000, FEBRUARY 1982.

U.S. DEPT. OF COMMERCE, 1980 OBERS BEA REGIONAL PROJECTIONS, JULY 1981.

NEW CASTLE COUNTY PLANNING BOARD, POPULATION PROJECTIONS 1980-2000, MARCH 1982.

0-10 MILES



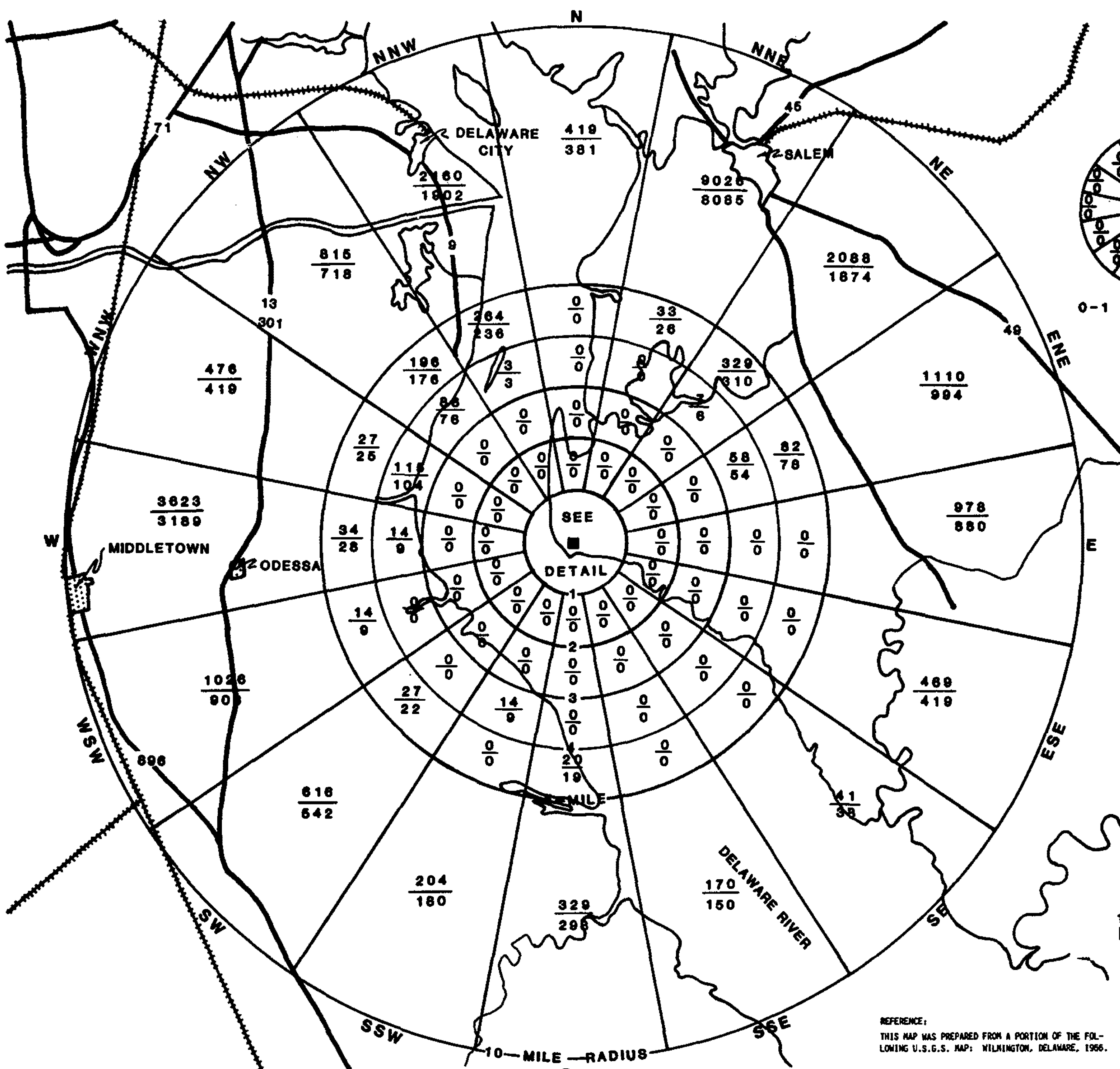
REVISION 0
APRIL 11, 1988

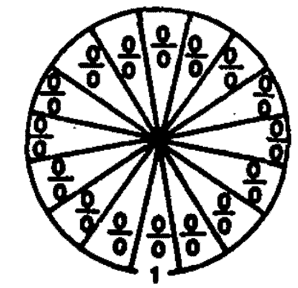
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

POPULATION DISTRIBUTION – YEAR 1987, WITHIN 0 TO 10 MILES

UPDATED FSAR

FIGURE 2.1-4





POPULATION PROJECTIONS & POPULATION DISTRIBUTION

1990

MILES	0-1	1-2	2-3	3-4	4-5	5-10	TOTAL
N	–	–	–	–	–	419/381	419/381
NNE	–	–	–	–	33/26	9026/8085	9059/8111
NE	–	–	–	7/6	329/310	2088/1174	2424/2190
ENE	–	–	–	58/54	82/78	1110/994	1250/1126
E	–	–	–	–	–	978/880	978/880
ESE	–	–	–	–	–	469/419	469/419
SE	–	–	–	–	–	41/38	41/38
SSE	–	–	–	–	–	170/150	170/150
S	–	–	–	–	20/19	329/298	349/317
SSW	–	–	–	14/9	–	204/180	218/189
SW	–	–	–	–	27/22	616/542	643/564
WSW	–	–	–	–	14/9	1026/903	1040/912
W	–	–	–	14/9	34/28	3623/3189	3671/3226
WNW	–	–	–	115/104	27/25	476/419	618/548
NW	–	–	–	88/76	196/176	815/718	1099/970
NNW	–	–	–	3/3	264/236	2160/1902	2427/2141
TOTAL	–	–	–	299/261	1026/929	23550/20972	24875/22162

1990 POPULATION / 1980 POPULATION: PROJECTED / EXISTING

RADIUS IN MILES	YEAR	0-1	0-2	0-3	0-4	0-5	0-10
ACCUMULATED POPULATION	1990	–	–	–	299	1325	24875
	1980	–	–	–	261	1190	22162

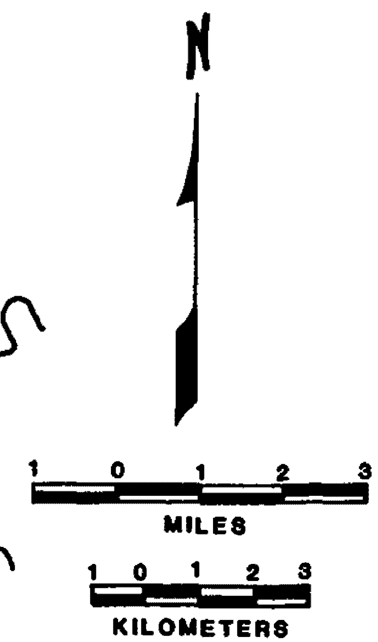
SOURCES: 1980 CENSUS OF POPULATION & HOUSING FOR NEW JERSEY AND DELAWARE

NEW JERSEY DEPT. OF LABOR, NEW JERSEY POPULATION PROJECTIONS 1980-2000, FEBRUARY 1982.

U.S. DEPT. OF COMMERCE, 1980 OBERS BEA REGIONAL PROJECTIONS, JULY 1981.

NEW CASTLE COUNTY PLANNING BOARD, POPULATION PROJECTIONS 1980-2000, MARCH 1982.

0-10 MILES



REFERENCE: THIS MAP WAS PREPARED FROM A PORTION OF THE FOLLOWING U.S.G.S. MAP: WILMINGTON, DELAWARE, 1956.

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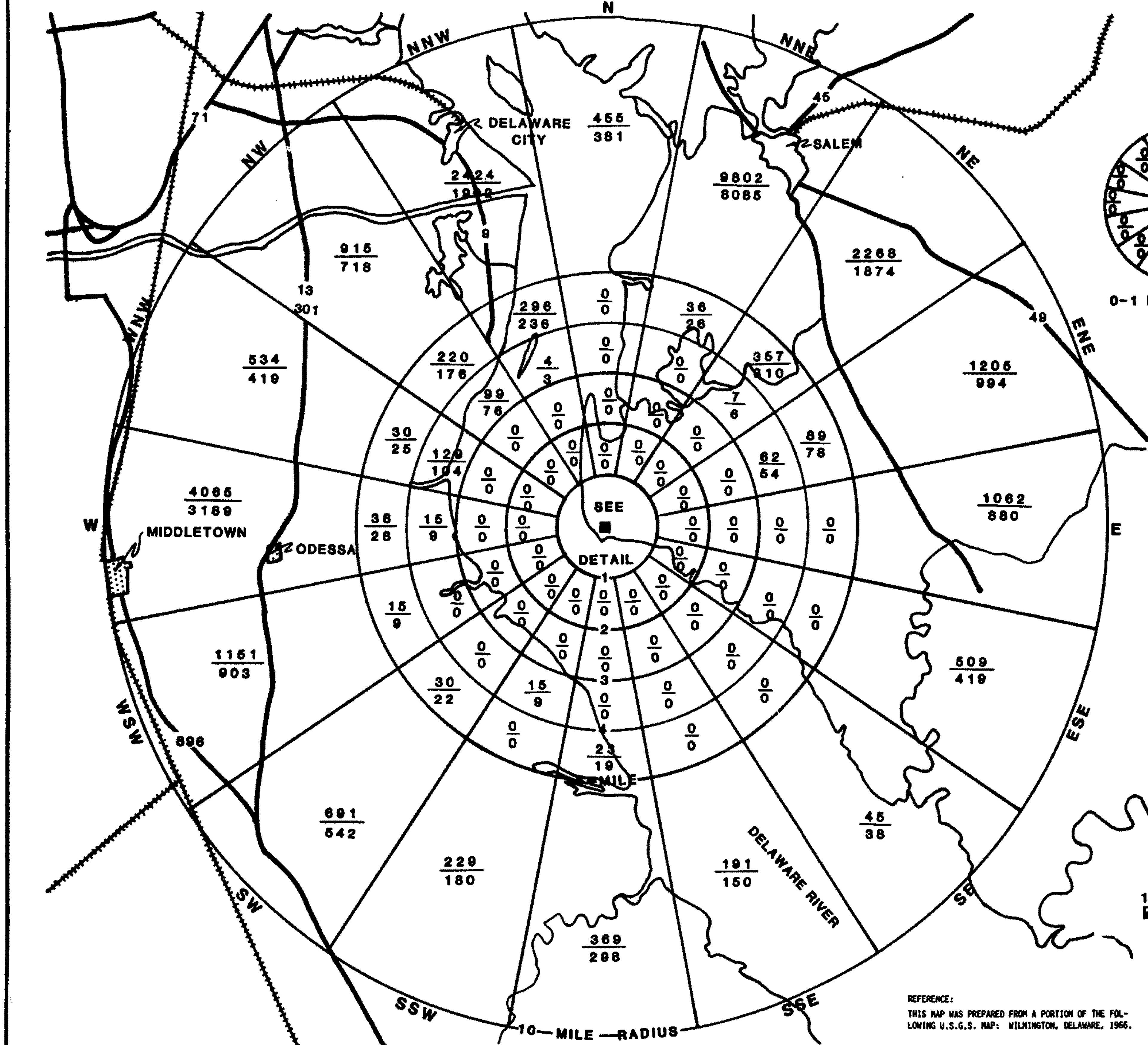
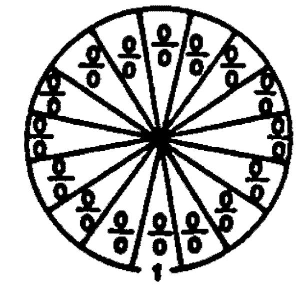
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

POPULATION DISTRIBUTION – YEAR 1990, WITHIN 0 to 10 MILES

UPDATED FSAR

FIGURE 2.1-5





0-1 MILE DETAIL

POPULATION PROJECTIONS & POPULATION DISTRIBUTION

2000

MILES	0-1	1-2	2-3	3-4	4-5	5-10	TOTAL
N	—	—	—	—	—	455/381	455/381
NNE	—	—	—	—	36/26	9802/8085	9838/8111
NE	—	—	—	7/6	357/310	2268/1874	2632/2190
ENE	—	—	—	62/54	89/78	1205/994	1356/1126
E	—	—	—	—	—	1062/880	1062/880
ESE	—	—	—	—	—	509/419	509/419
SE	—	—	—	—	—	45/38	45/38
SSE	—	—	—	—	—	191/150	191/150
S	—	—	—	—	23/19	369/298	392/317
SSW	—	—	—	15/9	—	229/180	244/189
SW	—	—	—	—	30/22	691/542	721/564
WSW	—	—	—	—	15/9	1151/903	1166/912
W	—	—	—	15/9	38/28	4065/3189	4118/3226
WNW	—	—	—	129/104	30/25	534/419	693/548
NW	—	—	—	99/76	220/176	915/718	1234/970
NNW	—	—	—	4/3	296/236	2424/1902	2724/2141
TOTAL	—	—	—	331/261	1134/929	25915/20972	27380/22162

2000 POPULATION — PROJECTED
1980 POPULATION — EXISTING

RADIUS IN MILES	YEAR	0-1	0-2	0-3	0-4	0-5	0-10
ACCUMULATED POPULATION	2000	—	—	—	331	1465	27380
	1980	—	—	—	261	1190	22162

SOURCES: 1980 CENSUS OF POPULATION & HOUSING FOR NEW JERSEY AND DELAWARE

NEW JERSEY DEPT. OF LABOR, NEW JERSEY POPULATION PROJECTIONS 1980-2000, FEBRUARY 1982.

U.S. DEPT. OF COMMERCE, 1980 OBERS BEA REGIONAL PROJECTIONS, JULY 1981.

NEW CASTLE COUNTY PLANNING BOARD, POPULATION PROJECTIONS 1980-2000, MARCH 1982.

N

0-10 MILES

1 0 1 2 3
MILES

1 0 1 2 3
KILOMETERS

REFERENCE:
THIS MAP WAS PREPARED FROM A PORTION OF THE FOLLOWING U.S.G.S. MAP: WILMINGTON, DELAWARE, 1966.

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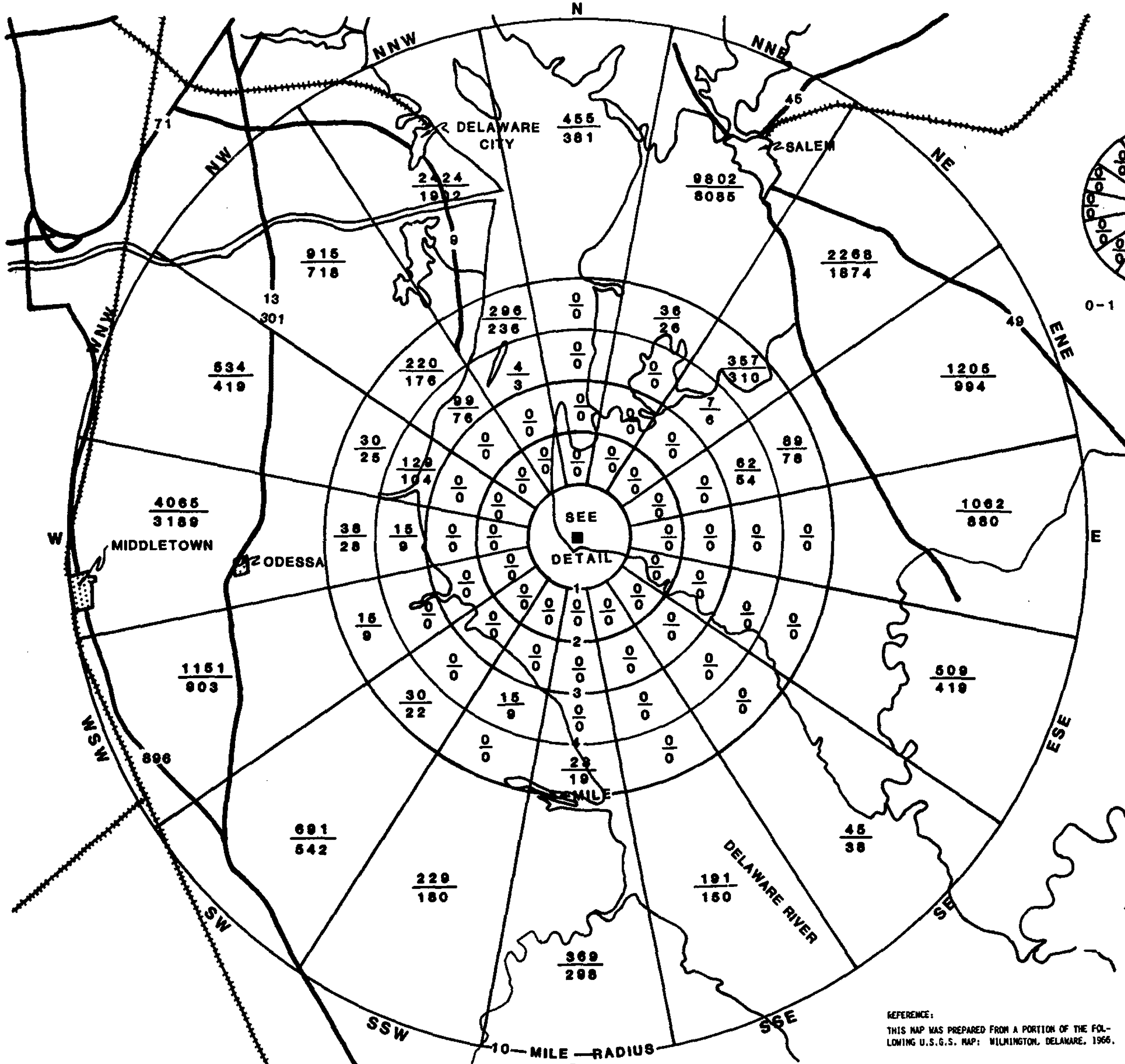
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

POPULATION DISTRIBUTION — YEAR 2000, WITHIN 0 TO 10 MILES

UPDATED FSAR

FIGURE 2.1-6

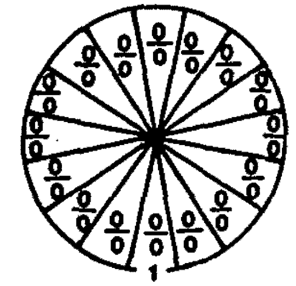


POPULATION PROJECTIONS & POPULATION DISTRIBUTION

2010

MILES	0-1	1-2	2-3	3-4	4-5	5-10	TOTAL
N	–	–	–	–	–	455/381	455/381
NNE	–	–	–	–	36/26	9802/8085	9838/8111
NE	–	–	–	7/6	357/310	2268/1874	2632/2190
ENE	–	–	–	62/54	89/78	1205/994	1356/1126
E	–	–	–	–	–	1062/880	1062/880
ESE	–	–	–	–	–	509/419	509/419
SE	–	–	–	–	–	45/38	45/38
SSE	–	–	–	–	–	191/150	191/150
S	–	–	–	–	23/19	369/298	392/317
SSW	–	–	–	15/9	–	229/180	244/189
SW	–	–	–	–	30/22	691/542	721/564
WSW	–	–	–	–	15/9	1151/903	1166/912
W	–	–	–	15/9	38/28	4065/3189	4118/3226
WNW	–	–	–	129/104	30/25	534/419	693/548
NW	–	–	–	99/76	220/176	915/718	1234/970
NNW	–	–	–	4/3	296/236	2424/1902	2724/2141
TOTAL	–	–	–	331/261	1134/929	25915/20972	27380/22162

2010 POPULATION / 1980 POPULATION — PROJECTED / EXISTING



0-1 MILE DETAIL

RADIUS IN MILES	YEAR	0-1	0-2	0-3	0-4	0-5	0-10
ACCUMULATED POPULATION	2010	–	–	–	331	1465	27380
	1980	–	–	–	261	1190	22162

SOURCES: 1980 CENSUS OF POPULATION & HOUSING FOR NEW JERSEY AND DELAWARE

NEW JERSEY DEPT. OF LABOR, NEW JERSEY POPULATION PROJECTIONS 1980-2000, FEBRUARY 1982.

U.S. DEPT. OF COMMERCE, 1980 OBERS BEA REGIONAL PROJECTIONS, JULY 1981.

NEW CASTLE COUNTY PLANNING BOARD, POPULATION PROJECTIONS 1980-2000, MARCH 1982.

0-10 MILES

N

1 0 1 2 3
MILES

1 0 1 2 3
KILOMETERS

REFERENCE:
THIS MAP WAS PREPARED FROM A PORTION OF THE FOLLOWING U.S.G.S. MAP: WILMINGTON, DELAWARE, 1966.

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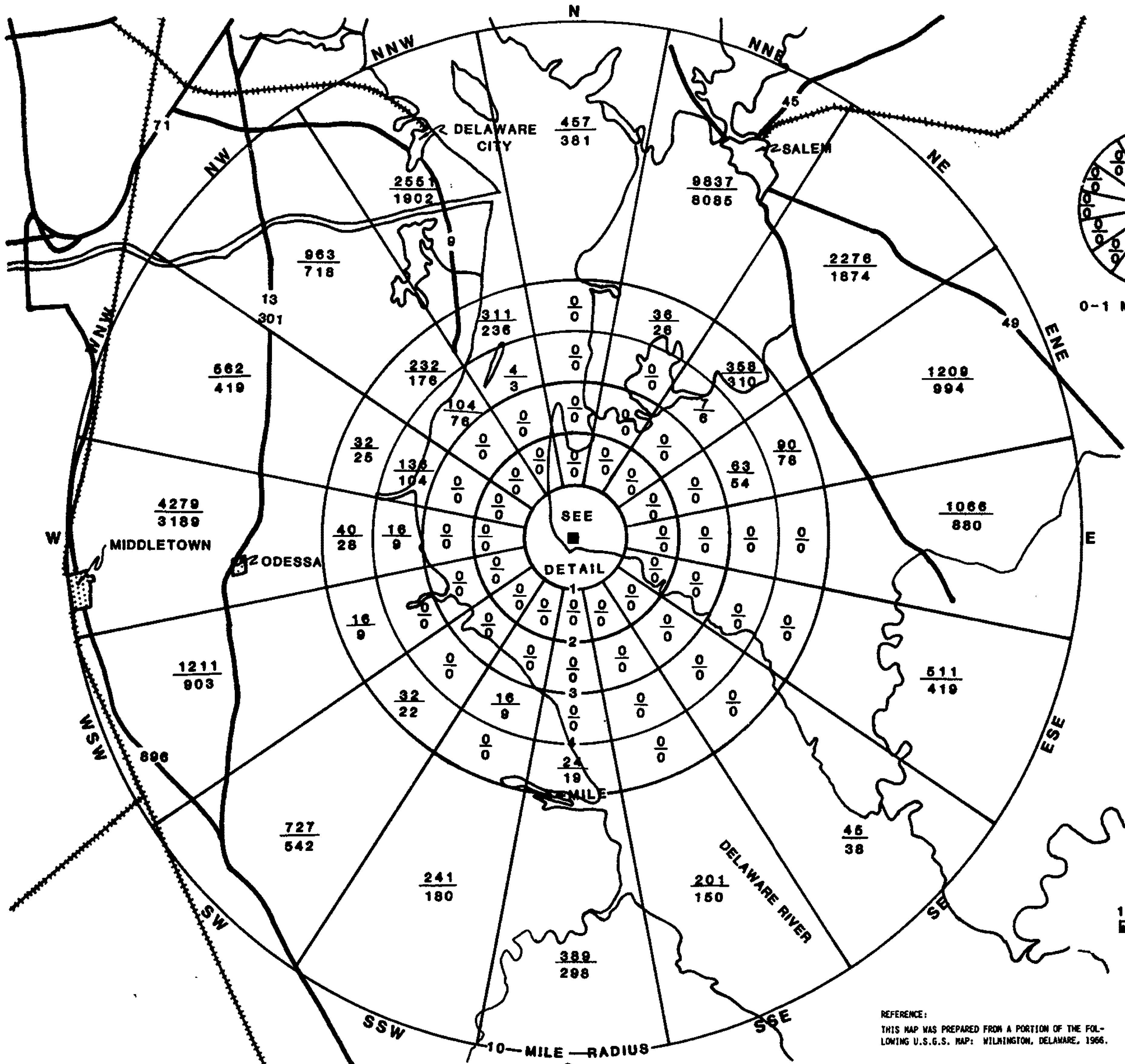
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

POPULATION DISTRIBUTION – YEAR 2010, WITHIN 0 TO 10 MILES

UPDATED FSAR

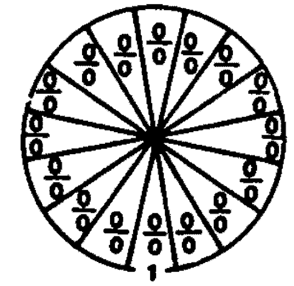
FIGURE 2.1-7



POPULATION PROJECTIONS & POPULATION DISTRIBUTION
2020

MILES	0-1	1-2	2-3	3-4	4-5	5-10	TOTAL
N	–	–	–	–	–	457/381	457/381
NNE	–	–	–	–	36/26	9837/8085	9873/8111
NE	–	–	–	7/6	358/310	2276/1874	2641/2190
ENE	–	–	–	63/54	90/78	1209/994	1362/1126
E	–	–	–	–	–	1066/880	1066/880
ESE	–	–	–	–	–	511/419	511/419
SE	–	–	–	–	–	45/38	45/38
SSE	–	–	–	–	–	201/150	201/150
S	–	–	–	–	24/19	389/298	413/317
SSW	–	–	–	16/9	–	241/180	257/189
SW	–	–	–	–	32/22	727/542	759/564
WSW	–	–	–	–	16/9	1211/903	1227/912
W	–	–	–	16/9	40/28	4279/3189	4335/3226
WNW	–	–	–	136/104	32/25	562/419	730/548
NW	–	–	–	104/76	232/176	963/718	1299/970
NNW	–	–	–	4/3	311/236	2551/1902	2866/2141
TOTAL	–	–	–	346/261	1171/929	26525/20972	28042/22162

2020 POPULATION / 1980 POPULATION — PROJECTED / EXISTING



0-1 MILE DETAIL

RADIUS IN MILES	YEAR	0-1	0-2	0-3	0-4	0-5	0-10
ACCUMULATED POPULATION	2020	–	–	–	346	1517	28042
	1980	–	–	–	261	1190	22162

SOURCES: 1980 CENSUS OF POPULATION & HOUSING FOR NEW JERSEY AND DELAWARE

NEW JERSEY DEPT. OF LABOR, NEW JERSEY POPULATION PROJECTIONS 1980-2000, FEBRUARY 1982.

U.S. DEPT. OF COMMERCE, 1980 OBERS BEA REGIONAL PROJECTIONS, JULY 1981.

NEW CASTLE COUNTY PLANNING BOARD, POPULATION PROJECTIONS 1980-2000, MARCH 1982.

0-10 MILES

N

1 0 1 2 3
MILES

1 0 1 2 3
KILOMETERS

REFERENCE:
THIS MAP WAS PREPARED FROM A PORTION OF THE FOLLOWING U.S.G.S. MAP: WILMINGTON, DELAWARE, 1966.

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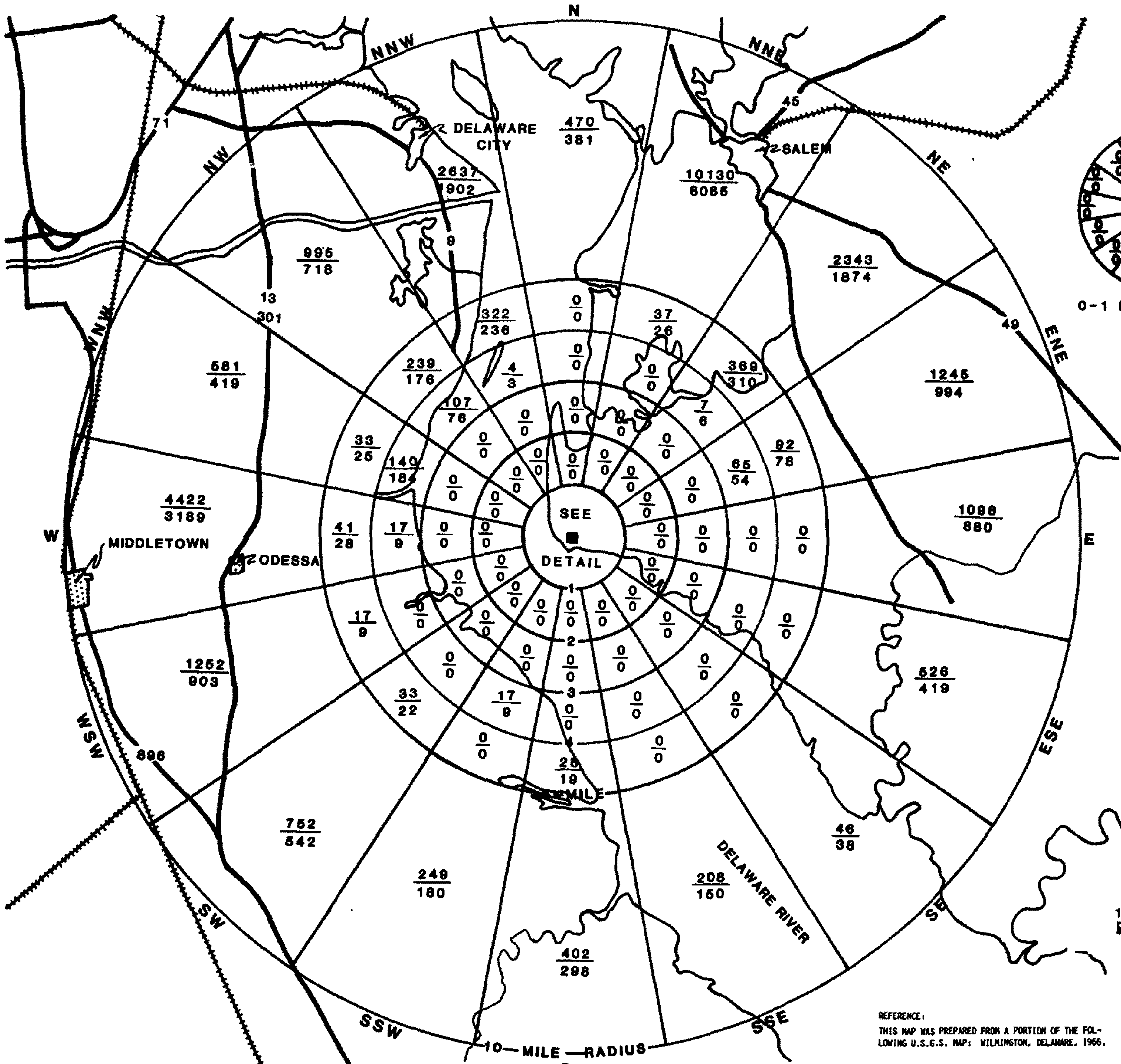
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

POPULATION DISTRIBUTION – YEAR 2020, WITHIN 0 TO 10 MILES

UPDATED FSAR

FIGURE 2.1-8

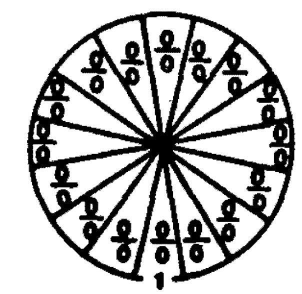


POPULATION PROJECTIONS & POPULATION DISTRIBUTION

2030

MILES	0-1	1-2	2-3	3-4	4-5	5-10	TOTAL
N	–	–	–	–	–	470/381	470/381
NNE	–	–	–	–	37/26	10130/8085	10167/8111
NE	–	–	–	7/6	369/310	2343/1874	2719/2190
ENE	–	–	–	65/54	92/78	1245/994	1402/1126
E	–	–	–	–	–	1098/880	1098/880
ESE	–	–	–	–	–	526/419	526/419
SE	–	–	–	–	–	46/38	46/38
SSE	–	–	–	–	–	208/150	208/150
S	–	–	–	–	25/19	402/298	427/317
SSW	–	–	–	17/9	–	249/180	266/189
SW	–	–	–	–	33/22	752/542	785/564
WSW	–	–	–	–	17/9	1252/903	1269/912
W	–	–	–	17/9	41/28	4422/3189	4480/3226
WNW	–	–	–	140/104	33/25	581/419	754/548
NW	–	–	–	107/76	239/176	995/718	1341/970
NNW	–	–	–	4/3	322/236	2637/1902	2963/2141
TOTAL	–	–	–	357/261	1208/929	27356/20972	28921/22162

2030 POPULATION PROJECTED
1980 POPULATION EXISTING



0-1 MILE DETAIL

RADIUS IN MILES	YEAR	0-1	0-2	0-3	0-4	0-5	0-10
ACCUMULATED POPULATION	2030	–	–	–	357	1565	28921
	1980	–	–	–	261	1190	22162

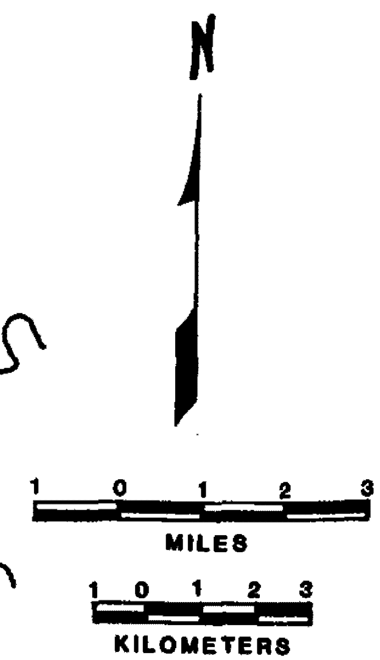
SOURCES: 1980 CENSUS OF POPULATION & HOUSING FOR NEW JERSEY AND DELAWARE

NEW JERSEY DEPT. OF LABOR, NEW JERSEY POPULATION PROJECTIONS 1980-2000, FEBRUARY 1982.

U.S. DEPT. OF COMMERCE, 1980 OBERS BEA REGIONAL PROJECTIONS, JULY 1981.

NEW CASTLE COUNTY PLANNING BOARD, POPULATION PROJECTIONS 1980-2000, MARCH 1982.

0-10 MILES



REFERENCE:
THIS MAP WAS PREPARED FROM A PORTION OF THE FOLLOWING U.S.G.S. MAP: WILMINGTON, DELAWARE, 1966.

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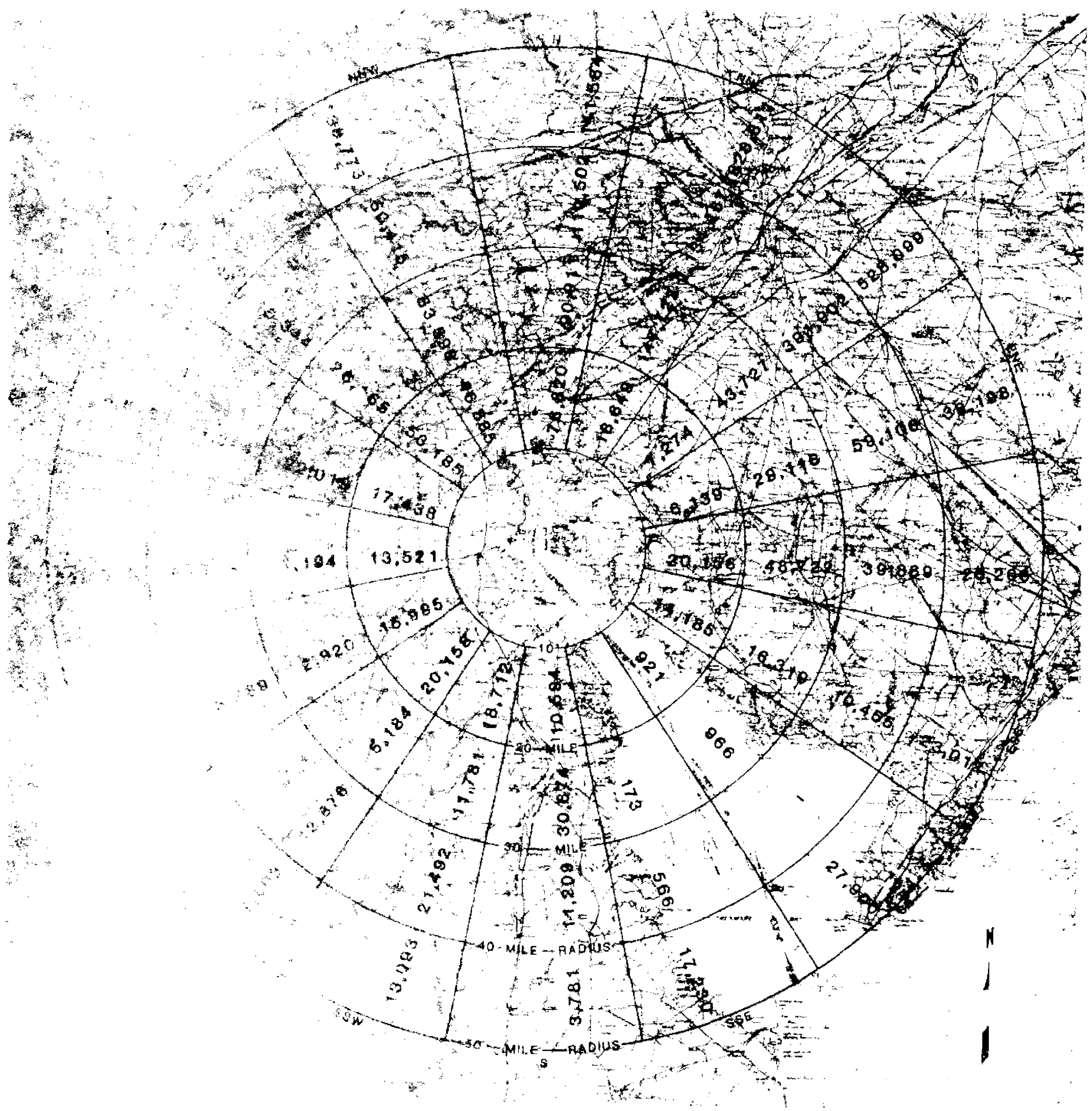
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

POPULATION DISTRIBUTION – YEAR 2030, WITHIN 0 TO 10 MILES

UPDATED FSAR

FIGURE 2.1-9



POPULATION DISTRIBUTION

1980

MILES	10-20	20-30	30-40	40-50	TOTAL
N	76.8	90.9	110.5	141.6	419.8
NNE	18.6	147.7	1134.2	928.8	2229.3
NE	7.3	43.7	391.9	528.1	971.0
ENE	6.1	29.1	59.1	39.2	133.6
E	20.2	48.7	39.9	23.2	132.0
ESE	14.2	16.3	10.5	23.0	64.0
SE	.9	1.0	—	27.9	29.8
SSE	—	.2	.6	17.2	18.0
S	10.6	30.7	11.2	3.8	56.3
SSW	18.7	11.8	21.5	13.1	65.1
SW	20.2	5.2	12.9	7.5	45.7
WSW	16.0	2.9	8.9	9.3	37.1
W	13.5	1.2	43.8	229.6	288.1
WNW	17.4	22.0	41.3	16.5	97.3
NW	53.2	26.2	15.3	29.4	124.2
NNW	46.9	63.6	50.4	38.8	199.7
TOTAL	340.6	541.2	1952.0	2077.1	4910.8

RADIUS IN MILES	YEAR	0-10	0-20	0-30	0-40	0-50
ACCUMULATED POPULATION	1980	22.2	362.8	904.0	2855.9	4933.0

POPULATION IN 000'S

10-50 MILES

SOURCE: 1980 CENSUS OF POPULATION & HOUSING FOR NEW JERSEY, DELAWARE, PENNSYLVANIA AND MARYLAND.

REFERENCE
THIS MAP WAS PREPARED FROM PORTIONS OF THE FOLLOWING SECTIONAL AERONAUTICAL CHART NEW YORK

5 0 5 20
MILES

5 0 5 20
KILOMETERS

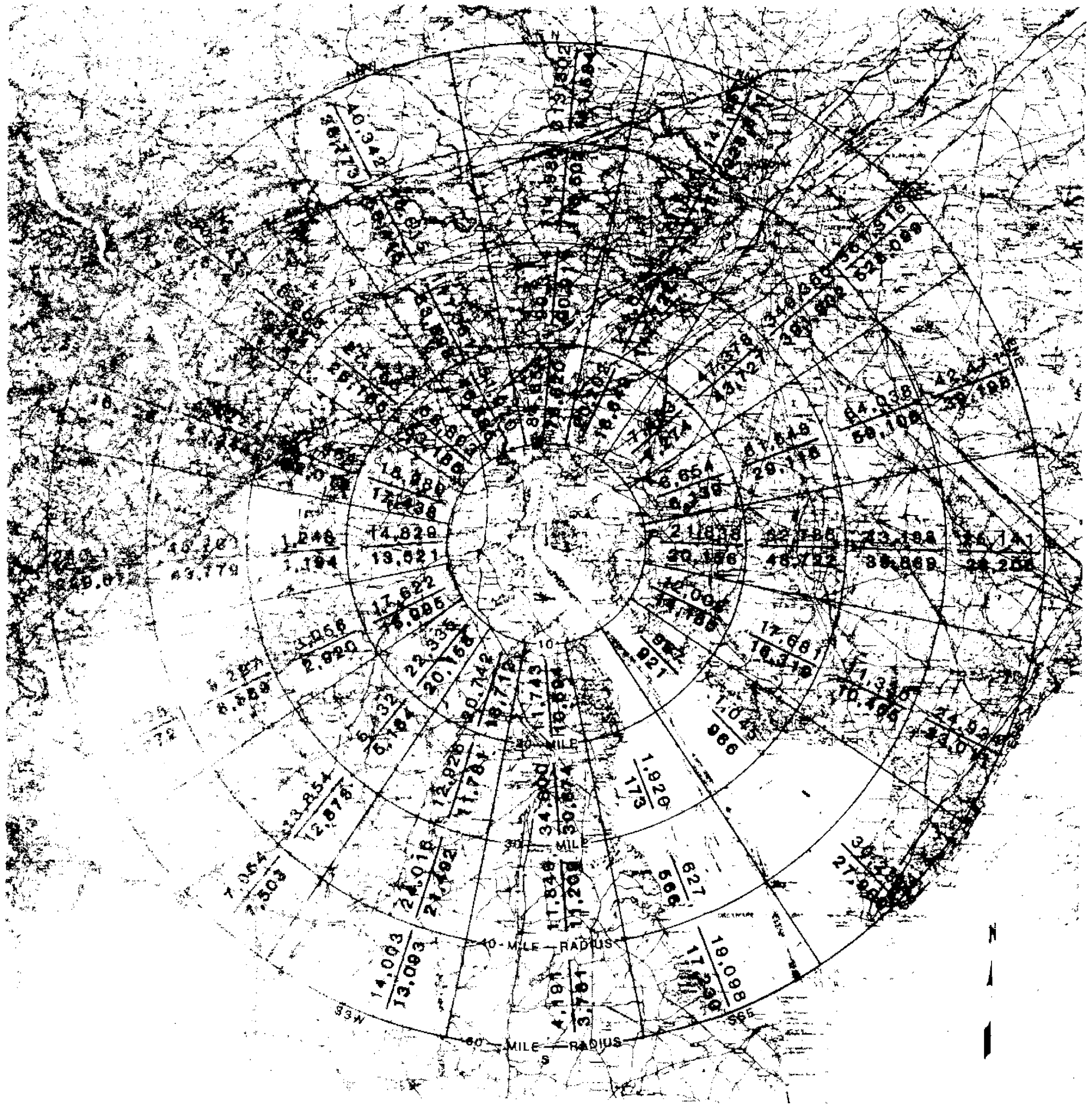
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

POPULATION DISTRIBUTION – YEAR 1980, WITHIN 10 TO 50 MILES

UPDATED FSAR

FIGURE 2.1-10



REFERENCE:
THIS MAP WAS PREPARED FROM PORTIONS OF THE FOLLOWING SECTIONAL AERONAUTICAL CHART NEW YORK

5 0 5 20
MILES

5 0 5 20
KILOMETERS

POPULATION PROJECTIONS & POPULATION DISTRIBUTION

1987

MILES	10-20	20-30	30-40	40-50	TOTAL
N	84.9 / 76.8	99.3 / 90.9	115.0 / 110.5	313.3 / 141.6	612.5 / 419.8
NNE	20.2 / 18.6	144.1 / 147.7	1171.3 / 1134.2	1141.7 / 928.8	2477.3 / 2229.3
NE	7.9 / 7.3	47.4 / 43.7	349.4 / 391.9	561.3 / 528.1	965.9 / 971.0
ENE	6.7 / 6.1	31.5 / 29.1	64.0 / 59.1	42.5 / 39.2	144.7 / 133.6
E	21.8 / 20.2	52.8 / 48.7	43.2 / 39.9	25.1 / 23.2	143.0 / 132.0
ESE	17.0 / 14.2	17.7 / 16.3	11.3 / 10.5	24.9 / 23.0	66.0 / 64.0
SE	1.0 / .9	1.0 / 1.0	—	30.2 / 27.9	32.2 / 29.8
SSE	—	1.9 / .2	.6 / .6	19.1 / 17.2	21.6 / 18.0
S	11.7 / 10.6	34.0 / 30.7	11.8 / 11.2	4.7 / 3.8	61.8 / 56.3
SSW	20.7 / 18.7	12.9 / 11.8	24.0 / 21.5	14.0 / 13.1	71.7 / 65.1
SW	22.3 / 20.2	5.4 / 5.2	13.9 / 12.9	8.0 / 7.5	49.6 / 45.7
WSW	17.6 / 16.0	3.1 / 2.9	9.3 / 8.9	9.7 / 9.3	39.7 / 37.1
W	14.8 / 13.5	1.2 / 1.2	45.8 / 43.8	240.1 / 229.6	302.0 / 288.1
WNW	19.0 / 17.4	22.0 / 22.0	43.2 / 41.3	17.2 / 16.5	101.4 / 97.3
NW	58.8 / 53.2	27.6 / 26.2	18.4 / 15.3	30.7 / 29.4	135.7 / 124.2
NNW	52.0 / 46.9	111.4 / 63.6	48.6 / 50.4	40.3 / 38.8	252.3 / 199.7
TOTAL	371.4 / 340.6	613.5 / 541.2	1969.9 / 1952.0	2522.5 / 2077.1	5477.3 / 4910.8

RADIUS IN MILES	YEAR	0-10	0-20	0-30	0-40	0-60
ACCUMULATED POPULATION	1987	24.2	395.6	1009.1	2979.0	5501.4
	1980	22.2	362.8	504.0	2855.9	4933.0

1987 POPULATION / 1980 POPULATION: 1987 POPULATION PROJECTED; 1980 POPULATION EXISTING

POPULATION IN 000'S

10-50 MILES

SOURCE: 1980 CENSUS OF POPULATION & HOUSING FOR NEW JERSEY, DELAWARE, PENNSYLVANIA AND MARYLAND.

NEW JERSEY DEPT. OF LABOR, NEW JERSEY POPULATION PROJECTIONS 1980-2000, FEBRUARY 1982.

U.S. DEPT. OF COMMERCE, 1980 OBERS BEA REGIONAL PROJECTIONS, JULY 1981.

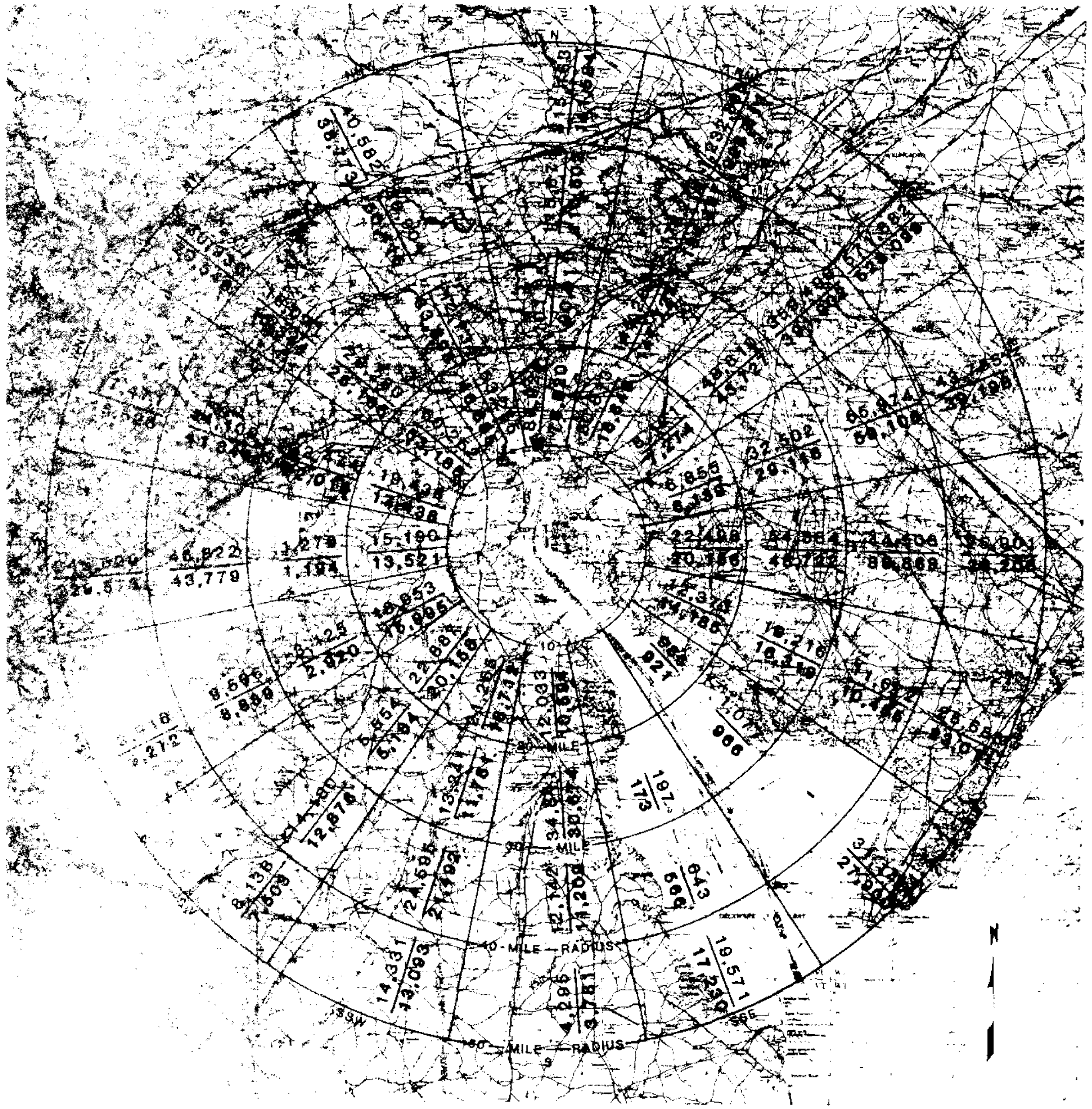
NEW CASTLE COUNTY PLANNING BOARD, POPULATION PROJECTIONS 1980-2000, MARCH 1982.

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

POPULATION DISTRIBUTION — YEAR 1987, WITHIN 10 TO 50 MILES

UPDATED FSAR | FIGURE 2.1-11



REFERENCE:
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NEW YORK

5 0 5 20
MILES

5 0 5 20
KILOMETERS

POPULATION PROJECTIONS & POPULATION DISTRIBUTION

1990

MILES	10-20	20-30	30-40	40-50	TOTAL
N	87.0 / 76.8	101.4 / 90.9	115.7 / 110.5	315.2 / 141.6	619.2 / 419.8
NNE	20.8 / 18.6	145.3 / 147.7	1178.3 / 1134.2	1232.9 / 928.8	2577.3 / 2229.3
NE	8.1 / 7.3	48.8 / 43.7	359.5 / 391.9	571.9 / 528.1	998.3 / 971.0
ENE	6.9 / 6.1	32.5 / 29.1	66.0 / 59.1	43.8 / 39.2	149.1 / 133.6
E	22.5 / 20.2	54.4 / 48.7	44.5 / 39.9	25.9 / 23.2	147.3 / 132.0
ESE	12.4 / 14.2	18.2 / 16.3	11.7 / 10.5	25.7 / 23.0	67.9 / 64.0
SE	1.0 / .9	1.0 / 1.0	—	31.1 / 27.9	33.2 / 29.8
SSE	—	.2 / .2	.6 / .6	19.6 / 17.2	20.4 / 18.0
S	12.0 / 10.6	34.8 / 30.7	12.1 / 11.2	4.3 / 3.8	63.3 / 56.3
SSW	21.3 / 18.7	13.2 / 11.8	24.6 / 21.5	14.3 / 13.1	73.4 / 65.1
SW	22.9 / 20.2	5.6 / 5.2	14.2 / 12.9	8.1 / 7.5	50.8 / 45.7
WSW	18.1 / 16.0	3.1 / 2.9	9.5 / 8.9	9.9 / 9.3	40.6 / 37.1
W	15.2 / 13.5	1.3 / 1.2	46.8 / 43.8	245.5 / 229.6	308.8 / 288.1
WNW	19.4 / 17.4	22.5 / 22.0	44.1 / 41.3	17.5 / 16.5	103.5 / 97.3
NW	60.3 / 53.2	28.3 / 26.2	16.1 / 15.3	30.9 / 29.4	135.5 / 124.2
NNW	53.3 / 46.9	112.7 / 63.6	48.9 / 50.4	40.6 / 38.8	255.5 / 199.7
TOTAL	381.0 / 340.6	623.3 / 541.2	1992.6 / 1952.0	2367.2 / 2077.1	5634.1 / 4910.8

RADIUS IN MILES	YEAR	0-10	0-20	0-30	0-40	0-50
ACCUMULATED POPULATION	1990	24.9	405.9	1029.2	3021.8	5659.0
	1980	22.2	362.8	904.0	2855.9	4933.0

1990 POPULATION / 1980 POPULATION — PROJECTED / EXISTING — POPULATION IN 000'S

10-50 MILES

SOURCE: 1980 CENSUS OF POPULATION & HOUSING FOR NEW JERSEY, DELAWARE, PENNSYLVANIA AND MARYLAND.

NEW JERSEY DEPT. OF LABOR, NEW JERSEY POPULATION PROJECTIONS 1980-2000, FEBRUARY 1982.

U.S. DEPT. OF COMMERCE, 1980 OBERS BEA REGIONAL PROJECTIONS, JULY 1981.

NEW CASTLE COUNTY PLANNING BOARD, POPULATION PROJECTIONS 1980-2000, MARCH 1982.

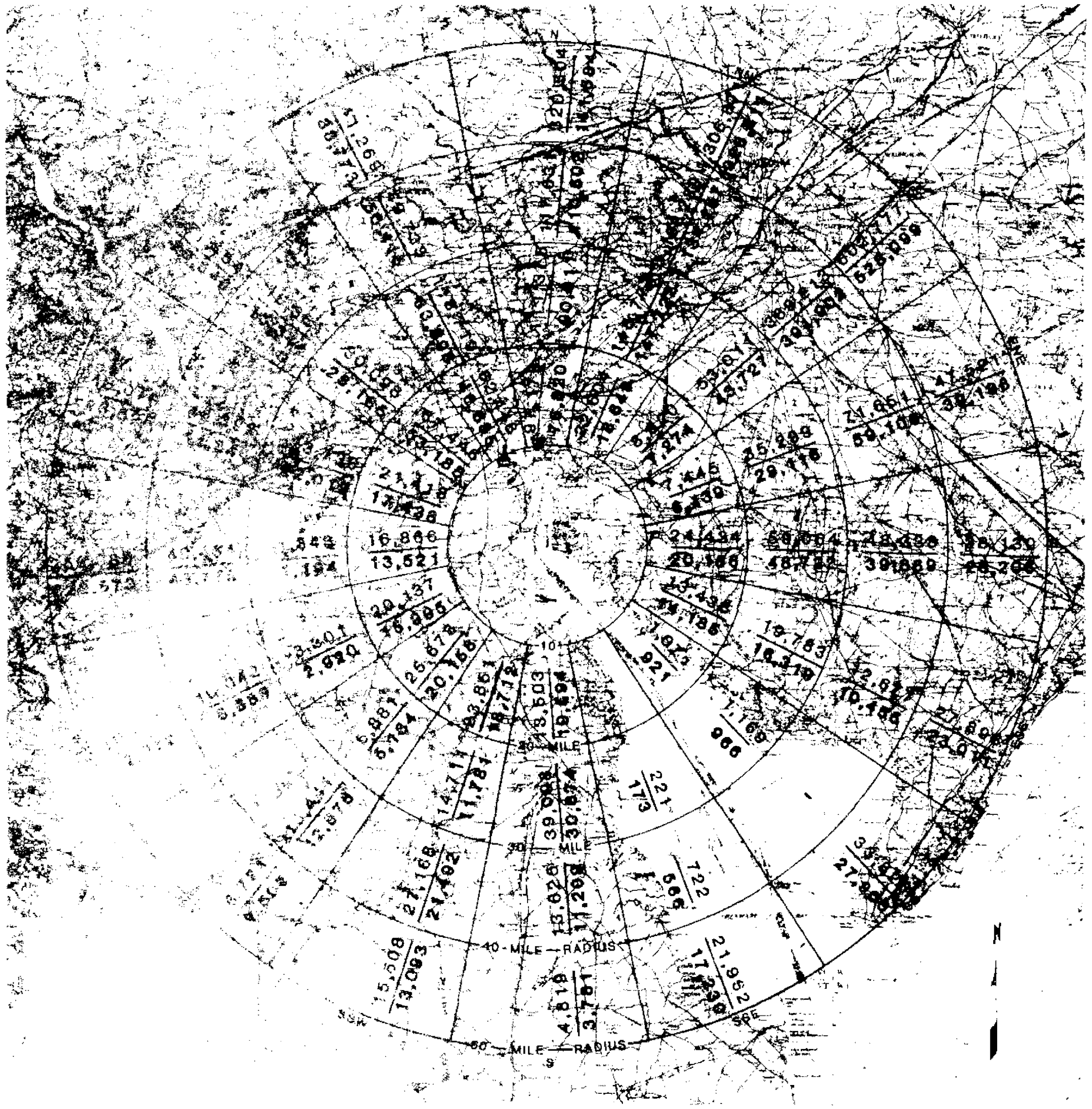
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

POPULATION DISTRIBUTION – YEAR 1990, WITHIN 10 TO 50 MILES

UPDATED FSAR

FIGURE 2.1-12



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NEW YORK

5 0 5 20
MILES
5 0 5 20
KILOMETERS

POPULATION PROJECTIONS & POPULATION DISTRIBUTION

2000

MILES	10-20	20-30	30-40	40-50	TOTAL
N	97.2 / 76.8	111.4 / 90.9	117.6 / 110.5	320.5 / 141.6	646.7 / 419.8
NNE	22.6 / 18.6	148.8 / 147.7	1198.2 / 1134.2	1306.9 / 928.8	2676.5 / 2229.3
NE	8.8 / 7.3	53.0 / 43.7	389.2 / 391.9	602.8 / 528.1	1053.8 / 971.0
ENE	7.4 / 6.1	35.3 / 29.1	71.7 / 59.1	47.5 / 39.2	161.9 / 133.6
E	24.4 / 20.2	59.1 / 48.7	48.3 / 39.9	28.1 / 23.2	160.0 / 132.0
ESE	13.4 / 14.2	19.8 / 16.3	12.7 / 10.5	27.9 / 23.0	73.8 / 64.0
SE	1.1 / .9	1.2 / 1.0	—	33.8 / 27.9	36.1 / 29.8
SSE	—	.2 / .2	.7 / .6	22.0 / 17.2	22.9 / 18.0
S	13.5 / 10.6	39.1 / 30.7	13.6 / 11.2	4.8 / 3.8	71.0 / 56.3
SSW	23.3 / 18.7	14.7 / 11.8	27.2 / 21.5	15.6 / 13.1	91.2 / 65.1
SW	25.7 / 20.2	5.9 / 5.2	15.4 / 12.9	8.7 / 7.5	55.7 / 45.7
WSW	20.1 / 16.0	3.3 / 2.9	10.0 / 8.9	10.5 / 9.3	44.0 / 37.1
W	15.9 / 13.5	1.3 / 1.2	49.9 / 43.8	259.4 / 229.6	237.1 / 288.1
WNW	21.4 / 17.4	23.7 / 22.0	46.4 / 41.3	18.1 / 16.5	109.6 / 97.3
NW	67.4 / 53.2	30.1 / 26.2	16.4 / 15.3	31.5 / 29.4	145.4 / 124.2
NNW	62.4 / 46.9	118.7 / 63.6	49.7 / 50.4	41.3 / 38.8	272.2 / 199.7
TOTAL	426.3 / 340.6	665.6 / 541.2	2066.7 / 1952.0	2779.2 / 2077.1	5937.8 / 4910.8

RADIUS IN MILES	YEAR	0-10	0-20	0-30	0-40	0-60
ACCUMULATED POPULATION	2000	27.4	453.7	1119.3	3186.0	5965.2
	1980	22.2	362.8	904.0	2855.9	4933.0

2000 POPULATION / 1980 POPULATION — PROJECTED / EXISTING — POPULATION IN 000'S

10-50 MILES

SOURCE: 1980 CENSUS OF POPULATION & HOUSING FOR NEW JERSEY, DELAWARE, PENNSYLVANIA AND MARYLAND.

NEW JERSEY DEPT. OF LABOR, NEW JERSEY POPULATION PROJECTIONS 1980-2000, FEBRUARY 1982.

U.S. DEPT. OF COMMERCE, 1980 OBERS BEA REGIONAL PROJECTIONS, JULY 1981.

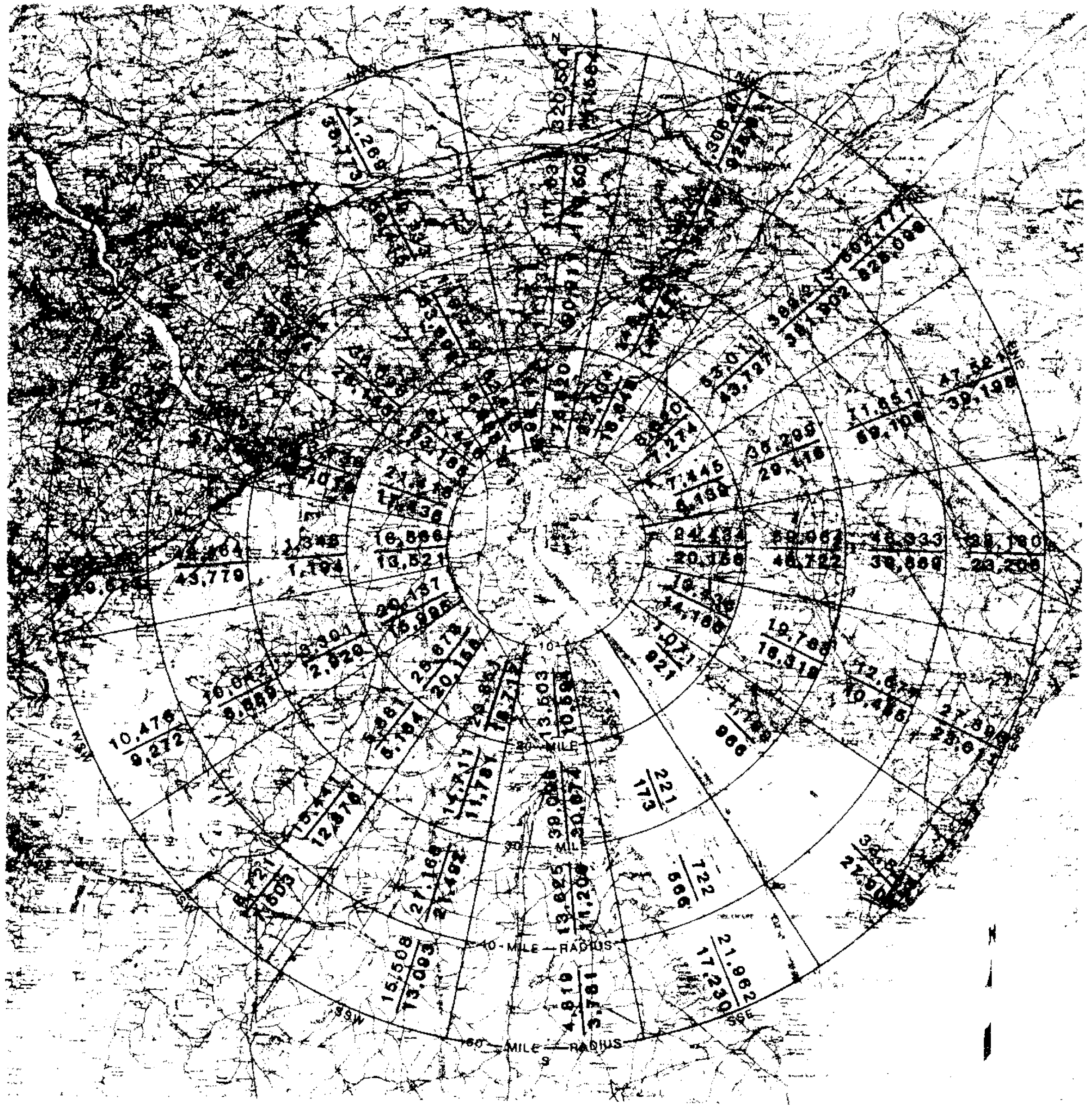
NEW CASTLE COUNTY PLANNING BOARD, POPULATION PROJECTIONS 1980-2000, MARCH 1982.

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

POPULATION DISTRIBUTION – YEAR 2000, WITHIN 10 TO 50 MILES

UPDATED FSAR — FIGURE 2.1-13



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NEW YORK

5 0 5 20
MILES

5 0 5 20
KILOMETERS

POPULATION PROJECTIONS & POPULATION DISTRIBUTION

2010

MILES	10-20	20-30	30-40	40-50	TOTAL
N	97.2 / 76.8	111.4 / 90.9	117.6 / 110.5	320.5 / 141.6	646.7 / 419.8
NNE	22.6 / 18.6	148.8 / 147.7	1198.2 / 1134.2	1306.9 / 928.8	2676.5 / 2229.3
NE	8.8 / 7.3	53.0 / 43.7	389.2 / 391.9	602.8 / 528.1	1053.8 / 971.0
ENE	7.4 / 6.1	35.3 / 29.1	71.7 / 59.1	47.5 / 39.2	161.9 / 133.6
E	24.4 / 20.2	59.1 / 48.7	48.3 / 39.9	28.1 / 23.2	160.0 / 132.0
ESE	13.4 / 14.2	19.8 / 16.3	12.7 / 10.5	27.9 / 23.0	73.8 / 64.0
SE	1.1 / .9	1.2 / 1.0	—	33.8 / 27.9	36.1 / 29.8
SSE	—	.2 / .2	.7 / .6	22.0 / 17.2	22.9 / 18.0
S	13.5 / 10.6	39.1 / 30.7	13.6 / 11.2	4.8 / 3.8	71.0 / 56.3
SSW	23.9 / 18.7	14.7 / 11.8	27.2 / 21.5	15.6 / 13.1	81.2 / 65.1
SW	25.7 / 20.1	5.9 / 5.2	15.4 / 12.9	8.7 / 7.5	55.7 / 45.7
WSW	20.1 / 16.0	3.3 / 2.9	10.0 / 8.9	10.5 / 9.3	43.9 / 37.1
W	16.9 / 13.5	1.3 / 1.2	49.9 / 43.8	259.4 / 229.6	237.1 / 288.1
WNW	21.4 / 17.4	23.7 / 22.0	46.4 / 41.3	18.1 / 16.5	109.6 / 97.3
NW	67.4 / 53.2	30.1 / 26.2	16.4 / 15.3	31.5 / 29.4	145.4 / 124.2
NNW	62.4 / 46.9	118.7 / 63.6	49.7 / 50.4	41.3 / 38.8	272.2 / 199.7
TOTAL	426.3 / 340.6	665.6 / 541.2	2066.7 / 1952.0	2779.2 / 2077.1	5937.8 / 4910.9

RADIUS IN MILES	YEAR	0-10	0-20	0-30	0-40	0-50
ACCUMULATED POPULATION	2010	27.4	453.7	1119.3	3186.0	5965.2
	1980	22.2	362.9	904.0	2855.9	4933.0

2010 POPULATION / 1980 POPULATION — PROJECTED / EXISTING — POPULATION IN 000'S

10-50 MILES

SOURCE: 1980 CENSUS OF POPULATION & HOUSING FOR NEW JERSEY, DELAWARE, PENNSYLVANIA AND MARYLAND.

NEW JERSEY DEPT. OF LABOR, NEW JERSEY POPULATION PROJECTIONS 1980-2000, FEBRUARY 1982.

U.S. DEPT. OF COMMERCE, 1980 OBERS BEA REGIONAL PROJECTIONS, JULY 1981.

NEW CASTLE COUNTY PLANNING BOARD, POPULATION PROJECTIONS 1980-2000, MARCH 1982.

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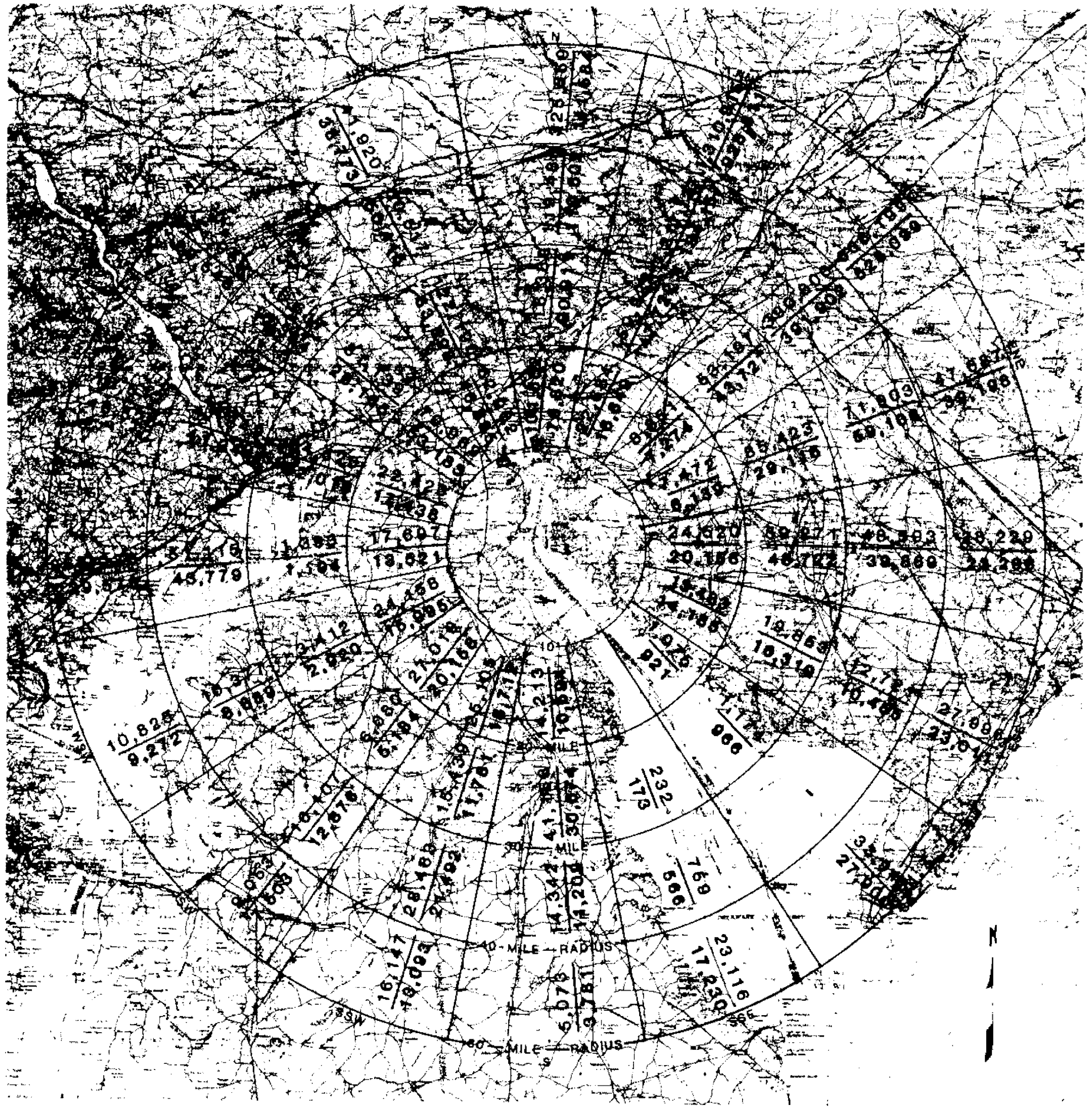
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

POPULATION DISTRIBUTION – YEAR 2010, WITHIN 10 TO 50 MILES

UPDATED FSAR

FIGURE 2.1-14



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FOLLOWING SECTIONAL
AERONAUTICAL CHART:
NEW YORK

5 0 5 20
MILES

5 0 5 20
KILOMETERS

POPULATION PROJECTIONS & POPULATION DISTRIBUTION

2020

MILES	10-20	20-30	30-40	40-50	TOTAL
N	101.6 / 76.8	116.4 / 90.9	119.5 / 110.5	325.6 / 141.6	663.0 / 419.8
NNE	22.7 / 18.6	150.9 / 147.7	1217.1 / 1134.2	1310.9 / 928.8	2701.7 / 2229.3
NE	8.9 / 7.3	53.2 / 43.7	390.8 / 391.9	608.2 / 528.1	1061.0 / 971.0
ENE	7.5 / 6.1	35.4 / 29.1	71.9 / 59.1	47.7 / 39.2	162.5 / 133.6
E	24.5 / 20.2	59.3 / 48.7	48.5 / 39.9	28.2 / 23.2	160.5 / 132.0
ESE	13.5 / 14.2	19.9 / 16.3	12.7 / 10.5	28.0 / 23.0	74.1 / 64.0
SE	1.1 / .9	1.2 / 1.0	—	33.9 / 27.9	36.2 / 29.8
SSE	—	.2 / .2	.8 / .6	23.1 / 17.2	24.1 / 18.0
S	14.2 / 10.6	41.2 / 30.7	14.3 / 11.2	5.1 / 3.8	74.8 / 56.3
SSW	25.1 / 18.7	15.4 / 11.8	28.5 / 21.5	16.1 / 13.1	85.2 / 65.1
SW	27.0 / 20.2	6.1 / 5.2	16.1 / 12.9	9.1 / 7.5	58.3 / 45.7
WSW	21.2 / 16.0	3.4 / 2.9	10.4 / 8.9	10.8 / 9.3	45.8 / 37.1
W	17.7 / 13.5	1.4 / 1.2	51.1 / 43.8	268.0 / 229.6	338.2 / 288.1
WNW	22.4 / 17.4	24.5 / 22.0	47.8 / 41.3	18.5 / 16.5	113.3 / 97.3
NW	70.9 / 53.2	31.1 / 26.2	16.6 / 15.3	32.0 / 29.4	150.6 / 124.2
NNW	62.9 / 46.9	122.2 / 63.6	50.5 / 50.4	41.9 / 38.8	277.5 / 199.7
TOTAL	441.1 / 340.6	681.8 / 541.2	2096.7 / 1952.0	2807.2 / 2077.1	6026.8 / 4910.8

RADIUS IN MILES	YEAR	0-10	0-20	0-30	0-40	0-50
ACCUMULATED POPULATION	2020	28.0	469.2	1151.0	3247.7	6054.8
	1980	22.2	362.8	904.0	2855.9	4933.0

2020 POPULATION / 1980 POPULATION: 2020 PROJECTED / 1980 EXISTING

POPULATION IN 000'S

10-50 MILES

SOURCE:

1980 CENSUS OF POPULATION & HOUSING FOR NEW JERSEY, DELAWARE, PENNSYLVANIA AND MARYLAND.

NEW JERSEY DEPT. OF LABOR, NEW JERSEY POPULATION PROJECTIONS 1980-2000, FEBRUARY 1982.

U.S. DEPT. OF COMMERCE, 1980 OBERS BEA REGIONAL PROJECTIONS, JULY 1981.

NEW CASTLE COUNTY PLANNING BOARD, POPULATION PROJECTIONS 1980-2000, MARCH 1982.

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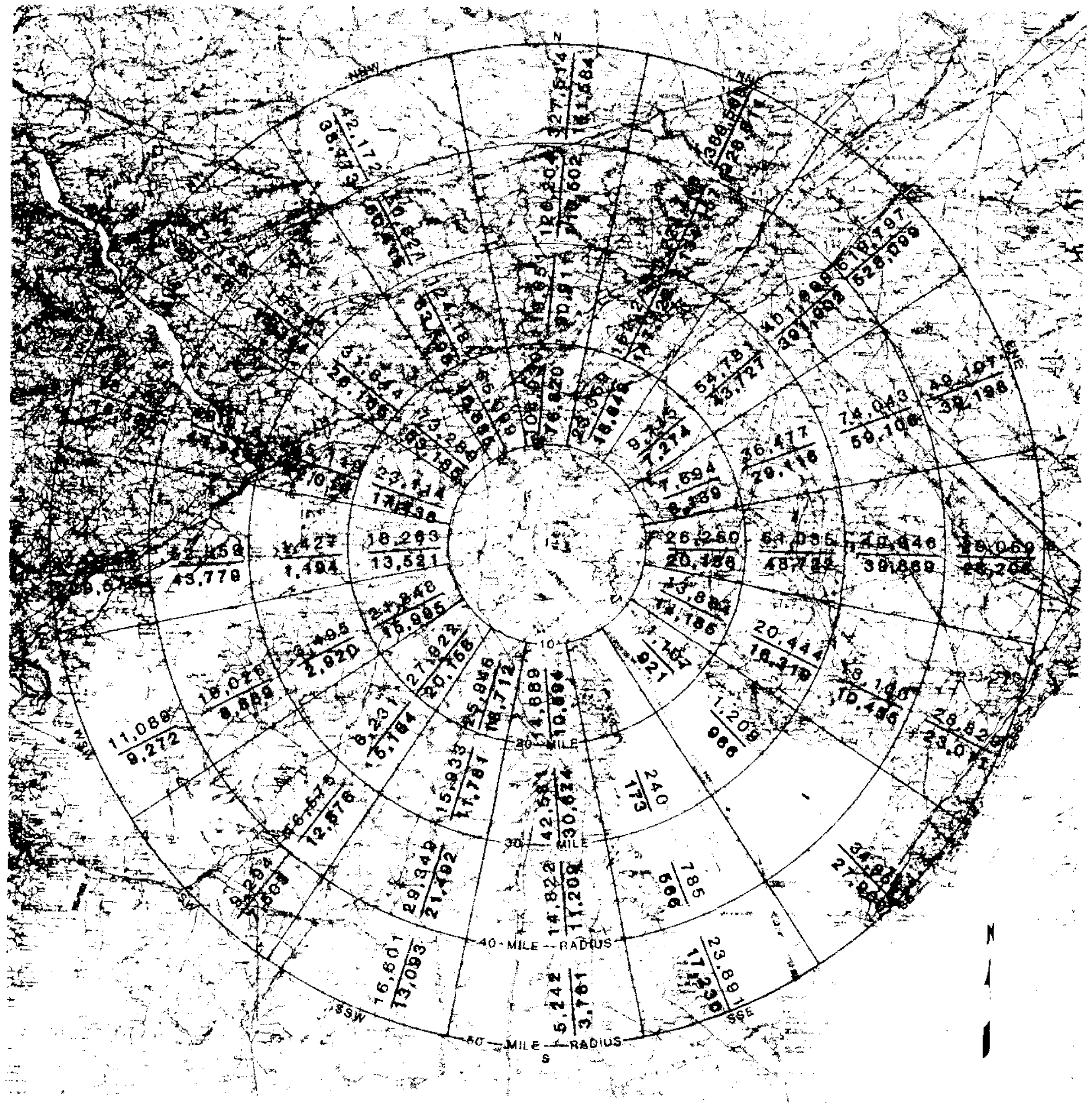
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

POPULATION DISTRIBUTION – YEAR 2020, WITHIN 10 TO 50 MILES

UPDATED FSAR

FIGURE 2.1-15



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5 0 5 20
MILES

5 0 5 20
KILOMETERS

POPULATION PROJECTIONS & POPULATION DISTRIBUTION

2030

MILES	10-20	20-30	30-40	40-50	TOTAL
N	104.9 / 76.8	119.7 / 90.9	120.2 / 110.5	327.5 / 141.6	672.3 / 419.8
NNE	23.4 / 18.6	152.2 / 147.7	1224.4 / 1134.2	1389.6 / 928.8	2789.6 / 2229.3
NE	9.1 / 7.3	54.8 / 43.7	402.0 / 391.9	619.8 / 528.1	1085.7 / 971.0
ENE	7.7 / 6.1	36.5 / 29.1	74.0 / 59.1	49.1 / 39.2	167.3 / 133.6
E	25.3 / 20.2	61.0 / 48.7	49.9 / 39.9	29.1 / 23.2	165.3 / 132.0
ESE	13.9 / 14.2	20.4 / 16.3	13.1 / 10.5	28.8 / 23.0	76.3 / 64.0
SE	1.1 / .9	1.2 / 1.0	—	35.0 / 27.9	37.3 / 29.8
SSE	—	.2 / .2	.8 / .6	23.9 / 17.2	24.9 / 18.0
S	14.7 / 10.6	42.5 / 30.7	14.8 / 11.2	5.7 / 3.8	77.3 / 56.3
SSW	25.9 / 18.7	15.9 / 11.8	29.3 / 21.5	16.6 / 13.1	87.8 / 65.1
SW	27.9 / 20.2	6.2 / 5.2	16.6 / 12.9	9.3 / 7.5	60.0 / 45.7
WSW	21.8 / 16.0	3.5 / 2.9	16.0 / 8.9	11.1 / 9.3	52.5 / 37.1
W	18.3 / 13.5	1.4 / 1.2	52.4 / 43.8	274.5 / 229.6	346.6 / 288.1
WNW	23.1 / 17.4	25.1 / 22.0	48.9 / 41.3	18.8 / 16.5	115.9 / 97.3
NW	73.3 / 53.2	31.8 / 26.2	16.7 / 15.3	37.1 / 29.4	115.9 / 124.2
NNW	65.0 / 46.9	124.7 / 63.6	50.8 / 50.4	42.2 / 38.8	282.2 / 199.7
TOTAL	455.4 / 340.6	696.8 / 541.2	2130.1 / 1952.0	2912.6 / 2077.1	6195.0 / 4910.8

RADIUS IN MILES	YEAR	0-10	0-20	0-30	0-40	0-50
ACCUMULATED POPULATION	2030	28.9	484.3	1181.2	3311.3	6223.9
	1980	22.2	362.8	904.0	2855.9	4933.0

2030 POPULATION / 1980 POPULATION — PROJECTED / EXISTING — POPULATION IN 000'S

10-50 MILES

SOURCE: 1980 CENSUS OF POPULATION & HOUSING FOR NEW JERSEY, DELAWARE, PENNSYLVANIA AND MARYLAND.

NEW JERSEY DEPT. OF LABOR, NEW JERSEY POPULATION PROJECTIONS 1980-2000, FEBRUARY 1982.

U.S. DEPT. OF COMMERCE, 1980 OBERS BEA REGIONAL PROJECTIONS, JULY 1981.

NEW CASTLE COUNTY PLANNING BOARD, POPULATION PROJECTIONS 1980-2000, MARCH 1982.

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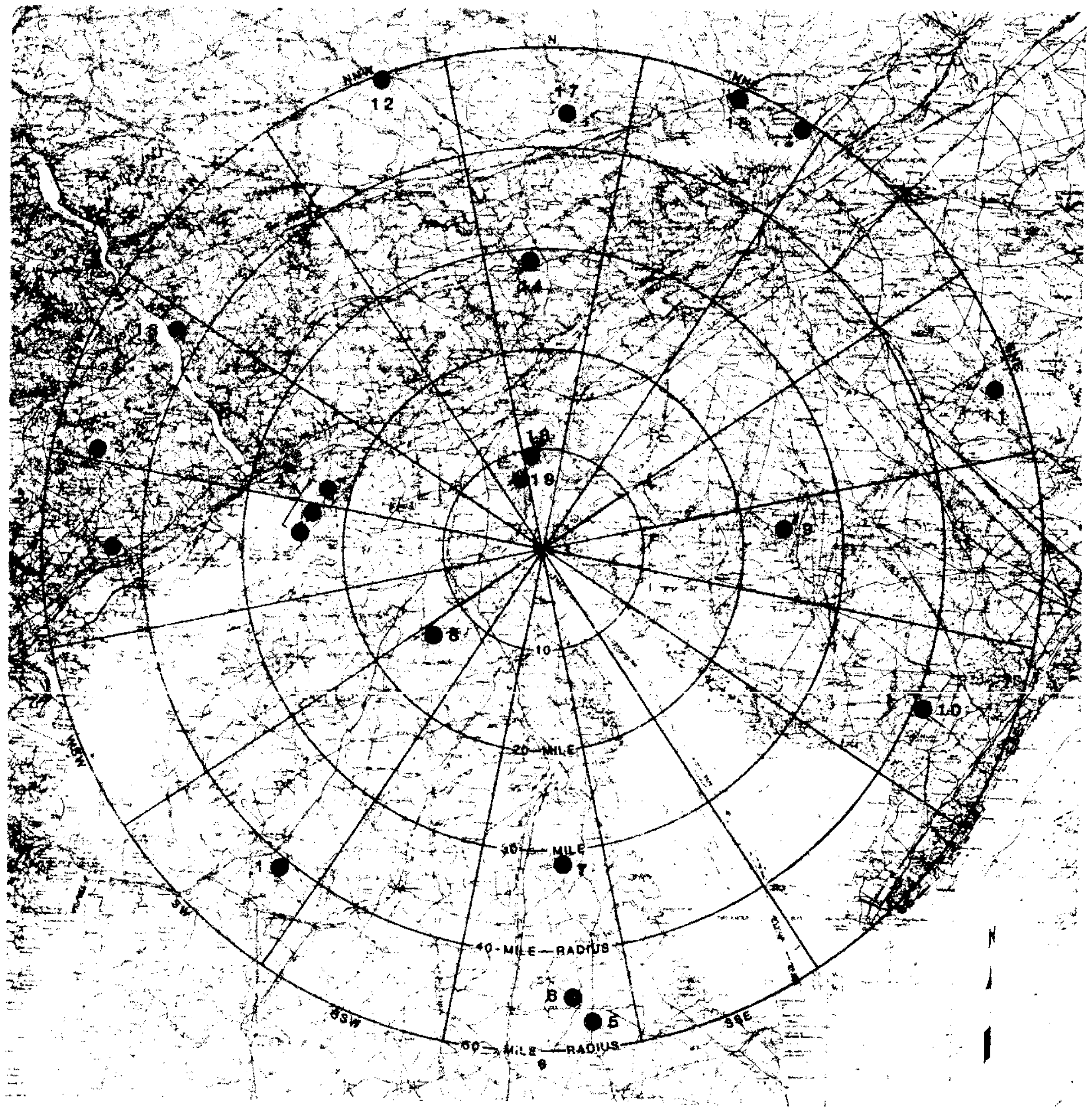
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

POPULATION DISTRIBUTION – YEAR 2030, WITHIN 10 TO 50 MILES

UPDATED FSAR

FIGURE 2.1-16



REFERENCE:
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5 0 5 20
MILES

5 0 5 20
KILOMETERS

STATE PARKS & FORESTS WITHIN 0-50 MILES 1981

		ANNUAL VISITATIONS
1.	TUCKAHOE STATE PARK, MD	50,149
2.	GUNPOWDER FALLS STATE PARK, MD	405,922
3.	ROCK STATE PARK, MD	218,608
4.	ELK NECK STATE PARK, MD	218,602
5.	REDDEN STATE FOREST, DE	4,000
6.	ELKENDALE STATE FOREST, DE	2,000
7.	KILLEN POND STATE PARK, DE	130,152
8.	BLACKBIRD STATE FOREST, DE	7,000
9.	PARVIA STATE PARK, DE	3,000
10.	BELLEPLAIN STATE FOREST, NJ	110,105
11.	WHARTON STATE FOREST, NJ	374,085
12.	FRENCH CREEK STATE PARK, PENN	361,121
13.	SUSQUCHANNOCK, PENN	32,383
14.	BRANDYWINE BATTLEFIELD STATE PARK, PENN	117,837
15.	FORT WASHINGTON, PENN	335,494
16.	INDEPENDANCE PARK, PENN	3,828,497
17.	VALLEY FORGE, PENN	12,181,740
18.	FORT MOTT STATE PARK, NJ	45,700
19.	FORT DELAWARE PARK, DE	12,200

SOURCE: DELAWARE STATE PARKS SERVICE, JUNE 1982

SOUTH JERSEY RESOURCE CONSERVATION & DEVELOPMENT AREA PLAN, U.S. DEPT. OF AGRICULTURE, SOIL CONSERVATION SERVICE, SOMERSET, NJ APRIL 1979

MARYLAND STATE PARK & RECREATION SERVICE, JUNE 1982

PENNSYLVANIA BUREAU OF STATE PARKS, JUNE 1982

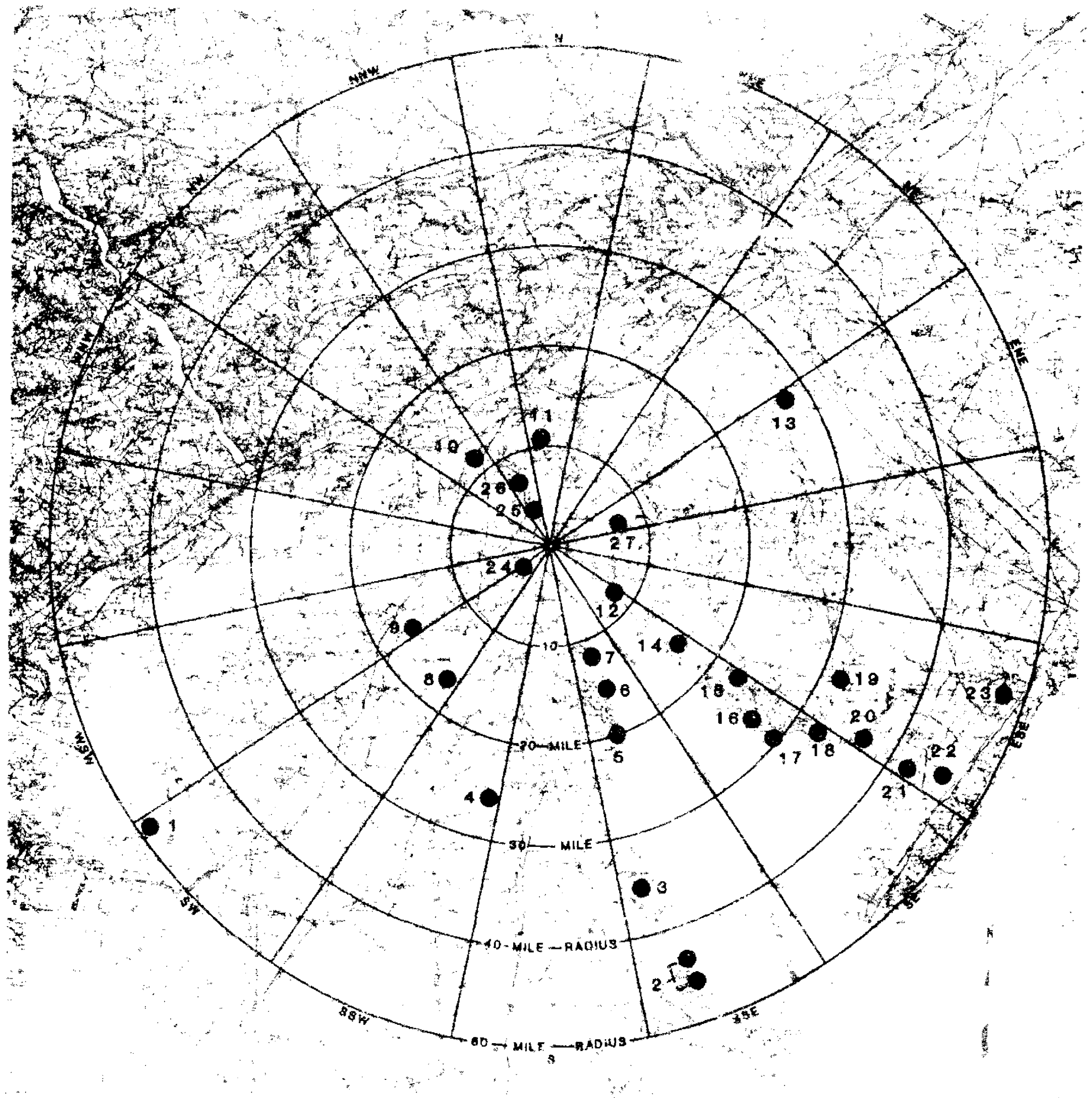
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

STATE PARKS & FORESTS
WITHIN 0 TO 50 MILES

UPDATED FSAR

FIGURE 2.1-17



REFERENCE
THIS MAP WAS PREPARED FROM PORTIONS OF THE FOLLOWING SECTIONAL AERONAUTICAL CHART
NEW YORK

5 0 5 20
MILES

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KILOMETERS

WILDLIFE MANAGEMENT AREAS WITHIN 0-50 MILES 1982

		ANNUAL VISITATION
1.	EASTERN NECK ISLAND NATIONAL WILDLIFE REFUGE, MD	45,400
2.	PRIME HOOK NATIONAL WILDLIFE REFUGE, DE	5,000
3.	MILFORD NECK WILDLIFE AREA, DE	2,000
4.	NORMAN G. WILDER WILDLIFE AREA, DE	1,500
5.	LITTLE CREEK WILDLIFE AREA, DE	2,500
6.	BOMBAY HOOK NATIONAL WILDLIFE REFUGE, DE	55,700
7.	WOODLAND BEACH WILDLIFE AREA, DE	8,000
8.	BLACKISTON WILDLIFE AREA, DE	2,500
9.	MILLINGTON WILDLIFE MANAGEMENT AREA, MD	4,000
10.	CANAL NATIONAL WILDLIFE REFUGE, DE	2,000
11.	KILLCOHOOK NATIONAL WILDLIFE REFUGE, DE	500
12.	MAD HORSE CREEK WILDLIFE MANAGEMENT AREA, NJ	1,000
13.	GLASSBORO FISH & WILDLIFE MANAGEMENT AREA, NJ	1,300
14.	DIX FISH & WILDLIFE MANAGEMENT AREA, NJ	700
15.	NANTUXENT FISH & WILDLIFE MANAGEMENT AREA, NJ	800
16.	FORTESCUE FISH & WILDLIFE MANAGEMENT AREA, NJ	300
17.	EGG ISLAND FISH & WILDLIFE MANAGEMENT AREA, NJ	1,200
18.	HEISLERVILLE FISH & WILDLIFE MANAGEMENT AREA, NJ	900
19.	EDWARD G. BEVON FISH & WILDLIFE MANAGEMENT AREA, NJ	500
20.	CORSON TRACT STATE WILDLIFE MANAGEMENT AREA, NJ	200
21.	DENNIS CREEK WILDLIFE MANAGEMENT AREA, NJ	700
22.	BEAVER SWAMP WILDLIFE MANAGEMENT AREA, NJ	500
23.	TUCKAHOE WILDLIFE MANAGEMENT AREA, NJ	1,200
24.	APPOQUINIMINK WILDLIFE AREA, DE	100
25.	REEDY ISLAND WILDLIFE REFUGE, DE	500
26.	AUGUSTINE CREEK WILDLIFE AREA, DE	500
27.	MASKELLS MILL POND WILDLIFE MANAGEMENT AREA, NJ	800

SOURCE: DELAWARE DIVISION OF PARKS & RECREATION, JUNE 1982
NEW JERSEY DIVISION OF PARKS & FORESTRY, JUNE 1982
DELAWARE DEPT. OF FISH & WILDLIFE, JUNE 1982
MARYLAND WILDLIFE ADMINISTRATION, JUNE 1982

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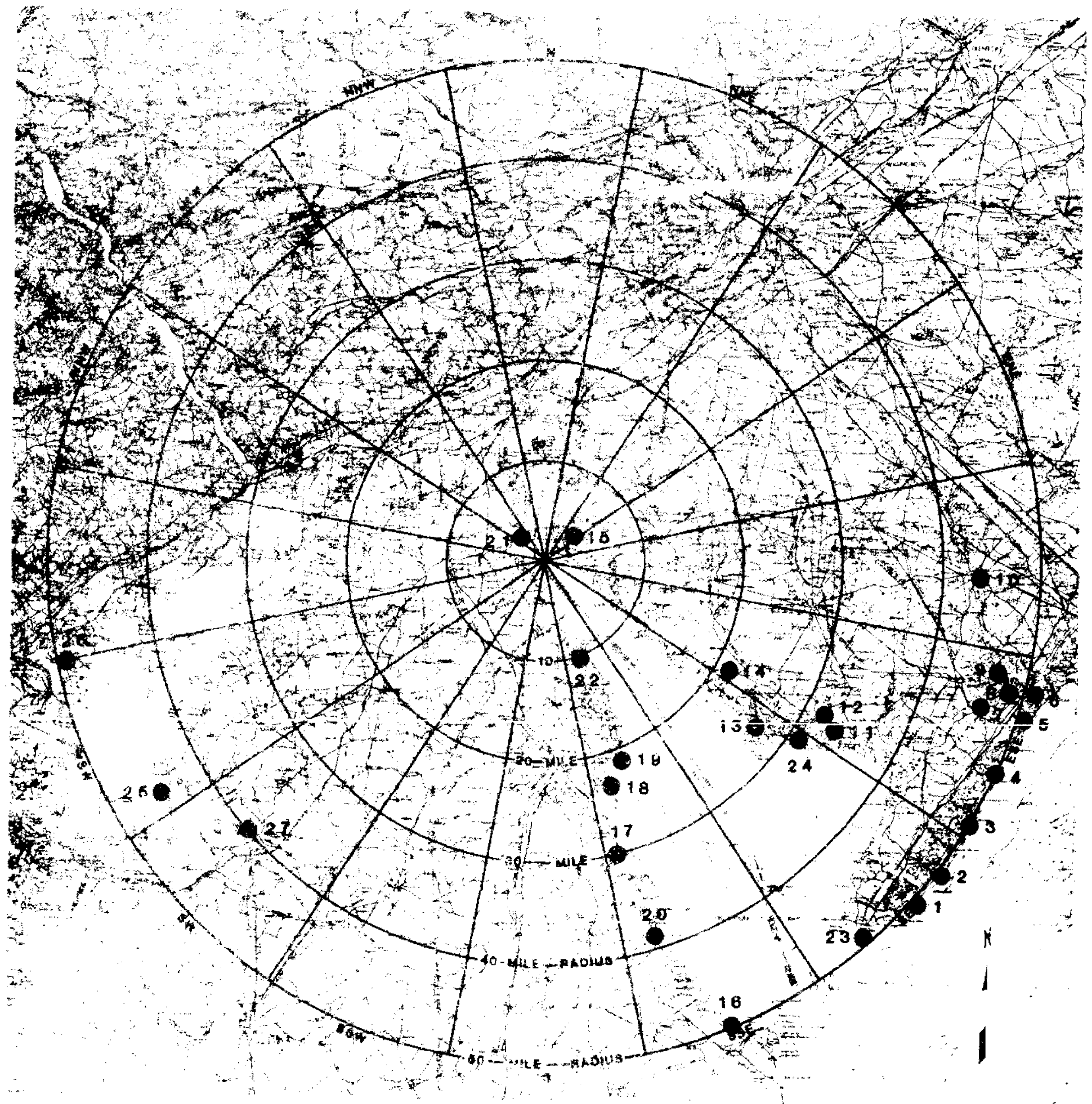
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

WILDLIFE MANAGEMENT AREAS – YEAR 1982, WITHIN 0 TO 50 MILES

UPDATED FSAR

FIGURE 2.1-18



PORTS OF LANDING FOR COMMERCIAL & RECREATIONAL SALTWATER FISHING WITHIN 0-50 MILES 1982

1. WILDWOOD, NJ
2. STONE HARBOR, NJ
3. AVALON, NJ
4. SEA ISLE CITY, NJ
5. MARMORA, NJ
6. OCEAN CITY, NJ
7. TUCKAHOE, NJ
8. SOMERS POINT, NJ
9. SCULLVILLE, NJ
10. MAYS LANDING, NJ
11. HEISLERVILLE, NJ
12. MATTS LANDING, NJ
13. FORTESCUE, NJ
14. NEWPORT, NJ
15. HANCOCK BRIDGE, NJ
16. LEWES, DE
17. BAUERS BEACH, DE
18. LITTLE CREEK, DE
19. PORT MAHON, DE
20. MISPILLION, DE
21. PORT PENN, DE
22. WOODLAND BEACH, DE
23. CAPE MAY, NJ
24. PORT NORRIS, NJ
25. ROCK HALL, MD
26. ESSEX, MD
27. CENTERVILLE, MD

SOURCES: TOWNSEND, R. GUIDE TO NEW JERSEY'S SALTWATER FISHING, DEP, DIVISION OF FISH, GAME & SHELLFISHERIES, 1974

DAREL CHRISTIAN, CHIEF FISHERIES STATISTIC INVESTIGATION NE REGION, NATIONAL MARINE FISHERIES SERVICE, JUNE 1982

MARYANN CARSON, BUDGET TECHNICIAN, ARMY CORPS OF ENGINEERS, PHILADELPHIA DISTRICT, JUNE 1982

REFERENCE
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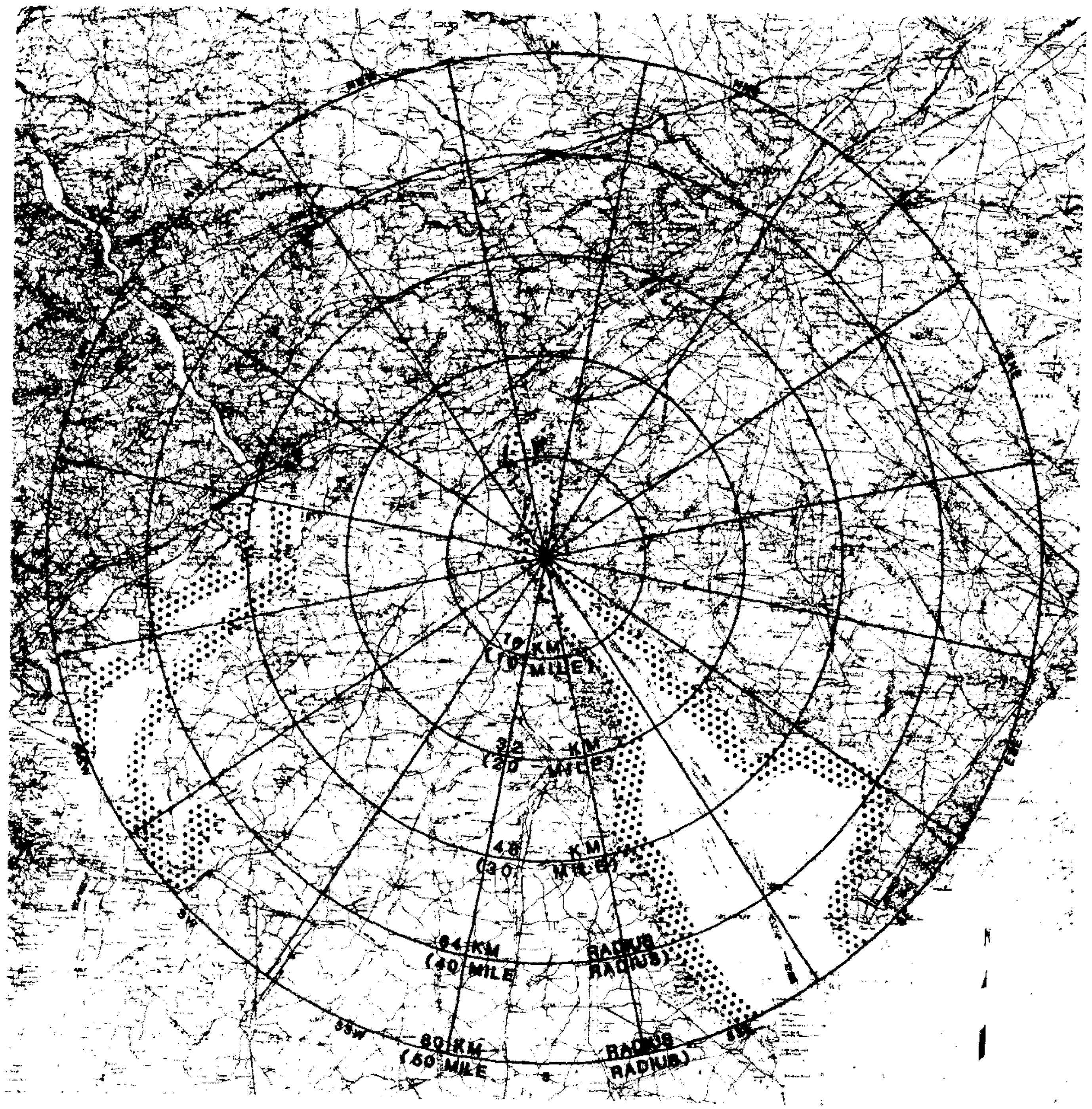
HOPE CREEK NUCLEAR GENERATING STATION

PORTS OF LANDING FOR COMMERCIAL AND RECREATIONAL SALTWATER FISHING WITHIN 0 TO 50 MILES

UPDATED FSAR

FIGURE 2.1-19

COMMERCIAL & RECREATIONAL FISHING & SHELLFISHING AREAS WITHIN 0-80 KILOMETERS 1982



RECREATIONAL AND COMMERCIAL FISHING AND SHELLFISHING AREAS

SOURCES: TOWNSEND, R, GUIDE TO NEW JERSEY'S SALTWATER FISHING, DED, DIVISION OF FISH, GAME & SHELLFISHERIES, 1974

DAREL CHRISTIAN, CHIEF FISHERIES STATISTIC INVESTIGATION NE REGION, NATIONAL MARINE FISHERIES SERVICE, JUNE 1982

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

COMMERCIAL AND REACREATIONAL FISHING AND SHELLFISHING AREAS WITHIN 0 TO 80 KM – YEAR 1982

UPDATED FSAR — FIGURE 2.1-20

LOW POPULATION ZONE
1982

TRANSPORTATION: HIGHWAY

DELAWARE	9
	420
	423
	424
NEW JERSEY	ALLOWAY CREEK NECK ROAD

RIVER

DELAWARE RIVER

WILDLIFE AREAS

1. AUGUSTINE CREEK WILDLIFE AREA
2. REEDY ISLAND WILDLIFE REFUGE
3. APPOQUINIMINK WILDLIFE AREA
4. MAD HORSE CREEK WILDLIFE MANAGEMENT AREA

BEACH AREAS

5. AUGUSTINE BEACH
6. BAY VIEW BEACH

SOURCE: NEW CASTLE COUNTY PLANNING BOARD, THE RED LION PLANNING DISTRICT PLAN, 1995, SEPTEMBER 1973

NEW CASTLE COUNTY PLANNING BOARD, THE MIDDLETOWN-ODESSA-TOWNSEND PLANNING DISTRICT PLAN, 1995, SEPTEMBER 1973

R. CHARTAWICH, DEPT. OF PLANNING, NEW CASTLE COUNTY PLANNING BOARD, MAY 1982

C. WARREN, DEPT. OF LAND USE & POPULATION, SALEM COUNTY PLANNING BOARD, APRIL 1982

U.S. DEPT. OF AGRICULTURE, SOUTH JERSEY RESOURCE CONSERVATION & DEVELOPMENT AREA PLAN, APRIL 1979

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KILOMETERS

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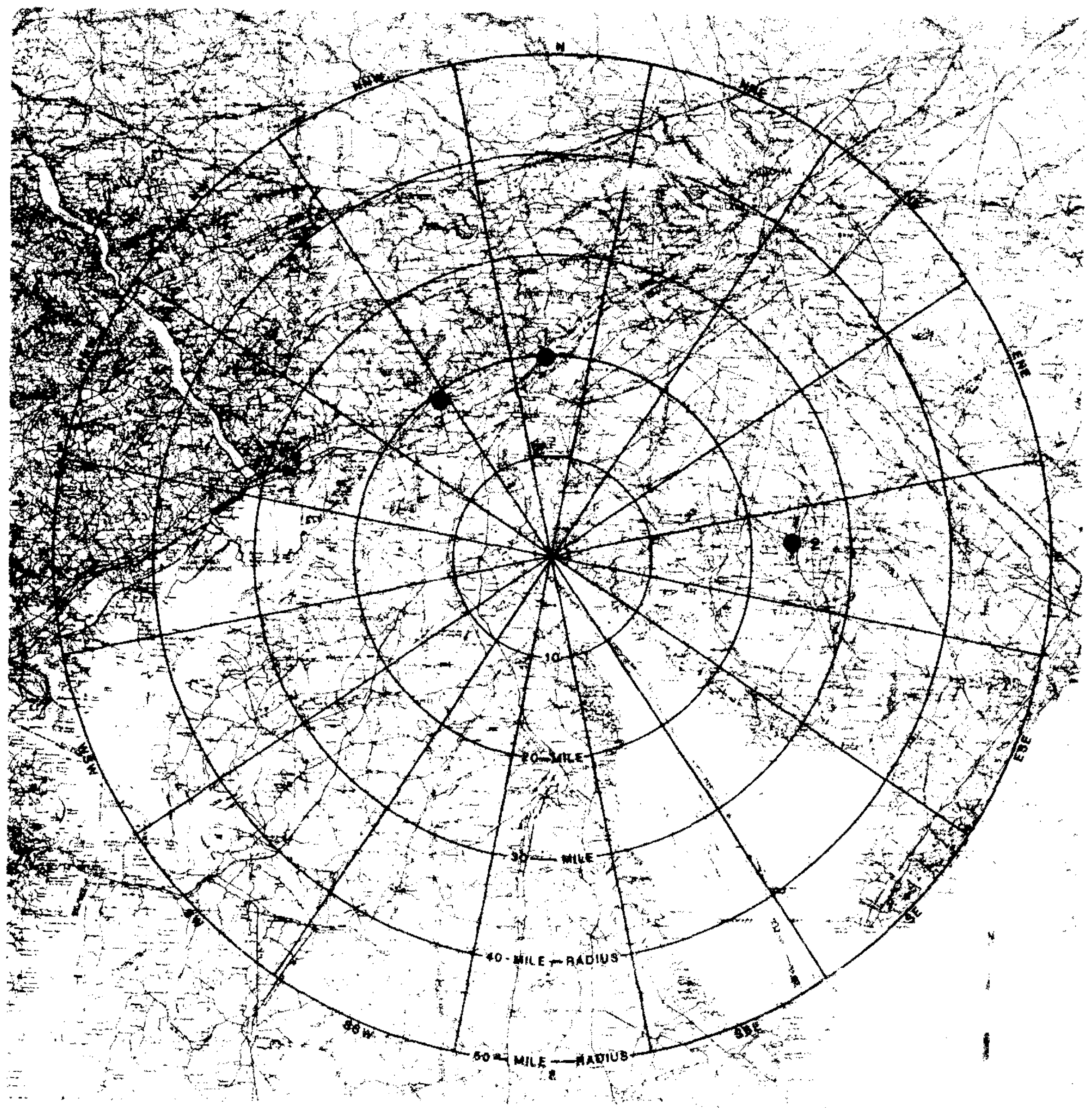
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOW POPULATION ZONE – YEAR 1982

UPDATED FSAR

FIGURE 2.1-21



POPULATION CENTERS WITHIN 0-30 MILES OF THE ARTIFICIAL ISLAND SITE 1980

● POPULATION CENTERS	POPULATION	DISTANCE/DIRECTION
1. WILMINGTON, DE	70,195	18.3 miles N
2. VINELAND, NJ	53,753	25.1 miles E
3. NEWARK, DE	25,247	17.8 miles NW

SOURCE: 1980 CENSUS POPULATION & HOUSING CHARACTERISTICS

REFERENCE:
THIS MAP WAS PREPARED FROM PORTIONS OF THE FOLLOWING SECTIONAL AERONAUTICAL CHART NEW YORK

5 0 5 20
MILES

5 0 5 20
KILOMETERS

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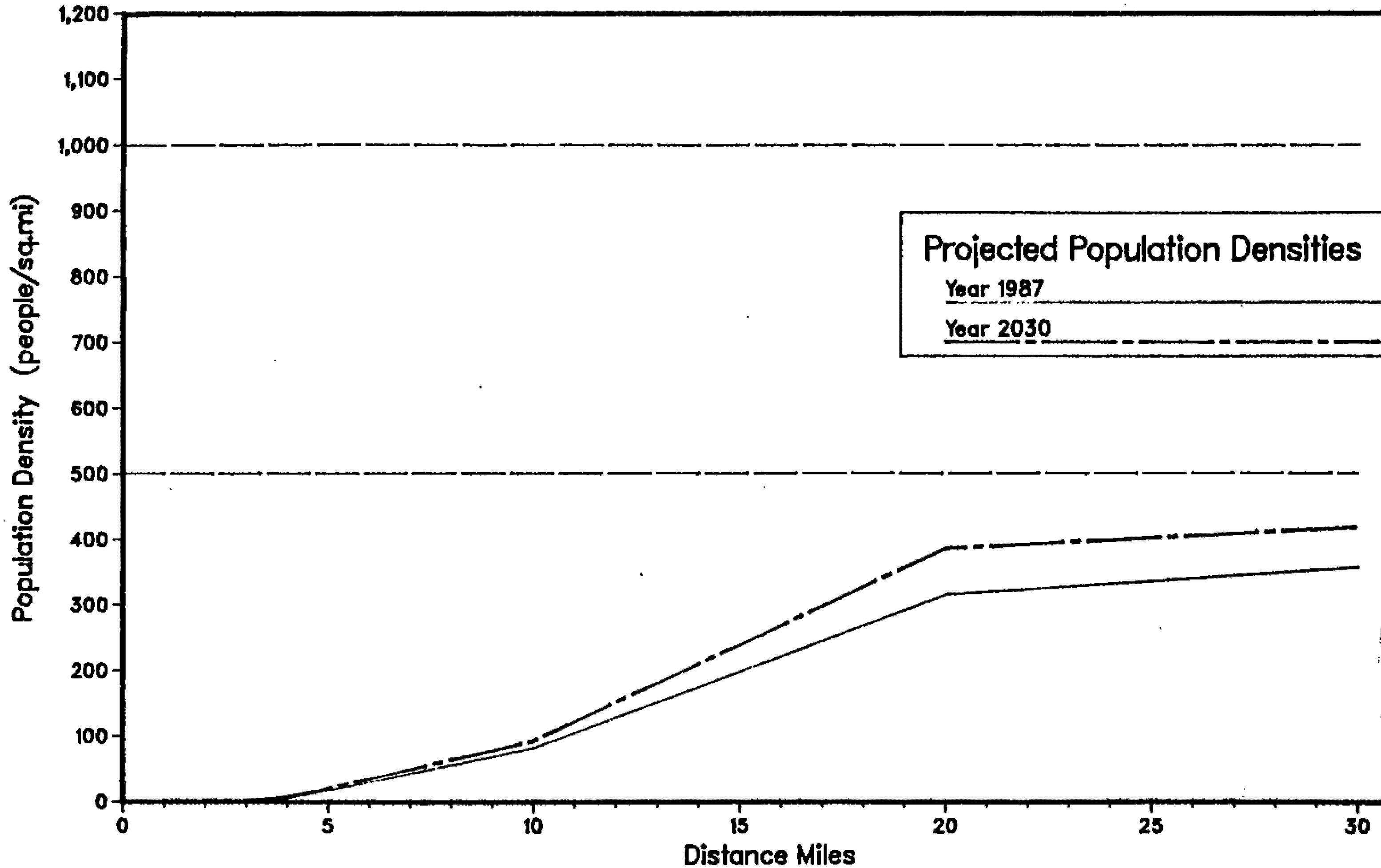
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

POPULATION CENTERS WITHIN 0 TO 30 MILES OF ARTIFICIAL ISLAND SITE – YEAR 1980

UPDATED FSAR

FIGURE 2.1-22



PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

REVISION 0
APRIL 11, 1988

UPDATED FSAR

POPULATION DENSITY

FIGURE 2.1-23

2.2 NEARBY INDUSTRIAL, TRANSPORTATION, AND MILITARY FACILITIES

All activities and facilities within 5 miles of the Hope Creek Generating Station (HCGS) site were considered.

2.2.1 Location and Routes

No significant manufacturing and chemical plants, oil refineries, storage facilities, transportation routes other than the Delaware River, and gas pipelines are located within 5 miles of the HCGS site, as found in Reference 2.2-1.

2.2.2 Descriptions

2.2.2.1 Description of Facilities

No manufacturing, industrial chemical plants, or storage facilities are located within 5 miles of the site. Nor are any military facilities located within 5 miles of the site.

2.2.2.2 Description of Products and Materials

No significant amounts of hazardous or toxic products and materials are regularly stored, manufactured, used, or transported within 5 miles of the Hope Creek Generating Station except as noted in either Tables 2.2-4, 2.2-5, or 2.2-6.

2.2.2.3 Pipelines

No pipelines are located within 5 miles of the HCGS site.

2.2.2.4 Waterways

The intake structure for the HCGS site is located on the east bank of the Delaware River on Artificial Island approximately 1 mile east of the Intercoastal Waterway. The waterway has a width of 800 feet in the location of the HCGS site.

The predominant types of river traffic are barges and freighters with a maximum draft range of 31 to 41 feet.

The Delaware River hydrographic chart indicates an anchorage zone northwest of Artificial Island. According to the U.S. Coast Guard's Safety Division, this is the only area within the vicinity of the HCGS site used for the anchorage of vessels carrying explosives. It has not been used in recent years.

2.2.2.5 <u>Airports</u>

2.2.2.5.1 Private Airports

There are three privately owned airports within approximately 10 miles of the site, as shown on Figure 2.2-1.

The Evergreen Airport is located approximately 5 miles west-northwest of the site. It has a 1400 foot grass runway, which is open only for emergency landings by small fixed-wing civil aircraft.

Salem Airport is located approximately 8 miles northeast of the site and has a 2200 foot grass runway. The airport owner operates a small agricultural spraying and dusting business. He also provides tie-down facilities for several privately owned and operated small fixed wing aircraft, as noted in Reference 2.2-3. A visual inspection revealed that neither the based aircraft nor the runway were in frequent use.

Summit Airport is located approximately 10 miles west-northwest of the site. It has a 4500-foot hard surfaced runway, 17/35 oriented 350°-170°6 magnetic, with lighting for nighttime operations and a perpendicular 3500-foot grass runway. There is a UNICOM (radio) available for aeronautical advisories, as noted in Reference 2.2-4. There are approximately 70 aircraft based at Summit Airport. The operators of these aircraft, including a company called Charter Service, may conduct a combined total of 72 operations (36 flights) per day. This number also includes the average 16 flights per day reported by Summit Aviation, which conducts a flight training school on the airport, Reference 2.2-4.

The inflight local training area for Summit Airport is west of U.S. Route 13. This boundary is approximately 6 miles west of the site and is vigorously enforced, in that to conduct any inflight training further east could interfere with aircraft on an extended final approach to runway 01 at the Greater Wilmington Airport and create potential mid-air collisions, References 2.2-4 and 2.2-5. There are no current plans for a major expansion of Summit Airport. However, there are normal maintenance programs and Charter Service is planning to increase operations. It is not anticipated that this would increase the overall number of operations by more than an additional 10 flights per day, Reference 2.2-4. The total is estimated to be approximately 35,000 operations per year.

There are no plans for construction of new runways or sufficient expansion of operations at these three private airports to cause, or contribute to, any hazard at the HCGS. Therefore, these airports will not be discussed further.

PSE&G plans approximately 700 annual operations at the helicopter landing pad located onsite. These helicopter operations are discussed further in Section 3.5.1.6.

2.2.2.5.2 Commercial Airports

Greater Wilmington Airport is the nearest commercial airport, located approximately 14 miles north-northwest of the plant site, see Figure 2.2-1. There are three crossing runways: runway 09/27 is 7165 feet long, 150 feet wide, and is oriented 088° and 268° magnetic; runway 14/32 is 5004 feet long, 150 feet wide, and is oriented 141° and 321° magnetic; and the primary instrument approach runway is 01/19, is 7002 feet long, 200 feet wide, and is oriented 015° and 195° magnetic, Reference 2.2-6.

There were 205,000 total operations at Greater Wilmington Airport in 1977, Reference 2.2-7. Based on this number and Department of Transportation predictions through 1992, Reference 2.2-7, the Greater Wilmington Airport local and itinerant operations will be discussed further in Section 3.5.1.6, even though the total number of operations during 1978 through 1981 have declined, Reference 2.2-8.

2.2.2.5.3 Airways

The VOR(V) airways service the low level air navigation structure. The V airways are 8 nautical miles wide (4.6 statute miles either side of center line), and extend from the minimum enroute altitudes up to but not including 18,000 feet mean sea level (MSL), Reference 2.2-6.

There are two low level V airways with centerlines that pass within approximately 7 miles of the site (see Figure 2.2-2). Airway V123-312 passes within 1 mile to the northwest, over the northern portion of Artificial Island. Airway V29-157 passes approximately 2 miles west of the site.

Jet routes (J) service the high level air navigation structure. The J airways are 10 nautical miles wide (5.7 statute miles either side of centerline), and extend from 18,000 feet up through 45,000 feet MSL, Reference 2.2-6. The airspace structure above 45,000 feet contains no airways or predetermined routes.

There is one high level airway J150, which is directly above V123-312, and also passes within 1 mile of the plant, as shown on Figure 2.2-3. Because J150 is directly above V123-312, the number of aircraft operations on the two airways are combined.

The number of operations on airways V123-312/J150 and V29-157 are considered in the next section and is discussed further in Section 3.5.1.6.

2.2.2.5.4 Itinerant, Federal Aviation Administration-Controlled Overflights

The Philadelphia Terminal Radar Approach Control (TRACON) controls all traffic under instrument flight rules (IFR) below 9000 feet, Reference 2.2-9, in the Philadelphia area, which includes the Greater Wilmington Airport. The Philadelphia TRACON provided flight strips which covered four days (96 hours) of operations, three weekdays, July 18, 19, and 21, and one weekend day, August 2, 1982, Reference 2.2-10. The flight strips were sorted to determine the total number of flights using each of the airways V123-312 and V29-157 and/or radio navigational fix-stations or radials that would overfly the HCGS site during the course of an arrival or departure route to any airport in the Philadelphia area. The weekday flight strips were averaged and multiplied by 261, the weekend number averaged and multiplied by 104, and the totals added. This results in an estimated annual number of 3000 operations below 9000 feet altitude for V123-312, and 16,000 operations below 9000 feet for V29-157, see Table 2.2-2, columns one and three.

The transition from Washington Air Route Traffic Control Center (ARTCC) to the New York ARTCC takes place in the airspace near the plant and varies with the assigned altitude of the aircraft traffic. All traffic above 9000 feet on V123-312, J150 and V29-157 is controlled by either Washington or New York ARTCC. To determine an estimate of the annual aircraft activity on the airways in the vicinity of Hope Creek, flight strips were obtained from both ARTCCs. The flight strips obtained from New York ARTCC covered five weekdays of operation, September 2 and 3, 1982 and September 6, 7, and 8, 1982. The flight strips obtained from Washington ARTCC covered eight days of operation, August 30, through September 5, and September 7, 1982, which included the Labor Day weekend. For each day, both sets of flight strips were sorted and tabulated according to the applicable airway and aircraft classification (air carrier, commuter air carrier and on-demand air taxi, general aviation small fixed wing, and military). Since there was no variance between the weekday count and the weekend count, the total annual number of operations was determined by multiplying the average daily count for each set by 365 days/year and summing the total count from each set.

The number of operations on these airways, as tracked by the New York and Washington ARTCCs, are shown in Table 2.2-2, columns two and four, and discussed further in Section 3.5.1.6.

2.2.2.5.5 Military Routes and Traffic Patterns

There are four military low level, slow speed, low altitude training routes, 844, 845, 846 and 847, in the area, see Reference 2.2-6 and Figure 2.2-4. These routes pass approximately 7 miles, NE, E, and SE of the site. The routes are flown in C-130 (four-turboprop engine) type aircraft by crews from the U.S. Air National Guard, 166th Tactical Airlift Group (TAG) stationed at Greater Wilmington Airport, New Castle, Delaware. These routes are open for training by any military organization but are controlled by, and used primarily by, the 166th TAG. The minimum altitude is 500 feet above ground level (AGL) for day and 1400 feet AGL for night operations. The ceiling, or maximum altitude for the routes, is 1500 feet AGL. The routes are flown approximately 200 times a year as a portion of a particular mission to train for, and test, combat readiness, Reference 2.2-11. The routes, as well as the entries and departures, are planned to avoid flying within 5 nautical miles (5.7 statute miles) of the site, References 2.2-6 and 2.2-11. U.S. Air Force Regulation 60-16 also restricts flight over any nuclear plant to 2000 feet vertically and 3 nautical miles horizontally.

The combination of the number of operations (200 per year), the route distance from the site (7 miles), the altitudes at which the aircraft are flown on these routes (500 to 1500 feet), and the highly reliable four-turboprop engine type aircraft, makes it extremely unlikely that this military operation poses any potential hazard to HCGS. Therefore, the four military low level, slow speed, low-altitude training routes will not be considered further.

The 166th TAG practices approximately 200 local airborne radar approaches (ARA) to runway 01 at Greater Wilmington Airport. This approach, controlled by the FAA and conducted only in VFR weather conditions, passes within one mile of the HCGS site, Reference 2.2-11. The 200 total number of operations is shown in Table 2.2-2, column five, and is discussed further in Section 3.5.1.6.

2.2.2.5.6 HCGS Site Helicopter Pad/Operations

Public Service Electric and Gas Company plans to fly a company owned helicopter to the HCGS site helipad approximately seven times a week, Reference 2.2-12. These flights will be controlled by the FAA, and approaches will be under visual flight rules (VFR) conditions. Helicopter VFR minimums are lower than fixed wing VFR minimum criteria. This average number of 700 annual operations (7x2x50 weeks) is shown in Table 2.2-2, column five, and is discussed further in Section 3.5.1.6.

2.2.2.5.7 Local VFR Over Flights

A radar survey was conducted from the Philadelphia Approach Control to determine the frequency of VFR flights within 5 miles of the HCGS site. The radar scope upper and lower altitude limits were set at 10,000 feet and 300 feet. Ten thousand feet was the upper limit, because these type aircraft normally do not have pressurized cabins and/or oxygen that would enable them to be flown at higher altitudes. The lower limit was set at 300 feet because the permanent ground returns at a lower setting would clutter the scope and preclude a count, and because even the most novice private aviator does not fly at altitudes below 500 feet except to perform specific proficiency training maneuvers.

The radar survey was conducted over a 3-day period. The weather was VFR-clear with good visibility on Thursday, August 5, 1982. From 4:00 to 7:00 p.m., there were no VFR operations observed within 5 miles of the site. On Friday, August 6, 1982, the weather was VFR, but there were some clouds at 2000 feet and the visibility was estimated at 5 miles. There was one VFR aircraft observed approximately 1 to 2 miles from the site between the hours of 9:00 a.m. and 7:30 p.m. On Saturday, August 7, 1982, the weather was VFR-clear with unlimited visibility. Between 8:00 a.m. and 2:00 p.m., 14 VFR aircraft were observed.

Even though HCGS is in a fairly remote location, as shown on Figure 2.2-1, the number of VFR flights observed within a 5-mile area of the HCGS site during the survey, multiplied by the respective weekdays and weekend days, produced an estimate of 1700 annual operations, shown in Table 2.2-3. These operations are given further consideration in Section 3.5.1.6.

2.2.2.6 Projections of Industrial Growth

Most of the area within 5 miles of the HCGS site lacks the infrastructure to support new industrial growth, as noted in Reference 2.2-1. Additionally, New Jersey and Delaware coastal protection legislation limits development in wetland areas. Much of the area within 5 miles of the site is wetland area within which industrial development is prohibited.

No significant changes from the past trends in the waterborne commerce and industry in the Delaware River are expected according to discussions with the U.S. Army Corps of Engineers, Philadelphia District, Reference 2.2-2.

2.2.3 Evaluation of Potential Accidents

This section provides an evaluation of potential accidents in nearby transportation and industrial facilities to determine what events need to be considered in the plant design. A description of design features to mitigate such events is also provided.

2.2.3.1 Determination of Design Basis Events

The information presented in Sections 2.2.1 and 2.2.2 shows that the

Hope Creek Generating Station (HCGS) Artificial Island site is located in a rural area consisting of marshes, meadowlands, and some farmland. There are no major manufacturing or chemical plants within 5 miles of the site. All such facilities are beyond 8 miles and would not interfere with the operation of HCGS. There are no military bases or missile sites within 10 miles of the site. There are no pipelines within 10 miles of the site. There are no major harbors, railway yards, or airports within 10 miles of the site. The only harbor facility of any significance is Getty Oil Company pipeline terminal in Delaware City, 9 miles north-northwest of the site. There are no railroads within 5 miles of the site.

[redacted] There are no petroleum wells, mines, or hard rock quarries within 10 miles of the site.

[redacted]

Reference 2.2-14 contains the details of these findings.

Based on the above information, it is concluded that the only events that could have an impact on the safety of HCGS relate to [redacted]

[redacted]

The estimates of probability of various accidents presented in this section were developed in accordance with the following steps:

1. The traffic history along the Delaware was established by using data from the U.S. Army Corps of Engineers, (Waterborne Commerce of the United States Data Base), the Philadelphia Maritime Exchange, the U.S. Coast Guard, and Poten and Partners, References 2.2-13, 2.2-14, and 2.2-15.

2. The probability of occurrence of a collision of sufficient severity to cause a major release of several types of cargo was estimated using the U.S. Coast Guard accident records, worldwide tanker experience, and a simplified statistic model, References 2.2-13 and 2.2-14.

3. For each major type of cargo, the distance to the plant within which the accident could pose a threat was estimated, References 2.2-13 and 2.2-14.

4. The probability of each type of cargo presenting a potential threat to plant safety was determined, References 2.2-13 and 2.2-14.

In evaluating the many estimates of the probability of potential impact, several simplifications and assumptions were made. However, the simplifications and assumptions were made in a conservative fashion so as not to underestimate the probability of occurrence of various events of concern.

The probability of potential impact to HCGS associated with the hazards from the SGS site are those relating to flammable liquid, corrosive liquid, flammable and toxic vapor dispersion, and liquid spills. Because of the land based nature of the chemical storage operation, only accidental spills on land are of concern, and ingestion of chemicals in the water intakes is not an issue. The spills on land were analyzed to determine if they presented any potential threat to the occupancy of the HCGS control room.

2.2.3.1.1 Explosions

[redacted]

[redacted]

2.2.3.1.3 Toxic Chemicals

Accidents involving the release of toxic chemicals from outside storage facilities and nearby mobile and stationary sources were considered.

Regulatory Guide 1.78, Position C.2, states that hazardous chemicals such as those indicated in Table C-1 of the guide must be included in the analyses if they are frequently shipped within a 5-mile radius of the plant. The guide also defines frequent shipments as 50 or more trips per year for barge traffic and specifies, in Position C.1, that chemicals stored or situated at distances greater than 5 miles from the facility need not be considered. Table 2.2-4 shows the chemicals stored at the adjacent SGS site. Table 2.2-5 shows estimates of hazardous chemical traffic in the vicinity of HCGS. An analysis of the SGS control room habitability, Reference 2.2-16, performed for a postulated hazardous chemical release occurring at the SGS site or within a 5-mile radius of the station demonstrated that the SGS control room personnel are adequately protected against the effects of accidental release of toxic gases. Due to the use of a sodium hypochloride biocide system at SGS, there is no onsite chlorine hazard. [redacted]
Calculations of the concentration of sulfuric acid and nitrogen that could reach the SGS control room air intakes show that they are well below the toxicity limits given in Table C-1 of Regulatory Guide 1.78. Calculations pertaining to the ammonia concentration that could reach the HCGS Control Room resulting from a postulated ammonium hydroxide release at SGS show that sufficient time exists for the control room personnel to take corrective actions to prevent exceeding the toxicity limit listed in Table C-1 of Regulatory Guide 1.78. Hydrazine stored at the SGS will not impact the HCGS control room as determined from calculations due to its low release rate and relatively small storage quantity. The

physical properties of sodium hydroxide preclude the formation of a plume and impacting the HCGS control room. It has a very low vapor pressure, and upon a release, mostly water will evaporate from the spill. Helium is stored in relatively small containers at the SGS. Upon a release, it will rapidly disperse and not pose a hazard to the HCGS control room. It is therefore clear that these chemicals will affect even less the HCGS main control room, since it is located further away from the source of any potential spill than the SGS control room.

There are several chemicals stored onsite at HCGS, as shown in Table 2.2-6. The effects of accidents involving these chemicals on the HCGS control room were studied, Reference 2.2-14. The study demonstrated that such accidents will not adversely affect the habitability of the control room, since even under the worst conditions it was not possible to generate unacceptable concentration of the chemicals of concern at the HCGS control room air intakes.

Table 2.2-5 shows that the mobile sources of hazardous chemicals shipped on the Delaware River are below the "frequent" criteria of Regulatory Guide 1.78, and are not required to be evaluated for impact on control room habitability due to the probability of such an accident.

Hazardous chemicals are also delivered to the HCGS and the SGS. Table 2.2-5 lists the shipments of hazardous chemicals to and near the Generating Stations. A review of the shipment deliveries were compared to the "frequent" shipment criteria as stated in Regulatory Guide 1.78. Aqueous sodium hydroxide, sodium hypochlorite, and ammonium bisulfite shipments are considered "frequent". As mentioned previously, a release of either sodium hydroxide or sodium hypochlorite will not impact the control room due to the physical properties of these chemicals. Ammonium bisulfite is also characterized as a chemical that will not readily evaporate and form a plume during a release due to its very low volatility. Therefore, a catastrophic failure of the tankers delivering these hazardous chemicals onsite will not impact control room habitability.

Ammonium hydroxide and sulfuric acid shipments delivered onsite also require an evaluation of their impact on control room habitability at the HCGS since their delivery schedule exceeds the criteria in Regulatory Guide 1.78. Calculations conclude that a release of ammonium hydroxide while onsite will not impact control room habitability at the HCGS. Also, calculations regarding the delivery of sulfuric acid to the SGS demonstrate that the control room will not be impacted during a catastrophic failure.

A more detailed analysis of the control room habitability is provided in Section 6.4.

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2.2.3.1.4 Fires

In addition to the flammable vapor clouds discussed earlier, events onsite and offsite that could lead to fires were evaluated. The offsite events relate to releases of flammable liquids from barge and ship traffic on the Delaware River. A pool fire as a result of a barge or ship accident would present a potential threat only to the water intakes, and only if the fire involved more than five million gallons of gasoline or oil. There have been very few tanker related spills of flammable material where cargo in excess of 5 million gallons was released, as mentioned in References 2.2-19 and 2.2-14. In the limited number of such spills that have occurred on a worldwide basis, there is no record of the spill having ignited. In attempting to determine the probability of ignition of a spill given a release of over 5 million gallons, discussions were held with several officers of the U.S. Coast Guard. The consensus was that the probability of ignition for such cases was under 5 percent as discussed in Reference 2.2-20. The probability of a fire, due to flammable material shipping, presenting a potential threat to HCGS was calculated taking into account the number of annual barge and tanker trips, the number of accidents per mile of trip, catchment distances, spills per accident, and the probability of ignition when a spill occurs. These calculations show that the risk of a large fire occurring at the water intake is in the order of 10^{-8} occurrences per year as mentioned in References 2.2-13, 2.2-14, and 2.2-20. Therefore, these fires do not have to be considered as DBEs.

Onsite events that could lead to fires are the release and ignition of the chemicals stored onsite and the release and ignition during their periodic resupply. Table 2.2-6 shows the chemicals stored at the HCGS site. An analysis of the potential for fires due to these chemicals and their periodic resupply method shows that these fires would be far too small in size and duration to affect the safety of the plant, and as such, do not have to be considered as DBEs, Reference 2.2-14.

2.2.3.1.5 Collisions With Intake Structure

Since the HCGS site is located near a navigable waterway, the probability and potential effects of collision impact of a ship or barge on the plant cooling water intake structure were considered. The size of the ships that could conceivably work their way to the river bank and ram into the water intake structure is limited by water depth and tidal conditions. Under normal tidal range, ships in excess of approximately 15,000 tons would likely ground on the shallow shoal areas outside the river channels before reaching the intake structure. Under extreme tidal conditions, however, such as the design high water level corresponding to hurricane conditions, the largest ships transiting the Delaware could reach the intake without grounding, as discussed in References 2.2-14 and 2.2-21.

The kinetic energy levels associated with the postulated rammings have been determined to be of the same order of magnitude as those from major collisions, as discussed in Reference 2.2-22. From ship collision studies, however, it can be argued that the expected structural damage from ship rammings of the intake structure will be mostly damage to the ship structure, and furthermore, that the damage will not be extensive enough to block the intake with structural rubble, as discussed in Reference 2.2-21. A further qualitative comparison of the seismic design input and the inertial loadings of the intake structure and its components caused by rammings indicate that the intake structure will suffer only local damage from the ramming accident. Its integrity would be maintained and equipment located inside would remain operable.

Our analysis in References 2.2-14 and 2.2-22 concluded that blockage of the intake structure opening by a runaway ship or barge is not possible. Under the most extreme low water conditions assumed in the design of the facility, consideration of the main intake area showed that the blockage to cause cavitating flow (97 percent of the area in the extreme low water condition) could not be accomplished by a conventional vessel with hull curvature, nor by any barge currently transiting the Delaware River near Artificial Island site, Reference 2.2-14. The most recent river bathymetry and National Oceanic and Atmospheric Administration charts were reviewed and it was determined that a smaller 2,000-ton displacement ship with a draft of 12 ft would be grounded 400 ft before it reached the intake structure. It was reconfirmed that the major damage would occur to the impacting vessel rather than to the intake structure. On the basis of past historical collision data on the Delaware River, the probability of a ship or barge impacting the intake structure during normal water levels is less than 10^{-7} per year.

Therefore, collisions with the intake structure do not have to be considered as DBEs for the HCGS site.

2.2.3.1.6 Liquid Spills

The accidental release of oil or liquids that may be corrosive, cryogenic, or coagulant was considered to determine if the potential exists for such liquids to be drawn into the plant intake structure and circulating water systems, or to affect the plant's safe operation.

Petroleum, oil products and cryogens floating on the Delaware River surface could approach the intake structure due to a spill upstream. The water intake itself occurs several feet under water, so these materials are excluded from entry into the service water supply line, even if the materials get past the intake surface skimmers. The most severe possible condition occurs at the design low water level condition, with water surface at plant elevation 76 feet. At this level, the service water pump intake is still submerged by 4 feet. In addition, Hope Creek Technical Specifications require a plant shutdown when river water level reaches 80 feet PSE&G datum. Thus, floating liquid spills do not have to be considered as DBEs for the HCGS site.

There are no known coagulants shipped on the Delaware River, and the only corrosive of potential concern is sulfuric acid. As discussed in Section 2.2.3.1.3 and in Reference 2.2-14, the probability of ingesting unacceptable concentrations of sulfuric acid is estimated to be about 10^{-9} occurrences per year. As a result, these types of liquid spills do not have to be considered as design basis events for the HCGS site.

2.2.3.2 Effects of Design Basis Events

From the foregoing discussion, it can be seen that no events arising from nearby industrial activities were identified as DBEs. The plant's safety-related components are designed to withstand the effects of potential accidents without endangering the health and safety of the public.

2.2.4 References

2.2-1 Personal communication with:

R. Chartawich, New Castle County Department of Planning, May 20, 1982.
C. Warren, Salem County Department of Planning, May, 20, 1982.

2.2-2 Personal communication with:

D. Chistian, National Marine Fisheries Service, June 21, 1982.
M. Carson, U.S. Army Corps of Engineers, Philadelphia District, June 21, 1982.

2.2-3 Personal communication with:

Elmer Grieves, August 2, 1982. (A drive-by visual inspection was conducted on August 6, 1982).

2.2-4 Personal communication with:

Edmund, M. Conaway, Summit Aviation, and K.J. Toth, NUS Corporation, letter dated August 31, 1982.

2.2-5 Personal communication with:

Summit Flight School chief flight instructor, August 6, 1982.

2.2-6 The Defense Mapping Agency Aerospace Center, "United States Government Flight Information Publications (FLIP)," June 1982.

2.2-7 T.F. Henry, Terminal Area Forecast, Department of Transportation, Federal Aviation Administration, FAA-APD-80-10, February 1981.

2.2-8 C. Zimmerman, PSE&G, to R.P. Douglas, PSE&G, Aircraft Hazard Analysis Data, memorandum dated February 25, 1982.

2.2-9 Personal communication with:

M. Isaacson, FAA, New York ARTCC, and K.J. Toth, NUS Corporation, letter dated August 31, 1982.

2.2-10 FAA, Terminal Radar Approach Control (TRACON) Flight Strips, 7/18/82, 7/19/82, 7/21/82, and 8/2/82 from John Furlong, Philadelphia TRACON.

2.2-11 Personal communication with:

J. Lanahan Lt. Col. USAF Reserve, 166th Tactical Airlift Group, and K.J. Toth, NUS Corporation, letter dated September 3, 1982.

2.2-12 Personal communication with:

J. James, PSE&G pilot, and K.J. Toth, NUS Corporation, letter dated September 13, 1982.

2.2-13 Salem Generating Station FSAR, July 1982, p. 2.2-5.

2.2-14 "An Update on the Analysis of Potential Effects of Waterborne Traffic on the Safety of the Control Room and Water Intakes at Hope Creek Generating Station," A.D. Little, Inc., March 1983.

2.2-15 Poten and Partners, Inc, "Summary of Gas Ships Transiting the Delaware River," reports to PSE&G, February 1982.

2.2-16 "Control Room Habitability Analysis," NRC Docket No. 50-311, Salem Generating Station, July 1, 1980.

2.2-17 A.D. Little, Inc, "Analysis of Potential Effects of Waterborne Traffic on the Safety of the Control Room and Water Intakes at Hope Creek Generating Station," September 1974, p. 6.

2.2-18 Ibid, p. 7.

2.2-19 Ibid, pp. 3, 5.

2.2-20 Ibid, p. 5.

2.2-21 Ibid, p. 7, 8.

2.2-22 Ibid, p. 8.

2.2-23 U.S. Coast Guard, Vessel Chemical Traffic Report, "Hazardous Traffic Passing Salem and Hope Creek Stations," dated July 15, 1993.

TABLE 2.2-1

NUMBER OF OPERATIONS AT GREATER WILMINGTON AIRPORT

Year	Total Operations(1)	Total Operations(2)
	ACTUAL	ACTUAL
1976	171,000	186,586
1977	205,000	202,242
1978	184,000	190,767
1979	176,000	171,497
	FORECAST	
1980		177,002
1981	190,000	155,024
1982	196,000	
1983	203,000	
1984	209,000	
1985	216,000	
1986	223,000	
1987	230,000	
1988	237,000	
1989	245,000	
1990	253,000	
1991	261,000	
1992	270,000	

(1) From FAA Terminal Area Forecasts 1981-1982, Reference 2.2-7

(2) From Andrew Nonnenmacher, FAA Facility Chief at the Greater Wilmington Airport, Reference 2.2-8

TABLE 2.2-2

NUMBER OF OPERATIONS OVER THE HCGS SITE
ITINERANT FAA CONTROLLED OVER FLIGHTS

Category of Aircraft	IFR-CONTROLLED V123-312 Philadelphia TRACON[1] Below 9000 ft	V123-312 & J150 Washington & New York ARTCC[2] Above 9000 ft	V29-157 Philadelphia TRACON Below 9000 ft	V29-157 Washington & New York ARTCC Above 9000 ft	VFR-CONTROLLED Traffic Pattern Within 5 Miles
SFW-GA, single-engine	610	1551	4500	460	-
SFW-GA, multi-engine (<12,500 lb)	200	4650	1500	1400	-
Air taxi/commuter/on-demand (>12,500 lb)	1900	1820	7020	1820	-
Air carrier (>12,500 lb)	180	116,000	1600	1460	-
Military	100	1460	1000	700	200 (ARA)[3]
Helicopter	-	-	-	-	700 (PSE&G)
Totals	3000	125,481	15,620	5840	

(1) Terminal Radar Approach Control.
(2) Air Route Traffic Control Center.
(3) Airborne Radar Approaches.

TABLE 2.2-3

NUMBER OF OPERATIONS OVER THE HCGS SITE
VFR OBSERVED ON RADAR FROM PHILADELPHIA APPROACH CONTROL

Period	Within 1 Mile	1-2 Miles	2-3 Miles	3-5 Miles
(Thur) 8/5/82				
4:00-6:30 p.m.	0	0	0	0
(Fri) 8/6/82				
9:45-11:59 a.m.	0	0	0	0
12:00-4:59 p.m.	0	1	0	0
5:00-7:00 p.m.	0	0	0	0
	0	1	0	0
		x261		
		261		
(Sat) 8/7/82 VFR				
8:00-10:59 a.m.	0	0	1	0
11:00-12:59 p.m.	1	1	0	6
1:00-2:00 p.m.	1	2	1	1
	2	3	2	7
	x104	x104	x104	x104
	200	312	200	700
Totals	200	575	200	700

TABLE 2.2-4

SECURITY-RELATED INFORMATION
WITHHELD UNDER 10 CFR 2.390

TABLE 2.2-5

SECURITY-RELATED INFORMATION

WITHHELD UNDER 10 CFR 2.309

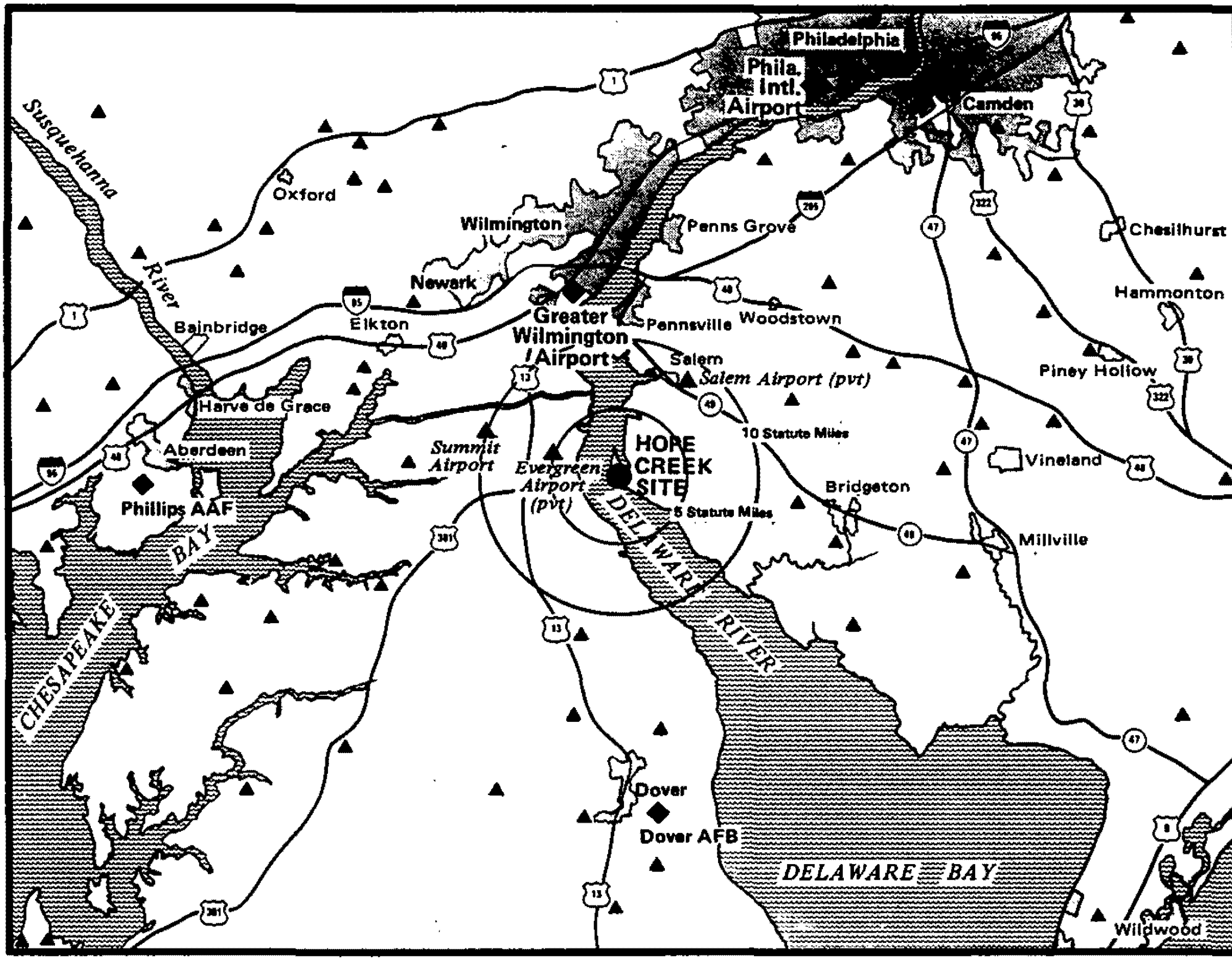
SECURITY-RELATED INFORMATION

WITHHELD UNDER 10 CFR 2.309

TABLE 2.2-6

SECURITY-RELATED INFORMATION

WITHHELD UNDER 10 CFR 2.309



Legend:

◆ Commercial/Military Controlled Airports

▲ Civil/Private Uncontrolled Airports

N

Kilometers
10 0 5 10

Statute Miles
10 0 5 10

Nautical Miles
10 0 5 10

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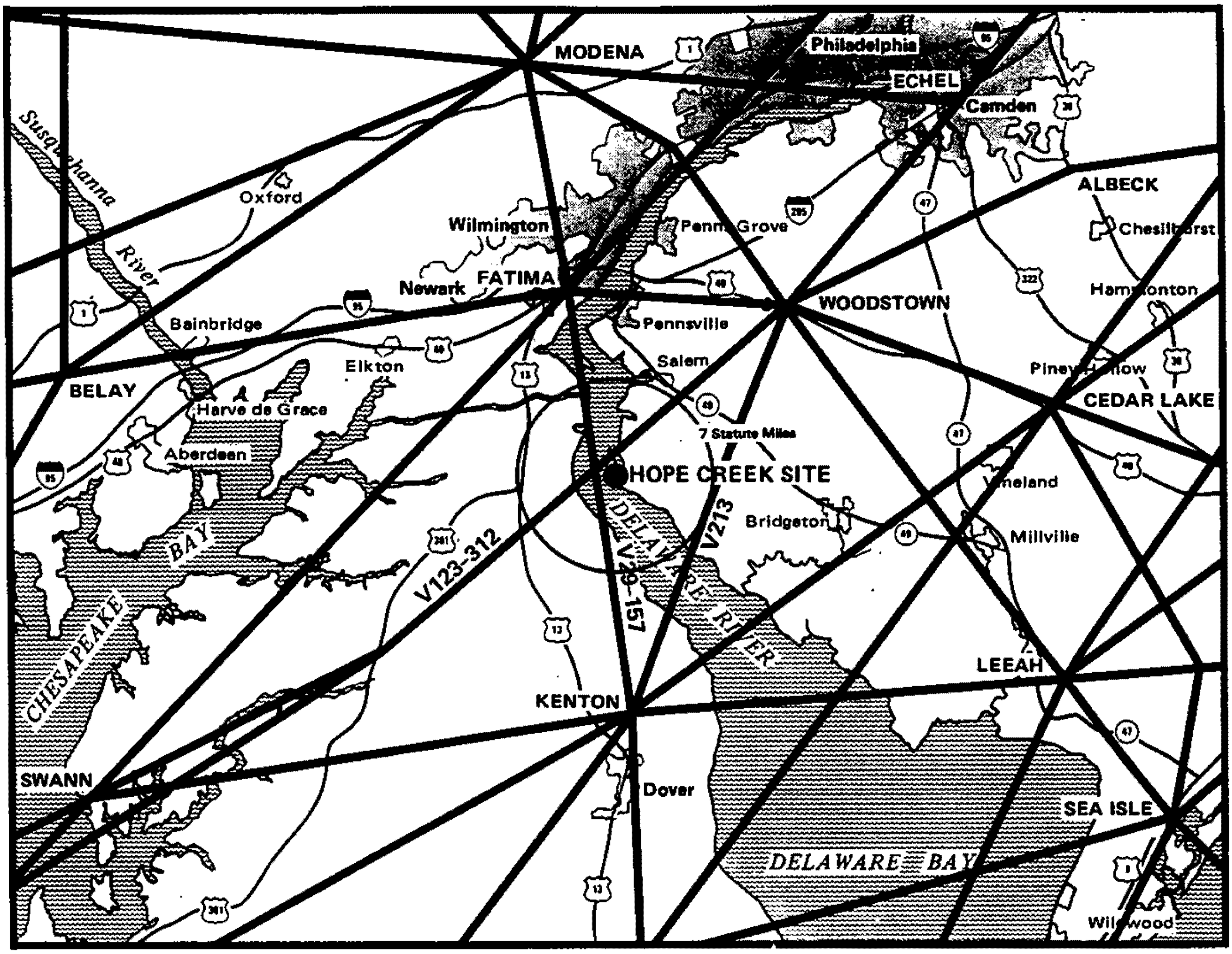
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

SITE MAP WITH AIRPORTS

UPDATED FSAR

FIGURE 2.2-1



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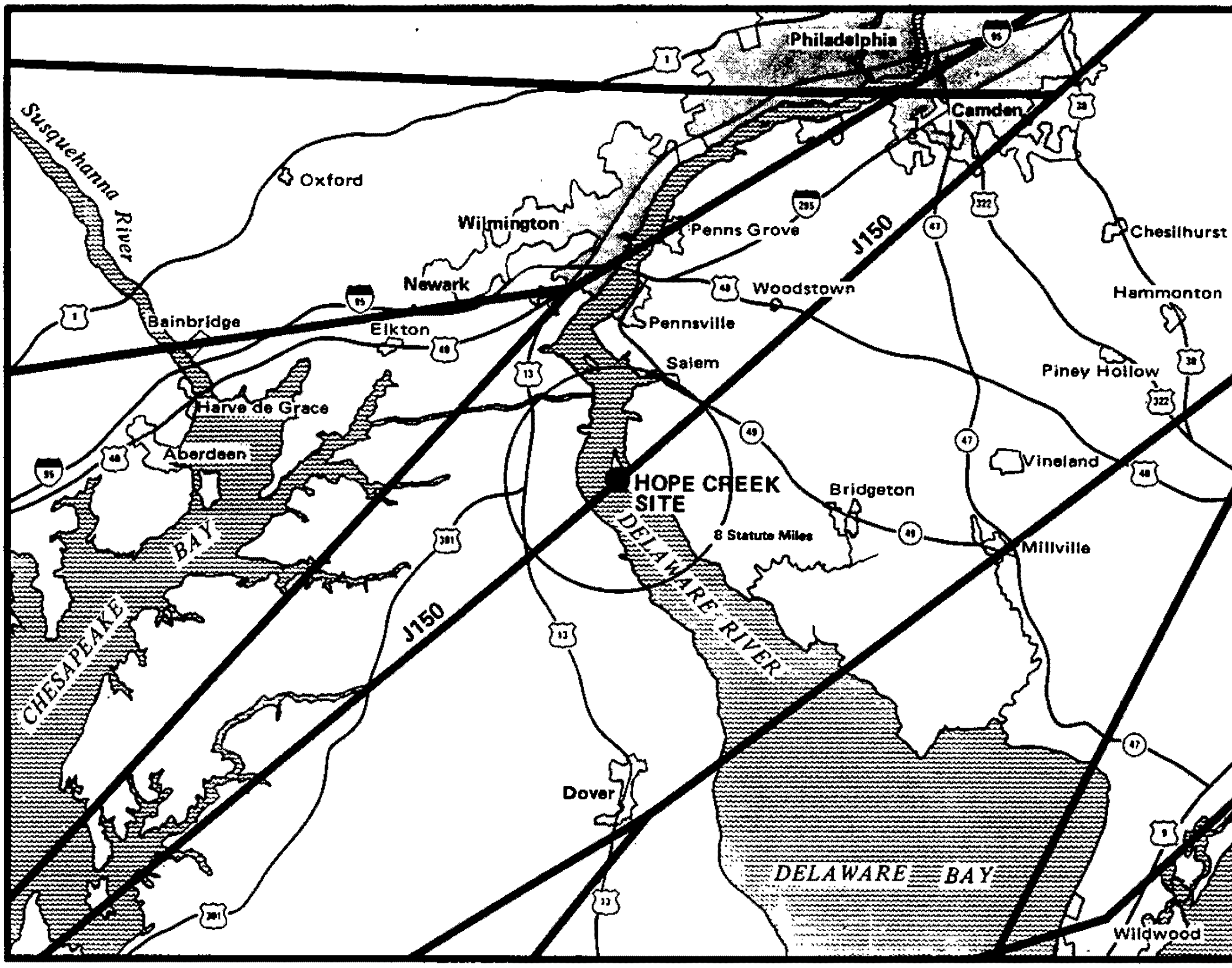
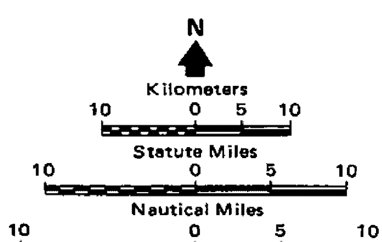
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOW LEVEL AIRWAYS
NEAR THE SITE

UPDATED FSAR

FIGURE 2.2-2





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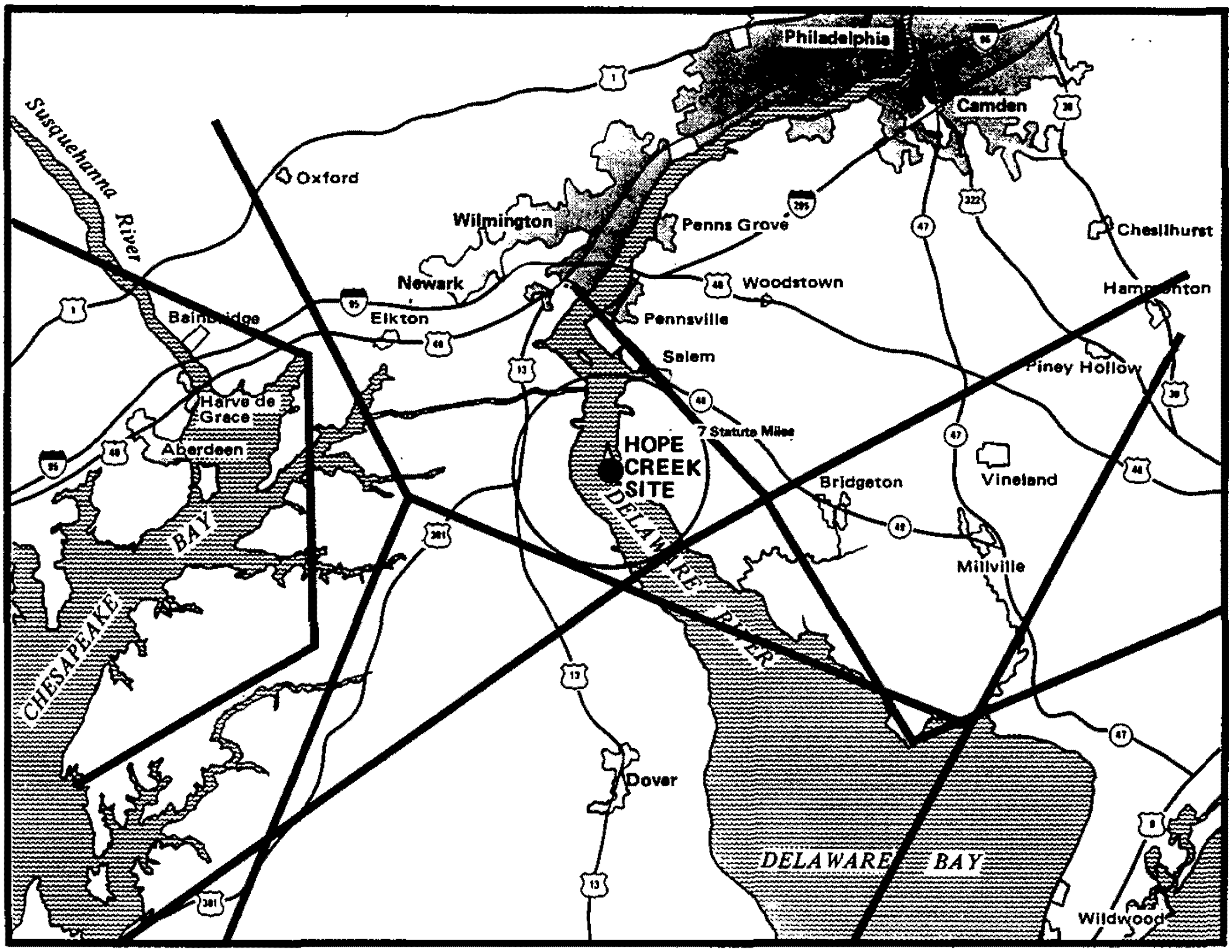
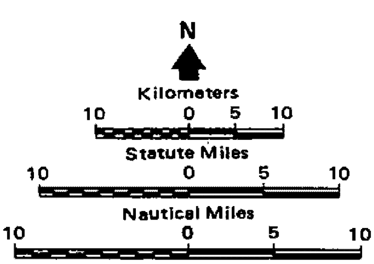
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

HIGH LEVEL AIRWAY
NEAR THE SITE

UPDATED FSAR

FIGURE 2.2-3





REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY HOPE CREEK NUCLEAR GENERATING STATION
MILITARY LOW LEVEL ROUTES NEAR THE SITE
UPDATED FSAR — FIGURE 2.2-4

2.3 METEOROLOGY

2.3.1 Regional Climatology

2.3.1.1 General Climate

Based on the Koeppen Climatic Classification System, the region intersects two climatic zones: humid continental and humid subtropical. Both zones have characteristics of warm summers and mild winters (Reference 2.3-1). Maximum summer average temperatures are near 80°F, and the coldest month is January with an average daily temperature of approximately 32°F. Examining a 30-year mean of precipitation amounts for Wilmington, Delaware National Weather Service (NWS) station shows that the most rainfall occurs in the summer months, followed by spring, fall, and winter (Reference 2.3-2).

Southern New Jersey is frequented by Polar Canadian air masses in the fall and winter and occasionally invaded by Arctic Canadian air late in winter. During the spring and summer, the dominant air mass is Maritime Tropical according to (Reference 2.3-1).

2.3.1.1.1 Precipitation

The frequency of precipitation events such as rain, snow, ice storms, thunderstorms, and hail are listed in Table 2.3-1.

The data in Table 2.3-1 were obtained from the Revised Uniform Summary of Surface Weather Observations, from Dover (Delaware) Air Force Base, during 1942 to 1965. The snowfall data presented in Tables 2.3-2 and 2.3-3 were obtained from Philadelphia International Airport and Trenton Airport, respectively.

2.3.1.1.2 Humidity, Winds

Humidity annually averages 70 percent, according to Reference 2.3-3. Prevailing winds on a monthly average during the winter (December through March) are from a northwest direction with a range of speeds from 9 to 13 mph. Average monthly winds for the spring and summer months (April through August) are from a southerly to southwesterly direction at speeds ranging from 7 to 10 mph. Winds during the fall are predominantly from the west-southwest veering to a west-northwest direction by December. The average wind speeds are higher during the winter months, as discussed in Reference 2.3-3.

2.3.1.2 Regional Meteorological Conditions for Design and Operating Bases

2.3.1.2.1 Seasonal and Annual Frequencies of Severe Weather

Severe weather is any destructive storm, such as tropical cyclones (hurricanes), tornadoes, waterspouts, thunderstorms, hail, and freezing rain or ice storms. The frequency and severity of these storms in the region surrounding Hope Creek have been assessed in the following sections.

2.3.1.2.1.1 Tropical Cyclones

Tropical cyclones originate over the tropical waters of the Atlantic Ocean, the Caribbean, and the Gulf of Mexico during early summer through fall. The most intense form is called a hurricane, which has wind speeds greater than 73 mph; however, other less destructive stages can exist. These are known as tropical depressions and tropical storms. The remnants of these cyclones, which dissipate over land, often become extratropical cyclones. Reference 2.3-4 shows that from 1899 through 1980, 12 extratropical cyclones, 8 tropical storms, and no hurricanes passed through the region. The average annual frequency of destructive tropical cyclones is less than 0.2 (less than 10 storms per 55 years), according to Reference 2.3-5.

The region is fairly well shielded from the most destructive forces of tropical cyclones, since it is not located directly on the Atlantic Coast. In fact, no hurricanes have been documented as having entered the state of Delaware directly from the Atlantic Coast, as shown in Reference 2.3-6.

2.3.1.2.1.2 Tornadoes

Tornadoes, although infrequent, do occur in the region, primarily during the spring and summer. Summaries prepared by Pearson, noted in Reference 2.3-7, indicate that there were 108 tornadoes reported within a 2°-latitude-by-2°-longitude area centered on the site during the period 1950 through 1981. This 2° by 2° area represents approximately 15,000 square miles. Of these 108 tornadoes, none had an estimated Fujita-Pearson force scale exceeding F3, 206 mph.

The closest reported tornado came within 10 miles of the site on July 1, 1954, across the Delaware River in Delaware. This tornado had a path area of 0.03 square miles.

Using the statistical methods of Thom, noted in Reference 2.3-8, the probability of a tornado striking any given point in the one degree latitude-longitude square centered on the Hope Creek site is once in every 10,229 years. This probability estimate is based upon a frequency of 1.4 tornadoes/year in the one degree square. The annual frequency of tornadoes was obtained from Pearson's summary, which showed that 44 tornadoes, of which three had multiple touchdowns, occurred in the one degree latitude-longitude square encompassing the site. The average path area for these 44 tornadoes was 0.26 square miles.

2.3.1.2.1.3 Waterspouts

Golden states in Reference 2.3-9, that a waterspout is an intense columnar vortex of limited horizontal extent, existing over a body of water, but not necessarily containing a funnel cloud. The basic differences between tornadoes and waterspouts are that tornadoes are generally more intense, larger, and longer lived.

The data on waterspouts is limited and concentrates on events occurring in the vicinity of the Florida Keys. Several studies on their size, duration, intensity, associated phenomena and effects have been made in the Keys area, yet little information is available to relate these storms to those occurring elsewhere, discussed in References 2.3-9 and 2.3-10.

Basically, the largest and most intense waterspouts can be of tornadic size. The upper limit to the rotational velocity is presently estimated as approaching 200 mph with typical translational velocities approaching 15 to 20 mph.

Estimates of the frequency of waterspouts at the Hope Creek site are extremely difficult to project. Waterspouts rarely leave evidence of their occurrence. The frequency of waterspouts occurring in the Hope Creek region, over the Delaware River or Bay, probably approaches that of tornadoes in the region.

2.3.1.2.1.4 Thunderstorms and Lightning

Thunderstorms are a seasonal phenomena in the region of the Hope Creek site. The Wilmington NWS records, found in Reference 2.3-11, shows an average of 31 thunderstorm days per year, with 26 of these days occurring during the warmer months of April through September.

Direct observation of lightning strikes is not a routine function at any of the standard observing stations. However, in Reference 2.3-12, Uman has developed a statistic that indicates that the number of lightning flashes (cloud to ground) per square mile, per year, is equal to between 0.05 and 0.8 times the number of thunderstorm days per year. A conservative estimate of the number of lightning strikes per year in the square mile area containing the Hope Creek site is 25.

The following provides monthly and annual estimates of lightning strikes at Wilmington NWS station as tabulated in NUREG/CR-2252, "National Thunderstorm Frequencies for the Contiguous United States.":

WILMINGTON NWS
MEAN NUMBER OF THUNDERSTORMS

Month

J	F	M	A	M	J	J	A	S	O	N	D	Annual
0.2	0.4	1	3	5	8	8	8	3	1	1	0.3	39

As given in NUREG/CR-2252 a thunderstorm day is defined as any day during which thunder is heard, whereas the aforementioned frequency estimates are based on surface data and special observations taken at the station. The summer months (June, July, and August) have the highest number of thunderstorms while the winter months have the fewest, which is expected. The annual number of thunderstorms is slightly higher than the aforementioned number derived from Uman's methodology, however, these frequencies are low compared to those for the Midwestern and Southeastern states.

The tallest structure on the Hope Creek site is the 512 ft cooling tower. This structure would be expected to attract the majority of the lightning strikes. Lightning protection is provided in the HCGS design and is not related to frequency of lightning strikes. A description of electrical protection of safety-related equipment is provided in Sections 7 and 8.

2.3.1.2.1.5 Hail

Severe hail storms are a relatively rare phenomenon in the region. Pautz, in Reference 2.3-13, reports only eight occurrences of hail with diameters of 0.75 inch or greater in New Jersey, and none in Delaware, during the period of 1955 through 1967. Of these eight occurrences in New Jersey, six storms had hail with diameters ranging from 0.75 to 1.50 inches, and two had diameters greater than 1.50 inches. Baldwin, in Reference 2.3-14, and Changnon, in Reference 2.3-15, report an annual frequency of approximately one hail storm per year in the Hope Creek region. Hail is generally associated with severe thunderstorms, and is most likely to occur during the late spring and summer.

Reference 2.3-16 shows six occurrences of hail during the period 1977 through 1981, which agrees well with frequencies reported by Baldwin and Changnon.

2.3.1.2.1.6 Ice Storms

A survey by Bennett, found in Reference 2.3-17, for 1928 through 1937, indicates that ice or freezing rain may occur up to one to three times per year in the site region. These occurrences are most frequent during the winter. Glaze accumulations greater than 0.25 inches are expected only once per year. A more recent summary of glaze statistics in Reference 2.3-14 indicates that during the 20-year period, 1950 through 1969, approximately four days of freezing rain annually occur through the region. Freezing rain has occurred on 25 days during 1977 through 1981 at the Wilmington NWS Station, as mentioned in Reference 2.3-16. The longest duration of freezing rain at the Wilmington NWS Station during this period lasted for 15 hours on February 15 and 16 of 1979.

2.3.1.2.1.7 High Air Pollution Potential

Episodes of limited atmospheric dispersion in the U.S. have been studied by Holzworth, in Reference 2.3-18 in terms of urban and area source problems. Holzworth has estimated approximately 20 forecast days of high potential for air pollution during a 5-year period in the vicinity of the site. Using a pressure gradient technique to define stagnating conditions, Korshover, in Reference 2.3-19, found between 150 to 175 stagnation days in the region during the 40-year period from 1936 through 1975. This converts to approximately four stagnation days per year, which is the same as Holzworth's estimate. The summer and fall experience the highest potential for stagnation days.

2.3.1.2.2 Maximum Snow Load

The weight on the ground of the 100-year mean recurrence interval snowpack at the Hope Creek site is 20 psf. This value was obtained from estimates made in References 2.3-20 and 2.3-21, both of which are based on the work of Thom, according to Reference 2.3-22. The extreme snow load may be conservatively estimated by adding the weight of the 48 hour probable maximum winter precipitation assumed to occur as snow, to the weight of the 100-year snowpack. From the work of Ho and Riedel, NUREG/CR-1486, as indicated in Reference 2.3-23, the 48-hour probable maximum winter precipitation is estimated to have a water equivalent of 19.8 inches, which has a ground force of 103 psf. Therefore, the extreme snow load on the ground at the Hope Creek site is estimated to be 123 psf. For roof design of Seismic Category I structures, this load is increased to 150 psf to account for building configuration and roof shapes and is considered a live load as discussed in Section 3.8.4.

It should be emphasized that this estimate is highly conservative and is presented only for building design purposes. The 48-hour probable maximum precipitation is based upon theoretical considerations, not measured values. The extreme load snowpack is equivalent to 24 inches of water or, using a typical ratio of snow to water (10:1), becomes 240 inches (20 feet) of snow.

2.3.1.2.3 Design Basis Tornado

The design basis tornado parameters at the Hope Creek site have been determined in accordance with the criteria given in Reference 2.3-24. These parameters are as follows:

Maximum wind speed	360 mph
Rotational speed	290 mph
Translational speed	
Maximum	70 mph
Minimum	5 mph
Radius of maximum rotational speed	150 ft
Pressure drop	3 psi
Rate of pressure drop	2 psi/s

2.3.1.2.4 Fastest Mile of Wind

The 100-year recurrence interval fastest mile of wind expected at the Hope Creek site is 93 mph, according to Reference 2.3-21 and 2.3-25. This is equivalent to a mean hourly wind speed of approximately 73 mph, as indicated in Reference 2.3-26.

The fastest mile of wind value is valid 30 feet above the ground. The vertical distribution of the fastest mile of wind is computed using the common power law, in the form:

$$U_Z = U_L(Z/Z_L)^P \qquad (2.3\text{-}1)$$

where:

U_Z = wind speed at height Z

U_L = wind speed at lower height Z_L

P = stability dependent exponent

Thom indicates in Reference 2.3-25 that a value for P of 1/7 is appropriate for high wind speeds in flat to rolling rural terrain, such as that at the Hope Creek site.

The following table presents the vertical distribution of the fastest mile:

Height Above Ground, ft	Fastest Mile, mph
30.0	93
100.0	110
150.0	117
200.0	122
300.0	129
400.0	135
500.0	139

A gust factor of 1.3 is commonly used at the 30-foot level. Since the gust factor is known to decrease both as a function of height and increasing wind speed, the use of 1.3 is conservative at higher heights. For design of Seismic Category I structures, a basic wind speed (V_{30}) of 108 mph is used as discussed in Section 3.3.1. The design wind velocities are given in Sections 2.3.2.3 and 3.3.1.1.

2.3.1.2.5 Ultimate Heat Sink

The ultimate heat sink (UHS) for the Hope Creek Generating Station (HCGS) is the Delaware River. Because of its large volume, a climatological analysis of maximum evaporation, drift loss, and minimum heat transfer is not necessary.

2.3.2 Local Meteorology

The analysis of the local meteorology in the vicinity of the Hope Creek Site is based upon 5 years of data, January 1977 through December 1981, collected at the onsite meteorological tower. The 300-feet tower is located on Artificial Island, approximately 1 mile southeast of Hope Creek. A full description of the meteorological tower and its instrumentation is given in Section 2.3.3.

The analysis of regional meteorology of the Hope Creek area is based upon data from the Wilmington, Delaware National Weather Service (NWS) first order station, 15 miles north of the site. This station provides both representative long term data for the region, as well as meteorological data concurrent with the onsite data period. The relevance of the meteorological data from the site tower and from the Wilmington NWS station to that over the long term meteorological conditions at the Hope Creek site is assessed by contrasting the two data sets. This assessment contrasts the extremes and distributions of the key meteorological parameters crucial to the safety, operation, and construction of HCGS.

Obviously, no two meteorological data sets collected at the same location during different time periods or at two different locations for the same time period are identical. However, the differences between the data sets must be assessed to ensure, in this application, that the onsite and/or long term data set reasonably represent the conditions that would be expected over the approximately 40-year lifetime of the plant.

2.3.2.1 Normal and Extreme Values of Meteorological Parameters

Meteorological data collected at the Artificial Island tower from January 1, 1977, through December 31, 1981, comprise the onsite data base. This 5-year data set includes the latest available annual cycle of meteorology.

Table 2.3-4 lists the 5-year data availability for each meteorological parameter with the incorporation of appropriate backup data.

Substitutions are made for missing wind data at the 150 and 300-foot tower levels. Concurrent wind direction data from the 150 and 300-foot levels are interchanged in all summaries if either is missing. The wind direction substitutions are only made if both corresponding wind speeds are at least 3 mph. The 3-mph minimum wind speed criteria is used to ensure the accuracy of the substitution. Similarly, concurrent wind speed data from 150 and 300-foot levels are interchanged if either is missing. The substituted wind speeds are adjusted to the proper level by the following stability dependent power law equation:

$$U_Z = U_L (Z/Z_L)^P \qquad (2.3\text{-}1)$$

where:

U_Z = wind speed at height Z

U_L = wind speed at lower height Z_L

P = stability dependent exponent

The values of the stability dependent exponent, P, are:

Pasquill Stability	P
A	0.10
B	0.15
C	0.20
D	0.25
E	0.25
F	0.30
G	0.30

Stability, determined from the 300 to 33-foot tower lapse rate, must be available to make any wind speed substitutions.

The Wilmington NWS data, concurrent with the onsite data, have been obtained from the National Climatic Center in North Carolina in several publications, as indicated in References 2.3-27 through 2.3-29, and References 2.3-11 and 2.3-16. Summaries of the

Wilmington NWS data over extended time periods of 10 years or more have also been obtained from the National Climatic Center.

2.3.2.1.1 Wind Flow

2.3.2.1.1.1 Wind Direction and Speed

Onsite wind measurements are made at three heights on the tower, 33, 150, and 300 feet. Annual wind direction and speed distributions have been computed by atmospheric stability class, according to the classification system recommended in Reference 2.3-30. These distributions are used in the diffusion models discussed in Sections 2.3.4 and 2.3.5, and are presented in Reference 2.3-31, Tables A-1 through A-3. In addition to the seven stability class wind direction and speed distributions, a summary for all stabilities is presented for each level directly following the individual stability distributions. Monthly distributions of wind direction and speed, by atmospheric stability for each level, are given in Reference 2.3-31, Tables A-4 through A-6 for the 33, 150, and 300-foot levels, respectively.

A summary of the annual wind direction distributions independent of stability for the three tower levels is presented in Table 2.3-5. These frequency distributions are extremely similar. The annual sector frequencies show differences of less than 2.8 percent between any two levels, with over 80 percent of the comparisons having less than a 1.0 percent difference. All three levels show primary frequency peaks in the northwest and west-northwest directions. Secondary wind direction frequency peaks are recorded in the southeast, south-southeast, and southwest sectors for the 33, 150, and 300-foot levels, respectively.

The Wilmington NWS wind direction and speed distributions are categorized by atmospheric stability class devised by Pasquill and modified by Turner in the STAR program, as discussed in Reference 2.3-32. The STAR program distributions are tabulated for the concurrent onsite data period in Reference 2.3-31, Table A-7.

and for the 10-year period, 1972-1982, in Reference 2.3-31, Table A-8. The concurrent and long term period wind direction distributions at Wilmington are very similar. All stability distributions are bimodal, with a primary peak in the west-northwest and northwest sectors, and a secondary peak in the south sector. A summary of the wind direction frequencies for the concurrent and long term periods are contrasted with the onsite wind direction frequencies in Table 2.3-6.

Although the onsite and NWS data are reasonably similar, there are some differences worth noting. The Wilmington NWS records for the concurrent time period, as well as the 10-year period also presented in this table, show a higher frequency of winds from the south and northwest compared to the site. However, the site distributions show a considerably higher frequency of winds from north-northeast and southeast. The frequencies in all other sectors at the site are within 2.5 percent, and most are within 2 percent of the concurrent and long term records at Wilmington. The low frequency of calms at the site is expected as a results of the excellent exposure in all directions.

Onsite monthly average wind speeds from the three tower levels are summarized in Table 2.3-7. The highest average wind speeds occur during the winter, while the lowest average wind speeds predominate in the summer months. The higher wind speeds measured at the tower usually occur with west-northwest, northwest, and southeast winds. The maximum hourly average wind speed measured during the five-year period was 40 mph at the 33-foot level, 54 mph at the 150-foot level, and 58 mph at the 300-foot level.

The monthly average wind speed distributions presented in Table 2.3-7 for the onsite data, and Table 2.3-8 for the long term Wilmington NWS data, are similar. Both locations report higher average wind speeds during the winter and spring months, while the summer months record the lowest seasonal wind speeds.

Annual mean wind speeds at the two locations are within 1 mph of each other at a height of 33 feet.

Annual wind direction frequencies at the 33 ft, 150 ft, and 300 ft levels observed during June 1969 to May 1971 (SGS preoperational data) are shown in Table 2.3-36. The 150 ft wind distribution was derived from January 1970 to May 1971 data. Annual wind direction distribution for the same three levels for the period January 1977 to December 1981 are presented in Tables 2.3-37, 2.3-38 and 2.3-39, respectively.

Data collection for the period of 1969 to 1971 was from a tower located 1400 feet north of the Hope Creek Reactor Building at a latitude of 39 degrees, 28 minutes, 13 seconds north, and a longitude of 75 degrees, 32 minutes, 12 seconds west. This tower was originally located to support preoperational data collection for the Salem units. The tower was relocated to the existing location to facilitate the construction of the Hope Creek Station and the cooling tower.

The comparison of annual wind direction frequencies at the 33 ft, 150 ft, and 300 ft levels for both Salem and Hope Creek for the available period of record is as follows:

33 feet

Highest wind direction frequencies from the period 1969 to 1971 (SGS) compare favorably with those from 1977 to 1981 (HCGS). The site has a bimodal distribution. SGS data shows the highest frequency of wind directions are SE-SSE-S and W-WNW-NW. HCGS data shows the same pattern. Frequencies other than these modes are evenly distributed throughout the compass points. For all individual years, the data recovery rates are above 90 percent. Variation among frequencies for individual years within the two data bases could be caused by overall synoptic conditions and related storm tracks. The bimodal distribution can be explained by synoptic conditions over the general area.

150 feet

Any comparison between the two data bases would be obscured because the SGS data starts in January 1970 rather than June 1969. In addition, the data recovery of the 150 ft wind direction for the period January 1970 to May 1970 is less than 65 percent due to installation and maintenance problems. The same bimodal distribution is observed between the two data bases at the 150 ft level as was observed at the 33 ft level.

300 feet

Any comparison between the two data sets shows the same bimodal distribution as was seen at the two lower levels. The data recovery rate for June 1969 to May 1970 was below 85 percent.

Minor differences between the two data sets are caused by seasonal variations. The SGS data was observed over a period of time that seasonally does not coincide with the HCGS data. Weather patterns are related to changes of seasons. The prevailing winter wind directions at Artificial Island for the period 1977 to 1981 were WNW-NW, during the summer were SSW-SW, during the spring were evenly distributed between SSE-SW-WNW, and during the fall were SSE-S and WNW-NW (Reference 2.3-31). These wind direction frequency results were the same for the 150 ft and 300 ft levels.

2.3.2.1.1.2 Wind Direction Persistence

Wind direction persistence at the Hope Creek site has been analyzed using a technique that determines the number of consecutive hours during which the wind direction at a given level remains within the same 22-1/2° sector. This analysis is performed using a sliding technique, so that the longest persistence is obtained incorporating a given hour. The results are summarized in tabulations of the number of times the wind direction at each level remains in the same sector for various time periods ranging from 2 to 3 hours to longer than 48 hours.

The 5-year annual summary of wind direction persistence is given in Tables A-9, A-10 and A-11 of Reference 2.3-31 for the three tower levels.

The wind persistence distributions in these tables are categorized by stability, in addition to those independent of stability. The unstable and neutral classes are combined as well as the slightly to very stable classes. The stability is defined in accordance with criteria described in Reference 2.3-30. It is based on the 300 to 33-foot temperature difference measured on the tower.

These tables indicate that wind direction persistence of more than 8 hours does not seem to be a function of stability. Numerous cases in all sectors and stabilities are associated with shorter persistence time. The southeast and northeast wind direction sectors at all levels show the highest frequencies of persistence. These are the only two sectors to record a case of wind direction persistence greater than or equal to 48 hours in duration.

The combined monthly distributions of persistence by level are presented in Tables A-12, A-13 and A-14 of Reference 2.3-31. The monthly summaries for the 5 years show that persistent winds greater than 12 hours occur more often in the winter than the other seasons. The upper wind level has the highest number of wind direction persistence cases greater than 12 hours in December.

2.3.2.1.2 Temperature and Dew Point

2.3.2.1.2.1 Temperature

The onsite monthly and annual means and extremes of temperatures are summarized in Table 2.3-9. January has the lowest mean monthly temperature of -2.1°C, and July has the highest mean monthly temperature of 23.8°C. The overall maximum hourly temperature at the site is 34.5°C, recorded on July 19, 1977, and on July 20, 1980. The lowest hourly temperature recorded during the period is -18.5°C, on February 18, 1979, and on January 17, 1977.

Monthly and annual frequency distributions of the hourly temperature data are shown in Table 2.3-10. These distributions demonstrate that larger temperature ranges exist during the winter months, compared to the relatively small temperature ranges of the summer months. Annually, temperatures below -5.0°C occur less than 5 percent of the time, and temperatures greater than or equal to 25.0°C occur less than 9 percent of the time.

The diurnal range of hourly temperatures for the period are summarized in Table 2.3-11. The average temperature for each hour of the day is listed on an annual and monthly basis.

Table 2.3-12 contrasts the monthly and annual mean temperatures at the site and Wilmington for the 5-year onsite data period. The Artificial Island temperatures are converted to degrees Fahrenheit for comparison with the Wilmington temperatures. This table shows a tendency for summer maximum temperatures at the site to be slightly lower than at Wilmington and winter minimums to be slightly higher.

Specifically, the maximum annual temperature at the site was 4°F cooler than Wilmington, while the minimum temperature at the site was 5°F warmer. During this concurrent period, the mean monthly and annual temperatures at the site compare very favorably with Wilmington.

Table 2.3-13 lists the Wilmington NWS long term temperatures means and extremes. These means are generally similar to the concurrent 5-year period mean temperatures measured at Wilmington. The mean temperatures at Wilmington during the 5-year period are considerably cooler than the long term Wilmington mean temperature for January and February. Both of these months have average temperatures more than 4°F lower than the long term means. This substantiates the extremely cold winters the Northeast has experienced during the last few years. The annual average temperature during the 5-year concurrent period in Wilmington is only 0.6°F lower than the long term record. The long term extreme temperatures are comparable to those measured during the 5-year period.

2.3.2.1.2.2 Dew Point

The means and extremes of dew point temperature are shown in Table 2.3-14. The highest dew point temperatures occur during the summer, with the overall maximum hourly dew point of 28.4°C recorded in August. The overall minimum hourly dew point temperature of -24.7°C occurs in December.

The monthly and annual frequency distributions of the hourly dew point temperature are given in Table 2.3-15. On an annual basis, 9.3 percent of the hours record dew point temperatures greater than or equal to 20.0°C and 9.6 percent of the hours have dew point temperatures less than -10°C.

Table 2.3-16 shows the monthly and annual diurnal range of dew point temperature. There is little diurnal variation of the dew point temperatures within each month.

2.3.2.1.3 Atmospheric Moisture

Onsite summaries of atmospheric moisture content are compiled from the 33-foot tower data of 1977 through 1981. These include relative and absolute humidity. These summaries are obtained by using the measured dew point temperature and the ambient temperature.

2.3.2.1.3.1 Relative Humidity

The monthly and annual means and extremes of onsite relative humidity are given in Table 2.3-17. There is little variation in the monthly mean relative humidities. The lowest monthly mean value, 62 percent, occurs in April, and the highest mean value is 76 percent in August. The overall lowest hourly relative humidity, 15 percent, occurred during April 1980.

Onsite monthly and annual frequency distributions of relative humidity are shown in Table 2.3-18. The annual frequency distribution shows 17.7 percent of the hourly relative humidities are greater than or equal to 90 percent, with approximately 1.5 percent of the relative humidities under 30 percent. April has the highest frequency of low relative humidities.

The diurnal ranges of relative humidity on a monthly and annual basis are presented in Table 2.3-19. These summaries show the highest relative humidities occur during the morning hours around the time of sunrise. The minimum relative humidity values are recorded during the afternoon hours.

The mean annual and monthly relative humidities at Artificial Island, shown in Table 2.3-19, and Wilmington, shown in Table 2.3-20, are very compatible. The mean relative humidity at Artificial Island tends to be slightly lower than the values at Wilmington at 0100 local time, and slightly higher at 1300 local time.

2.3.2.1.3.2 Absolute Humidity

Absolute humidity is summarized into monthly and annual means and extremes in Table 2.3-21. Frequency distributions of absolute humidity are given in Table 2.3-22. Absolute humidity is defined as the ratio of the mass of water vapor present to the volume occupied by the mixture, the density of the water vapor component according to Reference 2.3-33. The maximum absolute humidity of 26.9 g/m^3 occurs in August, and the minimum of 0.6 g/m^3 in December and January. The maximum annual frequency distribution is skewed toward the lower absolute humidity categories.

There is also a large seasonal variation of absolute humidity, as the monthly distributions show. This is expected, since the ability of air to hold water vapor is temperature dependent. Warm air can hold more water vapor than cold air. Table 2.3-23 shows there is very little diurnal variation of absolute humidity within each month.

2.3.2.1.4 Precipitation

Onsite precipitation measurements are summarized monthly for the five 1-year periods, as well as annually in Table A-15 of Reference 2.3-31. Combined monthly and the overall precipitation distributions for the five years are also tabulated in Table A-16 of Reference 2.3-31. The hourly measurements are categorized by the amount of water equivalent precipitation which fell during each hour. In addition, maximum hourly, daily, and monthly total precipitation amounts are highlighted in summarizations.

Time Period (hours)	Date	Maximum Measured Water Equivalent Precipitation (inches)
1	August 3, 1981	1.60
2		1.90
3		2.10
6		2.40
12		2.80
24	October 25, 1980	3.10
1 month	January 1979	7.90

During the 5-year period, the precipitation was fairly evenly distributed throughout the year. The precipitation intensity was generally light. More than 70 percent of the recorded precipitation hours during the 5 years occurred at rates of less than or equal to 0.10 in/h. The monthly distributions show that the summer months exhibit higher hourly precipitation rates than the winter months.

The duration of precipitation and the accumulated amounts are shown in Reference 2.3-31, Tables A-17 and A-18, as frequency distributions for the entire 5-year period, and by month. The majority of precipitation events last less than 6 consecutive hours. There was only one case of precipitation lasting between 12 and 23 consecutive hours during the 5 years.

Those hours with recorded precipitation are distributed by wind direction and speed, recorded at the three tower levels and categorized by precipitation rates. In addition, all precipitation hours are grouped into frequency distributions by wind speed and direction. Eight hourly rate categories are used and range from the smallest, an amount equal to 0.10 inches, to the largest, amounts exceeding 1.50 inches. Reference 2.3-31, Tables A-19 to A-21, present the distributions for the entire period, and Reference 2.3-31, Tables A-22 to A-24, list the combined monthly distributions.

These distributions indicate that precipitation is most frequently associated with winds containing an easterly component at all levels. Furthermore, winds at the 33-foot level are most frequently southeast during precipitation, and northeast and east-northeast winds are the most common 150 and 300-foot wind directions during precipitation.

A comparison of the onsite and Wilmington precipitation extremes during January 1977 through December 1981 reveals that the onsite extremes are comparable to those at Wilmington. The 1-hour, 24-hour, and 1-month maximum precipitation totals are shown in the following table:

Time Period (hours)	Artificial Island (inches)	Wilmington NWS (inches)
1	1.60	1.35
24	3.10	3.94
1 month	7.90	8.41

The long term maximum 24-hour and monthly precipitation totals measured at the Wilmington NWS are 6.53 inches in August 1945 and 14.91 inches in August 1911, respectively. These maximums significantly exceed those measured at the site.

The monthly and annual means, along with the monthly extremes and 24-hour maximums of precipitation at the Wilmington NWS station, are given in Table 2.3-24. The mean monthly precipitation totals range from a minimum of 2.60 inches in October to a minimum of 4.31 inches in July. The mean annual total precipitation for the 1941 through 1970 period was 40.25 inches.

These long term climatological monthly precipitation means are shown in Reference 2.3-31, Table A-16.

Unlike wind frequency and temperature (including stability indicators) where random missing data has only a limited effect in describing site annual meteorological characteristics, missing precipitation data does have an impact on annual averages. This fact is obvious as precipitation is an accumulative measurement. Missing precipitation data is attributed to three main causes: 1) equipment failure or malfunction, 2) system outage due to calibration or maintenance, 3) deletion of suspect data by the meteorologist reviewing the data collected from the data base. With respect to precipitation a conscientious effort to invalidate suspect data results directly in a potential underestimate of annual precipitation.

The digital Meteorological Data Acquisition Systems provide increased data recovery. It should be noted, that the Meteorological Data Acquisition System was designed to meet the requirements of Regulatory Guide 1.23 and precipitation data was not mentioned in this Regulatory Guide. However, one would expect statistical differences to occur regardless of any small precipitation data loss.

Yearly precipitation totals and precipitation statistics for the period 1977 to 1981 are presented in Tables 2.3-34 and 35 for Hope Creek, Wilmington NWS Station, Glassboro and Woodstown cooperative stations. Wilmington NWS Station is located 15 miles north of the HCGS. The Glassboro Station is located approximately 27 miles northeast of the HCGS. The Woodstown Station is located 17 miles northeast of the HCGS.

The statistics indicate that the Wilmington precipitation data has the greatest standard deviation and standard error. The means do not show good agreement in precipitation data from Wilmington and onsite data.

Further analysis shows that during the year 1980 precipitation data was not collected during July and August due to an instrument equipment problem.

Difference of more than five inches in a year is not unusual as shown in the table. The greatest difference between Wilmington and the three other stations occurred in 1978. Wilmington precipitation was 11.74 inches higher than observed at Glassboro and 6.84 inches higher than observed at Woodstown. The distance from Woodstown to Glassboro is 10 miles. The largest difference in observed precipitation between Woodstown and Glassboro is in 1981 (6.21 inches).

These differences in observed precipitation can be attributed to spatial differences between stations, frequency and intensity of localized convective storms (generally observed during the summer months) and the accuracy of precipitation measurements. NWS stations observed precipitation to 0.01 inches while onsite data was to the nearest 0.10 inches. The onsite rain gauge has been replaced with instrumentation that has an accuracy of 0.01 inches.

Precipitation data changes with distance in areas where localized short lived convective storms occur. The higher the frequency of occurrence, the greater the precipitation differences between stations. Precipitation data is not a good measure of representativeness between stations such as HCGS and Wilmington NWS Station. On a larger scale, almost approaching synoptic, measured parameters such as wind direction and speed, absolute humidity and stability provide a more precise measure of representativeness between stations.

Since snowfall is not measured at the site, the Wilmington NWS records are presented. Monthly and annual means, as well as the monthly and 24-hour maximum snowfall, are given in Table 2.3-25. February had the highest mean monthly snowfall of 6.4 inches. The maximum monthly snowfall of 27.5 inches occurred in February 1979. The maximum 24-hour snowfall of 22.0 inches was recorded in December 1909.

2.3.2.1.5 Fog and Haze

Table 2.3-26 presents the monthly and annual summary of fog, haze, and/or smoke for Wilmington. At Wilmington, between 1965 and 1974, light fog (visibility less than 7.0 miles) occurs on an average of 156 days per year and is rather evenly distributed throughout the year, with the exception of a slight relative minimum during the winter, as indicated in Reference 2.3-29. Heavy fog, (visibility less than or equal to 0.25 miles), is far less frequent, occurring on an average of 34 days per year.

Haze and/or smoke is reported on an average of 167 days per year, as found in Reference 2.3-29. Most of these days are between June and September.

2.3.2.1.6 Atmospheric Stability

Determinations of atmospheric stability are made from the temperature difference measured between the 300 to 33-foot and the 150 to 33-foot levels on the onsite tower. These temperature difference data are grouped into seven stability classes, A through G, according to the NRC lapse rate criteria shown in Reference 2.3-30.

Delta temperature stability distributions for the 300 to 33 ft and 150 to 33 ft intervals in the Artificial Island meteorological tower are given in Tables 2.3-27a and 2.3-27b, respectively. The 300 to 33 ft delta temperature distribution shows a majority of hours with neutral and stable conditions. The 150 to 33 ft delta temperature distribution also shows this pattern.

The 150 to 33 ft delta temperature was designed to provide backup data if the upper delta system was inoperable. The interval of measurement is only 117 ft. and to the nearest 0.1 degrees Celsius as required by Regulatory Guide 1.23 (Safety Guide 23). Therefore, when converting the C/100m, stability Class B does not exist. This is a further reason to use the 150-33 ft delta temperature data for backup purposes only.

Monthly and annual summaries of atmospheric stability have been incorporated into the wind roses previously presented in Tables A-1 through A-6 of Reference 2.4-31. The onsite distribution of only atmospheric stability for the 5-year record is summarized in Table 2.3-27. The temperature difference scheme classifies 43.9 percent of the hours as unstable neutral (Pasquill Classes A through D) and 56.1 percent of the hours as stable (Pasquill Classes E through G).

Table 2.3-27 also shows the Wilmington stability distributions for the concurrent onsite data period, as well as the long term 1972 through 1981 record. The Wilmington NWS stability is determined from the STAR program methodology consistent with Turner's scheme, found in Reference 2.3-32.

Because of different techniques used to determine atmospheric stability, e.g., the (temperature difference method for onsite data and the STAR method for the Wilmington data, the stability distributions given in Table 2.3-27 are also expectedly different. Differences have been documented in several papers and should not be alarming. See References 2.3-34 and 2.3-35.

Annual atmospheric stability distributions (Pasquill stability classes A-G) based on measured 300 to 33 ft and 150-33 ft delta temperature for the period June 1969 to May 1971 are presented in Table 2.3-43. Annual stability distributions for the period January 1977 to December 1981 are presented in Tables 2.3-27a and 2.3-27b. The 1969 to 1971 data shows the same predominantly neutral to slightly stable conditions both at the 150-33 ft and 300-33 ft levels as is shown in the 1977 to 1981 data set.

2.3.2.1.7 Monthly Mixing Height Data

The table below was derived from isopleths of mixing height data by season, presented in U.S. EPA Publication 101 entitled "Mixing Heights, Wind Speeds, and Potential for Urban Air Pollution Throughout the Contiguous United States."

	Mixing Height (meters)	
Season	Morning	Afternoon
Winter	850	1000
Spring	750	1700
Summer	600	1800
Autumn	700	1300
Annual	700	1300

2.3.2.1.8 Temperature Inversion Persistence

The frequency and duration of inversion conditions defined by the NRC delta temperature stability scheme are presented in Table 2.3-28.

Table 2.3-28 was derived from hourly 300 to 33-foot delta temperature data for the period January 1977 through December 1981. The NRC delta temperature stability scheme was used in conjunction with a duration - frequency program. Table 2.3-28 shows frequencies and duration of hourly stability greater than 0.0°C/100 meters and greater than 1.5°C/100 meters on a monthly and annual basis. The following are the results of the analysis.

All Inversions - Lapse Rate

>0.0°C/100m (Stability E, F, and G)

1. The duration of inversions, (NRC stabilities E, F, and G) lasting up to 12 hours were observed for over 50 percent of all cases, regardless of month.

2. For the months May through September, 75 percent of all these inversions had a persistence duration of less than or equal to 12 hours. 89.8 percent of the cases in August, the month with the highest number of cases, had a persistence less than or equal to 12 hours.

3. The highest percentage of cases when the duration of inversions was 13-24 hours was in November 36.3 percent), the lowest was in June (10.8 percent), and on an annual basis, it is 24.1 percent.

4. The highest number of inversions was in August (156 cases) and the lowest in January (83 cases). Averaged over a 5-year period, the number of occurrences equals 30 cases for each August and 15 cases for each January.

5. The highest percentage of cases, by month, that persisted for longer than 4 hours (2 days) was in March (1.7 percent of 121 cases or 2 occurrences).

Strong Inversions - Lapse Rate

>1.5°C/100 meters (F, and G)

6. Strong inversion persistence with durations less than or equal to 12 hours comprised the majority of all cases (75 percent of all cases for each month). Durations of up to seven hours occurred for 50 percent of the cases regardless of month. Analyzing the percentages of each case shows that, from April to October the duration was to 12 hours for over 90 percent of all monthly cases. On an annual basis, 90.8 percent of the cases had a duration of 12 hours or less and 86.6 percent had a duration of two to seven hours.

7. The highest number of persistence occurrences were in April (95) and the lowest in January (39). Averaged over a 5-year period, this translates into 20 cases during any April and 8 cases during any January.

2.3.2.1.9 Onsite Meteorological Data Tape

Hourly averages of wind speed and direction at the 300, 150, and 33-foot levels of atmospheric stability, determined by the 300 to 33-foot delta temperature, can be derived from the hourly data (January 1977 through December 1981) supplied to the NRC on magnetic tape.

2.3.2.2 Potential Influence of the Plant and Its Facilities on Local Meteorology

An EPRI study by Laurmann has concluded that, although quantitative predictions of the meteorological effects resulting from power plant operation cannot be made, evidence and theory indicate that plants of conventional size (up to 4000 MWe) rarely produce noticeable weather changes; see Reference 2.3-36. Minor effects on local meteorology, which might occur, are divided into two distinct categories: those attributable to the turbulent wakes associated with the plant structures, and those attributable to waste heat dissipation systems.

2.3.2.2.1 Turbulent Wake Effects From Plant Structures

As part of the technical support for the tall stack regulations in the 1977 Clear Air Act Amendments, the U.S. EPA has published a comprehensive review and literature search on the aerodynamic effects caused by building structures, as indicated in Reference 2.3-37. The consensus of this review is that a structure produces a cavity of increased turbulence on its leeward side, 1.5 building height deep, and persists for approximately five building heights downwind. Based upon these criteria, it is estimated that the Hope Creek turbine/reactor enclosure complex produces a turbulent wake on its leeward side, extending approximately 90 meters vertically and persisting 305 meters downwind.

Halitsky has shown through wind tunnel testing that the turbulent effects produced by rounded structures are not as large or severe as those produced by sharp edged buildings, as indicated in Reference 2.3-38. This is consistent with the result of a combined wind tunnel-field measurement study conducted by Smith and Mirabella on the cooling tower induced wake at the Rancho Seco Plant mentioned in Reference 2.3-39. Their results indicate that the cooling towers produce a turbulent wake only when wind speeds exceed 2 meters/s. They estimate that the wake would be 1.5 structure heights deep, and would persist for 2 to 3 tower diameters downwind. According to these criteria, the maximum wake produced by the Hope Creek cooling tower would be turbulent region extending 235 meters vertically and persisting 395 meters downwind.

2.3.2.2.1.1 Effect of the Turbulent Wake on the Gaseous Reactor Effluent

The primary effect of the structurally induced wakes on the reactor effluent is to enhance dispersion, and is discussed briefly in Section 2.3.5.

2.3.2.2.1.2 Effect of the Turbulent Wake on the Meteorological Tower Measurements

The turbulent wakes produced by the turbine/reactor enclosure and the cooling tower do not extend far enough to affect the meteorological tower. The tower is located approximately 1 mile from the Turbine/Reactor Buildings and the cooling tower, well beyond the distorted flow region in the lee of the plant.

2.3.2.2.2 Potential Effects of the Waste Heat Dissipation System on the Local Meteorology

The natural draft cooling tower at the Hope Creek site is the only source of effluents capable of influencing the local meteorology. Warm moist air is released from the cooling tower containing its moisture in both vapor and liquid forms. The liquid or visible forms of moisture are either very small droplets formed when the vaporous plume interacts with cooler ambient air, or drift droplets. The drift droplets originate when the high velocity air flowing through the cooling tower entrains small water droplets from the circulating water falling through the fill section of the cooling tower. The possible effects of both the vaporous and liquid forms of the cooling tower effluent are discussed in the following sections.

2.3.2.2.2.1 Visible Plume Occurrence

The cooling tower plume only becomes visible if the water vapor contained within the plume condenses. The plume will remain visible until the droplets evaporate into the drier ambient air. Whether or not the plume will be visible at any particular time depends primarily on the temperature and moisture content of the ambient air. Studies have shown that ambient saturation deficit is the best indicator of visible plume persistence, mentioned in References 2.3-40, and Reference 2.3-41. Saturation deficit is defined as additional water vapor required to produce moisture saturation at a given ambient temperature and pressure. Saturation of the air by the cooling tower plume results in a visible plume of condensed water droplets. Observational evidence has shown that the vast majority of visible plumes from natural draft cooling towers do not persist downwind for more than 0.6 miles, as indicated in Reference 2.3-42.

2.3.2.2.2.2 Cooling Tower Drift

When the heated brackish circulating water falls through the fill section of the cooling tower, small water droplets are entrained by the relatively high velocity of the air flowing through the tower. The entrained water droplets and salt particles, called cooling tower drift, are carried from the tower and, subsequently, fall to the ground downwind from the tower.

The very efficient drift eliminators installed on the Hope Creek cooling tower insure that the drift emitted from the tower is the minimum achievable under current technology.

Experiences at natural draft cooling towers have shown that the fallout of water and chemicals under the majority of weather conditions is too small to be observed or measured except in the immediate tower vicinity, and no significant offsite environmental effects are created, as indicated in Reference 2.3-42.

2.3.2.2.2.3 Ground Fogging and Icing

Several studies have shown that natural draft cooling tower plumes never intersect the ground, thus they do not cause ground fogging or icing, as derived from References 2.3-40 and 2.3-42. The height of release and buoyancy of natural draft cooling tower plumes ensures this. Ground icing due to cooling tower drift is also negligible, since the total surface accumulation of water drift from natural draft towers is insignificant. Measurements done in England downwind from a natural draft cooling tower complex of eight towers for a 2000 MW fossil plant, with efficient drift eliminators, indicate a maximum drift deposition rate of 0.02 mm/h of liquid water, found in Reference 2.3-43. This rate is too low to cause any ground icing or wetting.

2.3.2.2.2.4 Cloud Enhancement and Shadowing

The extent to which natural draft cooling tower plumes contribute to cloud formation can be qualitatively assessed based on observational studies conducted at three operating cooling tower sites, as mentioned in Reference 2.3-40. At each of these sites, cooling tower plumes were observed to occasionally cause broken cloud decks to become overcast and to enhance thin clouds. Separate cloud formations were occasionally observed to result from visible plume formation from the cooling towers, but usually at altitudes of several thousand feet above ground. Based on the above observations, the potential for increased cloud development due to cooling tower operation appears to be minimal compared to the potential for development due to natural causes.

The cooling tower does have the potential to cause slight decreases in the amount of solar radiation received at a point on the ground due to visible plume shadowing. A study conducted on a natural draft cooling tower from a 1500 MW fossil plant in Europe found that on a cloudy day, the maximum shadowing effect is a 20 percent reduction in total radiation for short time periods as discussed in Reference 2.3-44. The effects of visible plume shadowing are obviously mitigated by the fact that the variability in wind direction causes the plume to move horizontally and does not remain over any one point for long periods of time. The relative rarity of long persistent visible plumes, detailed in Section 2.3.2.2.2.1, also minimizes the effects of plume shadowing.

2.3.2.2.2.5 Precipitation Modification

Observations of precipitation falling from natural draft plumes are very limited. Kramer et al. have documented an observation of light rain falling from a natural draft plume, and several observations of light snowfall, mentioned in Reference 2.3-45.

Though it may be possible for a cooling tower to modify the precipitation pattern immediately downwind of the tower, it would not significantly alter the total precipitation in the region, as the water vapor emissions from the towers are small compared to natural fluxes, as indicated in Reference 2.3-42.

During the winter of 1975-1976, Kramer et al. observed light snow from several different cooling towers on ten separate days, as found in Reference 2.3-46. This effect was found only during stable atmospheric conditions, with temperatures below 10°F at the height of the plume centerline. In the 1-year summary of Philadelphia upper air soundings on 22 days, for short periods, the temperature criteria necessary for snowfall were met. This should not be interpreted as a prediction of snowfall frequency. There are several other variables, such as atmospheric stability, blowdown water chemistry, drift eliminator condition, and condensation nuclei availability, which play a role in snowfall formation. The height to which the plume rises is such that, in most cases, the snow crystals would sublimate before reaching the ground.

2.3.2.2.2.6 Relative Humidity Increases

Observational studies have shown that the changes in ground level relative humidity should not be expected as a result of natural draft cooling tower operation. Spurr, in a study of a 2000 MW eight tower complex in England, found no ground level relative humidity increases either upwind or downwind of the plant; see Reference 2.3-43.

2.3.2.2.2.7 Interaction with Other Plumes

There are no significant sources of pollutants within 1.2 miles of the Hope Creek Plant. Therefore, there is no concern for any chemical interactions between the cooling tower and other industrial plumes.

2.3.2.2.3 Topographic Features

The terrain that surrounds the Hope Creek site is extremely flat within the first 5 miles. The maximum terrain within 5 miles of the plant is less than 60 feet above plant grade. The only appreciable terrain features (greater than 500 feet above grade) are located northwest of the site at distances of over 15 miles. Figures 2.3-1 and 2.3-2 are detailed 5 and 50 mile topographic maps, respectively.

Figure 2.3-3 illustrates the terrain features around the Hope Creek site. These figures give the maximum terrain in each of the 16 directional sectors out to 50 miles.

2.3.2.3 Local Meteorological Conditions for Design and Operating Bases

Meteorological conditions used for design and operating basis considerations, and their bases, are discussed and referenced in HCGS-FSAR Section 3.3, Wind and Tornado Loadings.

Design Wind Velocity

Wind velocities at 30 feet of 108 mph and 100 mph were used as design bases for Seismic Category I and Non-Seismic Category I structures respectively. Recurrence interval is at least 100 years.

Design Basis Tornado

Refer to HCGS Sections 2.3.1.2.3 and 3.3.2.1. The design numbers were derived from Regulatory Guide 1.76, mentioned in Reference 2.3-24.

2.3.3 Onsite Meteorological Measurements Program

2.3.3.1 Meteorological Data Collection Program

To arrive at atmospheric dispersion factors for use in calculating radiological exposures from both low level routine and accidental releases, an extensive data collection program was undertaken at the site. This data collection program is described in detail in the following paragraphs. The present meteorological monitoring program is in conformance with the recommendations of Regulatory Guide 1.23 (Safety Guide 23 - February 7, 1972), and all requirements in Standard Review Plan Section 2.3.3 (Revision 2) and Regulatory Guide 4.2 (Revision 2 - July 1976).

2.3.3.2 Operational Data Collection Program

A detailed representation of the meteorological facility is not necessary because of the simplicity of the terrain. The tower data used are primarily those from the 33 and 300-foot levels, although data were obtained at the intermediate 150-foot elevation. The wind instrumentation consisted of anemometers and wind vanes, and the temperature difference measurements were obtained from aspirated thermometers. Precipitation, humidity, and solar radiation measurements are on record for possible use in general environmental applications.

The system became operational in April 1976.

The location of the tower, a 300-foot guy wire supported structure, is latitude 39° 27', 48.9" north, and longitude 75° 31' 11.76" west. The site and nearby sources of data are presented in Figure 2.3-4.

The data collection program also includes an additional tower, identified as a backup meteorological tower, consisting of a 10-meter telephone pole. The backup tower is located approximately 500 feet south of the primary meteorological monitoring tower. Backup meteorological data provides wind speed, wind direction, and a computed sigma theta.

Wind speed and direction instrumentation is located at 300, 150 and 33-foot elevations on the primary tower and at the 33-foot elevation on the backup tower.

Temperature measurement includes ambient temperature taken at the 33-foot elevation and temperature difference taken between T300 to T33 and T150 to T33. Temperature sensors consist of thermistors in a motor aspirated solar radiation shield. The vertical temperature gradient is determined from paired, matched thermistors.

The dew point is measured at the 33 foot level. Rainfall and barometric pressure are measured at approximately 3 and 6 feet, respectively. Solar radiation is measured at a height of 8 feet above ground level. Figure 2.3-5 depicts the heights of these instruments on the tower.

All meteorological parameters are electronically recorded in the Meteorological Instrument Building at the base of the tower.

The data acquisition system includes capabilities for remote interrogation in addition to data acquisition. The data acquisition systems consist of primary and backup data acquisition systems (DAS) located at the Meteorological Instrument Building. A diagram of the system configuration is provided on Figure 2.3-6. The rain gauge uses a tipping bucket.

The primary and backup DAS, shown in Figure 2.3-6, are configured with identical hardware.

Each DAS is provided with communication ports, including one as a link to the Safety Parameter Display System (SPDS), and one for direct dial-up capability. Each DAS provides storage for at least 7 days of 15-minute averages.

The primary DAS collects wind speed and direction from the primary tower. The backup DAS collects wind speed and direction from the backup meteorological tower. Each DAS calculates a sigma theta for its respective meteorological tower (each of the three level wind directions on the primary tower, one level on the backup tower). The host computers acquire the meteorological data collected by the data loggers.

The calculations of the sigma thetas use samples of horizontal wind direction at each elevation/location.

Data interrogation is possible through dial up connection to the digital data acquisition systems. The digital data acquisition systems provide data to the SPDS. The SPDS supports display units in the EOF, the Hope Creek Control Point, the Salem and Hope Creek TSCs, the Hope Creek OSC, the Hope Creek Control Room, and the Salem OPS Ready Room.

Additional sources of meteorological data to provide a description of airflow trajectories from the site out to a distance of 50 miles include Wilmington and Philadelphia National Weather Service (NWS) stations.

Hourly wind, temperature, and cloud cover data are readily available from these NWS stations.

Monthly and annual joint frequency distributions of wind speed and direction, based on atmospheric stability classes, are referenced in Section 2.3.2.1.1. The 5-year database containing hourly site meteorological data from January 1977 to December 1981 was used as input in the analysis.

2.3.3.3 Operational Data Display

Several meteorological parameters are incorporated in the database of the Control Room Integrated Display System (CRIDS) computer.

The Hope Creek Safety Parameter Display System (SPDS) provides 15-minute average meteorological monitoring system parameters. The parameters available for display are 33-ft wind speed, direction, sigma theta, and horizontal stability class; 150-ft wind speed, direction, sigma theta, and horizontal stability class; 300-ft wind speed, direction, sigma theta, and horizontal stability class; delta temperature between 300 and 33-ft; delta temperature between 150 and 33-ft; vertical stability class for each delta temperature; precipitation; barometric pressure; solar radiation; and ambient and dew point temperatures.

Atmospheric transport and diffusion is calculated by the Meteorological Information and Dose Assessment System (MIDAS) computers installed in both Salem and Hope Creek. A method for determining atmospheric transport and diffusion throughout the plume exposure emergency planning zone during emergency conditions has been developed.

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2.3.4 Short Term Diffusion Estimates

2.3.4.1 Objective

The objective is to provide conservative and realistic short term estimates of relative concentration (X/Q), at both the site boundary and the outer boundary of the low population zone (LPZ) following a hypothetical release of radioactivity from HCGS. The assessment is based on the results of atmospheric diffusion modeling and onsite meteorological data.

A ground level accidental radionuclide release from HCGS is analyzed at various distances. Conservative and realistic X/Q values at the exclusion area boundary (EAB) are derived for the 0 to 2-hour period following a postulated accident. Conservative and realistic estimates of the X/Q value at the outer boundary of the LPZ are computed for 2, 8, 16, 72, and 624 hours following a postulated accident. For this modeling assessment, the EAB is assumed to be a circle with a radius of 901 m, which is the shortest actual distance to the EAB bearing north.

2.3.4.2 Accident Assessment

The short term, 0 to 2-hour X/Q values for ground level releases are calculated with the sector dependent model described in Regulatory Guide 1.145 Reference 2.3-47. Annual accident X/Q values are also required to derive the intermediate time period X/Q values. These annual accident X/Q values are derived using the long term diffusion model described in Regulatory Guide 1.111, Revision 1, Reference 2.3-48, and in Section 2.3.5, Long Term (Routine) Diffusion Estimates.

2.3.4.2.1 Methodology

The procedure used to estimate the X/Q values for the appropriate time periods following a postulated accident are described in Regulatory Guide 1.145. The diffusion model generates a cumulative frequency distribution of X/Q values for each sector-distance combination representing the first 2 hours after the postulated accident. These 2-hour X/Q values are based on 1-hour averaged data, but are assumed to apply for 2 hours. The frequency distributions are plotted on a log probability scale for each sector distance combination, and are then enveloped in accordance with the methodology described by Markee and Levine in Reference 2.3-49.

The X/Q value that is equalled or exceeded 0.5 percent of the time at each sector percent distance combination is then determined from the intersection of the envelope and the 0.5 percent probability level. The highest sector dependent X/Q value is then compared with the "overall" 5 percent accident X/Q value. The highest value represents the conservative 2-hour accident X/Q. The realistic 2-hour accident X/Q is evaluated at the 50 percent probability level. The X/Q value that is equalled or exceeded 50 percent of the time at each sector distance combination is determined from the intersection of the envelope and the normalized (probability normalized to 100 percent in each sector) 50 percent probability level. The highest sector dependent X/Q value is then compared with the "overall" 50 percent accident X/Q. The highest value represents the realistic 2-hour accident X/Q.

The overall 5 percent and 50 percent X/Q values are determined by summing the sixteen sector dependent X/Q distributions for each distance into a cumulative frequency distribution representing all sectors and again enveloping the data points. The 5 percent and 50 percent values are determined by the intersection of the envelope with the 5 percent and 50 percent probability levels, respectively.

The X/Q accident values for time periods of up to 30 days following an accident are derived by logarithmic interpolation between the 2-hour 0.5 percent and 50 percent accident X/Q values, and the annual accident X/Q value at each sector distance combination. The intermediate time periods for the overall 5 percent and 50 percent X/Q values are determined by logarithmic interpolation between the overall 2-hour 5 percent and 50 percent X/Q values and the maximum

X/Q values and the maximum annual X/Q. The maximum X/Q value for a given five distance is the maximum sector 0.5 percent X/Q, or the overall 5 percent X/Q, whichever is higher, for the conservative assessment. The realistic assessment compares the maximum sector 50 percent X/Q and the overall 50 percent X/Q. The higher X/Q value is chosen again.

2.3.4.2.2 Meteorological Data

2.3.4.2.2.1 Representativeness

The Artificial Island meteorological tower data from January 1977 through December 1981 are employed in the accident assessment. The data collected at the tower are representative of the meteorological conditions under which effluents are released, since both are located on the Delaware River shoreline. Furthermore, the proximity of the 300-foot tower to HCGS ensures that the data are representative of the conditions used in an accident evaluation.

2.3.4.2.2.2 Joint Frequency Distributions

Joint frequency distributions of wind speed and direction by atmospheric stability class are used as input to the diffusion calculations. Wind speed and direction data from the 33-foot level are used in the assessment of diffusion for the ground level releases.

Atmospheric stability is determined for the 33-foot distributions by the vertical temperature difference between the 300 and 33-foot levels. Joint frequency distributions of wind speed and direction by atmospheric stability class are computed for 22.5° sector using the wind speed groups and atmospheric stability classes suggested in Regulatory Guide 1.23. The 5-year frequency distributions are shown in Section 2.3.2.1, and in Reference 2.3-31, Table A-1 for the 33-foot level.

With the exception of the calm and 25+ mph wind speed groups, the highest wind speed in each group is used to represent that group in the diffusion calculations. For conservatism, a wind speed of 0.5 mph is used to represent calms at the 33-foot level. This value represents a conservative threshold wind speed for the 33-foot wind instrumentation. Due to the high wind speeds associated with this site, a wind speed of 30 mph is used to represent the 25+ mph wind speed group.

2.3.4.3 Atmospheric Diffusion Model

The Reactor Building vent is treated as a ground level source for both short term and long term calculations. This implies that no plume rise is calculated and no terrain corrections are applied. A building wake correction factor is used, in accordance with the methodology discussed in Regulatory Guide 1.145 for vent releases. The building wake correction factor takes into account the initial mixing of the plume within the building cavity.

The vent release X/Q values are calculated with the following equations from Regulatory Guide 1.145:

$$X/Q = \frac{1}{U_{10}\ (\pi S_Y\ S_Z + A/2)} \qquad (2.3\text{-}2)$$

$$X/Q = \frac{1}{U_{10}\ (3\pi S_Y\ \ S_Z)} \qquad (2.3\text{-}3)$$

$$X/Q = \frac{1}{U_{10}\pi \Sigma_Y\ S_Z\ )} \qquad (2.3\text{-}4)$$

where:

X/Q = relative concentration, s/m^3

U_{10} = wind speed at 10 m level, m/s

S_Y - lateral plume speed, m

Σ_y - lateral plume spread with meander and building wake effects, m

S_Z - vertical plume spread, m

A - smallest vertical plane cross sectional area of the reactor building, and adjacent structures m^2.

A building wake correction factor (A/2) of 2915 m^2 is used for calculations of the short term X/Q.

The calculation of the "A" term in Equation 2.3-2 is based on Figure 2.3-7. The "smallest vertical-plane cross-sectional area," also happens to be the orientation of the station with respect to the minimum exclusion distance boundary formed by land (Section 2.1.2.1). A calculation of the Reactor Building cross-sectional area only, yields an area of 3341 m^2. If the service and radwaste area of the Auxiliary Building are added, this yields an additional 646 m^2. It should also be noted that a different building wake correction factor is used in calculating the long term X/Q, due to the different assumptions inherent in the guidance in Regulatory Guide 1.111. The smaller building wake correction factor used in this short term calculation is more conservative. As can be seen from Figure 2.3-7 the Auxiliary Building and Turbine Building are contiguous adjacent structures.

It must be pointed out however, that at 901 m exclusion area boundary where the critical X/Q occurs, the "A" term is of small importance. The reason for this is as follows. Under unstable conditions, the Pi, Sy, and Sz terms dominate the denominator of the calculations (Equation 2.3-2). Under stable conditions and low wind speeds Equation 2.3-4 becomes the dominate equation. The Turbine Building contributes 1843 m^2. The total smallest vertical-plane cross-sectional area "A" is 5830 m^2. The above reasoning supports the use of 1.9E-4 sec/m^3 as the short term exclusion area boundary.

For neutral or stable conditions combined with wind speeds less than 6.0 m/s, calculations of X/Q values are made using Equation 2.3-4. For all other meteorological conditions, X/Q values are calculated using Equations 2.3-2 and 2.3-3.

The values computed from Equations 2.3-2 and 2.3-3 are compared, and the higher value is selected. For neutral and stable conditions with a wind speed less than 6 m/s, the value from Equation 2.3-4 is compared with the value chosen from Equations 2.3-2 and 2.3-3, and the lower value is chosen to represent these conditions.

2.3.4.4 Diffusion Estimates

2.3.4.4.1 Exclusion Area Boundary

The maximum conservative 2-hour X/Q at the EAB, 0.56 miles from the Reactor Building vent, is 1.9×10^{-4} s/m^3. This is the overall 5 percent value at this distance. This value is larger than each of the 16 sector dependent X/Q values. The maximum realistic (50 percent) 2-hour X/Q at the EAB is 6.3×10^{-5} s/m^3. This is the normalized 50 percent X/Q value in the WNW sector. Conservative and realistic X/Q values for the EAB (0.56 miles) are given in Table 2.3-30. Conservative 2-hour X/Q values at the LPZ are given in Table 2.3-30a.

2.3.4.4.2 Low Population Zone

The maximum conservative and realistic X/Q values, 0.5 percent and 50 percent, respectively, given in Table 2.3-30 represent the maximum X/Q values (sector value used if greater than the overall value) for the Reactor Building vent at the LPZ boundary, 5.0 miles.

2.3.5 Long Term (Routine) Diffusion Estimates

2.3.5.1 Objective

The objective is to provide realistic annual average estimates of relative concentration (X/Q), relative concentration depleted by deposition (depleted X/Q), and relative deposition per unit area (D/Q) at appropriate distances from all routine gaseous releases of radioactive materials from HCGS. The assessment is made with the use of an atmospheric model.

2.3.5.2 X/Q and D/Q Estimates

Radionuclides are routinely emitted to the atmosphere from two locations at HCGS. They are the south and north vents, located adjacent to the turbine buildings. Estimates of annual average X/Q, depleted by deposition X/Q, and D/Q have been made for receptor locations out to 50 miles in each of 16 radial sectors. These annual average values are presented in the following tables for compliance with 10CFR50, Appendix I:

1. Table 2.3-31 - vent ground level release - X/Q

2. Table 2.3-32 - vent ground level release - depleted X/A

3. Table 2.3-33 - vent ground level release - D/Q

2.3.5.3 Methodology

The analysis of the atmospheric transport and diffusion properties is based on the onsite meteorological data, the source configuration, the terrain, and a sector average diffusion model.

2.3.5.3.1 Meteorological Input

Joint frequency distributions of wind speed and direction by atmospheric stability class are used for the diffusion calculations.

The meteorological tower is located approximately 1.0 mile southeast of HCGS. All meteorological data are from the Artificial Island meteorological tower. The flat, uncomplicated terrain that surrounds the site for a considerable distance in every direction, ensures excellent representation of the regional airflow measured by the Artificial Island meteorological tower.

Wind speed and direction data from the 33-foot tower level are used as input for the joint frequency distributions.

Joint frequency distributions of wind speed and direction by atmospheric stability class are computed for 22.5° sectors using the wind speed groups and atmospheric stability classes suggested in Regulatory Guide 1.23. The 8-year joint frequency distributions of wind direction, speed, and stability from the 33-foot level are used as input for both vents.

With the exception of the calm and the 25+ mph groups, the median speed from each wind speed group is used to represent the group in the diffusion calculations. For conservatism, a wind speed of 0.38 mph, equal to one-half of the highest threshold of the vane and propeller is assigned to the calms. A wind speed of 26 mph is used to represent the 25+ mph group.

2.3.5.3.2 Source Configuration

Radionuclides are routinely released from two sources, the south and north vents. Their source characteristics are given as follows:

Parameter	South Vent	North Vent
Height above grade, m	35.05	35.05
Exit diameter; m,the equivalent circ diam for rectangular vents	4.13	2.23
Exit velocity m/s		
Summer (Apr - Sept)	15.54	5.08
Winter (Oct - Mar)	10.82	5.08

Both vents, pointing upward, are adjacent to the tops of the turbine buildings, below the level of the reactor containment dome. Therefore, the vents are affected by the nearby building aerodynamics with moderate to strong winds.

The release is assumed to be a ground level, and a building wake correction factor (Reactor Building height squared) of 3819 m^2 is used in accordance with the methodology of Regulatory Guide 1.111, Revision 1. The building wake correction factor takes into consideration the initial mixing of the plume within the building cavity.

Regulatory Guide 1.111 states "For effluents released from points less than the height of adjacent solid structures, a ground level release should be assumed" (Reference 2.3-48).

The exit velocities for the south plant vent are significantly higher than those for the north plant vent. The assumption made in Regulatory Guide 1.111 (Revision 1) about the height of adjacent solid structures (i.e., Reactor Building dome) is simplistic in the case of effluents released at high exit velocities from vents oriented upward (versus horizontal). With the consideration of the Reactor Building dome as an adjacent structure, the projected effluent path becomes complicated because the transport wind and associated entrainment will be sector dependent. Therefore, the X/Q, depleted X/Q and D/Q values (Tables 2.3-31, 2.3-32, and 2.3-33) are conservative when based on a ground level release.

2.3.5.3.2.1 Site Impact on Vent Releases

The final consideration of the source configuration is to determine the effects, if any, of the natural draft cooling tower on the effluent released from the two vents. The natural draft cooling tower is located approximately 1250 feet northeast of the south vent and 920 feet northeast of the north vent. The physical dimensions of the natural draft cooling tower are:

1. Height above grade - 156.1 m

2. Base diameter - 130.8 m

3. Throat diameter - 75.9 m

4. Exit diameter - 82.6 m.

Field data obtained at Rancho Seco, especially during stable conditions, were used to determine the flow perturbations generated by natural draft cooling towers. The report, noted in Reference 2.3-50, states, "The overall interpretation of ground level concentrations, i.e., crosswind integrated concentrations and sigma-y values, are probably not severely distorted even when the observations are influenced by the by the cooling tower wakes,".

Thus, the effects of the natural draft cooling tower for the vent releases during stable conditions are neglected.

The effect of the cooling tower on the relatively low level vent releases during neutral and unstable atmospheric conditions would be to enhance the vertical diffusion through increased mechanical turbulence and thus reduce ground level concentrations. Therefore, to be conservative in the estimation of ground level concentrations for neutral and unstable conditions, the wake effect of the cooling tower has been neglected.

2.3.5.3.3 Diffusion Model

The sector average Gaussian plume equation, as expressed in Regulatory Guide 1.111, Revision 1, is used for all X/Q calculations.

The straight line Gaussian diffusion model did not need to be modified to calculate long term annual relative concentration values for the 80-kilometer (50-mile) region surrounding the HCGS site.

Consideration of temporal and spatial airflow changes in the site vicinity would insignificantly alter the long term diffusion estimates. In addition, the lack of local mesoscale circulations in the site region during the summer months eliminates the necessity of modifying the straight line diffusion model.

NRC Regulatory Guide 1.111 recognizes three basic situations that would require the consideration of temporal and spatial airflow changes (Reference 2.3-48). These are: 1) recirculation of airflow during periods of prolonged atmospheric stagnation at inland sites located in open terrain; 2) valley airflows at sites located in pronounced river valleys; and 3) sea or lake breeze flows at sites located along coasts of large bodies of water.

Recirculation of airflow during periods of prolonged atmospheric stagnation seldom occurs at the HCGS site. The airflow in the region is dominated by large-scale meteorological patterns. There are no terrain-induced alterations in the airflow since the region is extremely flat and uniform out to a distance of ten miles in all directions. Past ten miles, the topography is either flat or gently rolling. An analysis of atmospheric stagnation by Korshover has found between 150 and 175 stagnation days in the region during the 40-year period of 1936 through 1975 (Reference 2.3-10). Consequently, there is an average of only four cases of stagnating conditions occurring annually in the region. This agrees with Holzworth's estimate of an annual frequency of four stagnation periods (Reference 2.3-18).

The annual average diffusion estimates, which are based upon climatology, would not be significantly altered by modifying the straight line Gaussian equation to attempt to simulate these infrequent air stagnation events.

The second consideration cited in Regulatory Guide 1.111 applies to sites located in pronounced river valleys. While the HCGS site is located on the shore of the Delaware River, the river "valley" is extremely flat and open in this area. Typical valley airflows that are associated with sharp "V-shaped" river valleys do not occur. The marshy land areas bordering the water are only slightly higher than the river level in the region.

The third situation in which Regulatory Guide 1.111 states that spatial and temporal airflow variations may need additional considerations concerns coastal locations. The HCGS site is located on a man-made island in the Delaware River. It is located at a point where the river gradually widens into the Delaware Bay. From the site northward, the river is less than five kilometers (three miles) wide. South of the site, the river opens up into the bay, which eventually empties into the Atlantic Ocean, approximately 72 kilometers (45 miles) to the south-southeast. Since this site is not located on the coastline of a large body of water, such as an ocean or the Great Lakes, it should not be considered a coastal location. The site is not subject to the frequent sea-breeze mesoscale circulations commonly observed at coastal locations, that arise from the differential heating of the land and water surfaces.

A PSE&G study showed that sea breeze regimes that were often present at regional sites directly on the Atlantic Ocean generally do not affect the HCGS area (Reference 2.3-52). There was no evidence of substantial alteration of the synoptic airflow or of closed mesoscale circulations at the site. Additionally, the meteorological characteristics of the air flowing over the site from the waters of the Delaware Bay to the south-southeast are not significantly altered by passing over the site. Artificial Island is small, marshy and flat, and only five kilometers (three miles) in length and 2.5 kilometers (1.5 miles) wide at the widest point. Airflows originating from directions other than south-southeast are not significantly affected because of their short over-water fetch. The PSE&G study concluded that the present meteorological tower location allows adequate and representative measurements of the airflows and atmospheric stability required to simulate atmospheric dispersion in the region.

2.3.5.3.4 Terrain Corrections

Changes in terrain elevation, though very small in the immediate vicinity of the plant, have been applied at each receptor. Terrain heights above plant grade, which is 4 meters mean sea level (MSL), are used in the calculations, where applicable. The terrain height correction applied to any particular receptor is the highest terrain between the source and the receptor.

2.3.5.3.5 Atmospheric Stability

Atmospheric stability classes are determined using the vertical temperature difference between the 300 and the 33-foot levels of the Artificial Island tower. The seven lapse rate classes are those recommended in Regulatory Guide 1.23 for stability classification.

2.3.5.3.6 Dispersion Coefficients

The horizontal and vertical dispersion coefficients, sigma y and sigma z for each turbulence class, are computed using analytical approximations to the P-G sigma curves given in Regulatory Guide 1.111 Revision 1. These dispersion coefficients were developed for flat to rolling terrain, similar to that surrounding the Hope Creek site.

2.3.5.3.7 Dry Deposition

2.3.5.3.7.1 Depleted X/Q Values

Relative depletion by dry deposition has been estimated in accordance with Regulatory Guide 1.111, Revision 1. The depleted by deposition X/Q values are obtained from the X/Q values by multiplying the X/Q values by the fraction remaining in the plume. These fractions are determined from Regulatory Guide 1.111, Revision 1, Figures 2 through 5.

2.3.5.3.7.2 D/Q Values

Relative dry deposition has been estimated in accordance with Regulatory Guide 1.111, Revision 1. The relative deposition per unit area, D/Q, is obtained by:

1. Determining the relative deposition rate at each receptor, which is a function of distance from the source, source height, and atmospheric stability. This rate is obtained for Regulatory Guide 1.111, Revision 1, ground releases.

2. Multiplying the relative deposition rate by the fraction of the release transported into the sector.

3. Taking this product and dividing by the appropriate crosswind distance, which is the arc length of the sector at the point being considered.

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TABLE 2.3-1

PERCENTAGE OF DAYS WITH VARIOUS HYDROMETERS

DOVER DELAWARE AIR FORCE BASE

1942-1965

Month	Fog	Snow and/or Sleet	Hail	Thunderstorms
Jan	43.7	4.1	0.4	0.6
Feb	45.0	3.4	0.2	0.9
Mar	48.4	2.7	-	3.7
Apr	44.4	0.3	0.2	8.9
May	49.0	-	0.9	16.6
Jun	55.3	-	0.4	17.1
Jul	54.3	-	0.2	19.6
Aug	66.3	-	-	17.4
Sept	59.0	-	-	6.8
Oct	53.8	-	0.2	3.0
Nov	47.6	0.6	0.2	1.2
Dec	44.5	2.5	0.2	0.5
Annual	51.2	1.2	0.3	8.2

TABLE 2.3-2

SNOWFALL

(inches)

PHILADELPHIA INTERNATIONAL AIRPORT

Month	Mean	Monthly Maximum
Jan	5.7	19.7
Feb	6.1	18.4
Mar	4.1	13.4
Apr	0.3	4.3
May	T(1)	T(1)
Jun	-	-
Jul	-	-
Aug	-	-
Sept	-	-
Oct	T(1)	T(1)
Nov	0.8	8.8
Dec	4.6	18.8
Annual	21.6	
Length of record (yr)	28	

(1) Trace of precipitation.

TABLE 2.3-3

SNOWFALL

(inches)

TRENTON INTERNATIONAL AIRPORT

Month	Mean	Monthly Maximum	24-Hour Maximum
Jan	5.8	16.1	10.1
Feb	6.7	23.1	13.0
Mar	4.4	21.5	14.3
Apr	0.4	4.2	4.2
May	T[1]	T[1]	T[1]
Jun	-	-	-
Jul	-	-	-
Aug	-	-	-
Sep	-	-	-
Oct	0.1	1.6	1.6
Nov	1.0	13.0	7.7
Dec	4.9	21.5	16.6
Annual	23.3		
Length of record (yr)	34		

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(1) Trace of precipitation.

TABLE 2.3-4

DATA AVAILABILITY FOR ONSITE METEOROLOGICAL PARAMETERS
JANUARY 1977 - DECEMBER 1981

Meteorological Parameter	Height Above Tower Grade, ft	Data Availability, percent
Wind direction	33	92.4
	150	97.3
	300	97.4
Wind speed	33	96.1
	150	97.3
	300	97.4
Temperature difference	150 to 33	90.4
	300 to 33	93.3
Temperature	33	94.0
Dew point	33	83.2
Precipitation	7	91.2
Pressure	3	99.9

TABLE 2.3-5

COMPARISON OF ANNUAL ONSITE WIND DIRECTION FREQUENCY DISTRIBUTIONS
JANUARY 1977 - DECEMBER 1981

Directional Sector	Wind Instrument Height Above Tower Grade 33 ft, percent	150 ft, percent	300 ft, percent
N	6.8	6.3	6.0
NNE	5.2	4.9	4.6
NE	4.8	5.2	4.6
ENE	3.1	3.1	3.1
E	2.8	2.8	2.7
ESE	2.6	2.1	1.9
SE	8.4	6.5	5.6
SSE	6.9	7.1	6.8
S	6.4	6.9	7.1
SSW	5.8	6.0	6.5
SW	6.0	6.7	7.4
WSW	5.8	5.7	5.7
W	7.8	8.5	9.2
WNW	9.5	9.2	8.9
NW	11.6	12.1	12.2
NNW	6.5	7.2	7.8
Calm	0.3	<0.1	0.1

TABLE 2.3-6

COMPARISON OF ANNUAL ONSITE WITH WILMINGTON NWS WIND DIRECTION FREQUENCY DISTRIBUTIONS

	5-Year Concurrent Period[1]		Long Term Wilmington NWS
Directional Sector	Onsite 33 feet, percent	NWS 20 feet, percent	1972 - 1981 20 feet, percent
N	6.8	7.6	7.4
NNE	5.2	2.5	2.8
NE	4.8	3.8	3.9
ENE	3.1	4.1	4.7
E	2.8	4.0	4.0
ESE	2.6	1.8	2.0
SE	8.4	3.6	3.2
SSE	6.9	4.4	4.5
S	6.4	10.1	10.2
SSW	5.8	4.0	4.2
SW	6.0	6.1	5.9
WSW	5.8	5.8	6.1
W	7.8	10.0	9.6
WNW	9.5	11.2	10.8
NW	11.6	14.9	13.6
NNW	6.5	6.4	7.1
Calm	0.3	6.0	5.6

(1) January 1977 to December 1981.

TABLE 2.3-7

ONSITE COMPARISON OF AVERAGE WIND SPEEDS
JANUARY 1977 - DECEMBER 1981

	Wind Instrument Height Above Tower Grade		
Combined Months	33 ft, mph	150 ft, mph	300 ft, mph
Jan	9.9	14.8	17.2
Feb	9.7	14.1	16.8
Mar	10.5	15.4	18.1
Apr	9.8	14.2	16.5
May	8.5	12.1	14.0
Jun	8.6	11.9	13.7
Jul	7.9	10.5	12.4
Aug	7.8	10.2	11.9
Sep	8.3	11.4	13.7
Oct	8.6	12.7	15.1
Nov	8.9	13.5	16.1
Dec	8.8	13.9	16.3
Annual	8.9	12.9	15.1

TABLE 2.3-8

WILMINGTON NATIONAL WEATHER SERVICE AVERAGE WIND SPEEDS (1) (2)

	Period of Record	
Months	1977-1981, mph	1972-1981, mph
Jan	11.4	10.4
Feb	10.8	10.7
Mar	11.4	11.4
Apr	10.6	10.6
May	9.1	9.2
Jun	8.8	8.9
Jul	8.2	8.1
Aug	7.7	7.7
Sep	8.4	8.2
Oct	8.8	8.6
Nov	9.8	9.6
Dec	10.0	9.9
Annual	9.6	9.4

(1) U.S. Department of Commerce, "Wind Direction by Pasquill Stability Classes, STAR Program, Wilmington, Delaware," NOAA, Environmental Data Services, National Climatic Center, 1982.

(2) Location of wind speed sensor height is at 20 feet above ground level.

TABLE 2.3-9

ONSITE TEMPERATURE MEANS AND EXTREMES
JANUARY 1977 - DECEMBER 1981

Combined Months	Temperature, $^{o}C^{(1)}$ Mean	Hourly Maximum	Hourly Minimum
Jan	-2.1	15.5	-18.5
Feb	-0.9	16.5	-18.5
Mar	5.4	24.5	-13.0
Apr	11.6	29.8	-2.6
May	16.6	29.0	4.0
Jun	20.8	32.0	9.5
Jul	23.8	34.5	12.5
Aug	23.6	32.5	12.0
Sep	19.7	31.5	7.0
Oct	12.4	24.5	0.5
Nov	7.9	21.5	-4.0
Dec	2.5	19.5	-15.5
Annual	11.7	34.5	-18.5

(1) Temperature is measured at the 33-foot level.

TABLE 2.3-10

ONSITE HOURLY TEMPERATURE FREQUENCY DISTRIBUTIONS
JANUARY 1977 - DECEMBER 1981

Temperature[1] °C	Jan Sum	Jan Percent	Feb Sum	Feb Percent	Mar Sum	Mar Percent	Apr Sum	Apr Percent	May Sum	May Percent	June Sum	June Percent
32.5 - 34.9	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0
30.0 - 32.4	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	30	0.9
27.5 - 29.9	0	0.0	0	0.0	0	0.0	11	0.3	43	1.2	129	4.0
25.0 - 27.4	0	0.0	0	0.0	0	0.0	20	0.6	121	3.5	332	10.2
22.5 - 24.9	0	0.0	0	0.0	16	0.4	41	1.2	205	5.9	643	19.8
20.0 - 22.4	0	0.0	0	0.0	18	0.5	114	3.3	471	13.6	917	28.3
17.5 - 19.9	0	0.0	0	0.0	30	0.8	226	6.4	704	20.3	592	18.3
15.0 - 17.4	6	0.2	12	0.4	80	2.2	379	10.8	758	21.8	386	11.9
12.5 - 14.9	13	0.4	31	0.9	202	5.5	647	18.5	529	15.2	165	5.1
10.0 - 12.4	30	0.8	89	2.7	320	8.8	821	23.4	403	11.6	46	1.4
7.5 - 9.9	67	1.9	194	5.8	620	17.0	653	18.6	166	4.8	1	0.0
5.0 - 7.4	181	5.1	265	8.0	737	20.2	424	12.1	67	1.9	0	0.0
2.5 - 4.9	389	11.0	379	11.4	655	17.9	105	3.0	5	0.1	0	0.0
0.0 - 2.4	607	17.1	440	13.3	498	13.6	55	1.6	0	0.0	0	0.0
-2.5 - -0.1	638	18.0	562	16.9	248	6.8	7	0.2	0	0.0	0	0.0
-5.0 - -2.6	681	19.2	598	18.0	145	4.0	2	0.1	0	0.0	0	0.0
-7.5 - -5.1	520	14.7	388	11.7	33	0.9	0	0.0	0	0.0	0	0.0
-10.0 - -7.6	219	6.2	182	5.5	32	0.9	0	0.0	0	0.0	0	0.0
-12.5 - -10.1	129	3.6	117	3.5	15	0.4	0	0.0	0	0.0	0	0.0
-15.0 - -12.6	49	1.4	50	1.5	3	0.0	0	0.0	0	0.0	0	0.0
-17.5 - -15.1	11	0.3	7	0.2	0	0.0	0	0.0	0	0.0	0	0.0
-20.0 - -17.6	7	0.2	5	0.2	0	0.0	0	0.0	0	0.0	0	0.0
Total	3,547	100.0	3,319	100.0	3,652	100.0	3,505	100.0	3,472	100.0	3,241	100.0

Month (January to June)

TABLE 2.3-10 (Cont)

Temperature[1] °C			Month (July to December)													
			Jul		Aug		Sep		Oct		Nov		Dec		Annual	
			Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent
32.5	-	34.9	52	1.5	6	0.2	0	0.0	0	0.0	0	0.0	0	0.0	58	0.1
30.5	-	32.4	136	3.9	137	4.0	13	0.4	0	0.0	0	0.0	0	0.0	316	0.8
27.5	-	29.9	381	11.0	366	10.7	79	2.4	0	0.0	0	0.0	0	0.0	1,009	2.4
25.0	-	27.4	791	22.9	797	23.2	264	8.1	0	0.0	0	0.0	0	0.0	2,325	5.6
22.5	-	24.9	957	27.7	991	28.9	555	17.1	36	1.0	0	0.0	0	0.0	3,444	8.4
20.0	-	22.4	684	19.8	616	18.0	776	23.9	131	3.7	7	0.2	0	0.0	3,734	9.1
17.5	-	19.9	299	8.7	332	9.7	640	19.7	303	8.6	85	2.6	12	0.3	3,223	7.8
15.0	-	17.4	117	3.4	149	4.3	551	17.0	523	14.9	207	6.3	32	0.9	3,200	7.8
12.5	-	14.9	36	1.0	35	1.0	234	7.2	805	22.9	416	12.6	46	1.3	3,159	7.7
10.0	-	12.4	0	0.0	2	0.1	103	3.2	681	19.4	449	13.6	126	3.6	3,070	7.4
7.5	-	9.9	0	0.0	0	0.0	31	1.0	641	18.2	595	18.0	336	9.5	3,304	8.0
5.0	-	7.4	0	0.0	0	0.0	3	0.1	298	8.5	628	19.0	616	17.5	3,219	7.8
2.5	-	4.9	0	0.0	0	0.0	0	0.0	93	2.6	513	15.5	687	19.5	2,826	6.9
0.0	-	2.4	0	0.0	0	0.0	0	0.0	7	0.2	293	8.9	664	18.9	2,564	6.2
-2.5	-	-0.1	0	0.0	0	0.0	0	0.0	0	0.0	94	2.8	495	14.1	2,044	5.0
-5.0	-	-2.6	0	0.0	0	0.0	0	0.0	0	0.0	14	0.4	272	7.7	1,712	4.2
-7.5	-	-5.1	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	160	4.5	1,101	2.7
-10.0	-	-7.6	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	47	1.3	480	1.2
-12.5	-	-10.1	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	20	0.6	281	0.7
-15.0	-	-12.6	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	6	0.2	108	0.3
-17.5	-	-15.1	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	2	0.1	20	0.0
-20.0	-	-17.6	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	12	0.0
Total			3,453	100.0	3,431	100.0	3,249	100.0	3,518	100.0	3,301	100.0	3,521	100.0	41,209	100.0

(1) Temperature is measured at the 33-foot level.

TABLE 2.3-11

ONSITE DIURNAL TEMPERATURE VARIATIONS
JANUARY 1977 - DECEMBER 1981

Hour	Temperature, ^oC (1)												
Hour	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1	-2.6	-1.9	4.2	10.0	14.9	19.2	22.1	22.1	18.4	11.3	7.0	1.8	10.4
2	-2.8	-2.2	3.8	9.6	14.5	18.8	21.8	21.8	18.1	11.0	6.8	1.7	10.1
3	-2.9	-2.4	3.6	9.3	14.1	18.5	21.5	21.5	17.8	10.7	6.6	1.5	9.9
4	-3.1	-2.6	3.4	9.0	13.8	18.2	21.2	21.3	17.6	10.5	6.5	1.3	9.7
5	-3.2	-2.8	3.2	8.8	13.6	18.0	21.0	21.0	17.4	10.3	6.3	1.2	9.5
6	-3.4	-3.0	2.9	8.6	13.5	17.9	20.9	20.9	17.2	9.9	6.3	1.0	9.3
7	-3.6	-3.1	2.7	8.9	13.8	18.2	21.1	20.9	17.1	9.8	6.2	0.9	9.3
8	-3.6	-3.0	3.2	9.9	14.6	18.9	21.8	21.4	17.4	9.8	6.2	0.8	9.7
9	-3.2	-2.4	4.2	11.0	15.5	19.8	22.8	22.4	18.2	10.5	6.9	1.1	10.5
10	-2.6	-1.5	5.2	12.0	16.6	20.8	23.9	23.5	19.4	11.8	7.9	1.9	11.4
11	-1.9	-0.6	6.1	12.8	17.6	21.8	24.9	24.4	20.5	13.0	8.8	2.8	12.4
12	-1.3	0.3	6.8	13.5	18.4	22.6	25.5	25.2	21.3	14.0	9.5	3.5	13.2
13	-1.0	1.0	7.5	14.1	19.2	23.1	26.1	25.8	21.9	14.7	10.0	4.1	13.8
14	-0.6	1.4	8.1	14.4	19.7	23.6	26.5	26.2	22.4	15.1	10.5	4.4	14.2
15	0.2	1.6	8.5	14.7	19.9	23.9	26.8	26.5	22.7	15.3	10.5	4.6	14.5
16	0.2	1.5	8.4	14.7	19.8	23.7	26.7	26.5	22.8	15.3	10.3	4.4	14.4
17	0.4	1.4	8.1	14.4	19.5	23.5	26.6	26.2	22.6	14.9	9.6	4.0	14.1
18	-1.0	0.8	7.4	13.9	19.1	23.3	26.3	25.8	22.1	14.4	9.0	3.5	13.6
19	-1.3	0.2	6.6	13.1	18.3	22.6	25.7	25.2	21.3	13.7	8.5	3.2	13.0
20	-1.6	-0.2	6.1	12.3	17.5	21.9	24.9	24.5	20.5	13.0	8.1	2.9	12.4
21	-1.9	-0.6	5.6	11.6	16.9	21.0	24.1	23.8	19.9	12.4	7.7	2.6	11.8
22	-2.2	-0.9	5.3	11.2	16.3	20.5	23.6	23.3	19.4	12.0	7.5	2.4	11.4
23	-2.4	-1.2	5.0	10.8	15.8	20.1	23.0	22.9	18.9	11.6	7.2	2.1	11.0
24	-2.5	-1.4	4.7	10.4	15.5	19.7	22.6	22.5	18.6	11.3	7.0	2.0	10.7

(1) Temperature is measured at the 33-foot level.

TABLE 2.3-12

TEMPERATURE MEANS AND EXTREMES

JANUARY 1977 - DECEMBER 1981

Combined Months	Artificial Island, °F(1) Mean	Artificial Island, °F(1) Hourly Maximum	Artificial Island, °F(1) Hourly Minimum	Wilmington NWS Station, °F(2) Mean	Wilmington NWS Station, °F(2) Hourly Maximum	Wilmington NWS Station, °F(2) Hourly Minimum
Jan	28.2	60	-1	27.4	64	-3
Feb	30.4	62	-1	29.2	69	-6
Mar	41.7	76	9	42.1	82	7
Apr	52.9	86	27	53.0	87	25
May	61.9	84	39	63.1	91	30
Jun	69.4	90	49	69.7	94	44
Jul	74.8	94	55	76.0	98	50
Aug	74.5	91	54	75.8	95	49
Sep	67.5	89	45	68.5	98	40
Oct	54.3	76	33	54.1	82	30
Nov	46.2	71	25	46.3	76	22
Dec	36.5	67	-4	35.2	70	2
Annual	53.1	94	-1	53.4	98	-6

(1) Sensor height 33 feet.

(2) Source, U.S. Department of Commerce, "Local Climatological Data Annual Summary with Comparative Data - Wilmington, Delaware," NOAA, Environmental Data Service, 1977-1981.

TABLE 2.3-13

WILMINGTON NWS TEMPERATURE MEANS AND EXTREMES[1]

Months	Temperature, °F		
	Mean[2]	Maximum[3]	Minimum[3]
Jan	32.0	75.0	-4.0
Feb	33.6	74.0	-6.0
Mar	41.6	86.0	-6.0
Apr	52.3	91.0	22.0
May	62.4	95.0	30.0
Jun	71.4	99.0	41.0
Jul	75.8	102.0	50.0
Aug	74.1	101.0	46.0
Sep	67.9	100.0	36.0
Oct	57.2	91.0	24.0
Nov	45.7	85.0	14.0
Dec	34.7	72.0	-2.0
Annual	54.0	102.0	-6.0

(1) Source, U.S. Department of Commerce, "Local Climatological Data Annual Summary with Comparative Data - Wilmington, Delaware," NOAA, Environmental Data Service, 1977-1981.

(2) Period of record 1941 to 1970.

(3) Period of record 1948 to 1981.

TABLE 2.3-14

ONSITE DEW POINT TEMPERATURE MEANS AND EXTREMES
JANUARY 1977 - DECEMBER 1981

Combined Months	Absolute Humidity, g/m^3 Mean	Hourly Maximum	Hourly Minimum
Jan	-7.5	12.8	-23.4
Feb	-6.6	12.5	-20.5
Mar	-1.5	14.7	-21.7
Apr	3.6	18.3	-13.1
May	10.4	21.1	-12.2
Jun	14.6	24.7	-1.6
Jul	18.3	26.7	4.4
Aug	18.8	28.4	6.1
Sep	14.4	26.1	-1.1
Oct	6.9	19.7	-5.6
Nov	2.3	18.3	-13.3
Dec	-3.6	15.8	-24.7
Annual	5.1	28.4	-24.7

TABLE 2.3-15

ONSITE HOURLY DEW POINT TEMPERATURE FREQUENCY DISTRIBUTIONS
JANUARY 1977 - DECEMBER 1981

Dew Point Temperature, °C	Month (January to June)											
	Jan		Feb		Mar		Apr		May		June	
	Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent
27.5 - 29.9	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0
25.0 - 27.4	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0
22.5 - 24.9	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	51	1.5
20.0 - 22.4	0	0.0	0	0.0	0	0.0	0	0.0	39	1.3	428	12.8
17.5 - 19.9	0	0.0	0	0.0	0	0.0	1	0.0	332	10.9	672	20.0
15.0 - 17.4	0	0.0	0	0.0	0	0.0	68	1.9	473	15.6	662	19.7
12.5 - 14.9	2	0.1	1	0.0	47	1.3	251	7.2	445	14.7	527	15.7
10.0 - 12.4	34	0.9	36	1.2	94	2.6	325	9.3	434	14.3	436	13.0
7.5 - 9.9	48	1.3	86	3.0	257	7.1	411	11.8	386	12.7	260	7.8
5.0 - 7.4	70	1.9	108	3.7	314	8.6	489	14.0	330	10.9	171	5.1
2.5 - 4.9	121	3.3	126	4.3	377	10.3	467	13.4	251	8.3	98	2.9
0.0 - 2.4	162	4.4	159	5.5	409	11.2	484	13.8	185	6.1	42	1.3
-2.5 - -0.1	334	9.2	286	9.9	614	16.9	311	8.9	97	3.2	7	0.2
-5.0 - -2.6	550	15.1	300	10.3	444	12.2	330	9.4	45	1.5	0	0.0
-7.5 - -5.1	486	13.3	318	11.0	397	10.9	203	5.8	10	0.3	0	0.0
-10.0 - -7.6	484	13.3	455	15.7	272	7.5	108	3.1	4	0.1	0	0.0
-12.5 - -10.1	519	14.2	468	16.1	231	6.3	43	1.2	2	0.1	0	0.0
-15.0 - -12.6	427	11.7	301	10.4	113	3.1	4	0.1	0	0.0	0	0.0
-17.5 - -15.1	259	7.1	162	5.6	44	1.2	0	0.0	0	0.0	0	0.0
-20.0 - -17.6	104	2.9	95	3.3	22	0.6	0	0.0	0	0.0	0	0.0
-22.5 - -20.1	45	1.2	2	0.1	8	0.2	0	0.0	0	0.0	0	0.0
-25.0 - -22.6	4	0.1	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0
Total	3,649	100.0	2,903	100.0	3,643	100.0	3,495	100.0	3,033	100.0	3,354	100.0

TABLE 2.3-15 (Cont)

Dew Point Temperature, °C	Month (July to December)													
	Jul		Aug		Sep		Oct		Nov		Dec		Annual	
	Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent
27.5 to 29.9	0	0.0	3	0.1	0	0.0	0	0.0	0	0.0	0	0.0	3	0.0
25.0 to 27.4	24	0.9	35	1.4	8	0.3	0	0.0	0	0.0	0	0.0	67	0.2
22.5 to 24.9	377	13.8	391	15.9	42	1.7	0	0.0	0	0.0	0	0.0	861	2.4
20.0 to 22.4	805	29.4	885	36.1	304	12.6	0	0.0	0	0.0	0	0.0	2,461	6.7
17.5 to 19.9	611	22.3	354	14.4	425	17.5	73	2.6	19	0.7	0	0.0	2,487	6.8
15.0 to 17.4	362	13.2	330	13.5	510	21.1	223	8.0	75	2.7	10	0.3	2,713	7.4
12.5 to 14.9	255	9.3	196	8.0	368	15.2	250	8.9	149	5.3	3	0.1	2,494	6.8
10.0 to 12.4	186	6.8	158	6.4	303	12.5	305	10.9	153	5.5	28	0.9	2,492	6.8
7.5 to 9.9	77	2.8	79	3.2	194	8.0	303	10.8	261	9.3	116	3.6	2,478	6.8
5.0 to 7.4	41	1.5	21	0.9	121	5.0	507	18.1	346	12.4	186	5.8	2,704	7.4
2.5 to 4.9	1	0.0	0	0.0	121	5.0	597	21.3	311	11.1	261	8.2	2,731	7.5
0.0 to 2.4	0	0.0	0	0.0	22	0.9	319	11.4	344	12.3	419	13.2	2,545	7.0
-2.5 to -.1	0	0.0	0	0.0	4	0.2	158	5.6	398	14.2	442	13.9	2,651	7.3
-5.0 to -2.6	0	0.0	0	0.0	0	0.0	60	2.1	414	14.8	479	15.0	2,622	7.2
-7.5 to -5.1	0	0.0	0	0.0	0	0.0	4	0.1	217	7.8	359	11.2	1,994	5.5
-10.0 to -7.6	0	0.0	0	0.0	0	0.0	0	0.0	92	3.3	263	8.3	1,678	4.6
-12.5 to -10.1	0	0.0	0	0.0	0	0.0	0	0.0	18	0.6	282	8.9	1,563	4.3
-15.0 to -12.6	0	0.0	0	0.0	0	0.0	0	0.0	1	0.0	244	7.7	1,090	3.0
-17.5 to -15.1	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	64	2.0	529	1.5
-20.0 to -17.6	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	14	0.4	235	0.6
-22.5 to -20.1	0	0.0	0	0.0	8	0.0	0	0.0	0	0.0	7	0.2	62	0.2
-25.0 to -22.6	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	7	0.2	11	0.0
Total	2,739	100.0	2,452	100.0	2,422	100.0	2,799	100.0	2,798	100.0	3,184	100.0	36,471	100.0

TABLE 2.3-16

ONSITE DIURNAL DEW POINT TEMPERATURE
JANUARY 1977 - DECEMBER 1981

Hour	Absolute Humidity, g/m^3												
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1	-7.5	-6.7	-1.3	3.9	10.5	15.1	18.5	19.0	14.6	7.2	2.6	-3.3	5.3
2	-7.5	-6.8	-1.5	3.8	10.5	15.0	18.5	19.1	14.5	7.2	2.5	-3.4	5.2
3	-7.5	-6.7	-1.6	3.7	10.4	14.9	18.5	18.8	14.4	7.1	2.5	-3.6	5.2
4	-7.6	-6.9	-1.8	3.7	10.5	14.8	18.3	18.8	14.4	6.9	2.3	-3.7	5.0
5	-7.6	-6.9	-1.8	3.7	10.5	14.8	18.3	18.8	14.4	6.9	2.3	-3.7	5.0
6	-7.7	-6.9	-1.8	3.7	10.4	14.7	18.3	18.8	14.3	6.8	2.2	-3.8	5.0
7	-7.7	-6.8	-2.0	3.8	10.4	14.8	18.4	18.8	14.2	6.7	2.2	-3.9	5.0
8	-7.7	-6.8	-1.9	3.7	10.4	14.7	18.3	18.8	14.3	6.8	2.2	-3.8	5.0
9	-7.6	-6.8	-2.0	3.4	10.1	14.7	18.3	18.8	14.4	7.0	2.2	-3.8	4.9
10	-7.7	-6.8	-2.1	2.9	10.1	14.5	18.2	18.6	14.2	7.0	2.2	-3.7	4.8
11	-7.6	-6.8	-2.0	2.8	9.9	14.2	17.8	18.5	14.1	6.8	2.1	-3.7	4.7
12	-7.7	-6.8	-2.0	2.9	9.8	14.3	17.8	18.4	14.2	6.6	2.1	-3.9	4.7
13	-7.7	-6.8	-1.9	3.0	9.8	14.1	17.7	18.4	14.2	6.6	2.1	-3.8	4.7
14	-7.5	-6.8	-1.7	2.9	9.7	13.9	17.7	18.3	14.5	6.3	2.0	-3.9	4.8
15	-7.3	-6.7	-1.6	2.9	9.9	14.0	17.8	18.4	14.6	6.5	2.0	-3.8	4.9
16	-7.3	-6.7	-1.5	3.0	9.8	14.1	17.9	18.5	14.4	6.6	2.2	-3.7	4.9
17	-7.3	-6.7	-1.4	3.1	10.0	14.2	18.0	18.6	14.4	6.8	2.5	-3.4	5.1
18	-7.3	-6.4	-1.1	3.6	10.1	14.3	18.4	18.7	14.7	7.3	2.7	-3.1	5.3
19	-7.1	-6.1	-0.8	3.9	10.7	14.6	18.7	18.9	14.8	7.3	2.6	-3.1	5.5
20	-7.1	-5.9	-0.7	4.3	11.0	14.9	18.8	19.1	14.8	7.3	2.6	-3.1	5.6
21	-7.2	-6.0	-0.6	4.3	11.3	15.2	18.8	19.2	14.7	7.2	2.6	-3.3	5.7
22	-7.3	-6.1	-0.7	4.3	11.5	15.3	18.9	19.2	14.7	7.1	2.5	-3.2	5.7
23	-7.4	-6.1	-0.8	4.3	11.3	15.2	18.8	19.1	14.6	7.1	2.4	-3.5	5.5
24	-7.5	-6.2	-0.8	4.1	11.0	15.1	18.8	19.0	14.4	7.1	2.4	-3.6	5.4

TABLE 2.3-17

ONSITE RELATIVE HUMIDITY MEANS AND EXTREMES
JANUARY 1977 - DECEMBER 1981

Combined Months	Relative Humidity, percent Mean	Hourly Maximum	Hourly Minimum
Jan	66	100	28
Feb	64	100	23
Mar	64	100	18
Apr	62	100	15
May	71	100	18
Jun	70	100	18
Jul	73	100	30
Aug	76	100	32
Sep	74	100	30
Oct	72	100	31
Nov	71	100	19
Dec	69	100	20
Annual	69	100	15

TABLE 2.3-18

ONSITE HOURLY RELATIVE HUMIDITY FREQUENCY DISTRIBUTIONS
JANUARY 1977 - DECEMBER 1981

Relative Humidity, %	Jan		Feb		Mar		Apr		May		June	
	Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent
100	23	0.7	45	1.6	12	0.3	72	2.1	150	5.0	54	1.8
95 - 99	165	4.7	148	5.2	184	5.1	217	6.4	280	9.4	158	5.2
90 - 94	271	7.7	173	6.0	264	7.4	240	7.0	299	10.1	256	8.4
85 - 89	157	4.5	156	5.4	239	6.7	227	6.6	301	10.1	310	10.2
80 - 84	188	5.4	126	4.4	209	5.8	199	5.8	258	8.7	302	9.9
75 - 79	205	5.9	126	4.4	259	7.2	197	5.8	218	7.3	310	10.2
70 - 74	270	7.7	173	6.0	258	7.2	221	6.5	212	7.1	266	8.7
65 - 69	401	11.4	259	9.0	255	7.1	231	6.8	166	5.6	285	9.3
60 - 64	471	13.4	287	10.0	257	7.2	194	5.7	179	6.0	244	8.0
55 - 59	453	12.9	370	12.9	317	8.8	216	6.3	165	5.5	210	6.9
50 - 54	434	12.4	372	13.0	312	8.7	246	7.2	157	5.3	169	5.5
45 - 49	281	8.0	287	10.0	327	9.1	263	7.7	165	5.5	174	5.7
40 - 44	132	3.8	177	6.2	270	7.5	243	7.1	111	3.7	142	4.7
35 - 39	38	1.1	107	3.7	237	6.6	196	5.7	105	3.5	91	3.0
30 - 34	12	0.3	39	1.4	118	3.3	183	5.4	102	3.4	57	1.9
25 - 29	3	0.1	19	0.7	51	1.4	160	4.7	78	2.6	19	0.6
20 - 24	0	0.0	3	0.1	13	0.4	91	2.7	267	0.9	4	0.1
15 - 19	0	0.0	0	0.0	3	0.1	21	0.6	4	0.1	1	0.0
10 - 14	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0
5 - 9	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0
0 - 4	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0
Total	3,504	100.0	2,867	100.0	3,585	100.0	3,417	100.0	2,976	100.0	3,052	100.0

Month (January to June)

TABLE 2.3-18 (Cont)

Relative Humidity, %	Month (July to December)													
	Jul		Aug		Sep		Oct		Nov		Dec		Annual	
	Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent
100	39	1.5	59	2.4	24	1.0	111	4.1	83	3.2	108	3.5	780	2.2
95 - 99	184	7.0	151	6.2	140	5.9	203	7.5	277	10.6	215	7.0	2,322	6.6
90 - 94	280	10.6	294	12.0	283	11.9	240	8.9	276	10.6	252	8.2	3,128	8.9
85 - 89	313	11.9	351	14.4	279	11.7	215	7.9	171	6.6	187	6.1	2,906	8.2
80 - 84	306	11.6	330	13.5	301	12.6	249	9.2	129	4.9	196	6.4	2,793	7.9
75 - 79	267	10.1	318	13.0	286	12.0	223	8.2	168	6.4	227	7.4	2,804	8.0
70 - 74	245	9.3	240	9.8	229	9.6	272	10.0	214	8.2	246	8.0	2,846	8.1
65 - 69	216	8.2	155	6.3	184	7.7	255	9.4	255	9.8	270	8.8	2,932	8.3
60 - 64	188	7.1	138	5.7	183	7.7	225	8.3	211	8.1	286	9.4	2,863	8.1
55 - 59	194	7.4	134	5.5	157	6.6	197	7.3	248	9.5	309	10.1	2,970	8.4
50 - 54	133	5.0	105	4.3	127	5.3	184	6.8	202	7.7	261	8.5	2,702	7.7
45 - 49	111	4.2	90	3.7	87	3.7	152	5.6	177	6.8	213	7.0	2,327	6.6
40 - 44	81	3.1	47	1.9	54	2.3	110	4.1	97	3.7	158	5.2	1,622	4.6
35 - 39	70	2.7	29	1.2	42	1.8	58	2.1	59	2.3	70	2.3	1,102	3.1
30 - 34	13	0.5	3	0.1	9	0.4	15	0.6	23	0.9	42	1.4	616	1.8
25 - 29	0	0.0	0	0.0	0	0.0	0	0.0	13	0.5	14	0.5	357	1.0
20 - 24	0	0.0	0	0.0	0	0.0	0	0.0	7	0.3	6	0.2	150	0.4
15 - 19	0	0.0	0	0.0	0	0.0	0	0.0	1	0.0	0	0.0	30	0.1
10 - 14	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0
5 - 9	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0
0 - 4	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0
Total	2,640	100.0	2,444	100.0	2,385	100.0	2,709	100.0	2,611	100.0	3,060	100.0	35,250	100.0

TABLE 2.3-19

ONSITE DIURNAL RELATIVE HUMIDITY VARIATIONS
JANUARY 1977 - DECEMBER 1981

	Relative Humidity, percent												
Hour	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1	68.4	67.4	68.6	68.9	77.8	77.7	80.7	83.3	81.0	77.6	75.9	72.9	74.5
2	69.3	68.2	69.5	70.0	79.3	79.0	82.1	84.7	82.4	78.6	76.1	73.0	75.4
3	70.3	70.2	69.8	71.1	80.4	79.9	83.1	85.7	82.8	79.7	76.8	73.0	76.3
4	71.0	70.9	70.1	72.5	81.4	81.3	84.6	85.8	83.8	80.8	77.4	74.0	77.2
5	71.2	71.0	70.5	72.8	82.6	81.8	84.8	87.1	84.7	81.2	77.4	74.3	77.7
6	71.7	71.8	71.5	73.7	82.4	81.7	85.4	87.7	85.3	82.1	77.4	74.9	78.2
7	72.5	72.0	69.9	68.0	77.5	77.6	82.3	86.6	83.9	82.8	77.4	74.7	76.4
8	72.5	72.0	69.9	68.0	77.5	77.6	82.3	86.6	83.9	82.8	77.4	74.7	76.4
9	70.7	69.0	65.4	62.7	72.4	73.5	77.0	82.4	81.0	79.9	74.4	73.5	72.8
10	67.2	65.0	61.2	57.4	68.0	68.7	72.3	76.4	74.2	73.7	69.4	70.3	68.0
11	64.5	60.8	57.7	54.5	63.7	64.4	67.0	71.3	69.2	67.5	66.0	66.4	63.9
12	62.0	57.8	55.7	52.5	61.2	61.5	63.9	67.8	65.6	63.2	62.7	63.2	61.0
13	60.2	55.4	54.0	51.8	58.8	59.0	61.5	66.1	63.9	60.8	61.2	61.5	59.1
14	60.2	53.8	53.8	50.6	57.4	57.3	60.0	64.4	62.9	58.5	60.1	59.5	57.9
15	59.5	52.9	53.0	50.4	57.7	56.7	58.9	63.2	62.1	58.5	60.0	58.9	57.3
16	59.3	53.4	53.6	50.8	58.4	57.8	59.7	63.6	61.2	58.6	61.4	60.1	57.9
17	60.4	54.1	54.9	51.8	59.8	58.6	60.5	64.7	62.1	60.6	65.3	62.7	59.3
18	62.6	57.4	58.3	54.5	61.9	59.8	63.0	66.7	64.9	64.6	68.0	65.7	62.0
19	64.5	61.0	62.0	58.0	66.2	63.1	66.0	69.4	68.3	67.5	69.9	67.1	64.9
20	66.1	62.9	64.1	61.9	70.5	66.6	69.6	73.0	71.5	70.0	71.4	68.6	67.7
21	66.8	64.4	66.0	64.6	73.3	71.0	73.3	76.3	74.2	72.4	72.8	69.1	70.0
22	67.5	66.0	66.7	66.3	76.5	73.3	75.8	78.1	76.7	73.3	73.4	70.2	71.5
23	67.6	67.4	67.8	67.6	77.5	74.7	78.0	79.6	78.3	75.1	73.8	70.8	72.7
24	67.8	67.4	68.8	68.4	77.6	76.0	79.7	81.3	79.1	76.8	74.7	71.4	73.6

TABLE 2.3-20

WILMINGTON NWS DIURNAL RELATIVE HUMIDITY VARIATIONS[1]

Months	Relative Humidity, percent Hour (local time) 01	07	13	19
Jan	73	75	60	68
Feb	72	74	57	65
Mar	71	73	53	62
Apr	72	72	50	59
May	79	76	53	64
Jun	82	78	54	65
Jul	83	79	54	66
Aug	85	83	56	70
Sep	84	85	55	71
Oct	82	84	53	70
Nov	77	80	56	69
Dec	75	76	59	69
Annual	78	78	55	66

(1) Source: U.S. Department of Commerce, "Local Climatological Data - Annual Summary with Comparative Data - Wilmington, Delaware," NOAA, Environmental Data Service, 1977-1981.

TABLE 2.3-21

ONSITE HOURLY ABSOLUTE HUMIDITY MEANS AND EXTREMES
JANUARY 1977 - DECEMBER 1981

Combined Months	Absolute Humidity, g/m^3 Mean	Hourly Maximum	Hourly Minimum
Jan	2.9	11.0	0.6
Feb	3.2	10.8	0.9
Mar	4.6	12.2	0.8
Apr	6.4	14.2	1.7
May	9.8	18.0	1.8
Jun	12.6	22.1	4.0
Jul	15.4	24.9	6.2
Aug	15.9	26.9	6.7
Sep	12.4	24.0	4.2
Oct	7.9	16.5	3.0
Nov	6.0	15.2	1.6
Dec	4.0	13.1	0.6
Annual	8.1	26.9	0.6

TABLE 2.3-22

ONSITE HOURLY ABSOLUTE HUMIDITY FREQUENCY DISTRIBUTIONS
JANUARY 1977 - DECEMBER 1981

Absolute Humidity, g/m^3	Month (January to June)											
	Jan		Feb		Mar		Apr		May		June	
	Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent
26.1 - 27.00	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0
25.1 - 26.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0
24.1 - 25.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0
23.1 - 24.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0
22.1 - 23.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	3	0.1
21.1 - 22.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	4	0.1
20.1 - 21.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	12	0.4
19.1 - 20.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	35	1.1
18.1 - 19.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	73	2.4
17.1 - 18.0	0	0.0	0	0.0	0	0.0	0	0.0	17	0.6	186	6.1
16.1 - 17.0	0	0.0	0	0.0	0	0.0	0	0.0	73	2.5	357	11.7
15.1 - 16.0	0	0.0	0	0.0	0	0.0	0	0.0	137	4.6	272	8.9
14.1 - 15.0	0	0.0	0	0.0	0	0.0	4	0.1	196	6.6	234	7.7
13.1 - 14.0	0	0.0	0	0.0	0	0.0	20	0.6	192	6.5	227	7.4
12.1 - 13.0	0	0.0	0	0.0	2	0.1	80	2.3	282	9.5	346	11.3
11.1 - 12.0	0	0.0	0	0.0	26	0.7	159	4.7	260	8.7	285	9.3
10.1 - 11.0	11	0.3	7	0.2	35	1.0	150	4.4	262	8.8	253	8.3
9.1 - 10.0	18	0.5	30	1.0	63	1.8	220	6.4	249	8.4	198	6.5
8.1 - 9.0	29	0.8	65	2.3	217	6.1	330	9.7	276	9.3	213	7.0
7.1 - 8.0	40	1.1	77	2.7	199	5.6	310	9.1	282	9.5	142	4.7
6.1 - 7.0	90	2.6	114	4.0	307	8.6	442	12.9	271	9.1	111	3.6
5.1 - 6.0	115	3.3	135	4.7	408	11.4	507	14.8	223	7.5	75	2.5
4.1 - 5.0	317	9.0	323	11.3	698	19.5	484	14.2	170	5.7	24	0.8
3.1 - 4.0	797	22.7	448	15.6	701	19.6	423	12.4	80	2.7	1	0.0
2.1 - 3.0	980	25.9	860	30.0	614	17.1	263	7.7	4	0.1	0	0.0
1.1 - 2.0	1,099	31.4	780	27.2	301	8.4	25	0.7	2	0.1	0	0.0
LE 1.0	80	2.3	28	1.0	14	0.4	0	0.0	0	0.0	0	0.0
Total	3,504	100.0	2,867	100.0	3,585	100.0	3,417	100.0	2,976	100.0	3,501	100.0

TABLE 2.3-22 (Cont)

Absolute Humidity, g/m^3	Month (January to June)													
	Jan		Feb		Mar		Apr		May		June			
	Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent	Sum	Percent		
26.1 - 27.0	0	0.0	3	0.1	0	0.0	0	0.0	0	0.0	0	0.0	3	0.0
25.1 - 26.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0
24.1 - 25.0	1	0.0	4	0.2	1	0.0	0	0.0	0	0.0	0	0.0	6	0.0
23.1 - 24.0	2	0.1	13	0.5	6	0.3	0	0.0	0	0.0	0	0.0	21	0.1
22.1 - 23.0	21	0.8	21	0.9	0	0.0	0	0.0	0	0.0	0	0.0	45	0.1
21.1 - 22.0	73	2.8	41	1.7	1	0.0	0	0.0	0	0.0	0	0.0	119	0.3
20.1 - 21.0	129	4.9	152	6.2	12	0.5	0	0.0	0	0.0	0	0.0	305	0.9
19.1 - 20.0	215	8.1	295	12.1	49	2.1	0	0.0	0	0.0	0	0.0	594	1.7
18.1 - 19.0	246	9.3	305	12.5	73	3.1	0	0.0	0	0.0	0	0.0	697	2.0
17.1 - 18.0	304	11.5	325	13.3	132	5.5	0	0.0	0	0.0	0	0.0	964	2.7
16.1 - 17.0	322	12.2	243	9.9	185	7.8	13	0.5	0	0.0	0	0.0	1,193	3.4
15.1 - 16.0	242	9.2	165	6.8	203	8.5	28	1.0	3	0.1	0	0.0	1,050	3.0
14.1 - 15.0	236	8.9	136	5.6	175	7.3	64	2.4	38	1.5	0	0.0	1,083	3.1
13.1 - 14.0	160	6.1	158	6.5	268	11.2	104	3.8	25	1.0	1	0.0	1,155	3.3
12.1 - 13.0	164	6.2	149	6.1	219	9.2	110	4.1	41	1.6	9	0.3	1,402	4.0
11.1 - 12.0	142	5.4	111	4.5	222	9.3	164	6.1	103	3.9	1	0.0	1,473	4.2
10.1 - 11.0	109	4.1	102	4.2	153	6.4	144	5.3	61	2.3	4	0.1	1,291	3.7
9.1 - 10.0	115	4.4	92	3.8	195	8.2	196	7.2	98	3.8	27	0.9	1,501	4.3
8.1 - 9.0	87	3.3	85	3.5	170	7.1	224	8.3	193	7.4	80	2.6	1,969	5.6
7.1 - 8.0	53	2.0	39	1.6	122	5.1	313	11.6	220	8.4	142	4.6	1,939	5.5
6.1 - 7.0	19	0.7	5	0.2	92	3.9	548	20.2	290	11.1	192	6.3	2,481	7.0
5.1 - 6.0	0	0.0	0	0.0	90	3.8	478	17.6	303	11.6	370	12.1	2,704	7.7
4.1 - 5.0	0	0.0	0	0.0	17	0.7	234	8.6	429	16.4	569	18.6	3,265	9.3
3.1 - 4.0	0	0.0	0	0.0	0	0.0	89	3.3	581	22.3	668	21.8	3,788	10.7
2.1 - 3.0	0	0.0	0	0.0	0	0.0	0	0.0	223	8.5	553	18.1	3,425	9.7
1.1 - 2.0	0	0.0	0	0.0	0	0.0	0	0.0	3	0.1	427	14.0	2,637	7.5
LE 1.0	0	0.0	0	0.0	0	0.0	0	0.0	0	0.0	17	0.6	139	0.4
Total	2,640	100.0	2,444	100.0	2,385	100.0	2,709	100.0	2,611	100.0	3,060	100.0	35,249	100.0

TABLE 2.3-23

ONSITE DIURNAL ABSOLUTE HUMIDITY VARIATIONS
JANUARY 1977 - DECEMBER 1981

	Absolute Humidity, g/m^3												
Hour	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1	2.9	3.2	4.6	6.6	9.9	12.9	15.6	16.2	12.5	8.0	6.0	4.1	8.2
2	2.9	3.1	4.6	6.5	9.9	12.8	15.7	16.2	12.5	8.0	6.0	4.1	8.1
3	2.9	3.2	4.5	6.5	9.8	12.8	15.6	16.1	12.4	7.9	6.0	4.0	8.1
4	2.9	3.2	4.5	6.5	9.8	12.8	15.6	16.0	12.4	7.9	6.0	4.0	8.1
5	2.9	3.2	4.5	6.5	9.8	12.7	15.5	16.0	12.4	7.8	5.9	4.0	8.0
6	2.9	3.2	4.4	6.5	9.8	12.7	15.4	16.0	12.4	7.8	5.9	4.0	8.0
7	2.8	3.2	4.4	6.5	9.8	12.7	15.5	16.0	12.4	7.8	5.9	4.0	8.0
8	2.8	3.2	4.4	6.5	9.8	12.7	15.6	16.1	12.4	7.8	5.9	4.0	8.0
9	2.8	3.2	4.4	6.4	9.6	12.7	15.4	16.0	12.5	7.9	5.9	4.0	8.0
10	2.8	3.2	4.4	6.2	9.6	12.6	15.4	15.8	12.4	7.9	5.9	4.0	7.9
11	2.8	3.2	4.4	6.1	9.5	12.4	15.0	15.6	12.3	7.9	5.9	4.1	7.9
12	2.9	3.3	4.4	6.1	9.5	12.3	14.9	15.5	12.2	7.8	5.8	4.0	7.8
13	2.8	3.3	4.5	6.2	9.5	12.2	14.7	15.5	12.3	7.9	5.9	4.0	7.9
14	2.9	3.3	4.6	6.1	9.5	12.2	14.7	15.4	12.5	7.7	5.8	4.0	7.9
15	3.0	3.2	4.6	6.2	9.6	12.1	14.7	15.4	12.6	7.8	5.9	4.0	7.9
16	3.0	3.2	4.6	6.2	9.5	12.2	14.8	15.6	12.4	7.8	5.9	4.0	7.9
17	3.0	3.2	4.6	6.2	9.6	12.3	14.9	15.7	12.5	7.9	6.0	4.1	8.0
18	2.9	3.3	4.7	6.4	9.7	12.4	15.3	15.7	12.5	8.1	6.1	4.2	8.1
19	3.0	3.4	4.8	6.5	9.9	12.6	15.6	16.0	12.7	8.1	6.1	4.2	8.2
20	3.0	3.4	4.8	6.7	10.2	12.8	15.8	16.2	12.6	8.0	6.1	4.2	8.3
21	3.0	3.4	4.8	6.7	10.3	13.0	15.9	16.2	12.6	8.0	6.1	4.1	8.3
22	2.9	3.3	4.8	6.7	10.4	13.1	16.0	16.2	12.6	7.9	6.0	4.1	8.3
23	2.9	3.3	4.8	6.7	10.3	13.0	15.8	16.2	12.4	7.9	6.0	4.0	8.2
24	2.9	3.3	4.8	6.6	10.1	12.9	15.9	16.1	12.3	8.0	6.0	4.0	8.2

TABLE 2.3-24

WILMINGTON NWS PRECIPITATION MEANS AND EXTREMES[1]

	Precipitation, in.			
Month	Mean[2]	Monthly[3] Maximum	Monthly[3] Minimum	24 Hour[3] Maximum
Jan	2.85	8.41	0.52	2.12
Feb	2.75	7.02	0.83	2.29
Mar	3.74	6.22	0.81	3.11
Apr	3.20	6.57	1.12	2.56
May	3.35	7.35	0.22	2.35
Jun	3.24	7.49	0.44	4.35
Jul	4.31	7.51	0.16	6.24
Aug	3.98	12.09	0.25	4.11
Sep	3.42	9.53	0.82	5.62
Oct	2.60	6.41	0.21	3.88
Nov	3.49	7.84	0.49	3.83
Dec	3.32	7.90	0.19	2.22
Annual	40.25	12.09	0.16	6.24

(1) Source: U.S. Department Commerce, "Local Climatological Data - Annual Summary with Comparative Data - Wilmington, Delaware," NOAA, Environmental Data Service, 1977-1981.

(2) Period of record 1941 to 1970.

(3) Period of record 1948 to 1981.

TABLE 2.3-25

WILMINGTON NWS SNOWFALL MEANS AND EXTREMES[1]

	Snowfall, in.		
Months	Mean[2]	Monthly[2] Maximum	24-Hour[2] Maximum
Jan	6.2	17.2	11.2
Feb	6.4	27.5	16.5
Mar	3.8	20.3	15.6
Apr	0.1	1.1	1.1
May	T[3]	T[3]	T[3]
Jun	0.0	0.0	0.0
Jul	0.0	0.0	0.0
Aug	0.0	0.0	0.0
Sep	0.0	0.0	0.0
Oct	0.1	2.5	2.5
Nov	1.1	11.9	11.9
Dec	3.6	21.5	16.5
Annual	21.3	27.5	16.5

(1) Source: U.S. Department of Commerce, "Local Climatological Data - Annual Summary With Comparative Data - Wilmington Delaware," NOAA, Environmental Data Service, 1980.

(2) Period of record 1948 to 1980.

(3) T = trace.

TABLE 2.3-26

MEAN NUMBER OF DAYS AT WILMINGTON NWS WITH FOG, HAZE, AND/OR SMOKE[1]

Months	Fog: Visibility <7 mi	Fog: Visibility <0.25 mi	Haze and/ or Smoke
Jan	12	4	11
Feb	10	4	10
Mar	12	3	10
Apr	11	3	10
May	14	2	14
Jun	16	2	18
Jul	14	2	21
Aug	16	2	20
Sep	15	2	18
Oct	13	4	14
Nov	12	3	12
Dec	13	4	11
Annual	156	34	167

(1) Source: U.S. Department of Commerce, "Airport Climatological Summary - Climatography of the United States No. 90 (1965 - 1974), Wilmington, Delaware," January 1979.

TABLE 2.3-27

COMPARISON OF ONSITE AND
WILMINGTON NWS STABILITY FREQUENCY DISTRIBUTIONS

	5-Year Concurrent Period[1]		10-Year Period[2]
Stability Class	Artificial Island,[3] Percent	Wilmington NWS,[4] Percent	Wilmington NWS,[4] Percent
Extremely unstable (A)	1.1	0.2	0.3
Moderately unstable (B)	2.2	4.0	4.0
Slightly unstable (C)	2.0	10.9	10.9
Neutral (D)	38.6	54.4	53.8
Slightly stable (E)	43.7	15.0	15.1
Moderately stable (F)	10.2	12.5	13.0
Extremely stable (G)	2.2	3.0	3.0

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(1) January 1977 to December 1981.

(2) January 1972 to December 1981.

(3) Stability determined from temperature difference data measured on the site tower (300 to 33 feet).

(4) Stability determined by the STAR program consistent with Turner's method.

TABLE 2.3-27a

DELTA TEMPERATURE

STABILITY DISTRIBUTION

300 TO 33 FT

1977 TO 1981

COUNTS/(PERCENT)(1)

Year	A	B	C	D	E	F	G	Missing	Total Hours
	NRC Stability Class								
1977	51	184	153	3792	3103	784	218	475	8760
	(0.6)	(2.2)	(1.8)	(45.8)	(37.5)	(9.5)	(2.6)		
1978	176	238	139	3175	3543	837	79	523	8760
	(2.1)	(2.9)	(2.3)	(38.5)	(43.0)	(10.2)	(1.0)		
1979	22	141	184	3093	3483	748	253	836	8760
	(0.3)	(1.8)	(2.3)	(39.0)	(44.0)	(9.4)	(3.2)		
1980	190	242	159	2682	3875	813	141	682	8784
	(2.4)	(3.0)	(2.0)	(33.1)	(47.8)	(10.0)	(1.7)		
1981	15	87	142	3044	3865	993	191	423	8760
	(0.2)	(1.0)	(1.7)	(36.5)	(46.4)	(11.9)	(2.3)		
Combined	454	892	827	15786	17869	4175	882	2939	43824
1977-1981	(1.1)	(2.2)	(2.0)	(38.6)	(43.7)	(10.2)	(2.2)		

(1) Percents based on all good hours.

TABLE 2.3-27b

DELTA TEMPERATURE
STABILITY DISTRIBUTION
150 TO 33 FT
1977 TO 1981
COUNTS/(PERCENT)[1]

Year	A	B	C	D	E	F	G	Missing	Total Hours
1977	923 (11.1)	(2)	512 (6.2)	3194 (38.4)	2213 (26.6)	977 (11.7)	506 (6.1)	435	8760
1978	1269 (15.2)	(2)	488 (5.8)	2805 (33.5)	2234 (26.7)	1081 (12.9)	494 (5.9)	389	8760
1979	1259 (15.9)	(2)	415 (5.2)	2773 (35.5)	2189 (27.6)	828 (10.4)	462 (5.8)	836	8760
1980	827 (12.4)	(2)	314 (4.7)	1893 (28.4)	2409 (36.2)	839 (12.6)	379 (5.7)	2123	8784
1981	1163 (13.9)	(2)	461 (5.5)	2551 (30.6)	2600 (31.2)	1030 (12.4)	532 (6.4)	423	8760
Combined 1977-1981	5439 (13.7)	(2)	2190 (5.5)	13216 (33.4)	11645 (29.4)	4755 (12.0)	2373 (6.0)	4206	43824

NRC Stability Class: columns A through G.

(1) Percents based on all good hours.

(2) For a 36m (150-33 ft) vertical distance, NRC Class A stability computes to a delta temperature of less than or equal to -0.7°C. Stability class C computes to -0.6°C.. Therefore, stability class B computes greater than -0.6°C but less than -0.7°C. Since delta temperature is measured in the nearest 0.1°C, no counts occur in NRC stability Class B.

TABLE 2.3-28

300-33 FT. ONSITE TEMPERATURE INVERSION PERSISTENCE
JANUARY 1977 - DECEMBER 1981

Combined Months	Lapse Rate Greater Than, C°/100m	Consecutive Hours 2-3	4-7	8-12	13-18	19-24	25-30	31-36	37-48	49+	Total Occurrences
Jan	0.0	22	26	15	14	4	0	0	2	1	83
	1.5	13	11	8	5	1	1	0	0	0	39
Feb	0.0	20	17	19	26	11	4	1	2	0	100
	1.5	24	18	10	11	6	0	0	0	0	69
Mar	0.0	24	24	22	36	4	2	2	5	2	121
	1.5	29	31	14	10	4	0	0	1	0	89
Apr	0.0	39	50	20	33	3	1	1	1	1	149
	1.5	48	24	17	6	0	0	0	0	0	95
May	0.0	40	38	36	23	1	0	0	0	0	138
	1.5	36	26	10	1	0	0	0	0	0	73
Jun	0.0	51	47	34	15	1	0	0	0	0	148
	1.5	33	24	8	1	0	0	0	0	0	66
Jul	0.0	31	38	39	25	2	0	0	1	1	137
	1.5	42	23	10	1	0	0	0	0	0	76
Aug	0.0	55	49	36	16	0	0	0	0	0	156
	1.5	31	11	1	0	0	0	0	0	0	43
Sep	0.0	38	36	31	34	0	0	0	0	0	139
	1.5	36	24	8	0	0	0	0	0	0	68
Oct	0.0	30	35	30	38	6	1	2	0	0	142
	1.5	35	25	16	6	0	0	0	0	0	82
Nov	0.0	27	21	22	34	7	0	1	0	1	113
	1.5	24	23	7	6	1	0	0	1	0	62
Dec	0.0	35	37	21	24	5	1	2	2	1	128
	1.5	31	19	9	6	3	0	1	1	0	70
Annual	0.0	408	415	320	331	44	8	9	13	8	1556
	1.5	379	258	118	56	15	0	1	4	0	831

TABLE 2.3-29

METEOROLOGICAL INSTRUMENTATION

Height Above Tower Base, ft	Sensed Parameter	Recorded Parameter
300	Wind speed	Wind speed
	Wind direction	Wind direction
	Temperature[1]	Temperature difference
150	Wind speed	Wind speed
	Wind direction	Wind direction
	Temperature[2]	Temperature difference

TABLE 2.3-29 (Cont)

Height Above Tower Base, ft	Sensed Parameter	Recorded Parameter
33	Wind speed	Wind speed
	Wind direction	Wind direction
	Temperature differential	
		$T_{300}-T_{33}$(1)
		$T_{150}-T_{33}$(2)
	Dew point	Dew point
	Temperature ambient	Temperature
6	Barometric pressure	Barometric pressure
3	Rainfall	Rainfall
Backup Tower 33	Wind speed	Wind speed

TABLE 2.3-29 (Cont)

Height Above Tower Base, ft	Sensed Parameter	Recorded Parameter
	Wind direction	Wind direction

(1) Temperature taken as part of temperature differential measurement T300 - T33.
(2) Temperature taken as part of temperature differential measurement T150 - T33.

TABLE 2.3-29a

THIS TABLE INTENTIONALLY DELETED

TABLE 2.3-29b

THIS TABLE INTENTIONALLY DELETED

TABLE 2.3-29c

THIS TABLE INTENTIONALLY DELETED

TABLE 2.3-30

ACCIDENT X/Q ESTIMATES

(s/m^3)

	Time Following Accident					
	2 Hours	8 Hours	16 Hours	3 Days	26 Days	Annual
EAB (0.56 Miles)						
Conservative Estimate	1.9E-04	9.2E-05	6.4E-05	2.9E-05	9.2E-06	2.3E-06
Realistic Estimate	6.3E-05	3.6E-05	2.8E-05	1.5E-05	6.5E-06	2.3E-06
LPZ (5.0 Miles)						
Conservative Estimate	1.9E-05	8.0E-06	5.2E-06	2.0E-06	5.2E-07	9.9E-08
Realistic Estimate	4.8E-06	2.5E-06	1.8E-06	9.1E-07	3.4E-07	9.9E-08

TABLE 2.3-30a

ACCIDENT X/Q VALUES AT LPZ BY SECTOR

Sector Bearing	0.5 percent[2] X/Q	Annual X/Q
NNE	9.0E-6	6.8E-08
NE	1.1E-5	7.7E-08
ENE	9.0E-6	6.4E-08
E	8.5E-6	7.2E-08
ESE	9.8E-6	8.4E-08
SE	9.3E-6	9.9E-08
SSE	8.2E-6	6.6E-08
S	1.9E-5[1]	9.0E-08
SSW	1.2E-5	7.6E-08
SW	1.3E-5	7.6E-08
WSW	1.1E-5	5.6E-08
W	9.9E-6	5.2E-08
WNW	1.1E-5	4.9E-08
NW	1.8-5	9.5E-08
NNW	1.2E-5	7.6E-08
N	9.0E-6	7.6E-08
Overall 5 percent	1.8E-5	

(1) 1.9E-5 is the maximum 0.5 percent X/Q (Conservative at the LPZ).

(2) Two hour value.

TABLE 2.3-31

VENT X/Q AT GROUND LEVEL
LONG TERM GROUND LEVEL ROUTINE GASOUS RELEASES
ANNUAL AVERAGE X/Q BY SECTOR

(s/m^3)
1977-1984

Downwind Sector

Distance, Miles	NNE	NE	ENE	E	ESE	SE	SSE	S
0.25	5.609E-06	6.006E-06	5.075E-06	5.796E-06	6.119E-06	7.924E-06	5.851E-06	8.205E-06
0.50	1.819E-06	1.942E-06	1.650E-06	1.900E-06	2.016E-06	2.584E-06	1.923E-06	2.665E-06
0.75	1.044E-06	1.110E-06	9.424E-07	1.088E-06	1.155E-06	1.490E-06	1.106E-06	1.518E-06
1.00	6.943E-07	7.361E-07	6.248E-07	7.216E-07	7.660E-07	9.935E-07	7.353E-07	1.005E-06
1.50	3.913E-07	4.153E-07	3.524E-07	4.042E-07	4.288E-07	5.589E-07	4.122E-07	5.697E-07
2.00	2.569E-07	2.730E-07	2.316E-07	2.643E-07	2.802E-07	3.662E-07	2.695E-07	3.756E-07
2.50	1.841E-07	1.960E-07	1.661E-07	1.890E-07	2.001E-07	2.619E-07	1.925E-07	2.700E-07
3.00	1.399E-07	1.492E-07	1.263E-07	1.433E-07	1.515E-07	1.983E-07	1.457E-07	2.055E-07
3.40	1.172E-07	1.252E-07	1.058E-07	1.198E-07	1.266E-07	1.658E-07	1.218E-07	1.724E-07
3.50	1.126E-07	1.203E-07	1.017E-07	1.151E-07	1.215E-07	1.592E-07	1.169E-07	1.657E-07
3.60	1.083E-07	1.158E-07	9.781E-08	1.107E-07	1.168E-07	1.531E-07	1.124E-07	1.594E-07
3.70	1.043E-07	1.115E-07	9.420E-08	1.066E-07	1.124E-07	1.473E-07	1.082E-07	1.536E-07
3.80	1.005E-07	1.075E-07	9.080E-08	1.027E-07	1.083E-07	1.419E-07	1.042E-07	1.481E-07
3.90	9.699E-08	1.038E-07	8.761E-08	9.903E-08	1.045E-07	1.369E-07	1.005E-07	1.429E-07
4.00	9.366E-08	1.003E-07	8.461E-08	9.560E-08	1.008E-07	1.321E-07	9.699E-08	1.380E-07
4.10	9.052E-08	9.692E-08	8.177E-08	9.236E-08	9.738E-08	1.276E-07	9.368E-08	1.334E-07
4.20	8.755E-08	9.377E-08	7.910E-08	8.931E-08	9.414E-08	1.234E-07	9.057E-08	1.291E-07
4.30	8.475E-08	9.079E-08	7.657E-08	8.643E-08	9.108E-08	1.194E-07	8.762E-08	1.250E-07
4.40	8.210E-08	8.797E-08	7.418E-08	8.370E-08	8.818E-08	1.156E-07	8.484E-08	1.211E-07
4.50	7.958E-08	8.530E-08	7.191E-08	8.111E-08	8.544E-08	1.120E-07	8.220E-08	1.174E-07
4.60	7.719E-08	8.277E-08	6.976E-08	7.866E-08	8.284E-08	1.086E-07	7.970E-08	1.139E-07
4.70	7.493E-08	8.036E-08	6.772E-08	7.633E-08	8.038E-08	1.054E-07	7.733E-08	1.106E-07
4.80	7.278E-08	7.807E-08	6.577E-08	7.412E-08	7.803E-08	1.023E-07	7.507E-08	1.074E-07
4.90	7.073E-08	7.589E-08	6.392E-08	7.202E-08	7.580E-08	9.942E-08	7.292E-08	1.044E-07
5.00	6.877E-08	7.381E-08	6.216E-08	7.001E-08	7.368E-08	9.664E-08	7.088E-08	1.016E-07
7.50	3.911E-08	4.217E-08	3.539E-08	3.964E-08	4.158E-08	5.459E-08	3.998E-08	5.798E-08
10.00	2.615E-08	2.829E-08	2.368E-08	2.643E-08	2.765E-08	3.632E-08	2.658E-08	3.886E-08
15.00	1.480E-08	1.608E-08	1.342E-08	1.490E-08	1.554E-08	2.041E-08	1.492E-08	2.206E-08
20.00	9.872E-09	1.076E-08	8.958E-09	9.919E-09	1.032E-08	1.356E-08	9.906E-09	1.475E-08
25.00	7.208E-09	7.872E-09	6.545E-09	7.231E-09	7.507E-09	9.863E-09	7.205E-09	1.078E-08
30.00	5.573E-09	6.098E-09	5.064E-09	5.584E-09	5.789E-09	7.605E-09	5.554E-09	8.346E-09
35.00	4.483E-09	4.913E-09	4.075E-09	4.488E-09	4.646E-09	6.103E-09	4.456E-09	6.720E-09
40.00	3.713E-09	4.074E-09	3.376E-09	3.713E-09	3.841E-09	5.044E-09	3.682E-09	5.570E-09
45.00	3.144E-09	3.454E-09	2.860E-09	3.142E-09	3.247E-09	4.263E-09	3.112E-09	4.719E-09
50.00	2.709E-09	2.979E-09	2.465E-09	2.706E-09	2.794E-09	3.668E-09	2.677E-09	4.069E-09

TABLE 2.3-31 (Cont)

Distance, Miles	SSW	SW	WSW	W	WNW	NW	NNW	N
	Downwind Sector							
0.25	6.501E-06	6.505E-06	4.714E-06	4.541E-06	3.910E-06	8.218E-06	6.465E-06	6.542E-06
0.50	2.116E-06	2.113E-06	1.531E-06	1.456E-06	1.243E-06	2.635E-06	2.115E-06	2.140E-06
0.75	1.206E-06	1.202E-06	8.741E-07	8.304E-07	7.003E-07	1.461E-06	1.195E-06	1.219E-06
1.00	7.990E-07	7.957E-07	5.795E-07	5.509E-07	4.618E-07	9.538E-07	7.868E-07	8.070E-07
1.50	4.514E-07	4.493E-07	3.276E-07	3.130E-07	2.630E-07	5.355E-07	4.403E-07	4.528E-07
2.00	2.970E-07	2.955E-07	2.157E-07	2.068E-07	1.742E-07	3.519E-07	2.881E-07	2.965E-07
2.50	2.133E-07	2.123E-07	1.549E-07	1.489E-07	1.260E-07	2.541E-07	2.066E-07	2.124E-07
3.00	1.624E-07	1.618E-07	1.179E-07	1.136E-07	9.661E-08	1.954E-07	1.574E-07	1.615E-07
3.40	1.362E-07	1.358E-07	9.892E-08	9.549E-08	8.147E-08	1.651E-07	1.321E-07	1.353E-07
3.50	1.309E-07	1.305E-07	9.508E-08	9.183E-08	7.841E-08	1.589E-07	1.269E-07	1.300E-07
3.60	1.259E-07	1.256E-07	9.149E-08	8.840E-08	7.554E-08	1.531E-07	1.221E-07	1.251E-07
3.70	1.213E-07	1.210E-07	8.813E-08	8.518E-08	7.285E-08	1.477E-07	1.177E-07	1.205E-07
3.80	1.169E-07	1.167E-07	8.497E-08	8.216E-08	7.032E-08	1.427E-07	1.135E-07	1.161E-07
3.90	1.129E-07	1.126E-07	8.200E-08	7.932E-08	6.793E-08	1.379E-07	1.095E-07	1.120E-07
4.00	1.090E-07	1.088E-07	7.921E-08	7.664E-08	6.569E-08	1.334E-07	1.058E-07	1.082E-07
4.10	1.054E-07	1.052E-07	7.657E-08	7.412E-08	6.357E-08	1.291E-07	1.023E-07	1.046E-07
4.20	1.019E-07	1.018E-07	7.408E-08	7.173E-08	6.156E-08	1.251E-07	9.894E-08	1.012E-07
4.30	9.870E-08	9.853E-08	7.173E-08	6.948E-08	5.967E-08	1.213E-07	9.580E-08	9.792E-08
4.40	9.563E-08	9.548E-08	6.950E-08	6.734E-08	5.787E-08	1.176E-07	9.283E-08	9.486E-08
4.50	9.272E-08	9.258E-08	6.739E-08	6.531E-08	5.616E-08	1.142E-07	9.001E-08	9.196E-08
4.60	8.996E-08	8.983E-08	6.538E-08	6.339E-08	5.454E-08	1.109E-07	8.734E-08	8.920E-08
4.70	8.733E-08	8.723E-08	6.347E-08	6.156E-08	5.300E-08	1.078E-07	8.481E-08	8.659E-08
4.80	8.484E-08	8.474E-08	6.166E-08	5.982E-08	5.153E-08	1.049E-07	8.239E-08	8.411E-08
4.90	8.246E-08	8.238E-08	5.994E-08	5.817E-08	5.013E-08	1.021E-07	8.010E-08	8.174E-08
5.00	8.020E-08	8.013E-08	5.829E-08	5.659E-08	4.880E-08	9.938E-08	7.791E-08	7.949E-08
7.50	4.576E-08	4.582E-08	3.328E-08	3.248E-08	2.832E-08	5.806E-08	4.458E-08	4.526E-08
10.00	3.067E-08	3.076E-08	2.231E-08	2.186E-08	1.920E-08	3.956E-08	2.995E-08	3.029E-08
15.00	1.741E-08	1.750E-08	1.267E-08	1.247E-08	1.107E-08	2.298E-08	1.707E-08	1.717E-08
20.00	1.163E-08	1.172E-08	8.467E-09	8.364E-09	7.476E-09	1.561E-08	1.144E-08	1.147E-08
25.00	8.507E-09	8.577E-09	6.191E-09	6.132E-09	5.510E-09	1.156E-08	8.389E-09	8.384E-09
30.00	6.585E-09	6.646E-09	4.793E-09	4.756E-09	4.292E-09	9.035E-09	6.508E-09	6.489E-09
35.00	5.302E-09	5.356E-09	3.860E-09	3.836E-09	3.474E-09	7.336E-09	5.251E-09	5.224E-09
40.00	4.395E-09	4.442E-09	3.199E-09	3.184E-09	2.892E-09	6.124E-09	4.359E-09	4.329E-09
45.00	3.724E-09	3.767E-09	2.711E-09	2.701E-09	2.460E-09	5.222E-09	3.699E-09	3.668E-09
50.00	3.211E-09	3.249E-09	2.337E-09	2.331E-09	2.128E-09	4.527E-09	3.194E-09	3.163E-09

TABLE 2.3-32

VENT DEPLETED X/Q AT GROUND
LONG TERM GROUND-LEVEL ROUTINE GASEOUS RELEASES
ANNUAL AVERAGE DEPLETED X/Q BY SECTOR

(s/m^3)
1977-1984

Downwind Sector

Distance, Miles	NNE	NE	ENE	E	ESE	SE	SSE	S
0.25	5.222E-06	5.591E-06	4.725E-06	5.395E-06	5.696E-06	7.376E-06	5.446E-06	7.637E-06
0.50	1.650E-06	1.762E-06	1.497E-06	1.724E-06	1.829E-06	2.344E-06	1.745E-06	2.418E-06
0.75	9.227E-07	9.806E-07	8.328E-07	9.611E-07	1.020E-06	1.317E-06	9.772E-07	1.342E-06
1.00	5.967E-07	6.326E-07	5.370E-07	6.201E-07	6.583E-07	8.539E-07	6.320E-07	8.641E-07
1.50	3.233E-07	3.432E-07	2.912E-07	3.340E-07	3.543E-07	4.618E-07	3.406E-07	4.708E-07
2.00	2.064E-07	2.194E-07	1.861E-07	2.124E-07	2.252E-07	2.943E-07	2.166E-07	3.018E-07
2.50	1.448E-07	1.542E-07	1.306E-07	1.486E-07	1.574E-07	2.060E-07	1.514E-07	2.123E-07
3.00	1.081E-07	1.153E-07	9.756E-08	1.107E-07	1.171E-07	1.532E-07	1.126E-07	1.588E-07
3.40	8.944E-08	9.556E-08	8.076E-08	9.148E-08	9.661E-08	1.265E-07	9.295E-08	1.316E-07
3.50	8.570E-08	9.160E-08	7.739E-08	8.762E-08	9.252E-08	1.212E-07	8.900E-08	1.261E-07
3.60	8.222E-08	8.790E-08	7.425E-08	8.403E-08	8.870E-08	1.162E-07	8.533E-08	1.210E-07
3.70	7.896E-08	8.445E-08	7.132E-08	8.067E-08	8.513E-08	1.115E-07	8.190E-08	1.163E-07
3.80	7.591E-08	8.121E-08	6.857E-08	7.754E-08	8.180E-08	1.072E-07	7.870E-08	1.118E-07
3.90	7.306E-08	7.818E-08	6.599E-08	7.459E-08	7.868E-08	1.031E-07	7.569E-08	1.076E-07
4.00	7.038E-08	7.533E-08	6.357E-08	7.183E-08	7.575E-08	9.928E-08	7.287E-08	1.037E-07
4.10	6.785E-08	7.265E-08	6.130E-08	6.924E-08	7.299E-08	9.568E-08	7.023E-08	1.000E-07
4.20	6.548E-08	7.013E-08	5.916E-08	6.679E-08	7.040E-08	9.229E-08	6.773E-08	9.654E-08
4.30	6.324E-08	6.775E-08	5.713E-08	6.449E-08	6.796E-08	8.909E-08	6.538E-08	9.326E-08
4.40	6.112E-08	6.550E-08	5.523E-08	6.231E-08	6.565E-08	8.608E-08	6.316E-08	9.016E-08
4.50	5.912E-08	6.337E-08	5.342E-08	6.026E-08	6.347E-08	8.322E-08	6.106E-08	8.723E-08
4.60	5.722E-08	6.135E-08	5.171E-08	5.831E-08	6.141E-08	8.053E-08	5.908E-08	8.445E-08
4.70	5.543E-08	5.944E-08	5.009E-08	5.647E-08	5.946E-08	7.797E-08	5.720E-08	8.182E-08
4.80	5.373E-08	5.763E-08	4.856E-08	5.472E-08	5.761E-08	7.555E-08	5.542E-08	7.932E-08
4.90	5.211E-08	5.591E-08	4.710E-08	5.306E-08	5.585E-08	7.325E-08	5.373E-08	7.695E-08
5.00	5.057E-08	5.427E-08	4.571E-08	5.148E-08	5.418E-08	7.106E-08	5.212E-08	7.469E-08
7.50	2.695E-08	2.905E-08	2.438E-08	2.731E-08	2.864E-08	3.761E-08	2.754E-08	3.995E-08
10.00	1.684E-08	1.822E-08	1.525E-08	1.702E-08	1.781E-08	2.339E-08	1.712E-08	2.503E-08
15.00	8.669E-09	9.418E-09	7.860E-09	8.730E-09	9.102E-09	1.196E-08	8.743E-09	1.292E-08
20.00	5.406E-09	5.891E-09	4.906E-09	5.432E-09	5.650E-09	7.424E-09	5.425E-09	8.075E-09
25.00	3.746E-09	4.092E-09	3.402E-09	3.758E-09	3.902E-09	5.127E-09	3.745E-09	5.604E-09
30.00	2.776E-09	3.037E-09	2.522E-09	2.781E-09	2.883E-09	3.788E-09	2.766E-09	4.157E-09
35.00	2.154E-09	2.360E-09	1.958E-09	2.156E-09	2.232E-09	2.932E-09	2.141E-09	3.228E-09
40.00	1.729E-09	1.897E-09	1.572E-09	1.729E-09	1.788E-09	2.349E-09	1.715E-09	2.593E-09
45.00	1.424E-09	1.564E-09	1.295E-09	1.423E-09	1.471E-09	1.931E-09	1.410E-09	2.138E-09
50.00	1.197E-09	1.317E-09	1.089E-09	1.196E-09	1.235E-09	1.621E-09	1.183E-09	1.798E-09

TABLE 2.3-32 (Cont)

Distance, Miles	SSW	SW	WSW	W	WNW	NW	NNW	N
	Downwind Sector							
0.25	6.052E-06	6.055E-06	4.388E-06	4.227E-06	3.640E-06	7.650E-06	6.018E-06	6.089E-06
0.50	1.920E-06	1.917E-06	1.389E-06	1.321E-06	1.128E-06	2.391E-06	1.919E-06	1.942E-06
0.75	1.066E-06	1.063E-06	7.724E-07	7.339E-07	6.189E-07	1.291E-06	1.056E-06	1.078E-06
1.00	6.867E-07	6.839E-07	4.980E-07	4.734E-07	3.969E-07	8.197E-07	6.762E-07	6.936E-07
1.50	3.730E-07	3.712E-07	2.707E-07	2.586E-07	2.173E-07	4.425E-07	3.639E-07	3.741E-07
2.00	2.387E-07	2.375E-07	1.733E-07	1.661E-07	1.400E-07	2.828E-07	2.316E-07	2.383E-07
2.50	1.678E-07	1.670E-07	1.218E-07	1.171E-07	9.908E-08	1.998E-07	1.625E-07	1.671E-07
3.00	1.254E-07	1.250E-07	9.112E-08	8.780E-08	7.465E-08	1.510E-07	1.216E-07	1.248E-07
3.40	1.039E-07	1.036E-07	7.551E-08	7.289E-08	6.219E-08	1.260E-07	1.008E-07	1.033E-07
3.50	9.962E-08	9.934E-08	7.238E-08	6.990E-08	5.968E-08	1.210E-07	9.662E-08	9.898E-08
3.60	9.559E-08	9.534E-08	6.945E-08	6.710E-08	5.734E-08	1.163E-07	9.272E-08	9.496E-08
3.70	9.183E-08	9.160E-08	6.672E-08	6.449E-08	5.515E-08	1.119E-07	8.908E-08	9.120E-08
3.80	8.831E-08	8.810E-08	6.417E-08	6.204E-08	5.310E-08	1.077E-07	8.567E-08	8.769E-08
3.90	8.501E-08	8.481E-08	6.177E-08	5.975E-08	5.117E-08	1.038E-07	8.248E-08	8.439E-08
4.00	8.190E-08	8.173E-08	5.952E-08	5.759E-08	4.936E-08	1.002E-07	7.948E-08	8.130E-08
4.10	7.899E-08	7.883E-08	5.740E-08	5.556E-08	4.765E-08	9.677E-08	7.665E-08	7.839E-08
4.20	7.624E-08	7.610E-08	5.540E-08	5.365E-08	4.604E-08	9.353E-08	7.399E-08	7.565E-08
4.30	7.364E-08	7.352E-08	5.352E-08	5.184E-08	4.452E-08	9.047E-08	7.148E-08	7.306E-08
4.40	7.119E-08	7.108E-08	5.174E-08	5.013E-08	4.308E-08	8.758E-08	6.911E-08	7.062E-08
4.50	6.888E-08	6.878E-08	5.006E-08	4.852E-08	4.172E-08	8.484E-08	6.687E-08	6.831E-08
4.60	6.668E-08	6.659E-08	4.846E-08	4.699E-08	4.043E-08	8.224E-08	6.475E-08	6.613E-08
4.70	6.460E-08	6.452E-08	4.695E-08	4.554E-08	3.921E-08	7.977E-08	6.273E-08	6.405E-08
4.80	6.263E-08	6.256E-08	4.552E-08	4.416E-08	3.804E-08	7.743E-08	6.083E-08	6.209E-08
4.90	6.075E-08	6.069E-08	4.416E-08	4.285E-08	3.694E-08	7.520E-08	5.901E-08	6.022E-08
5.00	5.897E-08	5.892E-08	4.286E-08	4.161E-08	3.588E-08	7.308E-08	5.729E-08	5.845E-08
7.50	3.153E-08	3.157E-08	2.293E-08	2.238E-08	1.951E-08	5.000E-08	3.072E-08	3.118E-08
10.00	1.975E-08	1.981E-08	1.437E-08	1.408E-08	1.237E-08	2.548E-08	1.929E-08	1.951E-08
15.00	1.020E-08	1.025E-08	7.419E-09	7.306E-09	6.484E-09	1.346E-08	9.998E-09	1.006E-08
20.00	6.372E-09	6.416E-09	4.637E-09	4.581E-09	4.094E-09	8.548E-09	6.267E-09	6.282E-09
25.00	4.422E-09	4.458E-09	3.218E-09	3.187E-09	2.864E-09	6.006E-09	4.360E-09	4.358E-09
30.00	3.280E-09	3.310E-09	2.387E-09	2.369E-09	2.138E-09	4.500E-09	3.242E-09	3.232E-09
35.00	2.547E-09	2.573E-09	1.854E-09	1.843E-09	1.669E-09	3.524E-09	2.522E-09	2.510E-09
40.00	2.046E-09	2.068E-09	1.489E-09	1.482E-09	1.347E-09	2.851E-09	2.030E-09	2.016E-09
45.00	1.687E-09	1.706E-09	1.228E-09	1.223E-09	1.114E-09	2.365E-09	1.676E-09	1.662E-09
50.00	1.419E-09	1.436E-09	1.033E-09	1.030E-09	9.406E-10	2.001E-09	1.411E-09	1.398E-09

TABLE 2.3-33

VENT D/Q AT GROUND LEVEL
LONG TERM GROUND-LEVEL ROUTINE GASEOUS RELEASES
ANNUAL AVERAGE D/Q BY SECTOR

$(1/m^2)$
1977-1984

Downwind Sector

Distance, Miles	NNE	NE	ENE	E	ESE	SE	SSE	S
0.25	3.271E-08	3.214E-08	3.079E-08	4.145E-08	4.812E-08	6.193E-08	3.763E-08	4.270E-08
0.50	1.104E-08	1.085E-08	1.040E-08	1.400E-08	1.625E-08	2.091E-08	1.271E-08	1.442E-08
0.75	5.933E-09	5.830E-09	5.586E-09	7.518E-09	8.728E-09	1.123E-08	6.826E-09	7.746E-09
1.00	3.597E-09	3.535E-09	3.386E-09	4.558E-09	5.291E-09	6.810E-09	4.138E-09	4.696E-09
1.50	1.777E-09	1.746E-09	1.673E-09	2.251E-09	2.614E-09	3.364E-09	2.044E-09	2.320E-09
2.00	1.077E-09	1.059E-09	1.014E-09	1.365E-09	1.585E-09	2.039E-09	1.239E-09	1.406E-09
2.50	7.306E-10	7.180E-10	6.879E-10	9.259E-10	1.075E-09	1.383E-09	8.407E-10	9.540E-10
3.00	5.321E-10	5.229E-10	5.009E-10	6.743E-10	7.827E-10	1.007E-09	6.122E-10	6.947E-10
3.40	4.280E-10	4.206E-10	4.029E-10	5.423E-10	6.296E-10	8.103E-10	4.924E-10	5.588E-10
3.50	4.069E-10	3.999E-10	3.831E-10	5.157E-10	5.986E-10	7.705E-10	4.682E-10	5.313E-10
3.60	3.875E-10	3.808E-10	3.648E-10	4.910E-10	5.700E-10	7.336E-10	4.458E-10	5.059E-10
3.70	3.694E-10	3.630E-10	3.478E-10	4.682E-10	5.435E-10	6.995E-10	4.251E-10	4.823E-10
3.80	3.527E-10	3.466E-10	3.321E-10	4.469E-10	5.188E-10	6.678E-10	4.058E-10	4.605E-10
3.90	3.371E-10	3.313E-10	3.174E-10	4.272E-10	4.959E-10	6.383E-10	3.879E-10	4.401E-10
4.00	3.226E-10	3.170E-10	3.037E-10	4.088E-10	4.746E-10	6.108E-10	3.712E-10	4.212E-10
4.10	3.090E-10	3.037E-10	2.909E-10	3.916E-10	4.546E-10	5.851E-10	3.555E-10	4.035E-10
4.20	2.963E-10	2.912E-10	2.790E-10	3.755E-10	4.359E-10	5.611E-10	3.410E-10	3.869E-10
4.30	2.844E-10	2.795E-10	2.678E-10	3.605E-10	4.185E-10	5.386E-10	3.273E-10	3.714E-10
4.40	2.733E-10	2.686E-10	2.573E-10	3.463E-10	4.020E-10	5.175E-10	3.145E-10	3.568E-10
4.50	2.628E-10	2.583E-10	2.474E-10	3.331E-10	3.866E-10	4.976E-10	3.024E-10	3.431E-10
4.60	2.530E-10	2.486E-10	2.382E-10	3.206E-10	3.721E-10	4.790E-10	2.911E-10	3.303E-10
4.70	2.437E-10	2.395E-10	2.294E-10	3.088E-10	3.585E-10	4.614E-10	2.804E-10	3.181E-10
4.80	2.349E-10	2.309E-10	2.212E-10	2.977E-10	3.456E-10	4.448E-10	2.703E-10	3.067E-10
4.90	2.266E-10	2.227E-10	2.134E-10	2.872E-10	3.334E-10	4.291E-10	2.608E-10	2.959E-10
5.00	2.188E-10	2.150E-10	2.060E-10	2.773E-10	3.219E-10	4.143E-10	2.518E-10	2.857E-10
7.50	1.081E-10	1.062E-10	1.018E-10	1.370E-10	1.590E-10	2.046E-10	1.244E-10	1.411E-10
10.00	6.553E-11	6.439E-11	6.169E-11	8.304E-11	9.640E-11	1.241E-10	7.539E-11	8.555E-11
15.00	3.237E-11	3.181E-11	3.047E-11	4.102E-11	4.762E-11	6.128E-11	3.724E-11	4.226E-11
20.00	1.983E-11	1.948E-11	1.867E-11	2.512E-11	2.917E-11	3.754E-11	2.281E-11	2.588E-11
25.00	1.321E-11	1.298E-11	1.244E-11	1.674E-11	1.943E-11	2.501E-11	1.520E-11	1.724E-11
30.00	9.478E-12	9.314E-12	8.924E-12	1.201E-11	1.394E-11	1.795E-11	1.091E-11	1.237E-11
35.00	7.159E-12	7.036E-12	6.741E-12	9.073E-12	1.053E-11	1.356E-11	8.237E-12	9.347E-12
40.00	5.615E-12	5.518E-12	5.286E-12	7.115E-12	8.260E-12	1.063E-11	6.460E-12	7.331E-12
45.00	4.531E-12	4.453E-12	4.266E-12	5.742E-12	6.666E-12	8.579E-12	5.214E-12	5.916E-12
50.00	3.740E-12	3.676E-12	3.522E-12	4.740E-12	5.503E-12	7.082E-12	4.304E-12	4.884E-12

TABLE 2.3-33 (Cont)

Downwind Sector

Distance, Miles	SSW	SW	WSW	W	WNW	NW	NNW	N
0.25	3.225E-08	2.970E-08	1.905E-08	1.669E-08	1.469E-08	4.887E-08	3.925E-08	3.728E-08
0.50	1.089E-08	1.003E-08	6.435E-09	5.638E-09	4.962E-09	1.650E-08	1.325E-08	1.259E-08
0.75	5.849E-09	5.387E-09	3.456E-09	3.028E-09	2.665E-09	8.865E-09	7.119E-09	6.761E-09
1.00	3.546E-09	3.266E-09	2.095E-09	1.836E-09	1.616E-09	5.374E-09	4.316E-09	4.099E-09
1.50	1.752E-09	1.613E-09	1.035E-09	9.069E-10	7.982E-10	2.655E-09	2.132E-09	2.025E-09
2.00	1.062E-09	9.781E-10	6.275E-10	5.498E-10	4.839E-10	1.609E-09	1.293E-09	1.228E-09
2.50	7.204E-10	6.635E-10	4.257E-10	3.729E-10	3.282E-10	1.092E-09	8.768E-10	8.327E-10
3.00	5.246E-10	4.831E-10	3.100E-10	2.716E-10	2.390E-10	7.950E-10	6.385E-10	6.064E-10
3.40	4.220E-10	3.886E-10	2.493E-10	2.184E-10	1.923E-10	6.395E-10	5.136E-10	4.877E-10
3.50	4.012E-10	3.695E-10	2.371E-10	2.077E-10	1.828E-10	6.080E-10	4.883E-10	4.638E-10
3.60	3.820E-10	3.518E-10	2.257E-10	1.978E-10	1.741E-10	5.789E-10	4.650E-10	4.416E-10
3.70	3.642E-10	3.355E-10	2.152E-10	1.886E-10	1.660E-10	5.520E-10	4.433E-10	4.210E-10
3.80	3.477E-10	3.203E-10	2.055E-10	1.800E-10	1.584E-10	5.270E-10	4.232E-10	4.019E-10
3.90	3.324E-10	3.061E-10	1.964E-10	1.721E-10	1.514E-10	5.037E-10	4.045E-10	3.842E-10
4.00	3.180E-10	2.929E-10	1.879E-10	1.647E-10	1.449E-10	4.820E-10	3.871E-10	3.676E-10
4.10	3.047E-10	2.806E-10	1.800E-10	1.577E-10	1.388E-10	4.617E-10	3.708E-10	3.522E-10
4.20	2.922E-10	2.691E-10	1.726E-10	1.513E-10	1.331E-10	4.428E-10	3.556E-10	3.377E-10
4.30	2.805E-10	2.583E-10	1.657E-10	1.452E-10	1.278E-10	4.250E-10	3.413E-10	3.242E-10
4.40	2.695E-10	2.482E-10	1.592E-10	1.395E-10	1.228E-10	4.084E-10	3.280E-10	3.115E-10
4.50	2.591E-10	2.387E-10	1.531E-10	1.341E-10	1.181E-10	3.927E-10	3.154E-10	2.995E-10
4.60	2.494E-10	2.297E-10	1.474E-10	1.291E-10	1.136E-10	3.780E-10	3.036E-10	2.883E-10
4.70	2.402E-10	2.213E-10	1.420E-10	1.244E-10	1.095E-10	3.641E-10	2.924E-10	2.777E-10
4.80	2.316E-10	2.133E-10	1.369E-10	1.199E-10	1.055E-10	3.510E-10	2.819E-10	2.677E-10
4.90	2.235E-10	2.058E-10	1.320E-10	1.157E-10	1.018E-10	3.386E-10	2.720E-10	2.583E-10
5.00	2.157E-10	1.987E-10	1.275E-10	1.117E-10	9.830E-11	3.269E-10	2.626E-10	2.494E-10
7.50	1.066E-10	9.814E-11	6.297E-11	5.517E-11	4.856E-11	1.615E-10	1.297E-10	1.232E-10
10.00	6.461E-11	5.950E-11	3.818E-11	3.345E-11	2.944E-11	9.791E-11	7.863E-11	7.468E-11
15.00	3.191E-11	2.939E-11	1.886E-11	1.652E-11	1.454E-11	4.836E-11	3.884E-11	3.689E-11
20.00	1.955E-11	1.800E-11	1.155E-11	1.012E-11	8.907E-12	2.962E-11	2.379E-11	2.259E-11
25.00	1.302E-11	1.199E-11	7.695E-12	6.742E-12	5.934E-12	1.974E-11	1.585E-11	1.505E-11
30.00	9.345E-12	8.607E-12	5.522E-12	4.838E-12	4.258E-12	1.416E-11	1.137E-11	1.080E-11
35.00	7.059E-12	6.501E-12	4.171E-12	3.654E-12	3.216E-12	1.070E-11	8.591E-11	8.159E-12
40.00	5.536E-12	5.098E-12	3.271E-12	2.866E-12	2.522E-12	8.389E-12	6.738E-12	6.399E-12
45.00	4.468E-12	4.115E-12	2.640E-12	2.313E-12	2.036E-12	6.771E-12	5.437E-12	5.164E-12
50.00	3.688E-12	3.397E-12	2.179E-12	1.909E-12	1.680E-12	5.589E-12	4.489E-12	4.263E-12

TABLE 2.3-34

YEARLY PRECIPITATION TOTALS,

1977 to 1981

(INCHES)

Year	Onsite	Wilmington	Glassboro	Woodstown
1977	37.5	40.13	43.88	41.38
1978	33.3	51.28	39.54	44.44
1979	43.4	53.31	54.16	57.66
1980	24.8	33.92	33.06	36.43
1981	35.4	35.28	44.27	38.06

TABLE 2.3-35

PRECIPITATION STATISTICS,

1977 to 1981

	Onsite	Wilmington	Glassboro	Woodstown
Mean (Inches)	34.9 (37.4)[1]	42.78 (45.00)	42.98 (45.46)	43.59 (45.39)
Standard Deviation (Inches)	6.8 (4.4)	9.01 (8.70)	7.71 (6.18)	8.45 (8.59)
Standard Error (Inches)	3.0 (2.2)	4.03 (4.35)	3.45 (3.09)	3.78 (4.29)

(1) Numbers in parenthesis are based on data from 1977 to 1979 and 1981 with 1980 data omitted from the analysis.

TABLE 2.3-36

WIND DIRECTION DISTRIBUTIONS
ARTIFICIAL ISLAND
JUNE 1969 TO MAY 1971

Wind Elevation	33 Feet		150 Feet		300 Feet	
Sector	6/69 to 5/70	6/70 to 5/71	1/70 to 5/70	6/70 to 5/71	6/69 to 5/70	6/70 to 5/71
NNE	6.5%	4.8%	5.6%	3.7%	4.9%	3.6%
NE	3.8	5.6	6.4	5.0	5.1	4.4
ENE	3.2	3.7	4.3	4.0	3.1	3.3
E	3.3	3.1	3.5	3.5	3.2	4.0
ESE	4.0	2.6	4.1	2.2	2.0	2.2
SE	8.2	6.8	10.9	6.1	3.5	4.9
SSE	6.8	7.9	6.8	8.3	7.6	6.4
S	7.7	7.9	4.5	7.7	8.5	8.8
SSW	5.5	6.0	3.8	6.0	5.1	6.1
SW	4.7	5.0	6.1	6.5	6.9	5.6
WSW	5.1	5.3	5.4	5.1	5.2	5.5
W	10.2	9.9	8.1	9.4	8.3	9.6
WNW	10.8	9.3	8.1	8.3	7.1	9.4
NW	8.0	10.6	12.2	11.2	11.9	12.6
NNW	4.9	6.5	4.5	7.3	9.5	7.0
N	7.2	5.0	5.6	5.7	8.2	6.7
Missing	6.9%	3.5%	38%	4.1%	16.1%	5.0%
Good Hours	8152	8434	2247	8375	7353	8295
Total Hours	8760	8736 (1)	3624	8736 (1)	8760	8736 (1)

(1) May 16, 1971 data omitted.

TABLE 2.3-37

WIND DIRECTION DISTRIBUTIONS
ARTIFICIAL ISLAND

January 1977 to December 1981
Wind Elevation = 33 Feet

Sector	1977	1978	1979	1980	1981
	Year				
NNE	3.9	5.8	6.0	5.6	4.9
NE	4.5	6.1	5.6	3.8	4.0
ENE	3.8	3.4	3.8	2.1	2.8
E	2.8	3.4	2.8	1.9	3.0
ESE	2.7	2.6	2.9	1.9	2.8
SE	8.0	6.4	11.1	7.3	9.2
SSE	6.7	6.4	9.1	6.7	5.2
S	6.9	6.6	6.0	6.6	6.0
SSW	5.4	5.5	5.8	6.9	5.4
SW	6.5	6.0	4.8	6.0	6.5
WSW	6.5	6.0	4.8	4.7	6.7
W	9.4	8.0	6.2	5.7	9.6
WNW	10.0	9.4	7.7	9.3	11.2
NW	11.5	10.3	10.9	14.2	11.3
NNW	5.6	6.3	6.7	8.4	5.8
N	5.7	7.9	5.9	9.0	5.7
Missing	6.1%	7.5%	6.3%	11.7%	6.3%
Good Hours	8226	8100	8209	7761	8212
Total Hours	8760	8760	8760	8784	8760

TABLE 2.3-38

WIND DIRECTION DISTRIBUTIONS
ARTIFICIAL ISLAND

January 1977 to December 1981
Wind Elevation - 150 Feet

Sector	1977	1978	1979	1980	1981
NNE	3.8	5.6	5.4	5.3	4.2
NE	4.2	6.1	5.8	4.9	5.0
ENE	3.5	3.6	3.1	2.6	2.6
E	2.9	3.4	2.6	2.0	2.9
ESE	1.9	2.5	2.1	1.6	2.5
SE	6.8	6.8	6.9	5.2	6.4
SSE	7.2	5.8	9.7	6.8	6.2
S	7.6	6.6	8.2	6.2	5.8
SSW	5.9	5.4	6.3	6.6	5.7
SW	7.1	6.4	5.8	7.4	6.8
WSW	6.0	5.5	4.9	6.5	5.5
W	10.3	8.1	7.3	7.3	9.2
WNW	10.8	8.9	7.7	7.7	10.7
NW	11.8	10.4	11.8	12.7	13.8
NNW	5.2	7.1	6.9	9.3	7.7
N	5.0	7.8	5.6	7.8	5.1
Missing	2.5%	2.0%	4.4%	3.2%	1.2%
Good Hours	8539	8583	8377	8502	8657
Total Hours	8760	8760	8760	8784	8760

TABLE 2.3- 39

WIND DIRECTION DISTRIBUTIONS
ARTIFICIAL ISLAND

January 1977 to December 1981
Wind Elevation = 300 Feet

	Year				
Sector	1977	1978	1979	1980	1981
NNE	3.5	5.4	4.8	4.9	4.4
NE	4.0	5.5	5.7	4.5	3.5
ENE	3.4	4.0	2.5	2.9	2.9
E	2.9	3.1	2.8	2.1	2.9
ESE	1.9	2.2	2.1	1.3	2.0
SE	5.5	6.0	7.0	4.0	5.3
SSE	6.8	5.4	8.7	6.5	6.6
S	7.4	6.4	8.6	6.7	6.2
SSW	6.4	6.1	6.9	6.4	6.7
SW	8.5	7.3	6.4	7.8	7.1
WSW	5.6	5.6	4.9	6.4	5.9
W	11.0	8.9	7.6	8.1	10.5
WNW	9.3	8.8	8.3	7.4	10.3
NW	14.0	10.8	11.3	11.8	12.8
NNW	5.4	7.6	7.2	10.8	8.4
N	4.4	7.2	5.4	8.5	4.7
Missing	2.5%	2.4%	4.0%	3.2%	1.0%
Good Hours	8539	8554	8410	8502	8670
Total Hours	8760	8760	8760	8784	8760

TABLE 2.3-40

STABILITY DISTRIBUTIONS
ARTIFICIAL ISLAND
JUNE 1969 - MAY 1971

150 ft to 33 ft Delta Temperature
Stability Class

Data Period	A	B	C	D	E	F	G	Missing	Total
June 1969	625	[1]	380	2582	3105	1068	496	504	8760
to									
May 1970	7.6%	[1]	4.6%	31.3%	37.6%	12.9%	6.0%	-	

Percents based on 8256 good hours.

Missing data - 5.6%

Data Period	A	B	C	D	E	F	G	Missing	Total
June 1970	533	[1]	316	2544	3525	1135	591	92	8736[2]
to									
May 1971	6.2%	[1]	3.7%	29.4%	40.8%	13.1%	6.8%	-	

Percents based on 8644 good hours.

Missing data - 1.1%

300 ft to 33 ft Delta Temperature
Stability Class

Data Period	A	B	C	D	E	F	G	Missing	Total
June 1969	243	197	198	3728	2950	817	213	414	8760
to									
May 1970	2.9%	2.4%	2.4%	44.7%	35.4%	9.8%	2.4%	-	

Percent based on 8346 good hours.

Missing data - 4.7%

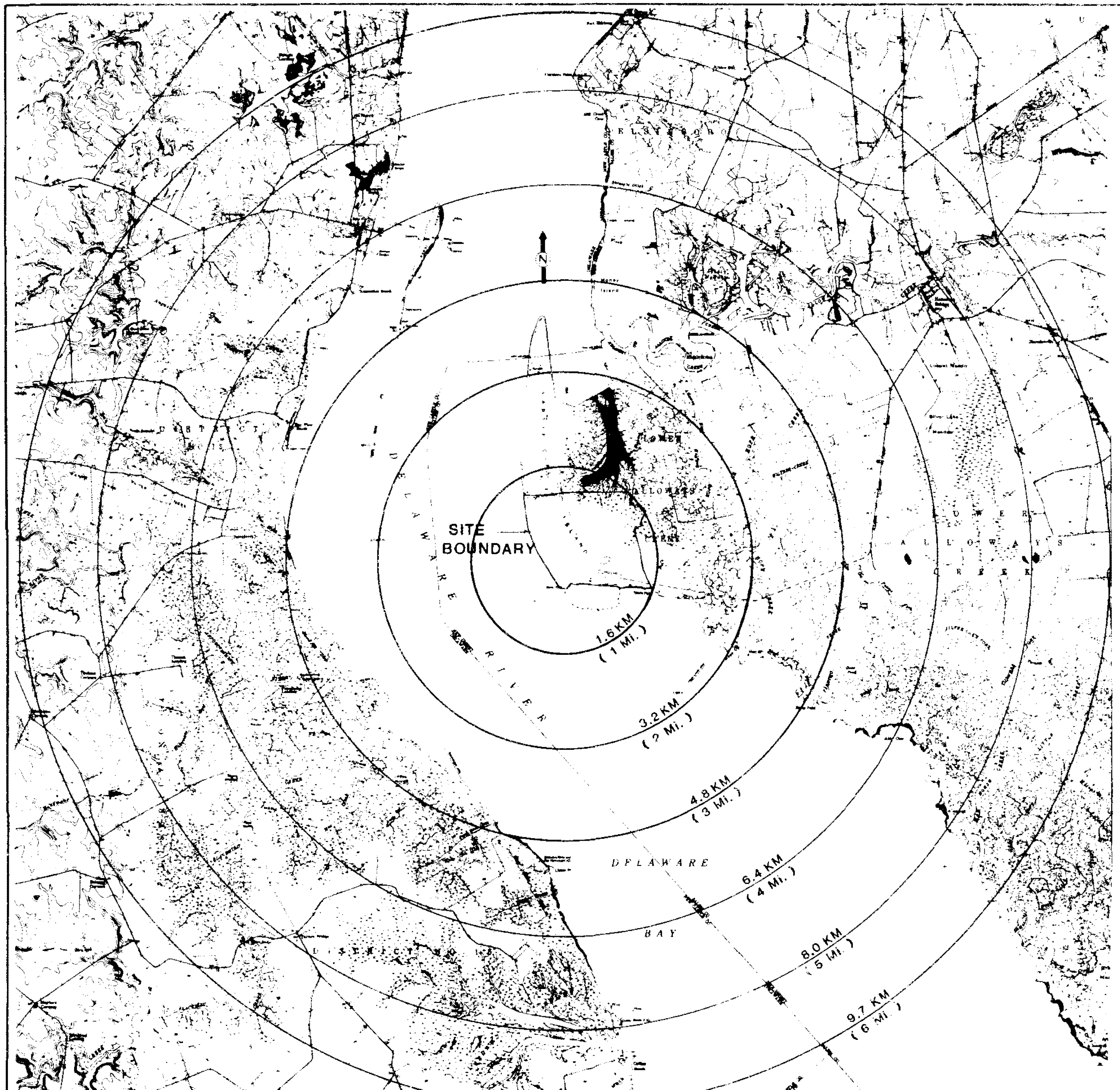
Data Period	A	B	C	D	E	F	G	Missing	Total
June 1969	546	272	221	3616	2985	773	230	93	8736[2]
to									
May 1970	6.3%	3.2%	2.6%	41.8%	34.5%	8.9%	2.7%	-	

Percents based on 8643 good hours.

---

(1) For a 36m (150-33 ft) vertical distance, NRC Class A stability computes to a delta temperature of less than or equal to -0.7°C. Stability Class C computes to -0.6°C. Therefore, stability Class B computes to greater than -0.6°C but less than -0.7°C. Since delta temperature is measured in the nearest 0.1°C, no counts occur in NRC Stability Class B.

(2) May 16, 1971 data omitted.



SCALE

1 0 1 2 3

MILES

1 0 1 2 3 4 5

KILOMETERS

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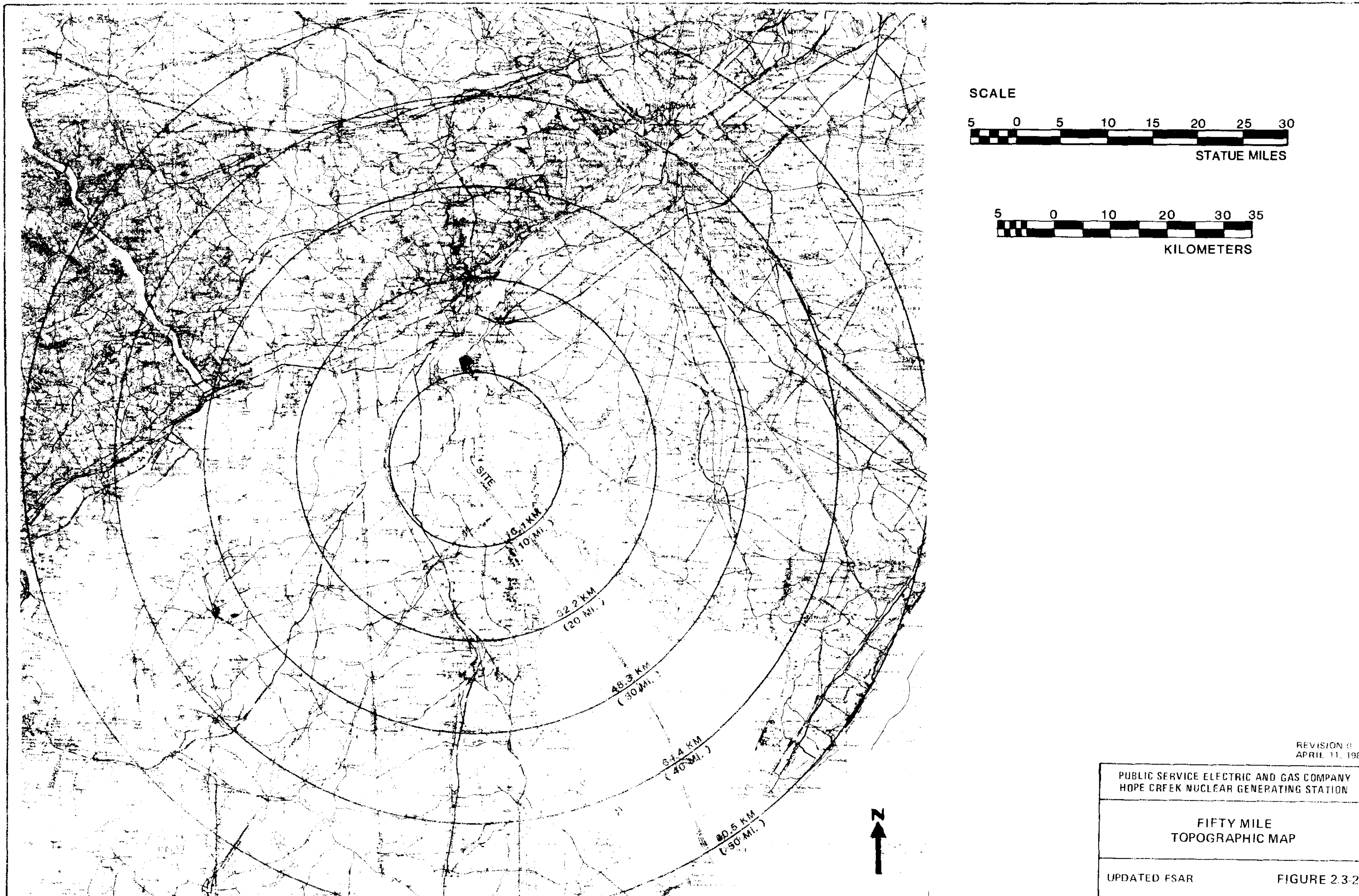
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

FIVE MILE
TOPOGRAPHIC MAP

UPDATED FSAR

FIGURE 2.3-1



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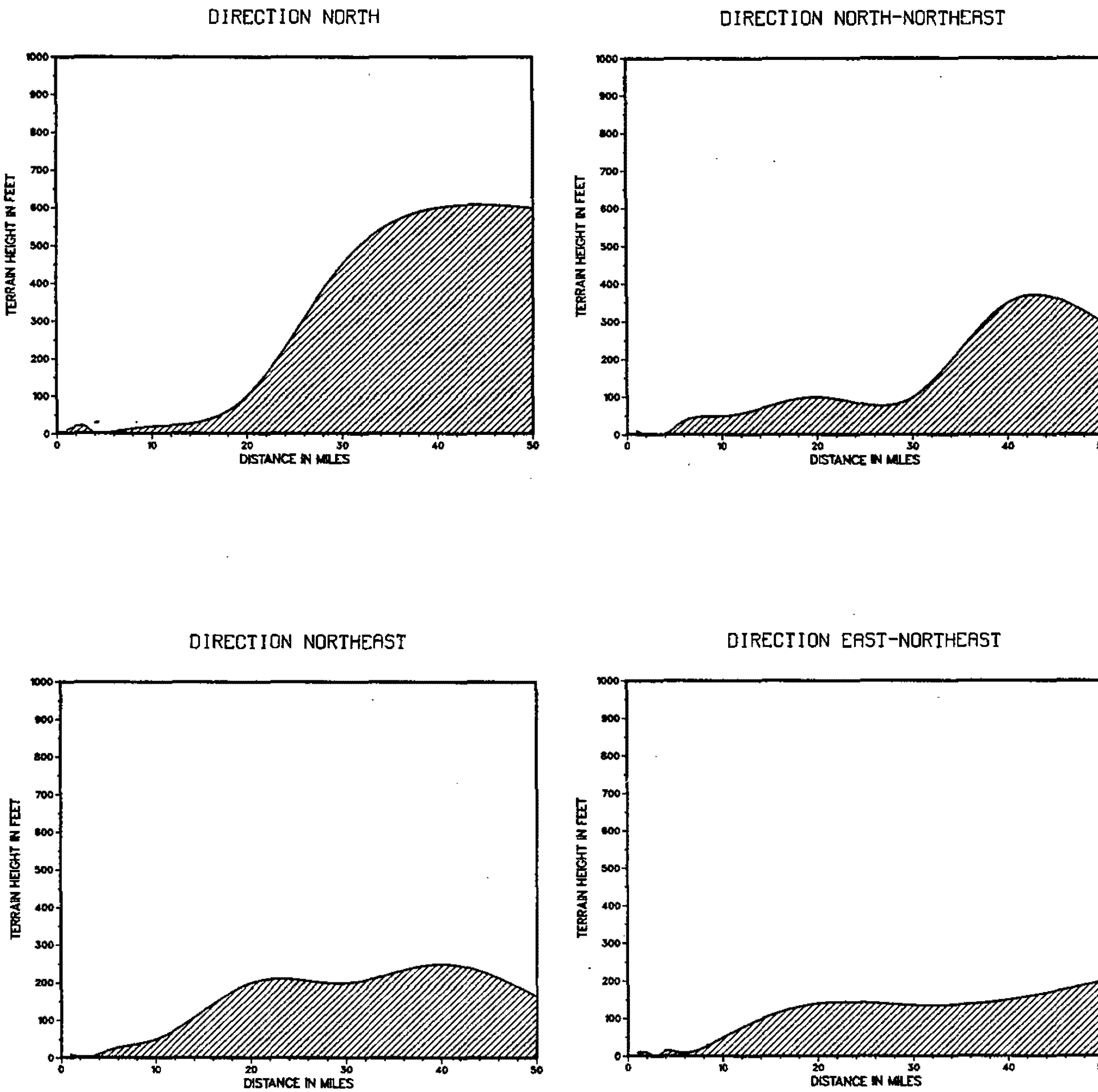
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

FIFTY MILE
TOPOGRAPHIC MAP

UPDATED FSAR

FIGURE 2.3-2



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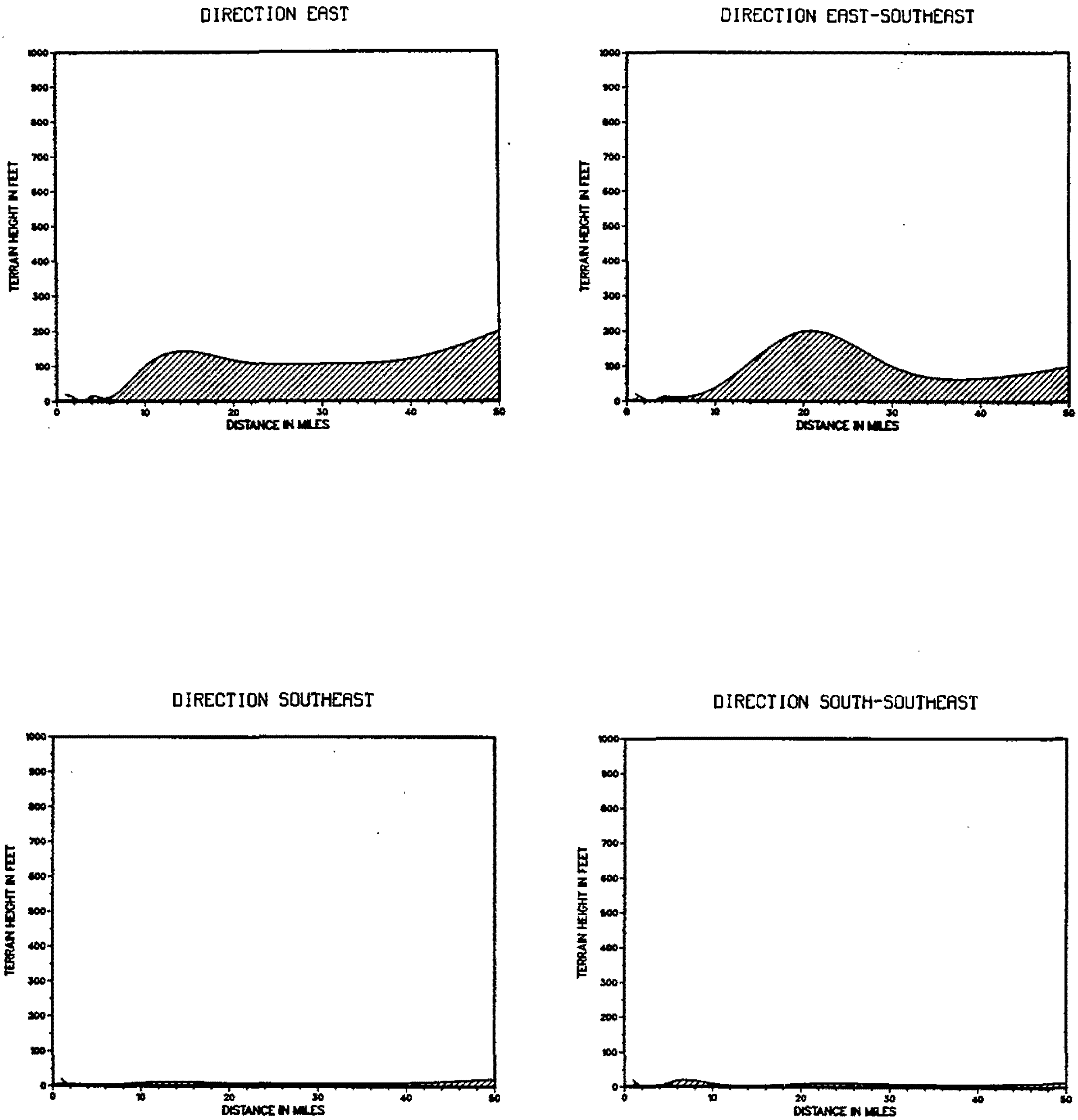
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

TERRAIN vs DISTANCE
DIRECTION N TO ENE

UPDATED FSAR

Sheet 1 of 4
FIGURE 2.3-3



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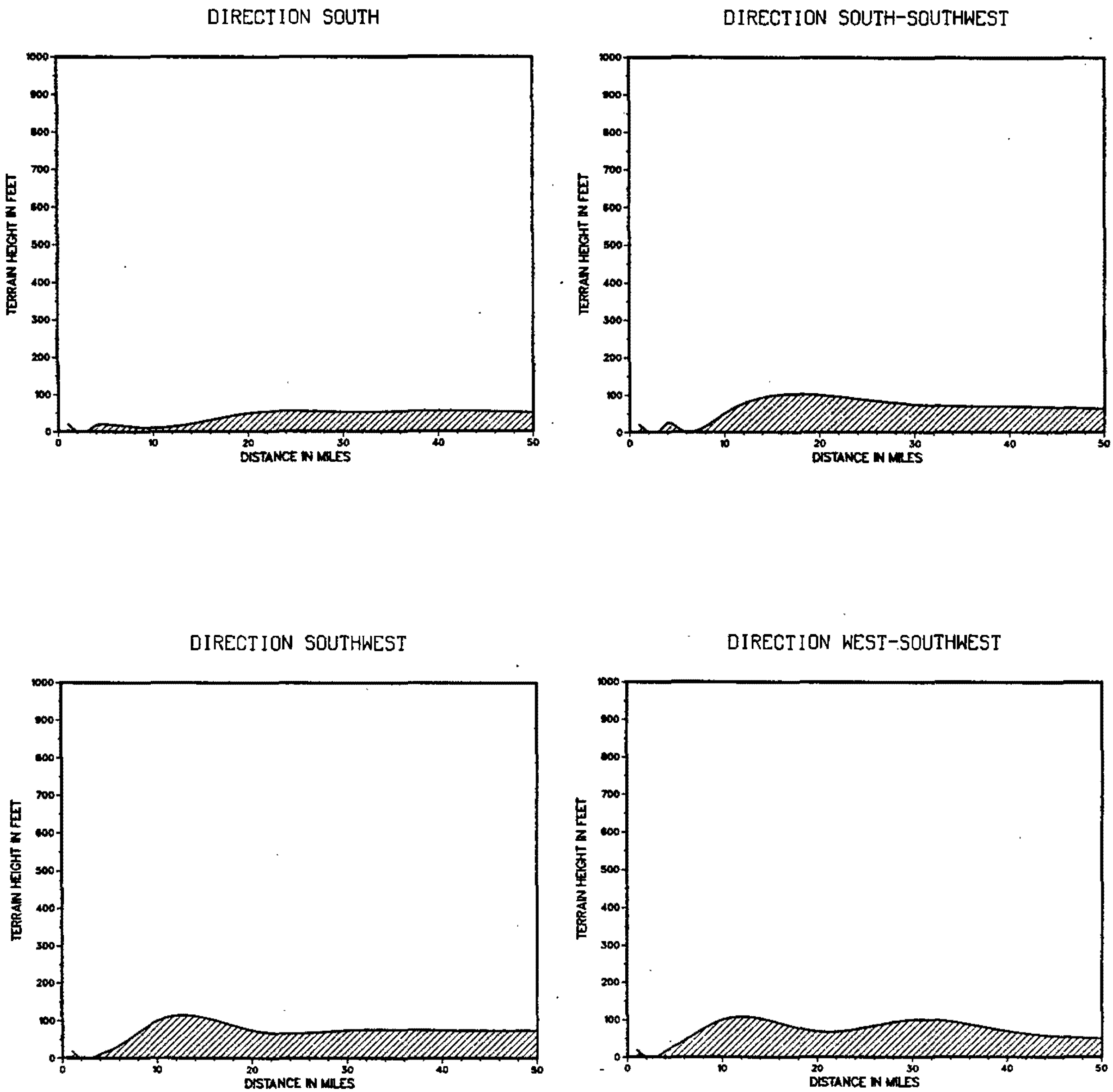
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

TERRAIN vs DISTANCE
DIRECTION E TO SSE

UPDATED FSAR

Sheet 2 of 4
FIGURE 2.3-3



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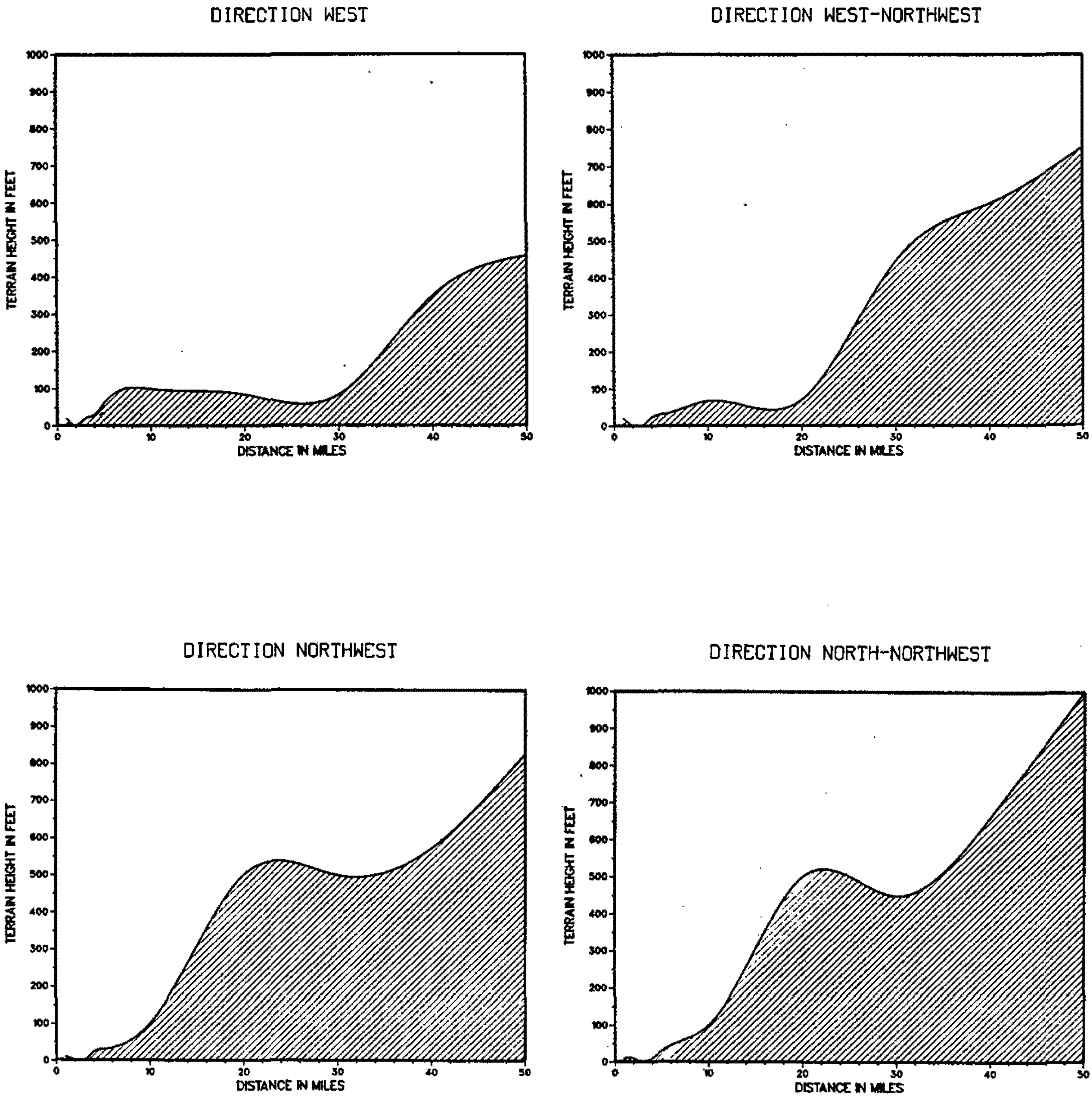
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

TERRAIN vs DISTANCE
DIRECTIONS S TO WSW

UPDATED FSAR

Sheet 3 of 4
FIGURE 2.3-3



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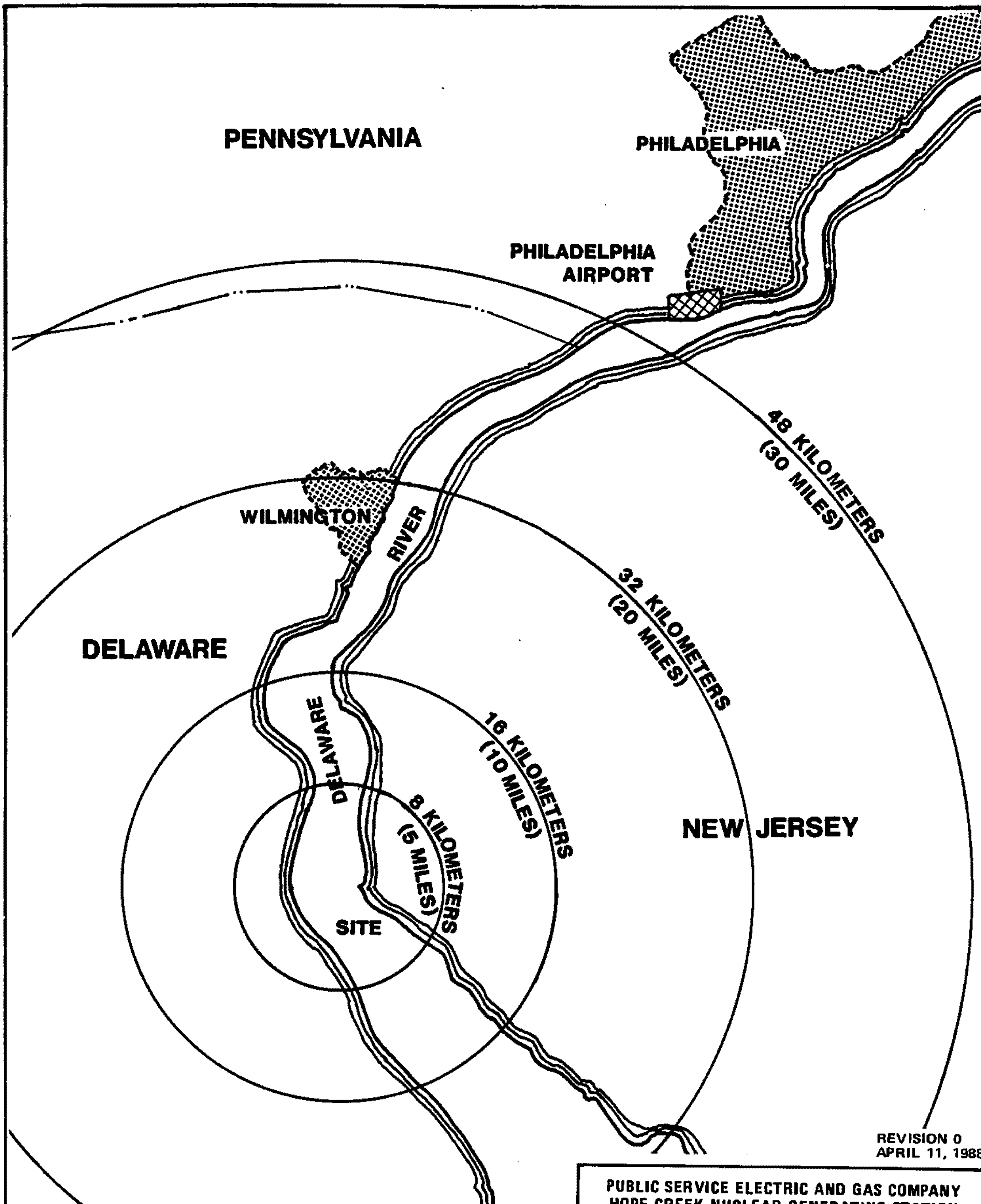
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

TERRAIN vs DISTANCE
DIRECTION W TO NNW

UPDATED FSAR

Sheet 4 of 4
FIGURE 2.3-3



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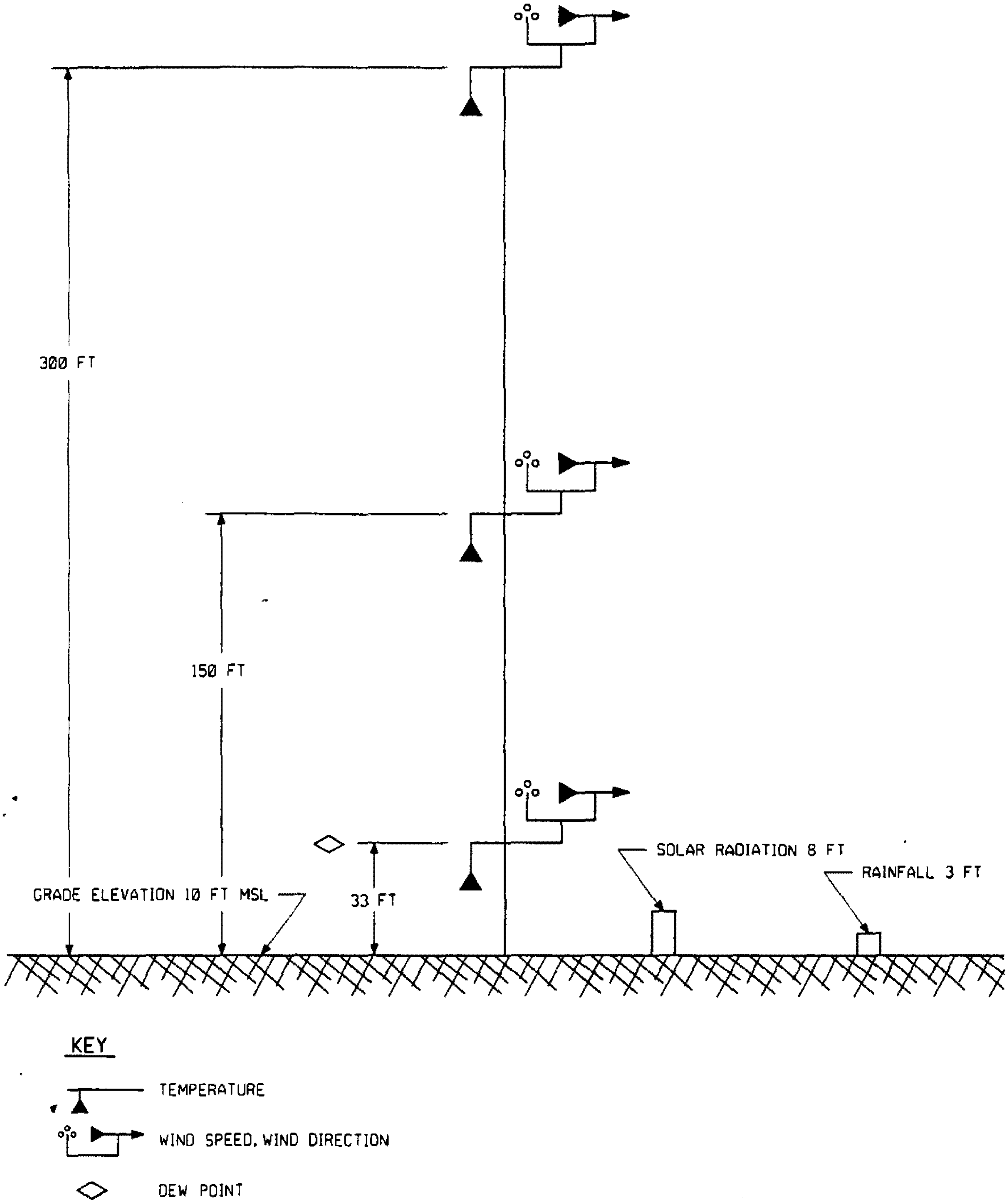
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

SOURCES OF DATA

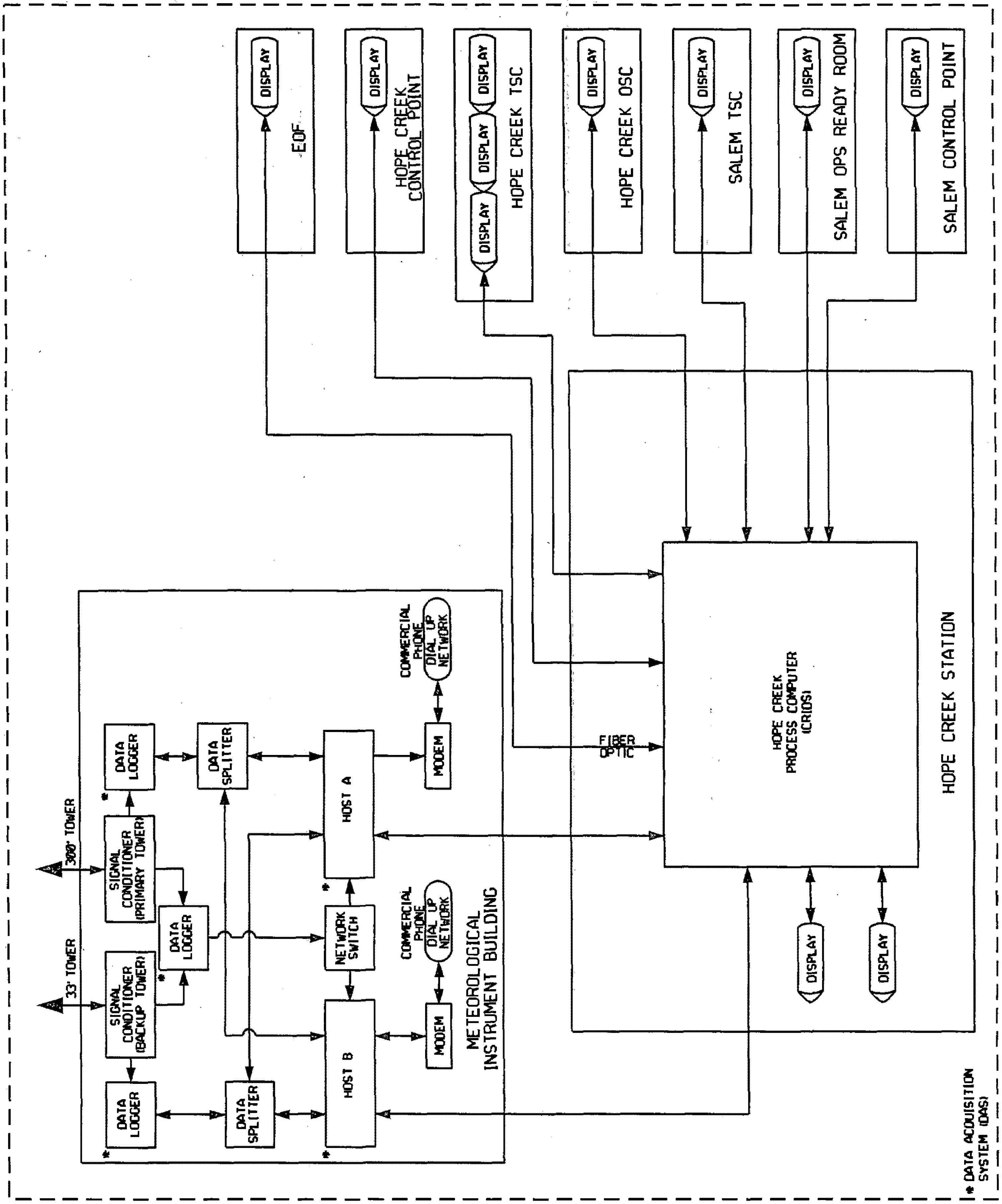
UPDATED FSAR

FIGURE 2.3-4



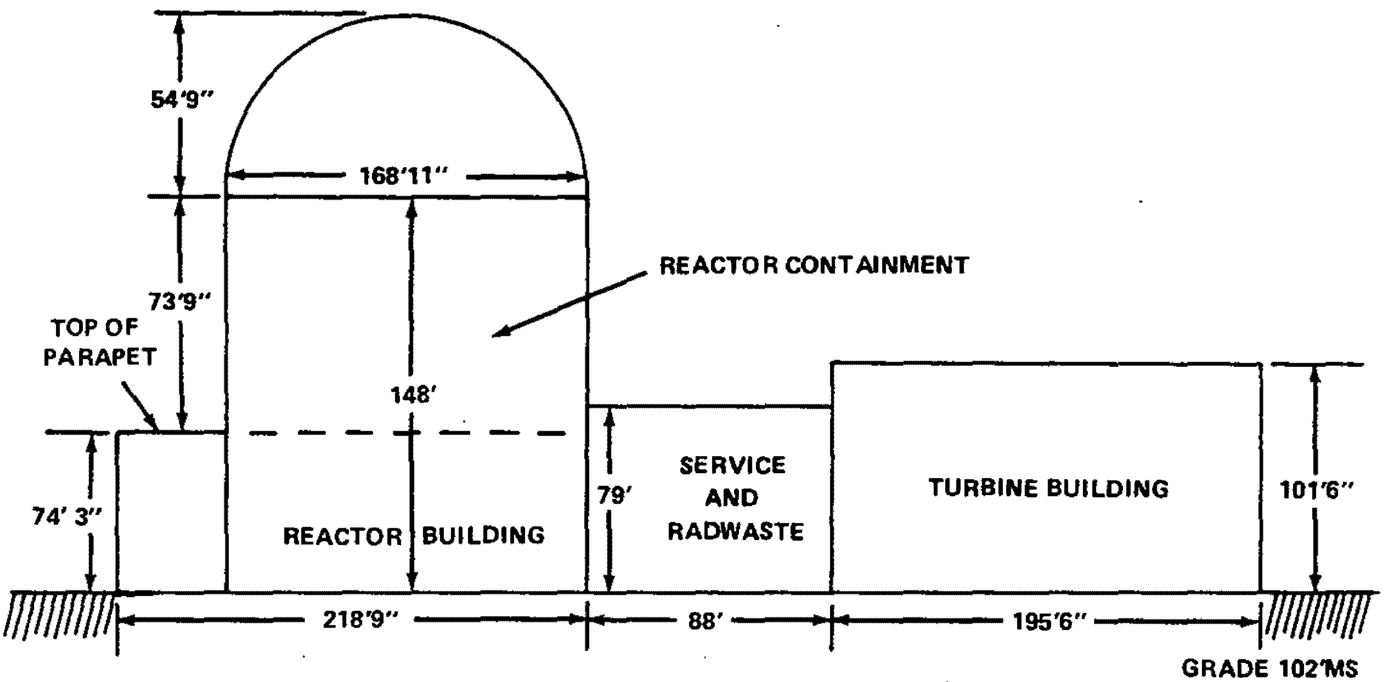
November 24, 2000

PSEG Nuclear, LLC HOPE CREEK NUCLEAR GENERATING STATION	Hope Creek Nuclear Generating Station METEOROLOGICAL TOWER SCHEMATIC
	Updated FSAR - Revision 11 — Figure 2.3-5



Revision 18, May 10, 2011

PSEG Nuclear, LLC HOPE CREEK NUCLEAR GENERATING STATION	Hope Creek Nuclear Generating Station METEOROLOGICAL DATA AQUISITION DISPLAY SYSTEM
	Updated FSAR Figure 2.3-6



REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

SMALLEST VERTICAL PLANE
CROSS-SECTIONAL AREA

UPDATED FSAR

FIGURE 2.3-7

2.4 HYDROLOGIC ENGINEERING

2.4.1 Hydrologic Description

2.4.1.1 Site and Facilities

The Hope Creek Generating Station (HCGS) is located on Artificial Island, an area that was previously a natural bar in the Delaware River, as shown on Figure 2.4-1 and Plant Drawing C-5018-0. The bar, and the area between the bar and the mainland, served as a disposal area for dredged material from the Delaware River. A wooden bulkhead constructed along the perimeter of the bar, around 1899-1901, initially contained the dredged material. Later, the addition of dikes connected Artificial Island to the mainland. Riprap, placed to stabilize the deteriorating bulkhead, now protects the shoreline of Artificial Island and its causeway.

Figure 2.4-9 shows the plant layout. The approximate grade of Artificial Island is 9 feet above mean sea level (MSL). In this document MSL refers to the National Geodetic Vertical Datum (NGVD) datum which at 0 feet is equal to the Public Service Electric and Gas Company datum of +89 feet. The plant is in an area of backfill with an elevation of 12.5 feet MSL. The Turbine and Auxiliary Building ground floor levels are at Elevation 13.0 feet MSL.

The site area is subject to flooding under the effects of a probable maximum hurricane surge. All Seismic Category I structures are flood proofed as indicated in Section 3.4.1 and Table 3.4-1 and are structurally designed to withstand the static and dynamic effects of the flood wave loading conditions as summarized in Table 2.4-11a.

The site area is generally flat with drainage flowing toward the Delaware River or into the marsh areas toward the mainland. The site drainage system consists of ditches that intercept runoff and lead it to below grade piping that, in turn, leads the runoff to discharge into the Delaware River. There are additional ditches that both intercept and convey runoff directly to the river. Section 3.4.1 provides further discussion of roof and site drainage.

2.4.1.2 Hydrosphere

2.4.1.2.1 Delaware River Estuary System

The HCGS Artificial Island location on the east shoreline of the Delaware River Estuary System is approximately 50.6 miles upstream of the estuary mouth. Cape May, New Jersey, and Cape Henlopen, Delaware, form the northern and southern limits of the estuary mouth, as shown on Figure 2.4-1. The Delaware River Basin Commission, as noted in Reference 2.4-1, defines the three major components of the estuary system as 1) Delaware Bay, 2) Delaware Estuary, and 3) Delaware River. Delaware Bay is legally defined as including the waters between the mouth of Delaware Bay (Cape May-Cape Henlopen transect) at River Mile 0 and Liston Point, Delaware, at River Mile 48.2. The Delaware Estuary extends from Liston Point up to the head of tide above Trenton at River Mile 133.4. HCGS is within the Delaware Estuary, being about 2.4 miles upstream of the Liston Point transect. The Delaware River extends upstream from the head of tide (River Mile 133.4).

The Delaware River Estuary System drains a basin of 12,765 mi^2, which includes parts of Delaware, New Jersey, Pennsylvania, and 8 mi^2 in Maryland, as shown on Figure 2.4-2. Of this area, the greater part (about 9700 square miles) consists of Appalachian Highlands with consolidated rock aquifers of generally low capacity, with the exception of certain valleys containing glacial outwashes from the Pleistocene era. The importance of these formations is that most of the basin tends to drain quickly, with highly fluctuating discharges, into the estuary at Trenton, New Jersey; see Reference 2.4-2.

The mean annual precipitation (1921-50) in the basin is 44 inches, with a runoff of 21 inches or 4.7×10^{12} gallons per year. The flow of the Delaware River at Trenton averaged over this same period is 11,810 cfs; see Reference 2.4-3. Fresh water drainage entering the river below Trenton augments this flow. Table 2.4-1 summarizes the flow contribution of the various tributary drainage systems.

Tidal flows dominate over fresh water discharges in the estuary portion of the system in which HCGS is located. In measurements made by the U.S. Geological Survey on August 21, 1957, noted in Reference 2.4-4, a peak downstream flow of about 400,000 cfs, and a peak upstream flow of about 600,000 cfs, occurred at Delaware Memorial Bridge (18.1 miles upstream of HCGS). The evening flood tide measured at the Delaware Memorial Bridge peaked at a slightly faster rate and was 1.4 feet higher than the morning ebb tide. A 1650 cfs mean daily discharge occurred at Trenton (82.8 miles above HCGS) on the same day. The maximum recorded discharge of 329,000 cfs occurred at Trenton on August 20, 1955, as mentioned in Reference 2.4-5. This flow rate is lower than the typical rates of tidal flow at the Delaware Memorial Bridge.

The tide in the Delaware Estuary is semi-diurnal in character. There are two high waters and two low waters in a tidal day, with comparatively little diurnal inequality. The mean range of the tide at the estuary mouth is about 4.3 feet and generally increases through the estuary to about 6.7 feet at Trenton, as mentioned in Reference 2.4-6. The variation in tidal amplitude with distance along the estuary is due to the opposing effects of convergence of the sides of the estuary, which tends to increase the amplitude, and friction that tends to decrease it; see Reference 2.4-2. Harleman, in Reference 2.4-7, constructed a one dimensional mathematical model for the tidal dynamics of the Delaware Estuary, based on the equation for a shallow water wave undergoing damping by friction. He estimates that the tidal wavelength in the estuary is 205 miles.

The Reedy Point Station is the tide gauge station nearest the site, as shown on Figure 2.4-1. The tides at the gauge have the following characteristics, noted in References 2.4-6 and 2.4-8:

1. Mean tide range = 5.5 feet

2. Spring tide range = 6.0 feet

3. Local mean sea level (MSL) = 2.8 feet above mean low water (MLW) = 1.45 NGVD

4. 10 percent exceedance high tide = 6.6 feet

The tides at HCGS have the following characteristics:

1. Mean tide range = 5.8 feet

2. Elevation 0 NGVD (MSL) = +89 feet PSE&G datum or 2.6 feet above MLW (+86.4 feet PSE&G datum)

Mean Sea Level (MSL), a tidal datum derived from the arithmetic mean of hourly water elevations observed over a specific 19-year Metonic cycle has not been measured at the site. The use of the abbreviation MSL as denoting the Sandy Hook, 1929 Adjustment, is technically speaking, a misnomer. However, the term MSL has been used throughout the HCGS FSAR and previous documents to represent this datum which should be called the National Geodetic Vertical Datum (NGVD). This practice was carried over from the Salem Nuclear Generating Station where MSL was also used to describe NGVD. MSL has also been used on occasion to describe, appropriately, local mean sea level.

At the HCGS, as well as SNGS, 0 feet NGVD does equal 89.0 PSD, or PSE&G plant datum which is also referred to as MSL (Sandy Hook 1929 Adjustment) U.S. Coast and Geodetic Survey Datum. This has been established by topographic survey. All design and construction is based on this topographically established datum. The mean tide level observed at the two sites is 0.3 feet NGVD or 89.3 PSD. This nomenclature discrepancy has no effect on design and does not adversely affect any safety-related designs.

Winds significantly influence the tidal fluctuations in the Delaware Estuary, producing the maximum recorded tidal fluctuations. The highest tide ever recorded occurred as a result of strong easterly winds and reached an elevation of +8.5 feet MSL on November 25, 1950. The lowest tide occurred as a result of north-northwesterly winds blowing downstream and reached a level of -8.0 feet MSL on December 31, 1962.

2.4.1.2.2 Surface Water Impoundments

Figure 2.4-2 shows the location of major existing impoundments in the Delaware Basin. Table 2.4-2 presents the hydrologic and hydraulic design features for these facilities.

The Delaware River Basin Commission (DRBC) has completed a Level B Water Resources Planning Study, found in Reference 2.4-1. A major purpose of the Level B Study is to provide a basis for updating the DRBC Comprehensive Plan. Proposed amendments include specific recommendations for inclusion and removal of projects from the Comprehensive Plan. Table 2.4-3 presents the impoundment projects recommended for addition or retention in the Comprehensive Plan. Figure 2.4-2 shows the locations of these potential projects in the Delaware Basin.

The Level B Study recommends that the environmental aspects of the following five projects be thoroughly investigated and, if found acceptable, construction or modification of these projects be expedited:

1. Merrill Creek

2. Francis E. Walter Modification

3. Prompton Modification

4. Hackettstown

5. Cannonsville Modification

The Aquashicola, Evansburg, Icedale, and Newark projects are retained in the Comprehensive Plan for future consideration. The Trexler project is retained only if needed to meet the future water supply needs of Allentown, Pennsylvania, and its environs. The Tocks Island project is retained in the Plan for consideration after the Year 2000, and remains as a project authorized by Congress for construction by the U.S. Army Corps of Engineers. Federal legislation to establish the Delaware Water Gap National Recreation Area, and to designate the middle Delaware River where Tocks Island Dam may be constructed as part of the National Wild and Scenic River System, does not de-authorize the Tocks Island project. It does, however, impose legal difficulties that must be overcome if the project is to be actively considered; see Reference 2.4-1.

2.4.1.2.3 Surface Water Users

The Delaware River is a major source of industrial and municipal water supply. Industrial surface water supplies are obtained directly from the river upstream of the site. The industrial use of river water below Marcus Hook (25 miles upstream of HCGS) is limited to cooling water applications due to salinity intrusion into the estuary. There are no industrial water users located downstream of HCGS, except for Salem Nuclear Generating Station, with its intake immediately downstream of HCGS.

The City of Salem obtains about 70 percent of its potable water from surface water supplies at Quinton on Alloways Creek about eight miles northeast of the site. This water supply is a dammed freshwater stream approximately nine miles upstream of the Delaware-Alloways Creek Confluence. There is no direct communication between the Delaware River and this potable water source in the absence of significant estuarine flooding.

The City of Philadelphia maintains a water treatment plant at Torresdale (River Mile 110.4 about 59.8 miles upstream of HCGS), which draws water from the Delaware River. Present DRBC water quality standards limit the maximum chloride concentration to 50 mg/l at any point in Zone 2 (River Mile 108.4 to 133.4). Thus, the Torresdale Plant is generally above the point of significant salinity intrusion into the estuary and, by direct correlation, it is also above the potential influence of the HCGS.

Groundwater users are described in Section 2.4.13.

2.4.2 Floods

2.4.2.1 Flood History

2.4.2.1.1 Riverine Floods

The river gage station at Trenton, New Jersey (River Mile 134.5), is the nearest station to HCGS, shown on Figure 2.4-1. It is the most downstream station on the main stem Delaware River. The Trenton gauge is located about 83.9 miles upstream of HCGS.

Table 2.4-6 presents the peak stage and discharge data for major floods on the Delaware River at Trenton.

Three significant floods occurred before the installation of the Trenton Gauge in 1902. The flood of February 27, 1692, reported 12 feet above the usual high water mark, may have been as great as or greater than the river flood of record, which occurred in August 1955. The August 1955 flood has a recorded discharge of 329,000 cfs and a stage of 28.6 feet mean sea level (MSL). The flood of January 8, 1841, was reported at that time to be the greatest since 1692. The ice jam flood of February 8, 1857, may have had a stage at Trenton equal to or higher than the ice jam flood of March 8, 1904, 30.6 feet MSL, the highest known stage at Trenton; See References 2.4-5 and 2.4-16.

The National Weather Service Analysis shows flow at Trenton of 654,000 cfs, or nearly double that of the 1955 flood, if tropical storm Agnes had traversed the Upper Delaware Valley; see Reference 2.4-17.

2.4.2.1.2 Tidal Floods

Tidal stations at Lewes, Delaware, near the mouth of the Delaware Bay and at Reedy Point, Delaware (about 8.3 miles upstream of HCGS), provide the most complete records of tidal floods in the vicinity of the site. Table 2.4-5 summarizes the tidal floods in the area.

The greatest tidal flood known to have occurred in the study area during the past 100 years occurred on November 25, 1950, along the Delaware Bay shore. The high tide during this storm reached 8.5 feet MSL near HCGS, as noted in Reference 2.4-18 and 2.4-18A. Strong easterly winds accompanied by heavy precipitation caused the highest tide of record in the area. A flood during the storm of October 1878 resulted in tides reported to be about the same magnitude as those of the November 25, 1950 storm. However, actual tide measurements for that storm are not available.

The hurricane of August 24-25, 1933, crossed inland over the Middle Atlantic Coast near Norfolk, Virginia. The Delaware Bay area was subjected to strong onshore winds from the northern portion of the storm pattern. Peak tide levels reached 7.9 feet MSL in the vicinity of HCGS.

The "northeaster" of March 6-8, 1962 also resulted in high tides in the Delaware Bay, reaching heights of 7.5 feet MSL at Reedy Point, Delaware, and 7.9 feet MSL at Lewes, Delaware. This storm produced abnormally high tide stages during five consecutive high waters; see Reference 2.4-18. Other significant floods directly affecting the area occurred during the storms of August 1934, September 1940, September 1944, and September 1960. These storms resulted in tides below that of November 1950, the tidal flood of record.

Because of the greater tidal flow compared to the freshwater discharge, tidal flooding tends to dominate riverine flooding, which may not be the major flood hazard at the HCGS site. Analysis of the maximum water levels associated with various combinations of flood-producing phenomena are presented in the following sections.

2.4.2.2 <u>Flood Design Considerations</u>

Sections 2.4.3 through 2.4.8 summarize and identify the individual types of flood producing phenomena and combinations of these events in order to establish the flood design basis for the plant safety-related features. Table 2.4-6 lists the postulated flood producing phenomena and the associated water levels at HCGS.

The most critical combination of flood producing phenomena results from the postulated occurrence of the probable maximum hurricane (PMH) surge with wave runup coincident with the 10 percent exceedance high tide. The maximum wave runup to 35.4 feet Msl occurs along the southeast face of the Reactor Building and a small corner face of the Auxiliary Building. These areas are flood proofed as indicated in Section 3.4.1 and Table 3.4-1 and structurally reinforced to withstand the static and dynamic effects of the flood and coincident waves as summarized in Table 2.4-11a.

Other sections of Seismic Category I power block structures are exposed to smaller waves, and are flood proofed as indicated in Section 3.4.1 and Table 3.4-1 and reinforced to withstand wave loading conditions as summarized in Table 2.4-11a.

The service water intake structure may be subjected to waves which could overtop the roof of the western portion at Elevation 39 feet MSL. Water would cascade onto the lower central roof surface at elevation 33 feet MSL and would run off the open ends to the north and south. Worst case water levels will not exceed the height of the reinforced concrete wall interior to the air intake screen at Elevation 39.5 feet MSL, thereby precluding entry into critical dry areas of the Intake Structure.

Security Related Information
Text Withheld under 10 CFR 2.390

the probable maximum precipitation (PMP) is as found in References 2.4-20 and 2.4-20a.

The yard drainage system subdivides the site area into discrete sub-basins, most of which have a storm water inlet. The individual inlets are connected by below grade piping. Exceptions to this general drainage plan occur along the north and west boundaries of the site. A narrow strip of land between the perimeter fence and the Delaware River at the southwest corner of the site is sloped toward and drains directly into the Delaware River. Drainage of areas along the sites north and northwest perimeter are drained by open ditches discharging to the Delaware River.

For additional descriptions of the roof and yard drainage system, refer to Section 3.4.

2.4.3 Probable Maximum Flood on Streams and Rivers

This section considers the probable maximum flood (PMF) event over the entire Delaware River Basin. The adopted PMF estimating procedure is the method presented in Appendix B of Regulatory Guide 1.59; see Reference 2.4-19.

2.4.3.1 Probable Maximum Precipitation

The methodology adopted in this study of the Delaware River Basin PMF does not require separate consideration of the probable maximum precipitation (PMP).

2.4.3.2 Precipitation Losses

The methodology adopted in this study of the Delaware River Basin PMF does not require separate consideration of precipitation losses in the drainage areas.

2.4.3.3 Runoff and Stream Course Models

The methodology adopted in this study of the Delaware River Basin PMF does not require separate consideration of runoff and stream course models.

2.4.3.4 Probable Maximum Flood Flow

The Delaware River Basin has a total drainage area of about 12,765 mi^2. Figure 2.4-4 presents the envelope curve of PMF peak discharge as a function of drainage area as developed from the PMF isoline charts provided in Appendix B of Regulatory Guide 1.59, and found in Reference 2.4-19.

The peak PMF discharge corresponding to the 12,765 mi^2 drainage area is estimated at 1,250,000 cfs. This PMF flood flow represents the peak discharge through the mouth of Delaware Bay. This peak discharge is conservatively translated without reduction to the HCGS site 50.6 miles upstream of the mouth.

2.4.3.5 Water Level Determinations

The PSAR for the Summit Generating Station, mentioned in Reference 2.4-21, discusses the PMF on the Delaware River. The estimated PMF discharge on the Delaware at the Chesapeake and Delaware Canal (8.3 miles above HCGS) is 1,000,000 cfs. A backwater analysis estimates that the maximum water surface elevation associated with that PMF discharge is less than 5 feet mean sea level (MSL) at the canal.

Given these results, it is clear that the PMF is a relatively minor flooding event in comparison to other postulated events evaluated in other portions of this study. Therefore, there is justification for adopting a simplified but conservative estimating approach for the PMF levels at the HCGS. The estimating procedure uses several assumptions and the results of the Summit Analysis mentioned in Reference 2.4-21. These assumptions are:

1. The Manning's equation is appropriate under the relatively uniform flow conditions of the PMF. The equation is:

$$Q = \frac{1.49}{n} AR^{2/3} S_F^{1/2} \qquad (2.4\text{-}1)$$

where:

Q	=	discharge, cfs
n	=	Manning's coefficient
A	=	cross-sectional area, ft^2
R	=	hydraulic radius (area/wetted perimeter of the channel)
S_F	=	slope of the total energy line, or the frictional slope

2. There is no abrupt change in channel cross sectional configuration for a distance of about 12 miles downstream of HCGS. Figure 2.4-5 shows the cross sections of the Delaware River at the Smyrna River and at the Cohansey River located about 6 miles and 12 miles downstream of the HCGS, obtained from the National Ocean Survey nautical charts of the Delaware River found in References 2.4-22 and 2.4-23. Given this condition, there is no sudden variation in water surface elevation or abrupt variation in the frictional slope.

3. The frictional slope of the HCGS PMF is similar to that of the Summit PMF since the flow condition is gradually varying, and the discharges, 1,250,000 cfs and 1,000,000 cfs, respectively, are of the same order of magnitude. Therefore, the variation in the peak discharge, Q, relates directly to the variation in the term $AR^{2/3}$ in Equation 2.4-1. The magnitude of the $S^{1/2}$ term is usually orders of magnitude less than the value of the $AR^{2/3}$ term.

4. The idealized channel cross section for the reach in the vicinity of the HCGS is a triangular section. The effective total depth of flow is about 30 feet below mean low water (MLW) for this ideal section.

5. For a triangular section, the term $AR^{2/3}$ is directly proportional to $h^{8/3}$, where h is the effective total depth of flow.

Applying these assumptions yields the following relationship:

$$\frac{Q_2}{Q_1} = \frac{(h_2)^{8/3}}{h_1} \qquad (2.4\text{-}2)$$

Using the Summit analysis results, found in Reference 2.4-21, of the maximum water level elevation of 5 feet MSL (7.6 feet MLW), the total depth of flow for the Summit PMF (1,000,000 cfs) is about 37.6 feet. Using this result with a PMF discharge at the HCGS (1,250,000 cfs) results in a maximum PMF water level elevation at HCGS of 8.3 feet MSL (10.9 feet MLW).

2.4.3.6 Coincident Wind Wave Activity

The coincident wind wave activity superimposed on the maximum stillwater level elevation at the plant site provides the conditions to evaluate the maximum wind wave effects on the plant structures. The method used is the shallow water wave generation with limited fetch length technique recommended in the Shore Protection Manual, 1977, found in Reference 2.4-24. Results give the significant wave heights and significant wave periods at the end of the fetch directions, i.e., the Artificial Island of HCGS. The estimated maximum wave height; is assumed to be at least 1.5 times the significant wave height; see References 2.4-8 and 2.4-34.

Figure 2.4-6 shows the nine fetch directions selected for this analysis. They generally radiate from HCGS in the downstream direction of the Delaware River towards the Delaware Bay. Starting from fetch No. 1, which is at 134 degrees azimuth from the North, each fetch direction is 15 degrees apart from an adjacent fetch. Fetch No. 2, at 149 degrees, is chosen to traverse the entire length of the Delaware Bay, from the bay entrance towards the plant site. The most critical fetch direction is determined after reviewing the average water depth and effective fetch depth along each fetch. Fetch No. 3 gives the most critical conditions for wind-generated waves. The National Ocean Survey nautical charts of the Delaware River are used to obtain water depth information. The wind condition used is the annual extreme mile, 30 feet above ground winds with 2-year mean recurrence intervals from H.C.S, Thom, 1968, in Reference 2.4-25. This wind speed is 48.5 mph for the Delaware Bay area. The effective fetch length and average water depth are determined using procedures recommended in Reference 2.4-24. Several assumptions are made for this analysis:

1. This analysis considers locally generated wind waves in the Delaware Estuary area. Wind generated deep water waves on the ocean side, after undergoing shoaling and refraction on the continental shelf as they propagate towards the bay entrance, may either break over the shallow water areas near the bay entrance or undergo significant energy dissipation and attenuation at the entrance to Delaware Bay.

2. The limit of fetch distance delineation is the Delaware Bay entrance or the land masses encountered along the fetch directions.

The maximum wave height estimated for the most critical fetch direction No. 3 is 9.9 feet; wave period is 5.0 seconds. The Sainflou method, found in Reference 2.4-24, estimates the wave run-up associated with the maximum wave height on safety-related structures. The maximum wave runup height thus estimated is 109.8 feet PSE&G datum or 20.8 feet MSL.

2.4.4 Potential Dam Failures, Seismically Induced

This section presents the analysis of potential dam failures. The analysis follows the guidance given in Appendix A of Regulatory Guide 1.59, indicated in Reference 2.4-19, and ANSI, ANS-2.8-1981 found in Reference 2.4-8, with respect to the selection of seismic failure models for the dams and coincident flow. The simplified and extremely conservative approach shows that the seismically induced dam failure flood is less severe than other flood producing mechanisms such as the probable maximum hurricane (PMH) induced flooding.

The ANSI, ANS-2.8-1981, found in Reference 2.4-8, provides two alternative combinations of earthquake and coincident flood. The higher of the two alternative combinations is an adequate design base for seismic dam failure floods. The alternatives are:

1. Alternative I

    a. 25-year flood

    b. Dam failure caused by safe shutdown earthquake (SSE) coincident with peak of flood

    c. 2-year extreme wind speed for critical direction and length of effective fetch

2. Alternative II

    a. One-half probable maximum flood (PMF)

    b. Dam failure caused by operating basis earthquake (OBE) coincident with peak of flood

    c. 2-year extreme wind speed for critical direction and length of effective fetch.

The postulated dam failure mode is the instantaneous disappearance of the dam. The disappearance of the dam is the least plausible condition of dam failure, and hence represents an extreme of conservatism. If the postulated failure occurs as a result of an OBE, it is clear that Alternative II represents the more severe single dam seismically induced flood.

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2.4.4.3 Water Level at Plant Site

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an initial rise sea level anomaly of one foot; and an astronomical spring high tide of 5.6 feet. The sum of the sea level anomaly and the astronomical spring high tide are exactly equivalent to the 10 percent exceedance high tide of 6.6 feet.

The computed maximum surge elevation at the mouth of Delaware Bay is 21.9 feet MLW. Figure 2.4-8 shows the computed surge hydrograph.

2.4.5.2.2 Delaware Bay Surge

The surge hydrograph at the mouth of Delaware Bay is routed to the site using the procedures developed by Bretschneider, as discussed in Reference 2.4-32. The process involves routing of the open coast surge through the entrance to Delaware Bay, allowing for convergence as the bay narrows, modifying of the surge due to friction, and additional wind stress on the surface of the surge.

Entrance losses at the mouth of Delaware Bay reduce the maximum surge elevation from 21.9 feet MLW (open coast value) to a peak surge elevation of 21.2 feet MLW within Delaware Bay. The value for the coefficient of discharge is 0.65 (maximum value). Routing of the surge as described above produces a peak surge level of 24.5 feet MLW. In reviewing the flood design considerations for the adjacent Salem Generating Station, Units 1 and 2, and discussed in Reference 2.4-28, the then Atomic Energy Commission (AEC) suggested that the computed surge levels be increased by 2.9 feet. The peak storm surge stillwater level after adjustment is 27.4 feet MLW. Figure 2.4-8 shows the portion of the hydrograph in the vicinity of the peak surge period. The stillwater level can be defined as the water level at the HCGS location as a result of PMH surge, which is the surge level at the site plus the cross-wind setup; see Table 2.4-10. It is also the level to which the rise in water levels due to wave action should be referenced.

2.4.5.3 Wave Action

2.4.5.3.1 Waves Approaching Artificial Island

For the purpose of maximizing the effects of hurricane surge and coincident wave activity at the site, the PMH is postulated to have a track along the west side of and generally parallel to Delaware Bay and River. Figure 2.4-7 shows this critical path. Wind speed and direction at the site change as the PMH moves along this path because of the effects of friction and filling overland and also because of the position of the hurricane center with respect to the site.

At the site, the winds shift in a clockwise direction from the southeast to the southwest, and blow directly across the Delaware River toward the plant. The analysis considers waves generated along nine fetches, each 15° apart. These fetch directions range from 134° to 254° measured from true north.

In the vicinity of the site, the hurricane moves on a circular path with a radius of 39 nautical miles. For a forward speed of 27 knots, the hurricane would require approximately 0.36 hours to cover an arc spanned by 15 degrees. The peak surge arrives at the site when maximum winds blow along fetch No. 2; see Table 2.4-10.

The procedures described in HUR 7-97, found in Reference 2.4-29, provide the wind speed along each fetch, based on the position of the storm center relative to the site and the time elapsed since landfall. The average water depth used for this analysis is the sum of the water depth at the center of the fetch, including the surge level above mean low water and the increase of computed surge level of 2.9 feet suggested by the then AEC.

The procedures described in Reference 2.4-24 are used for the estimation of significant wave conditions in shallow water. The ratio of maximum wave height to significant wave height is conservatively chosen to be 1.5 for design purposes. This design valve represents a conservative upper-bound limit which includes approximately 20 percent increase above the estimated value 1.23 for all fetch directions. The alternative approach as described in References 2.4-8 and 2.4-34 is used to derive the value 1.23.

The effect of viscous damping was also considered for incident waves towards the shoreline. Viscous damping effect was only considered for the waves approaching from fetch directions 2 to 6 which would be subjected to interference by the land mass near the headland of the Artificial Island. Methods described in Reference 2.4-7 is used for the analysis.

The attenuated maximum wave heights are then used for wave run-up and wave loading analyses for the west-facing vertical wall of the intake structure. The effects of a range of wave periods associated with the attenuated maximum wave height for each fetch direction is investigated. Although the range of wave fluid could vary from 0.5 to 1.9 times the significant shallow water wave fluid, an alternate approach is adopted to define the range to be analyzed. The lower wave period is calculated for the limiting steepness of the progressive waves as recommended by Reference 2.4-24 and the higher wave fluid can be estimated to be 1.20 times the significant wave fluid as recommended by ANSI ANS-28-1981, Reference 2.4-8.

Non-breaking wave conditions are assumed for wave run-up and loading analyses on the vertical wall. The steeper waves result in higher run-up heights and the longer waves result in higher wave loading on the vertical wall based on the Sainflon method for estimating wave run-up and wave forces. The results of computation are presented in Table 2.4-10a and 2.4-11a. The maximum wave runup height theoretically is at Elevation 134.4 feet PSE&G datum which indicates that the waves would overtop the vertical wall of the intake structure at Elevation 128.0 feet PSE&G datum. The maximum wave loading is 81.4 kips/ft of which the dynamic component is 27.6 kip/ft.

2.4.5.3.2 Incident Waves on Plant Structures

The distance from the edge of Artificial Island to the plant buildings varies from about 2700 feet for fetch No. 1 to about 500 feet for fetch No. 7 through 9, as shown on Figure 2.4-9. Wave transformation occurs as the incident waves encounter the earth dikes and fill areas in the vicinity of the plant. The dikes extend to Elevation 106.5 feet PSE&G datum, and the plant grade is at Elevation 101.5 feet PSE&G datum. Large waves break before they reach the plant buildings. The maximum wave height incident on safety-related plant facilities is depth limited for all fetches, except fetch Nos. 2 and 3. The breaking or maximum wave height for the depth limited fetches is equivalent to 0.78 of the water depth; see Reference 2.4-24. The analysis assumes that the corresponding wave period is unchanged by the process of wave transformation. Table 2.4-10 includes the characteristics of the transformed incident waves.

The adjacent Salem Nuclear Generating Station effectively prevents waves from directly reaching the HCGS safety-related buildings along fetch Nos. 2 and 3; see Figure 2.4-9. The effective fetch length for these two fetch directions is about 1000 feet, which is the distance between the HCGS and Salem buildings. The maximum wave heights are 1.31 and 1.29 times as large as the corresponding significant waves for fetch Nos. 2 and 3, respectively. An empirical formula derived from field observations of steady state wind wave generation over shallow water provides the two ratios; see Reference 2.4-34. Table 2.4-10 also presents the wave parameters for fetch Nos. 2 and 3.

Similar approach is taken for the wave runup and wave loading analyses on the plant structure. A range of wave periods and corresponding wave lengths associated with the maximum wave height is examined. The wave run-up height is estimated by the Sainflou method. Results indicate the controlling wave runup height to

Elevation 124.4 feet PSE&G datum. Results of the wave run-up computation is presented in Table 2.4-10. Breaking wave loading on plant building is estimated by Minikin method described in Reference 2.4-24. The controlling wave loading is 46.8 kip/ft.

2.4.5.3.3 Maximum Wave Crest Elevation

Standing waves, or clapotis, form as waves impinge on the vertical walls of safety-related facilities. The Sainflou method, discussed in Reference 2.4-33, estimates the maximum wave crest elevations under such conditions. The analysis indicated that the surge level and coincident wind wave action along fetch No. 1 result in a maximum wave run-up elevation of 35.4 feet MSL at the power block. The plot plan, shown on Figure 2.4-9, shows that the southeast face of the Unit No. 1 Reactor Building, and a small corner face of the Auxiliary Building, have exposure to wave action along fetch No. 1. For other fetch directions, the maximum wave runup elevations at the power block are equal to or lower than 30.0 feet MSL. Maximum wave run-up elevations at the service water intake structure are addressed in Table 2.4-10a.

2.4.5.4 Resonance

Local changes in atmospheric pressure and wind, as well as oscillations transmitted through the mouth of a partially enclosed water body, can generate seiches and resonance within that body. A periodic causative force that sets the water basin in motion can generate standing waves of large amplitude. This is especially true if the frequency components of the excitation approach any of the natural frequencies of the basin.

If the Delaware Estuary is idealized as a simple rectangular basin with an open end, i.e., the Atlantic Ocean, the longest period of free oscillation corresponding to the fundamental mode can be roughly estimated, assuming one dimensional disturbances, by use of Merian's equation, mentioned in Reference 2.4-24, which is:

$$T_o = \frac{4L}{(gd)^{1/2}} \qquad (2.4.5\text{-}1)$$

where:

T_o = natural free oscillating period for the fundamental mode, s

L = length of basin, ft

g = gravitational constant

d = basin depth, ft

Under normal stillwater conditions, the average depth of the Delaware Estuary is 21 feet and the length to the head of tide is 133 miles. Thus, the fundamental period of free oscillation for these conditions is on the order of 30 hours.

With maximum stillwater conditions that occur during a PMH surge, the average depth increases to about 45 feet and the length remains the same. Under these conditions, the natural period of oscillation decreases to roughly 20 hours.

Direct application of Merian's equation is difficult for natural basins, due to the unusually complex geometry and variable depths. However, it does serve as a useful first conservation approximation.

The possible forces and expected periods that could cause resonance in the Delaware Estuary are listed and discussed below:

1. The periods of wind generated waves in the Delaware Estuary could range between one and seven seconds. Since these periods are very much shorter than the fundamental period of free oscillation for the Delaware Estuary, no wave resonance would occur.

2. The astronomical tide has a period on the order of 12 hours, which is approximately one half to one third of the maximum oscillation period of the Delaware Estuary. Thus, the astronomical tide would not provide the forcing mechanism to generate resonance.

Based on the analyses made, it is concluded that seiche or resonance flooding is not a problem at the HCGS. Large amplitude oscillations are not possible, because the most probable forcing mechanisms identified lack either a period of oscillation close enough to the fundamental period of the Delaware Estuary to be of concern, or a magnitude and duration great enough to supply a significant amount of energy into the basin. In addition, energy dissipation of any water level oscillation occurs by frictional damping and reflection along the banks of the estuary.

2.4.5.5 Protective Structures

All safety-related facilities of HCGS require flood protection against the static and dynamic effects of PMH-induced surge flooding and coincident wave activity. Hardened flood protection, as defined by Regulatory Guide 1.59 and Reference 2.4-19, is discussed in Section 2.4.10.

Although conservatively ignored in the analyses, protection of safety-related facilities against flooding and coincident wave action is also provided by the construction of an earth dike along the shoreline near the site. This dike extends to an elevation of

106.5 feet PSE&G datum. Further, structures of the Salem Generating Station will provide additional protection against waves from the southerly direction, as discussed in Section 2.4.5.3. Sheetpile retaining walls and riprap construction, extending 100 feet on both sides of the intake structure, will provide protection against slope failure and minimize shoreline recession.

2.4.6 Probable Maximum Tsunami Flooding

Tsunamis are seismic sea waves generated by earthquakes, volcanic explosions, or large submarine landslides. However formed, the surface deformation spreads radially outward as a series of confocal waves, and its subsequent history is governed only by the topography of the sea floor. If, as is true for most seismic displacements, the source deformation is elongate, the waves radiating perpendicular to the principal source axis will be substantially higher than those traveling parallel to it. They are long period (5 to more than 60 minutes) waves, the amplitude of which varies inversely with distance from the source. Their length (crest to crest) may exceed a hundred miles, and their height at sea rarely exceeds about two feet, but they move at speeds of up to 600 miles per hour (speed is a function of water depth). As the wave enters shoaling water, its velocity decreases and its height increases.

Tsunamis are believed to be generated by rapid tectonic displacement of the ocean floor in very deep water. They are associated with major shocks (6.5 magnitude-Richter scale) and with shallow focal depths (25 mi). The tectonic generation of a tsunami requires a substantial vertical component of movement. This component would be most efficiently generated by dip slip movement, but it is conceivable that it could be generated by strike slip movement under certain conditions. Most destructive tsunamis are believed to originate from movements of large crustal fault blocks along the slopes of an oceanic trench system, as a result of readjustments of the moving sea floor crustal plates. The great Alaskan earthquake of March 28, 1964 involved the uplift of about 40,000 mi^2 of the shelf bordering the Gulf of Alaska by 6 to 65 feet.

Tsunamis also may be caused by submarine avalanches, as indicated in Reference 2.4-35, which may themselves be triggered by an earthquake. However, such a mechanism is a very inefficient generator, so a very large landslide would be required to generate a tsunami. Laboratory tests using sliding blocks and plates have been run to model tsunamis generated in this manner; see References 2.4-36 and 2.4-37.

The wave patterns generated by this tectonic activity, or submarine landslide and radiating into deep water, remain fairly circular until they encounter shallow water along the continental margins, where they become altered almost beyond recognition. Depending upon the angle of wave approach and the offshore continental profile, part of the wave energy is reflected and part transmitted across the coastal shelf in the form of a complicated edge wave system that bears little resemblance to the incident waves offshore.

Along a continental sea coast, a large tsunami is manifested by a series of quasi-periodic surges and withdrawals at intervals ranging from a few minutes to more than one hour, superimposed upon prevailing tide and wind/wave action. Low lying areas are inundated, breaking bores may develop in estuaries, and strong, oscillating currents are often observed in shallow water.

2.4.6.1 Probable Maximum Tsunami

Brandsma, et al, in Reference 2.4-38, has developed potential tsunami histories at various stations offshore of the US coasts. They define a large hypothetical earthquake from historical data and tectonic theory which corresponds to an earthquake magnitude 9.0. This value agrees with that taken by King and Knopoff, in Reference 2.4-39, as a reasonable upper limit to earthquake magnitude. This canonical source serves as input for computation of the resulting wave history anywhere within the ocean basin. The procedure is repeated for a number of potential source locations, chosen according to degree and type of seismic activity. Hypothetical coastal histories of great tsunamis emanating from any potential source area are simulated. The model predicts tsunami wave heights at offshore stations where the water depths are approximately 600 feet. The wave height predictions at these offshore stations include both the incident and reflected wave components. The wave characteristics at the site are the result of the transformation of the waves by their interaction with near shore features as they propagate shoreward.

Brandsma, et al, in Reference 2.4-38, estimated the peak tsunami wave heights and time histories for two stations bracketing the Delaware Bay entrance. These stations are located offshore of Atlantic City, New Jersey and offshore of Assateague Island, Maryland. The peak wave heights and arrival times are 1.5 feet at 9511 seconds, and 2.8 feet at 9370 seconds, respectively. The tsunami source is a hypothetical earthquake occurring near Haiti, as described in Section 2.4.6.3.

2.4.6.2 Historical Tsunami Record

Records of Atlantic tsunamis are relatively rare. Within the recorded history, a total of about 30 large tsunamis have occurred in the Atlantic, all shown in Table 2.4-11. Four tsunamis (Jamaica, 1696; Portugal, 1755; Virgin Islands, 1867; and Puerto Rico, 1918) involved wave systems large enough to be observable at transoceanic distances. The most recent Atlantic tsunami occurred as a result of the Grand Banks (Newfoundland) earthquake of 1929; see Table 2.4-12. This tsunami reached a height of some 98 feet near its source, but along the New Jersey coastline, it attained a height of less than one foot. The most destructive historic Atlantic tsunami swept the Portuguese coast in 1755. A remnant of that phenomenon achieved a height of about 20 feet in the West Indies. There is no record of this tsunami's impact on the Atlantic Coast of the United States.

2.4.6.3 Source Generator Characteristics

A hypothetical distant earthquake located near Haiti produces the maximum wave displacement at the offshore stations. The earthquake is located at 19°N and 67°W. The canonical bottom displacement has a dipolar shape with a major axis characteristic length of 575 nautical miles, and a minor axis characteristic length of 260 nautical miles. The major axis gives a bearing or principal wave propagation direction of 0°. The peak bottom displacement is 30 feet, as indicated in Reference 2.4-38.

There is no historical basis for a locally generated tsunami. Although earthquakes frequently occur in the eastern United States, these earthquakes have all occurred inland from the coastline. The probability of an earthquake having an epicenter in a location that would cause a tsunami, either on the coastline or in an estuary, cannot be determined from available data in Reference 2.4-40. Data presented in Section 2.5 substantiates this observation; therefore, the hypothetical earthquake occurring near Haiti is considered to be the controlling tsunami generator.

2.4.6.4 Tsunami Analysis

Brandsma, et al, in Reference 2.4-38, estimates the total peak tsunami wave heights at the two offshore stations as 1.5 feet (offshore Atlantic City), and 2.8 feet (offshore of Assateague Island). A coefficient provides a measure of the extent of reflection of the incident waves as a function of the incident wave angle relative to a baseline connecting the adjacent offshore stations. The arrival times of the peak wave displacements at the stations, the tsunami wave celerity, and the linear distance between the stations are factors in the coefficient estimation. The empirical equations for coefficient estimation, as indicated in Reference 2.4-36, are:

$$A = COS^{-1}\left(\frac{CdT}{B}\right) \qquad (2.4\text{-}6)$$

$$K = \frac{Hw}{Hi} = 2 \text{ if } 45° \leq A \leq 90° \qquad (2.4\text{-}7)$$

or

$$\left(1 + \frac{A}{45}\right) \text{ if } 0° < 45°$$

where:

A = incident wave angle with respect to the baseline

B = linear distance between the stations along the baseline

C = wave celerity

dT = difference in the arrival times

K = ratio of total wave height to incident wave height

Hi = incident peak wave height

Hw = total peak wave height

The estimated incident wave angle is approximately 90°, so that the tsunami waves approach the baseline almost perpendicularly. The peak incident wave heights are 0.7 feet and 1.4 feet offshore of Atlantic City and Assateague Island, respectively. The interpolated peak incident wave height of 1.1 feet applies to a point offshore of the Delaware Bay entrance.

Changes in water depth can transform the incident wave as it propagates across the continental shelf. The wave transformation analysis conservatively ignores the effect of the slope change that occurs along the continental shelf/slope boundary. The analysis uses techniques presented in References 2.4-24 and 2.4-40, using small amplitude, shallow water wave equations to route the waves from the offshore stations (600 feet water depth) inshore to the Delaware Bay entrance (30 feet water depth). The incident wave height at the Delaware Bay entrance is 1.6 feet when refraction and shoaling are considered. The wave length of the tsunami is 56 miles. Analysis of wave transmission estimates the tsunami wave conditions within the Delaware estuary. Green's law estimates wave transmission effects for long waves propagating into gradual transitions assuming negligible reflection and frictional attenuation. Green's law is stated as:

$$\frac{H_2}{H_1} = \frac{(b_1)^{1/4}}{b_2} \quad \frac{(h_1)^{1/4}}{h_2} \qquad (2.4\text{-}8)$$

where:

H_1, H_2 = wave heights at sections 1 and 2, respectively

b_1, b_2 = average channel widths at the sections

h_1, h_2 = average water depths at the sections

The National Ocean Survey (NOS) nautical charts of the Delaware River in References 2.4-22 and 2.4-23, provide the appropriate section characteristics. The transmission analysis indicates that the wave height at the site is 4.0 feet.

2.4.6.5 Tsunami Water Levels

The maximum water level at plant site resulting from tsunami waves considers the effect of coincident wind-wave activity, as described in Section 2.4.3.6. The coincident wave height with a 2-year extreme wind associated with the tsunami wave height and 10 percent exceedance high tide is 9.6 feet. The resulting wave run-up height estimated by Sainflou method is 107.1 feet PSE&G datum or 18.1 feet MSL.

The minimum water level at the plant site resulting from tsunami waves considers the effect of coincident wind-wave activity, as described in Section 2.4.3.6. The coincident wave generated by a 2-year extreme wind associated with the tsunami wave and 10 percent exceedance low tide result in a minimum water level of -10.3 feet MSL or 78.8 feet PSE&G datum. This level is 2.8 feet above the service water pump minimum design operating level of 76 feet PSE&G datum. In addition, Hope Creek Technical Specifications require a plant shutdown when river water level reaches 80 feet PSE&G datum.

2.4.6.6 Hydrography and Harbor or Breakwater Influences on Tsunami

The estimate of incident tsunami wave height at the Delaware Bay entrance resulting from a distant canonical source, and the routing of the tsunami wave from the offshore area to the HCGS site location, are described in Sections 2.4.6.1 to 2.4.6.5. Therefore, separate discussion in this section is not required.

2.4.6.7 Effects on Safety-Related Facilities

The estimated maximum wave height coincident with 10 percent exceedance high tide and 2-year extreme wind condition at the HCGS site location is 9.6 feet above the maximum stillwater level, which is at 6.0 feet MSL. The plant grade is at 12.5 feet MSL. Therefore, the effect of the maximum wave's height on safety-related facilities above the plant grade is insignificant. The maximum wave run-up elevation on safety-related facilities below the plant grade, such as the service water intake structure, is at 18.1 feet MSL.

2.4.7 Ice Effects

In ordinary winters, there is usually sufficient ice in the Delaware Bay and River to be of some concern to navigation. Thin ice has been known to form early in December below Philadelphia, but heavier ice does not begin to run before January. The tidal currents keep the ice in motion, except where it packs in the narrower parts of the river. Ice breakers from Philadelphia keep these parts of the river open. The ice usually packs heavier than elsewhere at Ship John Shoal, at Pea Patch Island, at Deepwater Point, and below Gloucester City. Ice is rarely encountered after the early part of March; see Reference 2.4-42.

A deicing system protects the service water intake against clogging by ice. The system provides hot water through a 24-inch horizontal deicing line with a series of 8-inch downcomer lines. The hot water is distributed along the intake opening outboard of the trash rack and prevents ice blockages at the trash rack. Ice barriers will be installed in front of the service water intake structure to prevent ice blockage.

2.4.8 Cooling Water Canals and Reservoirs

The design of the HCGS does not include any safety-related canals or reservoirs. Further discussion is therefore not necessary.

2.4.9 Channel Diversions

There is no evidence of channel diversions of significance in the Delaware River Basin. Since the HCGS is located in a tidally affected portion of the basin, sources of cooling water are located both upstream and downstream of the site. In the highly unlikely event that either the river flow or the tidal flow is temporarily interrupted by a channel diversion event, the other source would continue to supply water to the site area.

2.4.10 Flooding Protection Requirements

Section 2.4.2.2 discusses the water level elevations under various combinations of flood producing phenomena at the HCGS site location. The most critical condition is the postulated occurrence of the probable maximum hurricane (PMH) surge with wave run-up coincident with the 10 percent exceedance high tide.

All safety-related facilities of the HCGS require flood protection against the static and dynamic effects of PMH induced surge flooding and coincident wave activity. Hardened flood protection, as defined by Regulatory Guide 1.59, in Reference 2.4-19, includes the following design features:

1. Static and dynamic flood resistance incorporated into the exterior wall and base slab designs of the Seismic Category I structures.

2. Double water stop application in all seismic joints to above maximum water level.

3. Full waterstop application at construction joints to above maximum wave run-up level.

4. Static and dynamic flood resistance incorporated into the exterior door designs of the Seismic Category I structures.

5. Waterproof penetrations.

6. Flood alarm system to warn of rising water at limit levels in watertight compartments.

7. Floor drainage systems to sumps.

8. Level actuated sump pumps discharging to holding tanks.

Shore protection is required in the vicinity of the service water intake structure to assure that no blockage to the water intake will occur and that erosion will not impede the operation of the service water pipes. A study by Dames & Moore, completed in June 1977 (Reference 2.4-15A), addressed shore protection and makes recommendations to assure the safe operation of the plant. In summary, shore protection will extend 100 feet north and south of the intake structure. Shore protection consists of sheet pile cellular cofferdams which will be stable under the design seismic event and design flood (PMH). Sheet piling and surface protection are being provided for the service water piping.

2.4.11 Low Flow Considerations

2.4.11.1 Low Flow in Streams

The HCGS is located within the tidally-affected portion of the Delaware Estuary System. Historical extremes in water-level occurred as a result of wind related tide level variations, and not as a result of fluvial discharges; see Section 2.4.1.1. The record low flow on the Delaware River occurred on October 31, 1963, with a discharge of 1180 cfs; see Reference 2.4-16. The tidal records of the Philadelphia Tide Station, found in Reference 2.4-43, show that a monthly low tide of 3.7 feet below mean sea level (MSL) occurred on October 30, 1963. However, lower monthly tides occurred during six other months of that year. An analysis of minimum low tide occurrences, found in Reference 2.4-44, during the period 1932-1962, indicates that the low tide of October 30, 1963 has an average recurrence interval of about 105 years. Therefore, low river flows do not have a significant effect on estuary tide levels. Unusual tide levels correspond to this low water event.

There are no existing or planned dams downstream of the HCGS. Therefore, there are no postulated dam failure incidents that could result in a low water condition at HCGS.

The cooling water intake supplies both safety and nonsafety-related water to the HCGS. The design basis of this intake is the hurricane generated probable minimum water level as discussed in Section 2.4.11.2. Any postulated drought condition would have a minimal effect on water levels as compared to design basis. Therefore, both safety and nonsafety-related water supplies are secure during any postulated drought event.

2.4.11.2 Low Water Resulting from Surges, Seiches, or Tsunami

A postulated large radius stationary probable maximum hurricane (PMH) is the mechanism that produces the minimum water levels at HCGS. Figure 2.4-10 shows the location of the hurricane that produces the maximum winds from the northwest coinciding with the axis of Delaware Bay between the HCGS and the mouth of the bay. The meteorological parameters of the hurricane, as taken from HUR 7-97, Reference 2.4-29, are:

1. Latitude of storm center = 30°N
2. Central pressure index = 27.09 in., Hg
3. Peripheral pressure = 30.72 in., Hg
4. Radius of maximum winds = 39 nautical miles
5. Forward speed of translation = 0 mph
6. Maximum wind speed = 124 mph.

The minimum still water level elevation at HCGS site resulting from the probable maximum surge (PMS) was computed using procedures based on Bretschnider, as described by Marinos and Woodward in Reference 2.4-30. Coriolis effects within Delaware Bay were neglected due to boundary conditions. Calculations were carried from beyond the continental shelf to the HCGS site. The minimum stillwater level estimated is 8.0 feet below the mean low water (MLW) level.

To estimate the probable minimum water level for the extreme low water condition, the effects of coincident storm-generated waves should also be considered. This is done using procedures described in the 1977 Shore Protection Manual, as indicated in Reference 2.4-24, for the generation of local wind waves under fetch limited conditions.

A wind speed of 85 mph associated with the probable maximum hurricane, corrected for landfall effects, was used for the analysis. This wind speed appears to be consistent with the wind velocity at a station immediately downstream of the HCGS site, while a 79 mph wind speed is consistent with a station upstream of the site used for the estimate of minimum stillwater elevation. The use of the higher wind speed would tend to produce a more conservative estimate. Further, the wind vector is aligned parallel with respect to the axis of the Delaware River channel, which is the Reedy Island Channel in the vicinity of the critical fetch direction; see Reference 2.4-23. The critical fetch direction selected parallels the Liston Point channel, which is the Delaware River channel downstream of the HCGS site; see Reference 2.4-23. The angle between the wind vector and the fetch direction is about 58.5°. The component of the wind speed along the fetch direction is then obtained. The fetch distance 7830 feet is obtained using the method described in Reference 2.4-24. An average water depth of 17.0 feet below MLW is also used. This water depth coincides with the value of still water depth at the stations used in the estimate of minimum water level elevation resulting from the PMS.

Using the values described above, the significant wave height and wave period were 2.0 feet and 2.8 seconds, respectively. The extreme wave height is 1.67 times that of the significant wave height, and the wave period can be estimated as 1.20 times the significant wave period, as recommended in ANSI ANS-2.8-1981, Reference 2.4-8, for storm waves associated with storm induced low water. The extreme wave height and wave period, therefore, are approximately 3.3 feet and 3.4 seconds.

The Sainflou equation estimates the wave drawdown on the safety-related structure. The maximum deviation of the clapotis from the still water level at the trough of the wave is calculated by:

$$D = H - ho \qquad (2.4\text{-}9)$$

where:

D = deviation of the clapotis from stillwater level

$$ho = \pi \frac{H^2}{L} \coth\left(\frac{2\pi d}{L}\right) \qquad (2.4\text{-}10)$$

where:

H = maximum wave height

d = still water depth

L = wave length

The value of ho is found to be 0.85 feet. Therefore, the maximum deviation of the stillwater level is 2.44 feet or approximately 2.4 feet below the stillwater level. The estimated minimum water level elevation is, therefore, 76.0 feet PSE&G datum (-10.4 feet MLW), considering the effect of minimum water level and the associated storm wave effects. The Hope Creek Technical Specifications, however, require a plant shutdown when river water level reaches 80 feet PSE&G datum.

2.4.11.3 Historical Low Water

The lowest known low tide in the recorded history of the Delaware River Estuary occurred on December 31, 1962. The primary cause of this low tide was the strong persistent wind from the northwest, which resulted from a stationary low pressure area over Maine and the maritime provinces, and a high pressure area over the Great Lakes. The direction of the wind, blowing downstream on Delaware Bay, forced huge volumes of water out of the Delaware Bay and, at the same time, lowered ocean tide levels along the Atlantic Coast. These effects combined to produce the lowest known tide level in the estuary. The minimum tide at Reedy Point, Delaware, 8.3 mi. upstream of HCGS, was 8.6 feet below MSL.

This tide level is 1.7 feet below the previous minimum low tide recorded on January 26, 1939. Analysis of minimum low tides at Philadelphia suggest that the recurrence interval of the December 31, 1962 low tide is about 180 years; see Reference 2.4-16.

2.4.11.4 Future Controls

As discussed in Section 2.4.11.1, minimum flow conditions on the Delaware River have minimal effect on the water levels at HCGS. Future uses of the Delaware River water, even during minimum flow conditions, will not affect the ability of safety-related facilities to function adequately.

Flow augmentation may be an institutional requirement by the Delaware River Basin Commission (DRBC) as a condition of water withdrawal during low flow conditions. Section 2.4.11.5 discusses this requirement.

2.4.11.5 Plant Requirements

2.4.11.5.1 Plant Usage

The Station Service Water System (SSWS) intake houses four service water pumps that supply the plant's water. The rated flow capacity for these pumps is provided in Table 9.2-1. The service water sump invert is at an elevation of 70 feet PSE&G datum, or -16.4 feet MLW, with a well pit configuration. The suction bells of the pumps are 1.5 feet above the sump invert. The service water pump minimum design operating water level is 76 feet PSE&G datum, or -10.4 feet MLW. This level corresponds to the low water condition resulting from a stationary PMH; see Section 2.4.11.2. The Hope Creek Technical Specifications require a plant shutdown when river water level reaches 80 feet PSE&G datum. The service water pump submergence elevations (operating heads) under various hydrologic conditions are:

1. Head of 20.7 feet at mean high water

2. Head of 17.5 feet at MSL

3. Head of 14.9 feet at MLW

4. Head of 8.5 feet at Technical Specification limit.

5. Head of 4.5 feet at design low water.

The service water intake draws water from a tidally affected reach of the Delaware Estuary System. Low flow situations have only a minimum influence on water levels and, therefore, water availability at the intake. Any postulated drought condition has little effect on water levels that are significantly higher than the design basis level. Adequate water supplies are secure during any postulated drought event.

2.4.11.5.2 Institutional Constraints

The DRBC maintains a requirement that electric utility companies provide supplementary water storage to ensure availability of water needed to replace depletive uses at the generating stations during periods of low flow. A discharge of less than 3000 cfs at the Trenton gauge defines a low flow period; see Reference 2.4-1.

The Delaware River Basin Electric Utilities Group (DRBEUG) submitted an application to build an off-stream storage impoundment on Merrill Creek in New Jersey, in response to the DRBC's supplementary storage requirement. The Merrill Creek project is scheduled for completion and reservoir filling during the spring of 1985, as indicated in Reference 2.4-1. The authorized withdrawal of river water under low flow conditions by HCGS is contingent on the completion and operation of the Merrill Creek Project.

2.4.11.6 Heat-Sink Dependability Requirements

The ultimate heat sink (UHS) for HCGS engineered safeguard equipment is the Delaware River, which provides cooling water to the Safety Auxiliary Cooling System (SACS) heat exchangers, through the intake structure and SSWS. The SACS provides demineralized cooling water in a closed loop to the engineered safeguard equipment. The water from SSWS is discharged into the cooling tower basin to provide makeup for the circulating water system. Design bases for the UHS are as follows:

1. To dissipate heat load during normal operation, thorough evaporation to the atmosphere by a natural draft cooling tower, with overflow going to the Delaware River

2. To provide makeup water for the Circulating Water System (CWS)

3. To provide a heat sink for the safeguard equipment during normal plant operation, loss-of-coolant accident (LOCA),

and/or loss-of-offsite power (LOP), and plant shutdown conditions

4. To withstand the most severe natural phenomena or site related event

5. To perform under the adverse meteorological conditions from resulting maximum water consumption and minimum cooling water availability.

Discussion related to Delaware River water temperature is included in the HCGS Environmental Report - Operating License Stage.

2.4.12 Dispersion, Dilution, and Travel Times of Accidental Releases of Liquid Effluents in Surface Waters

The Delaware River is the only surface water body in the vicinity of the station that could potentially be affected by the highly unlikely postulated spillage of liquid radwastes, onsite spills, or operating discharge.

The average freshwater river discharge at the site is 16,000 cfs; the average tidal flow, measured at Wilmington, Delaware, 20 miles upstream of the plant, is 400,000 cfs. Therefore, the major factor determining estuarine velocities in the vicinity of the site is the tidal flow. At the plant site, the Delaware River behaves like a well mixed estuary, with the vertical salinity gradient at a given point usually varying minimally. Longitudinally, the salinity can vary from 10 to 15 parts per thousand, depending on the time of the year, the phase of the tidal cycle, and the river freshwater runoff; see Reference 2.4-59.

As the river segment adjacent to HCGS is relatively shallow (approximately 8 feet deep at MLW, 10 feet offshore), vertically fully-mixed conditions are achieved a short distance from the outlet for all plant discharge conditions. The discharge then follows the estuarine current direction. This structure is shown schematically in Figure 2.4-42.

During normal station operations, plant discharge to the Delaware River has been estimated to range from 18,800 to 23,250 gpm (41.9 to 51.8 cfs). The effluent will flow through an underground conduit to the Delaware River at River Mile 51.1, terminating in a 48-inch diameter horizontal pipe approximately 10 feet offshore at mean tide. The centerline of the opening will be about 6.0 feet below mean low water (MLW). At 20,500 gpm, the discharge velocity will be about 3.5 fps.

The average net tidal flow produces a relatively high current velocity in the station vicinity and is summarized in Figure 2.4-44. On this figure, main channel current measurements have been superimposed for additional insight into the overall current regime. Tide tables indicate that the average maximum velocity is about 3.0 ft/s during maximum flood tide, and 2.7 ft/s during maximum ebb; see Reference 2.4-62.

Based on the average net tidal flow, the HCGS/river flow could produce an average short-term dilution of 1 to 8750, if the flow were mixed uniformly, vertically and laterally, across the river. Based on the thermal plume modeling results, the average short term dilution ranges from 14 to 40-fold within the 3500-foot mixing zone. The presence of ambient cross currents tend to improve overall dilution. The plume boundary is always less than 2300 feet under simulated worst case conditions. Model results indicate that HCGS discharge will be predominantly negatively buoyant. This is a result of the relatively low discharge velocity and large density excess one ambient, as shown on Figure 2.4-43.

The modeling study was conducted by Roy F. Weston, Inc. to determine seasonal mixing zone characteristics and temperature differentials resulting from interaction between the HCGS heat dissipation system and the Delaware River; see Reference 2.4-61. Near field and far field dilution and dispersion behavior of the discharge was simulated using MIT near-field and MIT transient plume models (TPM). The near field model was modified to incorporate shallow water effects (predominantly from bottom interactions), and the TPM model was directly applied for far-field shallow water analysis using boundary conditions supplied by the near-field analysis.

In the near-field region, inertia force dominates the behavior of plume velocity and dilution. As the center line velocity of the plume decreases due to dilution and lateral spreading, inertial effects become dominated by buoyancy, and the jet stratifies towards the bottom or the surface depending on its density relative to the ambient; see Reference 2.4-60. Mixing in the far field is achieved primarily through the process of ambient diffusion and dispersion, as distinct from jet entrainment, which characterizes near field mixing; see Reference 2.4-61.

In each region, the appropriate conservation equations for momentum, thermal energy, and continuity were used along with empirical and/or assumed expressions for relevant physical parameters, such as the drag coefficient between the plume and the ambient current; the interfacial and bottom friction factors; lateral and vertical entrainment coefficients; lateral spread rate for the plume; and heat transfer coefficient. In each region, these conservation and continuity equations were solved using various numerical integration techniques.

To establish a conservative set of model input conditions, the station discharge maximum temperatures (predicted to be exceeded less than 5 percent of the time) and maximum ambient salinity concentrations were used, as well as a cooling tower salinity concentration factor of 1.8, which is 7 percent above the maximum and 28 percent above the minimum monthly concentration. Maximum river current velocity values of 1.76 fps at maximum flood, and 1.84 fps at maximum ebb, were used to conservatively approximate the frictional draft effect of the shoreline. These values are 65 and 61 percent of the river velocities at maximum flood and maximum ebb.

Extensive studies have been performed by the University of Delaware using drifters and drogues near the mouth of the Delaware River. The results indicate that there is a net residual seaward transport in the lower bay area moving at a rate approximately 2 miles per day.

Delaware River water in the region of HCGS site is not used for domestic or agricultural water supply purposes, and the industrial usage is limited to cooling applications. Virtually all domestic, agricultural, and industrial water supplies in this region are obtained from groundwater sources. Therefore, any accidental releases of liquid effluents in surface waters at or in the vicinity of HCGS would have negligible effects on potable water supplies in the area.

2.4.13 Groundwater

2.4.13.1 Description and Onsite Use

The Hope Creek Generating Station (HCGS) site is about 18 miles south of the Fall Line. The Coastal Plain is underlain by a sequence of interbedded sands and silts that comprise a series of aquifers, aquitards, and aquicludes of Cretaceous, Tertiary, and Quaternary ages. The geologic strata generally thicken and dip gently to the southeast. A generalized geologic cross section of the site is shown in Figure 2.4-11. The strata consist (in descending order from the land surface) of approximately 30 to 40 feet of blackish-gray clayey silt (hydraulic fill) overlying 5 to 10 feet of riverbed sand and gravel. The latter deposit is referred to as the shallow aquifer. Approximately 5 to 25 feet of grayish-brown clay underlie the riverbed sand and gravel. This clay is subdivided into an upper inorganic Quaternary Age unit and a lower organic clay unit belonging to the Kirkwood Formation (Section 2.5.1.2.2). The Kirkwood clay is underlain by a second aquifer, referred to as the Vincentown aquifer, composed of the basal sand of the Kirkwood Formation, all of the underlying Vincentown Formation, and upper sands of the underlying Hornerstown

Formation. The Vincentown aquifer is referred to as the "deep aquifer", although there are other aquifers at the site which are deeper.

The basal sand of the Kirkwood Formation generally consists of 5 to 10 feet of reddish brown micaceous, fine to medium grained sand with varying amounts of silt. The underlying Vincentown Formation consists of greenish gray, silty fine to medium grained glauconitic sand, with some highly cemented zones. The basal sand of the Kirkwood Formation is in direct hydraulic contact with the underlying sand of the Vincentown Formation. In addition, the upper sands of the Hornerstown Formation are in direct hydraulic connection with the overlying sand of the Vincentown Formation. Therefore, for analytical purposes, the combination of these three sand units is referred to as the Vincentown Aquifer.

The excavation for HCGS extended into Vincentown Formation to about Elevation +28 PSE&G datum, as shown on Figure 2.4-11. The planned bottom of the excavation was below the natural piezometric level of the Vincentown aquifer (about Elevation +93 to +97 PSE&G datum). Therefore, it was necessary to have dewatering and monitoring operations that would allow the excavation to proceed without adversely affecting the integrity of the foundation soils or the stability of the excavation slopes; see Reference 2.4-49. Plans showing the monitoring wells, dewatering wells, and the well point system are shown on Figures 2.4-12 through 2.4-14, respectively; see Reference 2.4-50.

The dewatering system for HCGS excavation significantly lowered the water table in the shallow aquifer in the area adjacent to the excavation and also lowered the piezometric surface in the Vincentown aquifer in a large area surrounding the site. The cones of depression resulting from the HCGS dewatering system significantly changed the nature of the groundwater flow regime in these two aquifers in the vicinity of the site.

A similar situation existed at the Salem Nuclear Generating Station (SNGS) site, which is directly adjacent to and south of the HCGS site. The dewatering system for the SNGS site had been decommissioned just prior to the start of the excavation operations for the HCGS site. It was expected that the groundwater levels at the HCGS site had substantially recovered from the effect of the prior years of dewatering at the SNGS site. However, water levels measured at the HCGS site prior to the start of the HCGS excavation and dewatering operations probably included residual effects of the previous SNGS dewatering operations.

The Mount Laurel and Wenonah sands, referred to herein as the Mount Laurel-Wenonah aquifer, are separated from the overlying Vincentown Formation by a 40-foot thick aquitard consisting of the Hornerstown and Navesink Formations. The Hornerstown and Navesink Formations are dark green, fine to medium grained clayey sand. The Mount Laurel Formation, where it was penetrated by the test borings, dewatering wells, and water supply wells, consists of greenish brown, fine to medium grained sand with varying amounts of silt. The Wenonah sand is mainly composed of fine to coarse-grained quartzose sands of white, yellow-red, rusty brown, and black hues, as indicated in Reference 2.4-51.

Underlying the Mount Laurel-Wenonah aquifer are the Marshalltown Formation, the Englishtown Sand, the Woodbury Clay, the Merchantville Clay, and the Raritan and Magothy Formations. From this group, only the Raritan and Magothy Formations constitute significant aquifers, although the Englishtown is a significant aquifer farther north in the state. The remaining formations are aquitards and aquicludes, as noted in Reference 2.4-51.

Groundwater is used onsite for industrial, sanitary, potable, and fire protection purposes. The water is pumped from wells screened in the Mount Laurel-Wenonah aquifer and the Raritan and Magothy aquifer. Details of production rates are given in Section 2.4.13.1.3.

2.4.13.1.1 Groundwater Aquifers

Groundwater aquifers are described in these sections, starting with the shallowest aquifer. Confining beds will be described along with the associated aquifers.

In the strictest sense of the word, all of the formations underlying the HCGS site are hydraulically connected, because none of the confining layers that separate the aquifers are completely impermeable. Permeabilities of the aquifers are in the range of 50 to 150 gpd/ft^2 for the shallow aquifer, 100 to 200 gpd/ft^2 for the Vincentown aquifer, and 100 gpd/ft^2 for the Mt. Laurel-Wenonah aquifer. The permeabilities of the confining beds are significantly less. The estimated vertical permeability for the Hornerstown and Navesink formations is 0.4 gpd/ft^2, and the permeability of the Kirkwood clay is estimated at less than 5 gpd/ft^2. Thus, the confining beds acts as aquitards and allow some leakage to occur between aquifers whenever there is a vertical hydraulic gradient between adjacent aquifers separated by an aquitard.

The aquifers present at the site are hydraulically connected to the Delaware River to some extent. Direct evidence of this is the outcrop pattern of the formations that indicates that they are in physical contact with the river. Pumping from onsite wells screened in the shallow Vincentown and Mt. Laurel-Wenonah aquifers has caused a hydraulic gradient to extend from the river toward the wells. In addition, the wells have been producing brackish water, suggesting brackish water intrusion from the Delaware River.

The degree of hydraulic connection of these aquifers with the Delaware River is somewhat reduced by the silting action, which naturally takes place in the river because the river silt is expected to have a lower permeability than the aquifer formations that outcrop under the silt in the river. Dredging of the river to deepen the river channel could, in some areas, remove the silt overlying the aquifer outcrops, thereby increasing the extent of hydraulic connection between the aquifers and the river. When the hydraulic gradient in the aquifers is toward the river, brackish water intrusion into the aquifers would not take place, regardless of the extent of hydraulic interconnection. However, whenever the hydraulic gradient is from the river at the aquifer outcrop zone toward a pumping well field, brackish water intrusion can occur.

The Delaware River channel intersects the dipping Coastal Plain formations in southern New Jersey such that the direction of the channel is approximately normal to strike of the formations. Wherever the river contacts a formation outcrop, there is a hydraulic connection between the river and the formation. Because the formations dip to the southeast, and the river channel is approximately oriented north-south in this area, zones of hydraulic contact between the river and the formations are progressively farther north for progressively deeper formations. The shallow aquifer contacts the river at the site, whereas the Upper Raritan aquifer contacts the river about 10 miles north of the site.

As shown in Figure 2.4-11, the Raritan aquifer has a zone of discharge in hydraulic connection with the river about 10 miles north of the site. If sufficiently heavy pumping were to occur from wells in the Upper Raritan aquifer at the HCGS site and from other wells that could be installed by other users in the future, the hydraulic gradient in the Upper Raritan aquifer could be reversed, thereby changing the Upper Raritan aquifer discharge zone into a recharge zone and allowing brackish water to intrude the aquifer from the Delaware River. Sufficient data are not available to determine when and if this may occur.

In addition, heavy pumping of the Upper Raritan wells could draw salt water from the downdip parts of the aquifer south of the site. The salt water that exists in the downdip parts of the aquifer is probably connate water that existed in the aquifer from the time of deposition and was never flushed out by fresh water because of insufficient hydraulic head in the recharge zones; see Reference 2.4-51.

2.4.13.1.1.1 Shallow Aquifer

The shallow aquifer is composed of riverbed sand and gravels overlain by fine grained hydraulic fill. The primary water producing zone of the shallow aquifer consists of the riverbed sands. Under natural conditions, water recharges the shallow aquifer by rainfall that infiltrates the land surface. Infiltration characteristics of the superficial soils, as measured by percolation tests conducted in accordance with the United States Army Corps of Engineers procedures, range from 1 to 4 gpd/ft^2, with an average rate of 2.7 gpd/ft^2, as indicated in Reference 2.4-52.

The upper soils at the site are dredged fill that was placed at about the turn by the century of the United States Army Corps of Engineers. The fill material is composed of a heterogeneous mixture of silt, fine sand, and organic material that apparently came from the channel of the Delaware River. Information obtained from test borings drilled at the site indicates that the hydraulic fill is generally 30 to 40 feet thick, and is underlain by sand and gravel. The permeability of the sand, estimated from particle size analyses, ranges from about 50 to 150 gpd/ft^2. The lateral extent of the sand member is unknown, but it appears to exist in most of the site area. It is hydraulically connected with the Delaware River, and near the river, water levels in this sand change in response to tidal fluctuations. Water levels in the shallow aquifer are essentially horizontal and, although changes in response to tides do occur, the horizontal component of groundwater movement under natural nonpumping conditions is probably small.

The shallow aquifer, consisting of river sands and gravels, together with the overlying hydraulic fill, is an unconfined or phreatic aquifer. However, because of the much higher permeability of the river sands compared to the overlying hydraulic fill, short term pumping tests on the river sands show a response characteristic of leaky artesian conditions. This is shown by the artesian type storage coefficients listed in Table 2.4-13.

Permeability values of the Pleistocene sands (riverbed sands), as evaluated from grain-size analyses, range from 50 to 150 gpd/ft^2, as noted in Reference 2.4-52. Permeability values based on pumping tests, given in Table 2.4-13, range from 524 to 556 gpd/ft^2. The thickness of the Pleistocene sands ranges from 5 to 10 feet. The data do not show whether the average thickness is closer to 5 or 10 feet. The investigator who conducted the pump testing referenced in Table 2.4-13, apparently assumed the average thickness to be 5 feet, in order to arrive at the permeability values given. However, if one assumes that the average thickness is closer to 10 feet, rather than 5 feet, the permeability range becomes 262 to 270 gpd/ft^2. Thus, by considering the uncertainty in the pumping test results, and the uncertainty in the average or effective aquifer thickness, the resultant uncertainty in the permeability gives a total range of 262 to 556 gpd/ft^2.

No direct measurements of porosity were made on the riverbed sands of the shallow aquifer, but, numerous sieve analyses were made that can be used to estimate the porosity and specific yield (effective porosity) using the graph given on page 24 of Todd, in Reference 2.4-53. The average of the samples analyzed gave a 90 percent size of 6.5 mm. This method gives a porosity of 35 percent and a specific yield (effective porosity) of 28 percent.

It should be noted that the use of the term "specific yield" is appropriate only after the potentiometric surface is lowered to below the top of the sand, thereby allowing gravity drainage to take place. The estimates of porosity and effective porosity are based only on sieve analyses and should be used with caution.

Figure 2.4-15, is a water level contour map for the shallow aquifer on May 17, 1978. It indicates that the drawdown effect on the shallow aquifer, resulting from the dewatering of the excavation, extends only a short distance from the limits of the excavation. This can be compared to a piezometric surface map of the Vincentown aquifer for the same date, shown on Figure 2.4-16. By projection, it can be seen that the cone of depression of the Vincentown aquifer, which is confined except in the immediate vicinity of the excavation, extends far beyond the immediate vicinity of the excavation; see Reference 2.4-50.

2.4.13.1.1.1.1 Effects of Dewatering and Dredging

The normal flow direction of water in the shallow aquifer is expected to be from the central part of Artificial Island in a westerly or southwesterly direction toward the Delaware River. Thus, the land surface would be the zone of recharge, and the Delaware River would be the zone of discharge. Figure 2.4-17 shows the water level contours in the shallow aquifer for January 16, 1976. At this time, the dewatering system at the SNGS site directly south of the HCGS site was no longer in operation. The SNGS dewatering system had been shut down on September 20, 1975. While the cone of degression from the SNGS dewatering system was recovering it can be expected that ground water was still flowing toward the SNGS site. As a result, the groundwater flow direction in the shallow aquifer at the HCGS site on January 16, 1976 was more to the south than to the southwest. On August 24, 1976, the SNGS dewatering system had been out of operation for 11 months. By this time, dredging had begun in the HCGS excavation with a resultant lowering of the water level in the excavation and in the shallow aquifer adjacent to the excavation, causing water in the shallow aquifer to flow radially toward the HCGS excavation. This condition is depicted on Figure 2.4-18; see Reference 2.4-50.

2.4.13.1.1.1.2 Effects of Sand Drains

During the period March 22 to June 21, 1976, a system of 733 vertical sand drains in three lines around the periphery of the excavation were installed with the intent of draining water from the shallow aquifer into the Vincentown aquifer. The locations of the sand drains are shown in Figure 2.4-13. The drains were installed by Moretrench American Corporation using a jetting method. The drains were drilled nominally 12 inches in diameter and approximately 70 feet deep. Each drain was filled with medium- to coarse-grained filter sand. The installation of the drains was observed by Dames and Moore to evaluate whether excessive amounts of filter sand were used (indicating the formation of a cavity by jetting), or whether blowouts occurred in adjacent drains. The drains were all observed to be installed correctly and no problems were reported.

The sand drains were designed to provide additional drainage of water from the shallow aquifer into the Vincentown aquifer, from which the water would be removed by pumping of the deep wells prior to dewatering of the HCGS excavation. In addition, the sand drains were designed to act as interceptors to decrease or eliminate water seeping from the shallow aquifer onto the upper slopes of the excavation during construction activities, prior to backfilling of the excavation. Primary drainage capacity was provided by the deep wells that were screened in both the shallow aquifer and the Vincentown aquifer, see Reference 2.4-50.

No measures were taken to determine the quantity of water being drained by the sand drains. However, significant seepage was not observed by Dames & Moore anywhere on the slopes after the excavation was completely dewatered and dressed, thus indicating that the system of sand drains was effective in intercepting seepage from the shallow aquifer.

It can be seen in Figure 2.4-18 that, even though the water level in the dredge excavation pool (93.8 PSE&G datum or +4.8 MSL) was higher than the elevation of the shallow water directly adjacent to the excavation (93 to 92 PSE&G datum or +4 to +3 MSL), the shallow groundwater flowed radially toward the excavation because of the effect of the sand drains and the dewatering wells. By comparing Figure 2.4-19 with Figure 2.4-18, which show August 24, 1976 piezometric levels for the Vincentown and shallow aquifers, respectively, it can be seen that the piezometric water levels are higher in the shallow aquifer than in the Vincentown aquifer throughout the site. This provided a downward hydraulic gradient allowing water to flow from the shallow aquifer through the sand drains and the dewatering wells into the Vincentown aquifer.

The effectiveness of the sand drain system was demonstrated when the water was completely removed from the excavation. Figure 2.4-20 shows a water level contour map for the shallow aquifer on June 21, 1977. The contours show a hydraulic gradient toward the excavation from all directions, indicating a groundwater flow toward the excavation. The river sand and gravels, which comprise the lowest part of the shallow aquifer, were exposed on all of the excavation cut slopes. No water was seen to be seeping from the sand and gravel because the water was all being intercepted by the system of sand drains and dewatering wells.

The sand drains form a permanent hydraulic connection between the shallow aquifer and the Vincentown aquifer at the HCGS site. The sand drains have been draining brackish water from the shallow aquifer to the Vincentown aquifer ever since they were constructed in 1976, and will probably continue to do so as long as there is a downward hydraulic gradient between the two aquifers. Because of the proximity of the sand drains to the deep dewatering wells, and because of the cone of depression created by the deep dewatering wells, it is expected that all the water drained from the shallow aquifer through the sand drains to the Vincentown aquifer was removed by the deep dewatering wells. It is expected that the sand drain water will continue to be removed from the Vincentown aquifer as long as the dewatering system is in operation. Because of these factors, it is expected that the sand drains have had no measurable effect upon the water quality of the Vincentown aquifer during the dewatering operations.

If the sand drains are not sealed when the dewatering system is decommissioned, they are not expected to have a significant impact on the water quality in the Vincentown aquifer, because the hydraulic gradient between the shallow and Vincentown aquifers is expected to be very small when the dewatering system is shut off. In addition, the water quality is brackish in both the shallow and deep aquifers, so that leakage from the shallow aquifer would not adversely affect water quality in the Vincentown aquifer. See Table 2.4-14, which compares water quality in the two aquifers.

The sand drains are located along the slope of the excavation near the top of the slope. During grading of the slopes in 1977, the surfaces of the sand drains were obliterated and covered to some extent with the fine-grained hydraulic fill. As a result, the sand drains are not readily accessible to allow infiltration of surface water into the sand drains. Therefore, if flooding of the site should occur, it is not expected that the sand drains will have any significant impact upon the Vincentown aquifer.

2.4.13.1.1.1.3 Effects of Backfill

The excavation at the HCGS site is backfilled with granular fill material. This granular backfill is expected to have a much higher permeability than the clay confining layer that separates the shallow and deep aquifers. As a result, the backfill is expected to provide a much more permeable hydraulic pathway than the system of sand drains connecting the shallow aquifer with the deeper aquifer. Thus, any impact that could potentially occur as a result of leakage from the shallow aquifer into the deeper aquifer would occur primarily as a result of the backfill material in the excavation, and only secondarily as a result of the sand drains.

The intrinsic permeability of the backfill of the HCGS excavation is estimated on the basis of sieve analyses of the material from Oldman's Borrow Source. The gradation curves for the material cover a narrow band of sizes. For example, the D_{50} sizes range from 0.52 to 1.2 mm. To be conservative, the gradation curve on the coarse side of the particle size envelope is used for the purpose of estimating the permeability with the Fair and Hatch equation,

$$K = \frac{1}{\left[M \frac{(1-\alpha)^2}{\alpha^3}\left[\frac{\theta}{100} \Sigma \frac{P}{d_m}\right]\right]} \qquad (2.4\text{-}11)$$

where:

k = intrinsic permeability

α = porosity (estimated to be 0.44)

m = packing factor (5)

θ = shape factor (estimated to be 6.0)

P = percentage of particles on a weight basis held between each pair of adjacent sieves

d_m = geometric mean $(d, d_2)^{1/2}$ opening of the sieve pair

The d sizes of the Oldman Borrow envelope are listed below:

Percent Passing	Sieve Size cm
100	7.62
90	1.905
80	0.50
70	0.22
60	0.14
50	0.11
40	0.09
30	0.072
20	0.050
10	0.025
0	0.004

The Fair and Hatch equation gives a value of 0.039 cm/s for k for this gradation curve. The coefficient of permeability K is given by

$$K = \frac{k}{\gamma\mu} \qquad (2.4\text{-}12)$$

where:

γ = density of water (1g m/cm^3 at 68.4°F)

μ = viscosity of water (1 centipoise at 68.4°F)

The coefficient of permeability for the HCGS backfill is conservatively estimated to be 0.039 cm/s (1.28 x 10^{-3} ft/s, 827 gpd/ft^2).

The area within the excavation, between the foundation mat and the top of the Vincentown Formation filled with granular backfill through which water could percolate vertically downward into the Vincentown Formation, is estimated to be about 260,000 ft^2. With unity hydraulic gradient and a permeability of 827 gpd/ft^2, the potential downward percolation rate through the fill would be about 215,000,000 gpd.

The permeability of the sand drains is estimated to be in the range of 5600 to 8,200 gpd/ft^2. The total cross sectional area of the 733 sand drains is about 1464 ft^2. With an estimated maximum average permeability of 8200 gpd/ft^2 for the sand and unity hydraulic gradient in the sand drains, the estimated maximum drainage through the sand drains is about 12,000,000 gpd.

Because of the larger cross sectional area of the fill compared to the sand drains, the potential for surface spills entering the Vincentown Formation is much greater through the fill than through the sand drains, event though the sand drain sand has a higher estimated permeability than the backfill material.

2.4.13.1.1.1.4 Water Quality

During the operation of the HCGS dewatering system, the cone of depression causes brackish water intrusion from the Delaware River into the shallow aquifer, thereby potentially degrading the water quality in the shallow aquifer. Because the shallow aquifer is overlain by hydraulic fill dredged from the nearby Delaware River, the shallow aquifer may have always contained brackish water that may not have been flushed out by rain water recharge. Flooding of the site by occasional high flood waters from the Delaware River prior to construction at HCGS may have caused some recharge of brackish water to the shallow aquifer.

Chemical analyses of water samples from the shallow aquifer are shown in Table 2.4-14. The shallow aquifer is characterized by about 1000 to 2000 mg/l chloride; 4.5 to 225 mg/l sulphate (water from well 323 had 1080 mg/l); 520 to 1600 mg/l sodium up to 220 mg/l total iron; up to 6.722 mg/l total dissolved solids (TDS); and 1250 to 1850 mg/l total hardness. Turbidity is moderate to high (up to 1050 turbidity units); see Reference 2.4-54.

A groundwater quality map for the shallow aquifer, also referred to as the Quaternary river bed aquifer, is shown on Figure 2.4-21. The map shows lines of equal chloride concentration in the shallow aquifer. Two sets of contour lines are shown: one for water samples taken before the pumping test, the other for samples taken after the pumping test. The contours show that a shift in the isochlors towards the pumping well occurred during the pump test. It should be noted that these isochlor shifts refer to the shallow aquifer pump test and not the Vincentown aquifer pump test.

Chemical analyses of water samples from the Vincentown aquifer are shown in Table 2.4-14. The Vincentown aquifer is characterized by about 2800 to 5100 mg/l chloride, 63 to 330 mg/l sulfate, 1200 to 2100 mg/l sodium, up to 267 mg/l total iron, up to 9100 mg/l TDS, 1280 to 3260 mg/l total hardness, and high turbidity (up to 720 turbidity units); see Reference 2.4-54.

A groundwater quality may for the Vincentown aquifer is shown in Figure 2.4-23. The map shows lines of equal chloride concentration. Two sets of contour lines are shown: one set for water samples taken before the pumping test, and one set for water samples taken after the pumping test on the deep aquifer. Contours show a shift in the isochlors that took place during the pumping test, as indicated in Reference 2.4-54.

2.4.13.1.1.1.5 Aquifer Parameters

A pumping test was performed on the shallow aquifer at the HCGS site to determine the aquifer transmissivity and storage coefficient. These aquifer parameters are listed in Table 2.4-13.

2.4.13.1.1.2 Vincentown Aquifer

The Vincentown aquifer is separated from the overlying shallow aquifer by the Quaternary and Kirkwood clay units; see Section 2.5.1. Because of hydraulic interconnections, the aquifer consists of the entire Vincentown Formation, the basal sands of the overlying Kirkwood Formation, and the upper sands of the underlying Hornerstown Formation.

Several pumping tests were performed on the Vincentown deep aquifer at the HCGS site to determine the transmissivity and storage coefficient. These aquifer parameters are listed in Table 2.4-14.

The Kirkwood Formation of Miocene Age underlies the Quaternary and extends to about 70 feet in depth. It consists of a gray silty clay and is an aquitard. Permeability values are less than 50 gpd/ft^2.

Prior to pumping of the SNGS and the HCGS dewatering systems, the direction of regional groundwater flow in the Vincentown aquifer was probably southwesterly toward the Delaware River. Operation of the SNGS dewatering system temporarily modified the groundwater flow regime in the aquifer, creating a radial flow pattern toward the SNGS site. Groundwater flow from the HCGS site was in a southerly direction toward the SNGS dewatering system until it was decommissioned. Shortly after the SNGS dewatering system was shut off, the HCGS dewatering system was put into operation, temporarily modifying the groundwater regime and creating a pattern of radial flow toward the HCGS excavation. The groundwater flow is expected to return to a southwesterly direction when the HCGS dewatering system is decommissioned.

The Vincentown Formation is about 65 feet thick and is encountered at a depth of about 70 feet. It consists of fine to medium grained sand, with occasional gravel, and is separated from the overlying shallow aquifer by about 35 feet of low permeability silty clay. Grain size analyses of this sand indicate an estimated permeability of about 200 gpd/ft^2. Pumping tests give values in the range of 100 to 148 gpd/ft^3.

No direct measurements of porosity were made on the Vincentown aquifer. However, numerous sieve analyses were made that can be used to estimate the porosity and specific yield (effective porosity) using the graph in page 24 of Todd in Reference 2.4-53. The average of samples analyzed from the basal sands of the Kirkwood (considered as the upper part of the Vincentown aquifer) gave a 90 percent size of 0.65 mm. This gives a porosity of 42 percent and a specific yield of 30 percent.

Numerous sieve analyses were made on samples from various depths within the Vincentown formation itself. These gave 90 percent sizes which ranged from 0.55 mm to 0.8 mm, indicating a range of 42 percent to 41 percent for porosity and 29 percent to 32 percent for specific yield, respectively.

Water levels in the Vincentown aquifer are affected by tidal fluctuations in the Delaware River. It was found that the tidal efficiency of the aquifer varies inversely with distance from the Delaware River. The average tidal efficiency at the site was found to be 7 percent, as indicated in Reference 2.4-55.

Water levels in the Vincentown aquifer are also affected by barometric pressure changes. The barometric efficiency of the Vincentown aquifer averages 68 percent at the site; see Reference 2.4-55.

Figure 2.4-22 shows a piezometric contour map for the Vincentown aquifer on April 22, 1975, with the SNGS dewatering system still in operation. It can be seen that the groundwater flow is primarily to the south, toward the SNGS site. The contours vividly show part of the cone of depression caused by the SNGS dewatering system; see Reference 2.4-50.

Figure 2.4-19 shows piezometric contours for the Vincentown aquifer on August 24, 1976. The SNGS dewatering system was shut off on September 20, 1975. The water levels in the Vincentown aquifer shown are, therefore, probably representative of normal or nearly normal groundwater conditions, but it is probable that there are some residual drawdown effects in the August 24, 1976 water levels. The data is insufficient to determine whether or not the Vincentown water levels had completely recovered by August 24, 1976; see Reference 2.4-50.

Operation of the HCGS dewatering system causes a large cone of depression in the Vincentown aquifer piezometric surface around the power complex. This is shown graphically by the piezometric level contour map for May 17, 1978, in Figure 2.4-15. This water level contour map shows that the water in the Vincentown aquifer was flowing radially toward the excavation as a result of the operation of the HCGS dewatering system. It is expected that the flow pattern will retain essentially the same configuration until the HCGS dewatering system is decommissioned.

The Hornerstown Sand conformably underlies the Vincentown Formation in New Jersey; see Reference 2.4-56. It consists of highly glauconitic, fine to medium, dark green quartz sand, with varying but large amounts of silt and traces of shell fragments.

At the site, the upper surface of the Hornerstown Sand is generally encountered at Elevation -30 PSE&G datum. Its thickness ranges between 14 and 25 feet. Hydrologically, the Hornerstown Sand is considered to be a leaky confining layer. Its field permeability is estimated to be about 30 gpd/ft^2; its vertical permeability to be about 0.4 gpd/ft^2, as indicated in Reference 2.4-56. Laboratory measurements of permeability are given in Table 2.4-15. The upper sand of the Hornerstown Formation is considered to be hydraulically part of the Vincentown aquifer.

The Navesink Formation unconformably underlies the Hornerstown Sand. It consists basically of fine glauconite sand, with varying amounts of silt and clay. Its thickness varies from about 35 feet in East-Central New Jersey to 5 feet in the southwest. At the site, its thickness is about 20 to 22 feet. Its top occurs at approximately Elevation -50 PSE&G datum. It does not extend to New Castle County, Delaware; see Reference 2.4-54. Hydrologically, the Navesink Formation is considered to be a leaky confining layer together with the lower part of the overlying Hornerstown Formation. Vertical permeability computed for both units in an aquifer performance at SNGS was 0.4 gpd/ft^2, as noted in Reference 2.4-56. Its average permeability is estimated to be 60 gpd/ft^2, also noted in Reference 2.4-56. Laboratory measurements of permeability are given in Table 2.4-15.

2.4.13.1.1.3 Mount Laurel - Wenonah Aquifer

The Mount Laurel Formation, together with the underlying Wenonah Formation, are considered to act as a single aquifer and are treated as such in this discussion; see Reference 2.4-54.

The Mount Laurel Sand underlies the Navesink Formation with a sharp, conformable contact. Its lithology varies laterally from interbedded clay and sand to massive sand beds, as mentioned in Reference 2.4-54. In the vicinity of the site, the top of the Mount Laurel Formation consists of brown, fine to coarse glauconite quartz sand and contains few shell fragments. The silt fraction makes up about 35 to 50 percent of the entire sediment, with only traces of clay.

The Wenonah Formation is characterized by gradational contacts both at the bottom into the Marshalltown Formation, and at the top into the Mount Laurel Sand. The Wenonah Formation consists basically of a very fine to a fine micaceous quartz sand and clayey silt. The formation has a maximum thickness of 60 feet near Trenton, New Jersey, and thins out both to the northeast and to the southwest. The formation either disappears or thins out in Delaware. An exact determination is extremely difficult due to the gradational contacts at the top and bottom; it is not reported in Delaware; see Reference 2.4-54.

Observation Well G, east of SNGS, shows about 20 feet of light gray-greenish fine to medium glauconitic quartz sand, forming a transition between a greenish gray medium grained sand of the Mount Laurel type and a dark gray fine to very fine clayey sand and silt of the Marshalltown Formation at about elevation -200 PSE&G datum.

The Mount Laurel Sand crops out about 4 to 5 miles north of the site, along a NE-SW trending band which intersects the Delaware River. At the site, the top of the Mount Laurel Sand is encountered at approximately Elevation -65 PSE&G datum. Its thickness, as determined by Bore holes 201 and 206, and by Observation Wells A and G, varies between 105 and 115 feet and increases to the southeast. The Mount Laurel Sand, together with the underlying Wenonah Formation, is one of the most important sources of water supply in New Jersey as noted in Reference 2.4-56. In Delaware, it is only a minor aquifer; see Reference 2.4-54. Downdip from the formation outcrop at the site, the aquifer is artesian; being confined by the Navesink and lower Hornerstown Formations.

Hydraulic characteristics for the aquifer were computed from tests in Salem County, about 8 miles northeast of the site, and showed values of transmissibility of about 9000 gpd/ft and permeabilities of about 100 gpd/ft^2; see Reference 2.4-56. In New Castle County, Delaware, the computed transmissibility was on the order of 1800 gpd/ft, noted in Reference 2.4-56. Laboratory permeabilities are listed in Table 2.4-15.

No direct measurements of porosity were made on the Mount Laurel-Wenonah aquifer. Rosenau, et al., in Reference 2.4-56, gave a porosity value for the aquifer of 44.1 percent. By using the graph on page 24 of Todd, in Reference 2.4-53, the specific yield is estimated to be about 30 percent.

Recharge to the Mount Laurel-Wenonah aquifer in Salem County is derived from vertical leakage from overlying aquifers. In the outcrop area, discharge to local streams, as well as some recharge, occurs. The quality of groundwater in the Mount Laurel-Wenonah aquifer appears to be quite sensitive to drops in water levels caused by heavy pumpage. This fact is shown by an increase in salinity of water from public supply wells in the Salem area during the 1961 and 1966 drought periods, due to intrusion from the Delaware River; see Reference 2.4-56. At the HCGS site, salinity increases toward the north, the northwest, and possibly the east in the Mount Laurel-Wenonah aquifer. The increase in salinity toward the margins of the site is interpreted as an active brackish water intrusion in progress, and/or vertical leakage from the overlying formations.

Regionally, the water quality of the Mount Laurel-Wenonah aquifer is quite variable, with TDS varying from less than 40 ppm to 700 ppm, with chloride varying from 5 to about 400 ppm in Salem County, with iron concentrations from 0.1 to 6.3 ppm, and with hardness varying from 12 to 345 ppm; see Reference 2.4-52. The range of TDS is related to the varying compositions of the recharge water and, possibly, to differences in the geochemical regime within the saturated portion of the aquifer.

At the site, water in the Mount Laurel-Wenonah aquifer is characterized by 29 to 7060 mg/l chloride, up to 212 mg/l sulphate, 20 to 3900 mg/l sodium, 0.03 to 195 mg/l total iron, 316 to over 7500 mg/l TDS, 217 to 3280 total hardness, and 0.14 to 273 turbidity units. The variations in salinity may be attributed to the presence of a brackish water intrusion taking place north of the site (probably where the Vincentown and the Mount Laurel-Wenonah Formations crop out under the Delaware River), or to vertical leakage through the Hornerstown and Navesink Formations. North of the HCGS site, the Kirkwood clay pinches out, and the Hornerstown and Navesink Formations appear to be sandier and more pervious. The increase in permeability reduces protection against salt water intrusion into the Mount Laurel Formation; see Reference 2.4-54.

Figure 2.4-24 shows the pattern of isochlors in the Mount Laurel-Wenonah Formations. The values were determined after a pumping test conducted to determine aquifer parameters at the HCGS site. Two sets of isochlors are present:

1. One set consists of values from 4250 to about 6000 mg/l, based on samples taken from Observation Wells 401, 402, and 404. Wells 402 and 404 were screened in the upper part of the Mount Laurel-Wenonah from Elevation -69 to -100 PSE&G datum. Well 401 was screened from elevation -111 to -131 PSE&G datum, indicating that at this location, the Mount Laurel-Wenonah Formations are brackish from the top to the middle portion of the aquifer.

2. The other set consists of values from 29 to 750 mg/l, based on samples from SNGS Production Wells 1, 2, 3, and 4, as well as Observation Wells C and D. These wells were all screened in the lower part of the aquifer, from Elevation -115 to -200 PSE&G datum.

The Marshalltown Formation underlies the Wenonah Formation with a sharp contact. It consists of greenish black, very fine to fine sand and sandy clay. It is generally 10 to 15 feet thick throughout the region, as noted in Reference 2.4-54, although a thickness of about 125 feet has been reported. Borings 201 and 206 encountered dark gray clayey silt, with minor quantities of quartz and glauconite at Elevation -174 PSE&G datum down to Elevation -212 PSD in Boring 201 and Elevation -218 PSE&G datum in Boring 206.

Hydrologically, the Marshalltown Formation is a leaky confining layer, used by only a few domestic wells, as mentioned in Reference 2.4-56. The water quality is generally fair to poor for human consumption because of its high iron content, turbidity, and an objectionable odor; see Reference 2.4-56.

2.4.13.1.1.4 Raritan and Magothy Aquifer

The Raritan and Magothy formations are considered as a unit, because there is reason to believe that the major aquifers in them are connected with each other at distance, if not locally, as noted in Reference 2.4-51. The discussion of these formations is included because the production wells at the HCGS draw their water from the upper Raritan.

The Raritan and Magothy Formations are separated from the overlying Mt. Laurel-Wenonah aquifer by a sequence of formations including the Merchantville clay, the Woodbury clay, the Englishtown sand, and Marshalltown Formation. The Merchantville, Woodbury, and Marshalltown Formations are aquicludes, whereas the Englishtown is an aquifer elsewhere in the state; locally, it is too fine-grained to be productive.

The aquifer sands and the interbedded clay layers of the Raritan and Magothy Formations appear to lack horizontal continuity, with the aquifers being somewhat more continuous than the clays. The Magothy Formation is overlain by a thick and apparently continuous layer of clay that prevents interconnection with higher aquifers. Likewise, the aquifers of the Raritan Formation are underlain by confining materials that appear to prevent interconnections with any lower aquifers.

The combined Raritan and Magothy Formations have a maximum thickness of about 475 feet near their outcrop area. They crop out, or are covered by a thin veneer of permeable deposits of Pleistocene or Recent age, in a band several miles wide that extends southwestward from Long Island across New Jersey and Delaware into Maryland. The outcrop area closely follows the general course of the Delaware River between Trenton, New Jersey, and Wilmington, Delaware. The top of the Magothy dips at an average rate of 45 feet per mile, and the basal sands of the Raritan dip at about 60 feet per mile to the southeast. Presumably, the formations extend beneath the coastal plain sediments from the outcrop area all the way to the continental shelf, about 100 miles offshore from the southern coast of New Jersey.

The porosity of the Magothy is about 45 percent, and its specific yield (effective porosity) is about 40 percent. The average porosity for the Raritan is 40 percent and the average specific yield (effective porosity) is 35 percent; see Reference 2.4-51.

Values of permeability, transmissivity, and storage coefficient for the Raritan aquifer at the site are not available. Values of these parameters for the aquifers are available at other locations, as given by Barksdale, et al., in Reference 2.4-51. These values are summarized in Table 2.4-18.

The Raritan Formation derives its name from the Raritan River in Middlesex County, New Jersey. Typically, it is composed of light colored medium to coarse grained quartzose sand containing some gravel and varicolored clays. Shades of white, yellow, brown, red, and light gray are characteristic of the materials. Lignite and pyrite occur in some beds. The Magothy Formation derives its name from the Magothy River in Maryland. Typically, it consists of beds of dark gray or black clay that are often lignitic and contain pyrite, alternating with beds of white micaceous fine-grained sand. Rapid lithologic changes in the Raritan and Magothy sediments are extremely common, so that it is difficult to trace one bed or layer very far.

In and near the outcrop area, the clay units in the two formations are generally composed of dense, tough, plastic clays. Exceptional beds appear to have been hardened until they approach the consistency of soft shales, but have not lost their plasticity. Many of the clays, particularly those of the Raritan Formation, are suitable for use in the ceramic industry and some for the manufacture of fine china. They have been extensively mined for these purposes.

The sand units are usually well sorted. They vary in texture from very fine grained sands in the Magothy, to medium or coarse grained sands and to fine grained gravel in the Raritan. Small boulders or large cobbles, several inches in diameter, are rarely observed in the outcrop of the Raritan or are recovered from wells that tap it.

2.4.13.1.2 Aquifer Recharge and Discharge

Natural recharge to the shallow aquifer occurs from direct infiltration of precipitation on the outcrop area within the site. Temporary induced recharge also occurs from the Delaware River as a result of the increased gradient toward the HCGS dewatering system. Discharge under normal conditions would be to the west and southwest toward the Delaware River. Some discharge occurs to the dewatering system that is currently in operation at the HCGS site. Some discharge occurs by leakage to the underlying Vincentown aquifer. Part of the leakage is by way of the sand drain system that connects the shallow aquifer with the Vincentown aquifer at the site. The sand drains were constructed in conjunction with the HCGS dewatering system.

Recharge to the Vincentown aquifer occurs primarily by leakage from the overlying shallow aquifer. Because of the lowering of the Vincentown aquifer piezometric surface to levels below mean sea level (MSL) by the dewatering operations of the HCGS and SNGS sites, some recharge to the Vincentown aquifer has occurred by salt water intrusion from the Delaware River. This is reflected by the high chloride content of the water discharged from the HCGS dewatering system. Under normal conditions, discharge from the Vincentown aquifer is expected to be toward the southwest to the Delaware River. The dewatering operation at HCGS has thus changed the Delaware River at the site from a zone of discharge into a zone of recharge, for the Vincentown aquifer.

Recharge to the Mount Laurel-Wenonah aquifers normally occurs by leakage from overlying aquifers. Under such conditions, discharge is expected to be toward the Delaware River. However, pumping of the Mount Laurel-Wenonah aquifer by SNGS water supply wells has caused a lowering of the piezometric surface and salt water intrusion from the Delaware River. The Delaware River at the site has thus been changed from a zone of discharge to a zone of recharge, for the Mount Laurel-Wenonah aquifer, as well. Recharge to the Raritan and Magothy aquifers occurs from precipitation in the outcrop area by induced infiltration from bodies of surface water that lie in their intake areas (mainly the Delaware River), and by leakage from the aquifers through the overlying or underlying aquicludes.

The natural zones of discharge for the Raritan-Magothy are the small streams that cross the high parts of the intake area, and the rivers in the low parts, such as the Delaware, Raritan, and South Rivers and Raritan Bay; see Reference 2.4-3.

2.4.13.1.3 Onsite Use of Groundwater

Groundwater is developed as a permanent source of water for the HCGS site and the adjacent SNGS site. The water is used for industrial, sanitary, potable, and fire protection purposes. No groundwater is used as a safety-related source of water; see Reference 2.4-57.

Cooling water is derived directly from the adjacent Delaware River.

The initial water production wells constructed on the SNGS site draw water from the Mount Laurel-Wenonah aquifer. These wells are an average of 300 feet deep and screened only in the Mount Laurel-Wenonah aquifer. Because of salinity problems, additional wells were drilled into the deeper Raritan-Magothy aquifer to produce water for use on the SNGS and the HCGS sites.

Locations of the production water wells for the HCGS site, and the observation wells and production water wells for the SNGS site, are shown on Plant Drawing C-5018-0.

All water wells on both sites, except the HCGS No. 1 Production Well, are double cased with 20- to 26-inch casing down to the first rock (sic) formation, and a full length, 16- to 18-inch casing down to the particular aquifer. New Jersey state approved sealing is provided at the top of each well, and cement grouting is installed around the full length of the casings. No. 1 Production Well has a 10-inch casing down to the aquifer. The pumps are of the vertical turbine multi stage type, and have above-ground motors. A screen and gravel filter are installed in the aquifer at the lower end of each casing; see Reference 2.4-54.

A tabulation of the pertinent data for each well is listed below; see Reference 2.4-57:

	SNGS[1]					HCGS	
Well No.	1	2	3	5	6	1	2
Depth, ft	300	300	300	800	1100	800	800
Pump setting, ft	170	170	170	370	400	270	270
Capacity, gpm[3]	250	300	600	800[2]	600	750	750

NOTES:

(1) No. 4 Well is sealed and abandoned because of high chloride readings.
(2) An older report lists this well with a capacity at 200 gpm (Table 2.4-17).
(3) Water for fire protection and potable use at the HCGS site is stored in two above-ground tanks with 300,000 gal capacity each, located at coordinates N1800, W600.

Well water withdrawal rates (gallons per minute, gallons per month, and gallons per year) are limited by permit from the State of New

Jersey. Historical use of water by the SNGS site is shown in Figure 2.4-26; see Reference 2.4-57.

2.4.13.2 Sources

Nearly all water used for consumptive purposes within 25 miles of the site is groundwater. With the exception of the highly industrialized Wilmington, Delaware area, the major uses of water are for domestic and agricultural application; see Reference 2.4-59.

2.4.13.2.1 Regional Use of Groundwater

Wells in the vicinity of the site were canvassed for the environmental study that was published in 1971; see Reference 2.4-58. The inventory was updated in 1982 by an examination of well permit applications on file with the New Jersey Department of Environmental Protection and the Salem County Department of Health.

There are six towns in New Jersey within 25 miles of the site that have public water supplies. There are five public water supplies in Delaware within 15 miles of the site. Data concerning these public water supplies are shown in Table 2.4-16, Public Water Supplies in the Vicinity of the Site. The locations of these supplies are shown on Figure 2.4-27, Public Water Supplies in Vicinity of the HCGS Site; see Reference 2.4-58.

Nearly all domestic water supplies in this region are obtained from private wells other than those serving the SNGS and HCGS sites. Most wells are 2 inches in diameter and greater than 75 feet deep. The aquifer commonly used in the vicinity of the site is the Mount Laurel-Wenonah Formation. Information pertaining to these domestic wells is presented in Table 2.4-17. Wells in the vicinity of the site are shown on Figure 2.4-28; see Reference 2.4-58.

2.4.13.2.2 Use of Groundwater in the Vicinity of the HCGS Site

Except for the PSE&G water production wells at the site that produce groundwater for use on the HCGS and SNGS sites, there are no known operating water wells within two miles of the site. The locations of operating wells are shown on Figure 2.4-28, derived from Reference 2.4-58. There are three abandoned wells near the site, reported to be several hundred feet deep.

The nearest residences to the site are about three miles distant. These are seasonal cottages inhabited primarily during summer weekends. A water supply is obtained from shallow driven wells, or in some cases, bottled water is carried in along with the other provisions; see Reference 2.4-58.

Most of the water wells inventoried are located three to four miles from the site. The nearest wells in Delaware are more than three miles from the site and were not canvassed since it is not believed that they would be affected by a change in the groundwater regimen at the site, because of the intervening Delaware River that acts as a constant head boundary for the shallow aquifer, Vincentown aquifer and Mount Laurel-Wenonah aquifer.

2.4.13.2.3 Projected Future Use of Groundwater

Because of the salinity of the water in the shallow aquifer, Vincentown aquifer, and the Mount Laurel-Wenonah aquifer in the vicinity of the site, it is not expected that groundwater use in these aquifers will be developed locally for domestic and public use in the future. Likewise, distant users of these aquifers would be unaffected because they would be beyond the zone of influence of the dewatering operation at the HCGS site.

All of the projected groundwater use by the HCGS site will be produced from the deeper Upper Raritan aquifer. No municipal wells are known to produce water from this very deep aquifer in the vicinity of the site. Therefore, groundwater use by the HCGS is not expected to affect existing groundwater users.

Possible future development of water supplies from the Upper Raritan aquifer for agricultural, industrial, or municipal use could be affected if they are developed within the zone of influence of the HCGS and SNGS wells.

2.4.13.2.4 Water Levels and Groundwater Movement

Water levels were measured in observation wells prior to and during foundation excavation of the HCGS site. Water levels were monitored in the shallow aquifer and the Vincentown aquifer to assess the effects of the dewatering operations at the HCGS site. Some measurements were made within the Mount Laurel-Wenonah aquifer during the same period to assess the effect on the deeper aquifer of dewatering the Vincentown aquifer.

2.4.13.2.4.1 Observation Wells

A network of permanent observation wells was installed in the shallow aquifer and the Vincentown aquifer in the vicinity of the excavation for the HCGS. Two observation wells screened in the Vincentown aquifer were also installed on the SNGS site to monitor water levels during HCGS excavation and dewatering operations. In addition to the observation wells, pressure cell piezometers were installed under the excavation to monitor the soil pore pressure. These pressure cell piezometers were operated remotely by nitrogen gas that flowed through tubes to the pressure cells from an instrumentation shed. The locations of the observation wells and the pressure cell piezometers are shown on Figure 2.4-12, and in Reference 2.4-50.

Four permanent observation wells are installed to monitor ground water level on Artificial Island. The locations of these observation wells are shown in Plant Drawing C-5018-0.

2.4.13.2.4.2 Groundwater Levels

The depth to water in the shallow observation wells ranged from 5 to 10 feet prior to the start of the excavation and dewatering operations. Details of water level fluctuations in the shallow aquifer are shown in the water level hydrographs on Figures 2.4-29 through 2.4-33 for the construction period. Water level contour maps are given in Section 2.4.13.1.1.1; see also Reference 2.4-50.

Records of natural seasonal fluctuations in the water levels of the shallow aquifer, beneath and in the vicinity of the site, as being unaffected by the dewatering system are not available. It is expected, however, that groundwater levels would respond to seasonal variations in rainfall recharge with a groundwater level decline occurring during the summer, and recoveries occurring during the spring rainy season. Because of the closeness of the land surface to sea level, the natural seasonal variation can not be expected to be greater than a few feet. Likewise, long term natural fluctuations in water levels of the shallow aquifer beneath and in the vicinity of the site are not expected to be greater than a few feet.

The depth to the piezometric surface of the Vincentown aquifer was approximately 10 feet below the ground surface prior to the start of the excavation and dewatering at the HCGS site excavation. Dewatering lowered the Vincentown piezometric surface by about 67 feet in the central part of the excavation. Details of piezometric level fluctuations in the Vincentown aquifer are shown in the water level hydrographs in Figures 2.4-34 through 2.4-40. Piezometric level contour maps are given in Section 2.4.13.1.1.2; see also Reference 2.4-50.

Records of natural seasonal and long term fluctuations of the water levels in the Vincentown aquifer in the vicinity of the site are not available. Because of the close relationship observed between the shallow and Vincentown aquifer water levels, it is expected that seasonal and long term fluctuations in the Vincentown aquifer would be on the order of only a few feet.

No records are available regarding seasonal and long term natural fluctuations of water levels in the Mt. Laurel-Wenonah aquifer in the vicinity of the site.

No records are available for seasonal and long term natural fluctuations of water levels in the upper Raritan aquifer beneath and in the vicinity of the site.

2.4.13.2.4.3 Direction of Groundwater Flow

Under normal conditions groundwater in the shallow aquifer is expected to flow to the southwest and discharge into the Delaware River. However, dewatering operations alter the groundwater flow pattern at the HCGS site. Groundwater flow in the Vincentown aquifer is radial toward the ring of dewatering wells and sand drains surrounding the excavation. Details of the dewatering systems are given in Section 2.4.13.1.1.1. When the dewatering system is decommissioned and groundwater levels return to normal, it is expected that the direction of groundwater flow in the shallow aquifer will be primarily to the southwest, toward the Delaware River. Part of the groundwater flow from the shallow aquifer will continue to leak downward into the Vincentown aquifer through the system of sand drains whenever the piezometric surface is higher in the shallow aquifer than in the Vincentown aquifer. The piezometric surface is not expected to be higher in the Vincentown aquifer than in the shallow aquifer, under natural conditions. Therefore, upward flow from the Vincentown aquifer to the shallow aquifer would not be expected. Due to poor water quality in the Vincentown aquifer, it is not expected that production wells will be installed in this aquifer in the foreseeable future.

Little data is available regarding the groundwater flow in the Mount Laurel-Wenonah aquifer at the site. Because of the pumpage of SNGS site production wells PW-1, PW-2, and PW-3, and the evidence of brackish water intrusion from the Delaware River, it can be expected that the groundwater flow in the aquifer would be radial toward the center of pumping of the three wells; see Plant Drawing C-5018-0. At the HCGS site, the groundwater flow component would thus be expected to be approximately in a southeast direction.

Groundwater for use at the HCGS site is withdrawn from two wells, HC-1 and HC-2, screened in the Upper Raritan Formation; see Plant Drawing C-5018-0. Regional groundwater flow, as given by Barksdale, et al, in Reference 2.4-51, is given in Figure 2.4-41. This regional map shows that the groundwater flow direction in the Raritan Formation in the area of the site is expected to be toward the northeast. But whenever HCGS wells HC-1 and HC-2 are pumping, local groundwater flow is expected to be radial toward the pumping wells. If the net withdrawal from the Upper Raritan aquifer is significant compared to the regional flow in the area, it may cause a permanent condition of radial flow toward the pumping wells.

A decline in water levels in the Raritan aquifer is expected to be associated with the removal of water from the aquifer by the HCGS production wells, and by existing and future users updip from the site. As long as the HCGS production wells are pumping, it is expected that the hydraulic gradients in the upper Raritan aquifer will be toward the HCGS production wells, and that the water levels will continue to decline gradually until the cone of depression meets a recharge boundary such that the rate of recharge balances the pumping rate. This will occur when the cone of depression reaches the aquifer intake area, or if leakage from an underlying and/or overlying aquifer balances the pumping rate. Sufficient data are not available to estimate when this may occur.

Pumping water from the Raritan aquifer is not expected to have any significant effect on water levels or gradients in overlying or underlying aquifers, because of the thick confining layer, the slow leakage rate, and the widespread area over which leakage generally occurs in artesian aquifers.

2.4.13.2.5 Aquifer Parameters

Pumping tests were performed on wells screened in the shallow aquifer, and on wells screened in the Vincentown aquifers, to estimate the formation transmissivity, storativity, and permeability. Laboratory tests of undisturbed soil samples were made to estimate the vertical permeability of the samples. The results are summarized in Tables 2.4-13 and 2.4-15.

2.4.13.2.6 Reversibility of Groundwater Flow

Dewatering operations at the HCGS site have caused significant reversals of groundwater flow directions within the shallow and Vincentown aquifers. Normal groundwater flow directions are expected to be to the southwest for both aquifers. But, dewatering created a radial flow pattern toward the excavation, thus reversing the flow direction in the region between the excavation and the Delaware River. When the dewatering system is decommissioned, it is expected that the radial flow pattern will dissipate and that the normal groundwater flow direction to the southwest will resume. Some minor adjustment in flow paths may remain, because of impermeable structures that penetrate the aquifers on the HCGS site.

Groundwater flows in the Mount Laurel-Wenonah aquifer and the Upper Raritan aquifer are expected to be primarily radial toward the high capacity production wells at both the HCGS and SNGS sites, as long as the wells are operating. During periods when the wells are not operating, it is expected that local changes of groundwater flow directions will take place as a state of equilibrium becomes re-established.

2.4.13.2.7 Water Quality

The water quality in the shallow and Vincentown aquifers is generally brackish because of brackish water intrusion from the Delaware River. Total dissolved solids in the shallow aquifer are in the range of 4000 to 6,700 ppm. Total dissolved solids in the Vincentown aquifer are in the range of 6000 to 9100 ppm. These data are summarized in Table 2.4-14; see Reference 2.4-54.

Water quality in the Mount Laurel-Wenonah aquifer is variable, possibly as a result of fluctuations in pumping rates, which could cause changes in the rates of brackish water intrusion. The data in Table 2.4-14 show a range of about 300 to over 7500 ppm of TDS in the various test wells. More recent analyses of water from the SNGS production wells (Wells No. 1, 2, and 3) in the Mount Laurel-Wenonah aquifer are shown in Tables 2.4-20, 2.4-21, and 2.4-22. Progressive brackish water intrusion is indicated by an increase in dissolved mineral content with time, for Wells No. 2 and No. 3; see Reference 2.4-57.

Water quality in the Upper Raritan aquifer is generally better than in the Mount Laurel-Wenonah aquifer. Typical analyses are shown in Table 2.4-23 (Well No. 5). It can be seen that concentrations of most constituents are lower in water from the Upper Raritan wells than in samples from the Mount Laurel-Wenonah wells (Wells 1, 2, and 3). The time sequence of analyses for the upper Raritan water (Well No. 5) indicates an increasing trend in dissolved mineral content starting with the October 15, 1980 analysis, suggesting that brackish water intrusion may have begun in the Upper Raritan aquifer at the site.

It should be noted that the position of the HCGS wells in the Upper Raritan is relatively close to the fresh water-salt water interface estimated by Barksdale, et al, in Reference 2.4-51. The water in the Raritan aquifer downdip of the HCGS site is salt water, which was probably never flushed out by recharging fresh water. Because of the relatively low elevation of the Raritan outcrop areas, there is not, and probably never was, sufficient hydraulic head available at the zone of recharge to drive the denser salt water out through the downdip part of the aquifer to the presumed outcrop zone on the edge of the continental shelf. Thus, pumping fresh water from the Raritan aquifer, at a rate in excess of the natural replenishment rate, could cause a migration of the salt water interface updip towards the HCGS production wells. This could eventually result in salt water intrusion into the HCGS production wells. Industrial development along the Delaware Valley is expected to cause increased demands upon the groundwater resources of the area, including the Raritan-Magothy aquifers. These added demands are expected to increase the potential for salt water migration toward the HCGS production wells. Sufficient data are not available to estimate when the salt water interface may reach the HCGS production wells.

2.4.13.3 Accidental Releases of Liquid Effluents in Ground and Surface Waters

2.4.13.3.1 Groundwater

There is no credible accident that releases radioactive liquid effluents to groundwater.

All tanks containing radioactive liquids within the power block are surrounded by dikes and/or drains connecting them with sumps that would automatically pump the contents to another tank. In addition, the radwaste tanks with the highest inventories are located on Elevations 54 and 77 ft. These locations are substantially below the groundwater table and would, therefore, experience groundwater inflow under postulated barrier accident conditions.

The only tank outside the power block containing radioactive liquids is the condensate storage tank (CST), which contains low level activity. It is surrounded by a 2-foot-thick reinforced concrete dike and foundation with waterstops at all construction joints. The dike is seismically designed (Category I) and would prevent leakage from the CST from reaching the groundwater.

2.4.13.3.2 Surface Water

There is no credible accident that releases radioactive liquid effluents to surface water.

All tanks containing radioactive liquids are surrounded by dikes to contain leakage and/or drains that would route any leakage to another tank and preclude transport to surface waters.

No analysis of a misaligned tank discharge is necessary. As discussed in Sections 11.2, 11.5, and 15.7.3, the radiation monitoring system would detect any radioactive discharge above certain limits and stop the discharge before 10CFR20 limits could be exceeded.

2.4.13.4 Monitoring or Safeguard Requirements

Groundwater is used as a permanent source of water for industrial, sanitary, potable, and fire protection purposes. No groundwater is used for safety-related purposes. Water is pumped from the Mt. Laurel-Wenonah and Raritan-Magothy aquifers, using wells ranging in depth from 300 to 1100 feet, see Section 2.4.13.1.3 for a more detailed description.

Except for the PSE&G water production wells at the site, there are no other known producing wells within 2 miles of the site. The nearest residences are 3 miles from the site. They are seasonal cottages and derive their water from shallow driven wells or carry it in along with other provisions; see Section 2.4.12.2 for additional detail.

Under plant operating conditions, groundwater in the shallow aquifer is expected to flow to the southwest and discharge directly into the Delaware River. Accidents effects are described in Section 2.4.13.3, and it concludes that concentrations are such that there is no dangerous exposure directly to humans, or indirectly through the food chain. We conclude that there is no need for plans, procedures, safeguards, and monitoring programs.

2.4.13.5 Design Bases for Subsurface Hydrostatic Loading

During the operating life of the plant, the natural groundwater table is expected to have a configuration that would be similar to that which it had in the natural state prior to the dewatering of the site. Prior to construction and dewatering of the site, the water table was a few feet below the land surface and within the hydraulic fill. During this pre-dewatering phase, water levels in the water table aquifer (upper aquifer) at the site were observed within the range of +86 to +96 PSE&G datum, with a hydraulic gradient of about 0.7 percent across the site toward the Delaware River.

Groundwater levels in the deeper aquifer (Vincentown aquifer) were observed to be within the range of +90.5 to +91.5 PSE&G datum, which figures are lower than the groundwater levels in the shallow aquifer at the time. The hydrologic regime, under normal conditions, is one of recharge through the land surface to the water table aquifer and leakage from the water table aquifer down to the underlying Vincentown aquifer. Structural fill emplaced around the various structures at the site is expected to form a permeable hydraulic conduit connecting the shallow aquifer with the Vincentown aquifer. Thus, when the excavation is completely backfilled and the dewatering system is shut off, water levels in the two aquifers are expected to return to normal. The new normal level is expected to be approximately the same for both aquifers under the site.

For purposes of foundation analysis, the maximum expected groundwater level at the site was conservatively estimated to be at Elevation +96 PSE&G datum. Due to the water table gradient, actual water elevations decrease toward the river. The assumption of a single design base across the site is thus slightly more conservative for the structures nearer the river.

As described in Section 2.4.13.1, the main power block area was dewatered prior to commencement of excavation, in order to facilitate construction and maintain the integrity of safety-related structures. By means of an extensive dewatering system, the water table aquifer (upper aquifer) was completely dewatered in the immediate vicinity of the excavation. The groundwater in the Vincentown aquifer, the second aquifer encountered below the land surface at the site, was drawn down to between Elevation +5 and +15 PSE&G datum. At all times, the water levels were maintained at least 3 feet below the base of the excavation, including test pits and test trenches dug in the base of the excavation. Upon completion of construction and backfilling in this area, dewatering will be discontinued and groundwater will be allowed to return to natural pre-existing levels described above.

When the site construction is completed, the natural groundwater recharge regime to the shallow aquifer will be somewhat altered from the natural pre-construction conditions. Areas which are paved or covered by structures will receive little or no recharge. Unpaved areas covered by permeable gravel or fill may receive additional recharge compared to pre-construction conditions. Because of these variable factors, a maximum expected water table elevation of +96 PSE&G datum was considered a reasonable and conservative design base for hydrostatic loading.

2.4.14 Technical Requirements Manual and Emergency Operation Requirements

The Technical Requirements Manual requires that watertight perimeter flood doors and hatches be closed in advance of a hydrological event that has a potential of producing water levels (including wave runup) above elevation 99.5 feet PSE&G datum at the Service Water Intake Structure.

The HCGS emergency plan will include emergency actions levels (EALs) related to the emergency plan classification system for hydrology-related events. Emergency instructions will provide operating personnel with detailed procedures to initiate actions necessary to cope with adverse hydrology related events.

Aspects of station design related to flood and low water conditions are discussed in Sections 2.4.10 and 2.4.11, respectively.

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TABLE 2.4-1

DRAINAGE AREAS AND GAUGED RIVER FLOW OF
STREAMS TRIBUTARY TO DELAWARE RIVER AND BAY[1][2]

River or Stream	Drainage Area, mi^2	Average Discharge ft^3/s	Average Discharge $ft^3/s/mi^2$
Delaware at Trenton	6,780	11,710	1.73
Crosswicks Creek	84	152	1.82
Neshminy	210	265	1.26
Rancocos, North Branch	111	162	1.46
Schuylkill - At Philadelphia	1,893	2,715	1.44
Chester Creek	61	78	1.27
Brandywine Creek	287	378	1.32
White Clay Creek	88	119	1.36
Maurice River	113	176	1.56
Total Gauged (69.25%)	9,627	15,755	1.64
Ungauged Area (30.75%)	4,273	7,010[3]	(1.64)
Total Drainage Area	13,900	22,765	(1.64)

(1) Source: Ketchum, 1953 (Reference 2.4-9)
(2) Drainage areas greater than 50 square miles.
(3) Ungauged area times 1.64 (average ft^3/mi^2).

TABLE 2.4-2

MAJOR EXISTING UPSTREAM SURFACE WATER IMPOUNDMENTS[1]

Impoundment	Owner	Purpose[2]	Total Storage, Acre-feet	Type of Construction	Dam Height, feet

Security Related Information
Text Withheld under 10 CFR 2.390

0

TABLE 2.4-2 (Cont)

Impoundment	Owner	Purpose(2)	Total Storage, Acre-feet	Type of Construction

Security Related Information
Text Withheld under 10 CFR 2.390

Y.

TABLE 2.4-3

MAJOR PROPOSED NEW OR MODIFIED UPSTREAM IMPOUNDMENTS[1]

Impoundment	Owner	Purpose[2]	Status[3]	Total Storage, acre-feet	Type of Construction	feet

Security Related Information
Text Withheld under 10 CFR 2.390

Revision 0
April 11, 1988

TABLE 2.4-3 (Cont)

(2) Purpose:
  a. WS = water supply
  b. FL = flood loss reduction
  c. REC = recreation
  d. PS = pumped storage (hydroelectric power)

(3) Status:
  a. To be constructed or modified
  b. Retained in Comprehensive Plan
  c. Retained in Comprehensive Plan for consideration after the year 2000

TABLE 2.4-4

PEAK DISCHARGE DATA FOR THE DELAWARE RIVER AT TRENTON, NEW JERSEY

Peak Discharges and Stages[1]

Date		Gauge Height, feet	Discharge, cfs
1904	October 11	20.7	295,000
1913	March 28	13.3	160,000
1925	February 13	13.0	154,000
1936	March 13	15.34	199,000
1936	March 19	16.66	227,000
1940	April 1	12.85	151,600
1942	May 24	13.35	161,200
1955	August 20	20.83	329,000

Peak Discharges at Selected Recurrence Intervals

Recurrence Interval, years	Peak Discharge, cfs
10	180,000
25	227,000
50	265,000
100	305,000

Location: Lat. 40°13'18", long. 74°46'38"

Drainage Area: 6,780 square miles

Gauge: Datum of gauge is 7.77 feet above 1929 mean sea level datum

Regulations:	Some effects on peak flood regulation from Lake Hopatcong, Lake Wallenpaupak, Toronto Reservoir, Swinging Bridge Reservoir, Cliff Lake Reservoir, Wild Creek Reservoir, Neversink Reservoir, Pepacton Reservoir, and other smaller reservoirs

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(1)Sources:

USGS 1981 (Reference 2.4-16)

Thomas, 1964 (Reference 2.4-5).

TABLE 2.4-5

TIDAL FLOODS ON DELAWARE RIVER

Station	Date	Tidal Flood Stage ft above MSL 1929 ADJ
Lewes, Del	March 1962	7.9
	November 1950	7.2
	August 1933	6.1
Cohansey River, N.J.	November 1950	8.6
	August 1933	7.0
Liston Front Light	November 1950	8.5
	August 1933	7.8
Reedy Point, Del	March 1962	7.5
	November 1950	8.6
	August 1933	8.0
New Castle, Del	November 1950	8.5
	August 1933	8.1
Philadelphia, Pa	August 1933	8.7

TABLE 2.4-6

POSTULATED FLOOD PRODUCING PHENOMENA

Phenomenon	Maximum Stillwater Level, feet, MSL	Maximum Wave Runup Elevation, feet, MSL

Security Related Information
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TABLE 2.4-6 (Cont)

Phenomenon	Maximum Stillwater Level, feet, MSL	Maximum Wave Runup Elevation, feet, MSL
e. Probable maximum tsunami with coincident 10 percent exceedance high tide	6.0	18.1
f. Ice jam flood	Negligible	

(1) 35.4/30.0 - Maximum wave runup elevation for power block structures along Fetch Number 1/Fetch Number 2.

TABLE 2.4-7

UNSTEADY FLOW ANALYSIS OF SINGLE DAM FAILURES

Tock's Island Dam	Francis E. Walter (Modified)

Security Related Information
Text Withheld under 10 CFR 2.390

TABLE 2.4-8

SECURITY-RELATED INFORMATION

WITHHELD UNDER 10 CFR 2.390

TABLE 2.4-9

UNSTEADY FLOW ANALYSIS OF MULTIPLE DAM FAILURES

	Cannonsville	Pepacton

Security Related Information
Text Withheld under 10 CFR 2.390

TABLE 2.4-10

PROBABLE MAXIMUM HURRICANE (PMH) DESIGN HIGH WATER LEVELS

AT POWERBLOCK

Still Water Level, ft PSE&G Datum	Fetch Number	Azimuth degree	Time Since Hurricane Makes Landfall hrs	Time Relative to Peak Surge hrs	Surge Level at the Site ft MLW	Fetch Length miles	Waves Approaching Artificial Island						
							Wind Speed mph	Average Water Depth ft	Cross Wind Setup ft	Significant Waves			Max Wave Height ft
										Height ft	Length ft	Period sec	
112.1	1	134	1.34	-0.6	25.7	7.3	108.6	42.2	0.0	12.21	163	5.9	20.4
113.8	2	149	1.94	0.0	27.4	7.7	113.3	42.2	0.0	12.92	164	5.9	21.6
112.1	3	164	2.50	0.56	25.7	8.1	112.2	40.9	0.0	12.69	162	5.9	21.2
109.4	4	179	2.90	0.96	23.0	6.1	108.6	40.8	0.1	11.73	153	5.7	19.6
107.2	5	194	3.34	1.40	20.8	4.8	106.6	38.5	0.3	10.34	143	5.5	17.3
104.7	6	209	3.67	1.73	18.3	3.4	106.0	40.3	0.3	9.53	141	5.4	15.9
103.4	7	224	4.03	2.09	17.9	2.38	100.0	37.3	0.4	9.03	117	4.9	15.1
103.0	8	239	4.39	2.45	16.6	2.36	105.0	36.0	0.4	8.91	116	4.9	14.9
102.4	9	254	4.75	2.81	16.0	2.43	104.5	35.5	0.4	8.93	115	4.8	14.9

TABLE 2.4-10

PROBABLE MAXIMUM HURRICANE (PMH) DESIGN HIGH WATER LEVELS
AT POWERBLOCK

Still Water Level, ft PSE&G Datum	Fetch Number	Azimuth degree	Incident Waves Toward Plant Buildings (1)			Water Rise From Wave Effect ft	Total Water Level, ft PSE&G Datem
			Max Wave Height ft	Length ft	Period sec		
112.1	1	134	8.3	163	5.9	11.7	123.8
113.8	2	149	4.5	38	2.8	6.2	120.0
112.1	3	164	4.2	36	2.7	5.8	118.0
109.4	4	179	6.2	153	5.7	8.7	118.2
107.2	5	194	4.7	143	5.5	6.6	114.1
104.7	6	209	2.7	141	5.4	3.8	108.8
103.4	7	224	2.5	117	4.9	3.5	108.2
103.0	8	239	1.5	116	4.9	2.1	105.5
102.4	9	254	1.0	115	4.8	1.4	104.5

(1) Mean low water level is at 86.4 feet PSE&G datum.
Existing plant grade is at 101.5 feet PSE&G datum.

TABLE 2.4-10a

(PMH) NON-BREAKING WAVE RUNUP ON THE VERTICAL WALL OF THE INTAKE STRUCTURE FACING THE DELAWARE RIVER

Fetch	Azimuth Degree	Stillwater Level @ Site PSE&G (1)	Fetch Length (Miles)	Wind Speed (mph)	Average Depth (ft)	Surge Level @ Site (MLW)	Incident Wave Conditions(4) Significant Waves Height (ft)	Length (ft)	Period (sec)	Max Wave Height (ft)	Viscous Damping Factor	Attenuated Wave Conditions(2) Maximum Wave Height (ft)	Length (ft)	Period (sec)	Elevation of Wave runup(3) (PSE&G)
1	134	112.1	7.3	108.6	42.2	25.7	10.5	180	6.4	15.8	-	Incident waves are sheltered by the shoreline			-
2	149	113.8	7.7	113.3	42.2	27.4	10.9	186	6.6	16.4	0.94				-
3	164	112.1	8.1	112.2	40.9	25.7	10.8	183	6.6	16.2	0.95	15.4	111	4.7	134.4(5)
4	179	109.5	6.1	108.6	40.8	23.0	9.9	170	6.2	14.9	0.97	14.5	106	4.5	130.1(5)
5	194	107.5	4.8	106.6	38.5	20.8	9.1	155	5.9	13.7	0.98	13.4	95	4.3	126.8
6	209	105.0	3.4	106.0	40.3	18.3	8.4	144	5.5	12.6	0.99	12.5	88	4.2	123.2
7	224	104.7	2.38	106.0	37.3	17.9	7.4	125	5.1	11.1	-	11.1	78	3.9	120.9
8	239	103.4	2.36	105.0	36.0	16.6	7.3	123	5.1	11.0	-	11.0	78	3.9	119.3
9	254	102.8	2.43	104.5	35.5	16.0	7.3	123	5.1	11.0	-	11.0	Not computed		

(1) Mean Low Water is at Elevation 86.4 feet PSE&G Datum.
(2) The attenuated wave conditions listed are for the limiting steepest waves.
(3) The wave runup elevations listed assume that the vertical wall extends to the highest elevation.
(4) These values and subsequent calculations are based on the U.S. Army Corps of Engineers, Shore Protection Manual, U.S. Army Coastal Engineering Research Center, 3 Volumes 1977.
(5) Waves would overtop the vertical wall at Elevation 128.0.

TABLE 2.4-11

OCCURRENCE OF ATLANTIC TSUNAMIS[1]

Dates	W.Indies	W.Europe	Azores	Other	Total
Before 1500		2			2
1500 to 1800	6	4	3		13
Since 1800	10	1	1	3	15
Total	16	7	4	3	30

(1) Source: N.H. Heck, 19847 (Reference 2.4-45).

TABLE 2.4-11a

SUMMARY OF WAVE LOADING CONDITIONS

(A) WAVE LOADING ON VERTICAL WALLS OF THE INTAKE STRUCTURE

Section	Criterion	Critical Fetch Direction	Critical Wave Conditions Wave Height (ft)	Length (ft)	Period (sec)	Wave Loading per Linear feet of Wall Static (kip/ft)	Dynamic (kip/ft)	Total (kip/ft)	Stillwater Level @ Site (PSE&G)	Height of Wave Runup (PSE&G)
West Wall	non breaking	4	14.5	228	7.4	53.8	27.6	81.4	109.5	127.6
South Wall	breaking(1)	2	15.4	111	4.7	13.5	77.0	90.5	113.8	Not computed
North Wall	breaking(1)	8(2)	11.0	78	3.9	2.0	8.6	10.6	103.4	Not computed

(B) WAVE LOADING ON VERTICAL WALLS OF THE POWER BLOCK

Criterion	Critical Fetch Direction	Critical Wave Conditions Wave Height (ft)	Length (ft)	Period (sec)	Wave Loading per Linear feet of Wall Static (kip/ft)	Dynamic (kip/ft)	Total (K/ft)
Breaking	1	8.3	114	4.8	7.0	39.8	46.8

(1) The Breaking wave loadings estimated for the south and north walls assume perfect breaking wave conditions occur.
(2) Fetch No. 8 is uses as the limiting case for the north wall.
(3) The wave loadings estimated for the power block can be conservatively applied to the east facing vertical wall of the SSWS.

TABLE 2.4-12

ATLANTIC TSUNAMIS OCCURRING BETWEEN 1891 AND 1961[1]

Date	Source	Remarks
Nov 29, 1897	West Indies	Large tsunami at Montserrat
Jan 14, 1907	Jamaica	Tsunami generated, main damage at Kingston
Oct 11, 1918	Puerto Rico	Tsunami caused fatalities and damage at Point Boringquen and Aguadilla; damage also caused at Mayaguez.
Jan 17, 1929	Cumana, Venezuela	Many boats wrecked
Nov 18, 1929	Grand Banks	Tsunami hit Newfoundland.
Dec 5, 1941	Panama-Costa Rica	Slight tsunami at Costa Rica; height of 0.75 feet
Aug 1946	Dominican Republic Republic	Town of Matanzas badly damaged and abandoned; more than 100 persons killed; minor damage on coast of Haiti
May 31, 1953	Near Dominican Republic	Very slight tsunami

(1) Source: F.E. Camfield, 1980 (Reference 2.4-40).

TABLE 2.4-13

SUMMARY OF RESULTS FROM AQUIFER TESTS CONDUCTED AT HCGS

Aquifer	Method	Coefficient of Transmissibility T, gpd/ft U[1]	A[2]	Storage Coefficient S U	A	Vertical Permeability of Confining Bed P, gpd/ft^2 U	A	Permeability K, gpd/ft^2 U	A	Remarks
Shallow										
Modified Theis semilog plot s versus r	t = 120 min	2640	2640	5.5×10^{-5}	5.5×10^{-5}	-	-	528	528	
	t = 180 min	2640	2640	2.9×10^{-5}	2.9×10^{-5}	-	-	528	528	
Theis nonleaky artesian aquifer type curve s versus r^2 on log-log paper	t = 120 min	2620	2620	6.0×10^{-5}	6.0×10^{-5}	-	-	524	524	
	t = 180 min	2700	2700	1.8×10^{-5}	1.8×10^{-5}	-	-	540	540	
Jacob steady-state leaky artesian type curve s versus r^2 on log-log paper	t = 120 min	2620	2620	-	-	0.085	0.085	524	524	Confining bed is Hydraulic Fill
	t = 180 min	2780	2780	-	-	0.023	0.023	556	556	
Deep										
Modified Theis semilog plot s versus r	t = 235 min	10,400	-	1.1×10^{-4}	-	-	-	130	-	
	t = 1080 min	10,400	-	2.6×10^{-4}	-	-	-	130	-	
	t = 1380 min	8000	9100	1.0×10^{-3}	6.9×10^{-4}	-	-	100	114	
Theis nonleaky artesian aquifer type curve s versus r^2 on log-log paper	t = 235 min	9880	-	1.3×10^{-4}	-	-	-	124	-	
	t = 1080 min	9550	-	3.6×10^{-4}	-	-	-	119	-	
	t = 1380 min	11,460	11,860	2.4×10^{-4}	2.8×10^{-4}	-	-	143	148	
Jacob steady-state leaky artesian type curve s versus r^2 on log-log paper	t = 235 min	9540	-	-	-	0.205	-	119	-	Confining bed is Kirkwood Clay
	t = 1080 min	9870	-	-	-	0.098	-	123	-	
	t = 1380 min	11,450	11,010	-	-	0.030	0.041	143	138	

(1) U = Using unadjusted drawdowns. No correction for barometric and tidal effects.
(2) A = Using adjusted drawdowns. Corrected for barometric and tidal effects.

TABLE 2.4-14
CHEMICAL ANALYSIS OF WATER SAMPLES[1]

Formation	Wells	Date, 1974	Analyzed By[2]	Remarks[3]	Alkalinity: Carbonates	Alkalinity: Bicarbonates	Anions, mg/l: Chloride	Anions, mg/l: Sulphate	Anions, mg/l: Nitrite	Anions, mg/l: Nitrate
Quaternary river bed (shallow)	302	3-22	D&M	- N	0	363	2700	215	0.001	5.5
	302	3-23	D&M	BP N	0	401	2670	215	0.001	0.036
	302	3-29	D&M	AP	0	273	2580	255	0.001	0.076
	312	3-23	D&M	BP N	0	142	1940	20	0.001	0.064
	312	3-26	D&M	AP	0	914	1930	4.5	0.44	0.015
	313	3-24	D&M	BP M	0	1050	2070	129	0.051	0.011
	313	3-29	D&M	AP	0	227	2000	123	0.001	0.012
	323	3-24	D&M	BP M	0	1050	1120	1080	0.36	0.01
	323	3-26	D&M	AP	0	1110	987	1070	0.14	0.009
	502	3-25	D&M	BP N	0	506	2750	114	0.001	0.11
	502	3-26	D&M	I M	0	1080	2930	41	0.51	0.025
	502	4-5	FW	AP	0	1290	2528	156	0	-
Vincentown	300	4-1	FW	I N		-	3610	-	-	-
	300	4-5	FW	AP	0	170	3820	63	0	-
	301	3-28	D&M	BP N	0	0	4640	198	0.012	0
	301	4-1	FW	I N		-	5100	-	-	-
	301	4-5	FW	AP	0	230	4660	135	0	-
	311	3-26	D&M	BP N	0	400	3780	149	0.001	0.025
	311	3-26	D&M	BP	0	474	3390	119	0.44	0.008
	311	4-5	FW	AP	0	300	2790	80	0	-
	314	3-27	D&M	BP M	0	556	3930	153	0.3	0
	314	4-5	FW	AP	0	590	4120	111	0	-
	321	3-14	D&M	BP	0	80.5	3939	266	0.011	0.035
	322	3-21	D&M	BP N	0	108	3660	137	0.001	0.018
	322	3-26	D&M	I	0	324	3320	110	0.51	0.008
	322	4-5	FW	AP	0	340	3140	91	0	-
	501	3-28	D&M	BP N	0	0	4370	135	0.007	0
	501	4-4	FW	AP	0	460	4030	106	0	-
	Dewatering well SNGS	3-19	D&M	BP	0	281	3660	329	0.053	0.037
Mount Laurel and Wenonah	401	3-19	D&M	BP C?	0	79	903	2000	0.26	0.19
	401	3-26	D&M	BP C?M	0	246	2690	35	0.11	0.001
	401	4-5	FW	AP	0	340	5970	36	0	-
	402	4-1	FW	BP C?N	-	-	1980	-	-	-
	402	4-5	FW	AP	0	470	4750	212	0	-
	404	3-28	D&M	BP M	0	382	7060	185	0.44	0.008
	404	4-5	FW	AP	0	430	4270	156	0	-
	C	4-3	FW	I	0	240	160	<10	10	-
	D	4-3	FW	I	0	160	178	<10	5	-
	PW-1	3-22	D&M	I N	0	0	38	25	0.001	0
	PW-2	3-22	D&M	I N	0	133	60	0	0.52	0
	PW-3	3-22	D&M	I N	0	0	29	16	0.001	0.01
	PW-4	3-28	D&M	BP M	0	161	739	25.1	0.031	0.008
	PW-4	4-5	FW	AP	-	160	624	14	0	-

		Cations, mg/l					
Formation	Wells	Sodium	Potassium	Calcium	Magnesium	Total Iron	Manganese
Quaternary river bed (shallow)	302	1340	58	100	380	24	3.5
	302	1340	50	100	360	13	3.5
	302	1600	50	160	370	41	3.5
	312	1080	42	160	285	123	3.0
	312	1080	42	100	285	46	2.0
	313	920	50	160	320	70	3.0
	313	1080	42	160	310	54	3.5
	323	520	42	300	320	220	11
	323	520	33	300	285	55	11
	502	1340	67	100	370	89	3.0
	502	1340	67	100	330	70	0.25
	502	1313	85.0	158	349	5.58	-
Vincentown	300	-	-	-	-	82.80	-
	300	1525	37.5	569	117	18.95	-
	301	1740	17	1410	140	48	1.5
	301	-	-	-	-	47.10	-
	301	1550	26.5	1250	82	4.49	-
	311	1600	67	660	360	267	0.25
	311	1480	67	340	330	53	0.2
	311	1210	62.3	148	216	3.28	-
	314	2000	25	200	310	48	0.5
	314	1950	55.0	228	285	1.22	-
	321	1410	37	1000	45	2.1	0.30
	322	1740	8	40	10	140	0.5
	322	1350	15	420	200	112	0.5
	322	1250	24.3	399	168	2.19	-
	501	2140	16	500	190	18	0.5
	501	1650	29.5	466	18	0.85	-
	Dewatering well SNGS	1880	42	340	200	4	0.5
Mount Laurel and Wenonah	401	520	16	410	285	40	32
	401	920	33	480	90	18	0.5
	401	2200	66.5	1139	103	1.38	-
	402	-	-	-	-	2.19	-
	402	1720	57.5	619	213	1.55	-
	404	3888	50	660	54	195	0.10
	404	1718	65.0	661	99	3.18	-
	C	122	17.2	37	9	0.77	-
	D	108	15.0	33	19	0.20	-
	PW-1	29	4.0	100	4.5	2	0.05
	PW-2	40	3.6	150	5.0	3	0.05
	PW-3	40	3.2	135	8.6	3	0.05
	PW-4	120	25	200	52	4	0.5
	PW-4	128	31.5	201	65	0.58	-

TABLE 2.4-14 (cont)
Other Parameters

Formation	Wells	Silica	Specific Conductance	pH	TDS	Turbidity[4]	Total Hardness as $CaCO_3$	Carbonate Hardness	Color[5]
Quaternary river bed (shallow)	302	56	11,000	6.1	6722	26	1520	396	-
	302	56	10,800	6.1	6562	125	1520	440	-
	302	57	10,900	5.6	6890	46	1650	311	-
	312	56	8,400	5.7	5314	22	1340	180	-
	312	51	7,800	6.8	4330	26	1250	1270	-
	313	52	8,300	6.7	4550	29	1380	112	-
	313	56	8,400	5.6	5174	44	1420	356	-
	323	41	6,400	6.9	4030	68	1750	113	-
	323	47	6,300	6.8	3770	26	1740	1216	-
	502	29	11,100	6.1	6412	55	1490	542	-
	502	17	11,000	6.8	6106	23	1350	1160	-
	502	-	>8,500	7.0	6375	1050	1850	-	HO
Vincentown	300	-	>10,000	2.3	>7500	-	-	-	-
	300	-	>10,000	6.5	>7500	720	1890	-	MO
	301	47	19,200	2.1	6110	7.9	3260	0	-
	301	-	>10,000	2.4	>7500	-	-	-	-
	301	-	>10,000	6.7	>7500	500	3490	-	MO
	311	32	13,800	6.2	9096	87	2720	454	-
	311	19	11,800	6.9	6760	12	1700	506	-
	311	-	8,000	7.2	6000	620	1280	-	MO
	314	31	12,500	7.5	7436	6.5	1600	620	-
	314	-	>10,000	7.0	>7500	150	1760	-	MO
	321	24	11,400	7.3	7648	200	2530	99	-
	322	31	13,800	5.7	8820	44	2560	131	-
	322	33	10,900	6.7	6610	26	1660	354	-
	322	-	10,000	6.8	7500	380	1700	-	MO
	501	42	15,700	2.5	9068	1.4	1910	0	-
	501	-	>10,000	6.8	>7500	147	1840	-	LYO
	Dewatering well SNGS	40	11,800	6.8	7510	45	1580	391	-
Mount Laurel and Wenonah	401	25	5,900	6.6	4350	6.9	2220	100	-
	401	18	9,000	7.3	5610	65	1660	258	-
	401	-	>10,000	6.7	>7500	160	3280	-	MO
	402	-	>10,000	1.8	>7500	-	-	-	-
	402	-	>10,000	6.8	>7500	273	2440	-	MG
	404	26	12,000	7.2	7780	18	1840	478.0	-
	404	-	>10,000	6.9	>7500	168	2070	-	MBG
	C	-	925	7.3	694	125	200	-	LO
	D	-	810	7.6	608	49	164	-	LY
	PW-1	15	6,500	2.3	370	0.14	328	0	-
	PW-2	15	7,200	6.5	404	2.8	298	126	-
	PW-3	14	6,800	2.1	316	0.24	217	0	-
	PW-4	15	2,800	7.1	1916	15	795	172	-
	PW-4	-	2,100	7.8	1575	140	770	-	LG

(1) Reference 2.4-54

(2) Laboratory:
FW = Feedwaters Inc
D&M = Dames & Moore

(3) BP = Before pumping
AP = After pumping
I = Intermediate
C = Contaminated
N = Sample container pretreated with nitric acid to stabilize metals in solution
M = Sample container pretreated with mercuric chloride to stabilize nitrogen in solution

(4) Turbidity units:
FW laboratory - JTU (Jackson turbidity units)
D&M laboratory - FTU (Formazin turbidity units)
JTU is equivalent to FTU.

(5) Color:
HO = heavy orange
MO = moderate orange
MG = moderate green
MBG = moderate blue-gray
LO = light orange
LY = light yellow
LG = light gray

TABLE 2.4-15

LABORATORY PERMEABILITY TEST DATA

Soil Well/Boring Number	Depth, Feet	Formation	Soil Type	Permeability gpd/ft^2	Permeability cm
319/254	150	Hornerstown	SM	$4.0x10^{-2}$	$1.9x10^{-6}$
401/232	150	Hornerstown	SM	$1.3x10^{-1}$	$6.1x10^{-6}$
319/254	160	Navesink	SM	$1.5x10^{-1}$	$7.1x10^{-6}$
401/232	160	Navesink	SM	$1.7x10^{-1}$	$8.0x10^{-6}$
319/254	170	(Top) Mt Laurel	SM	$8.0x10^{-1}$	$3.8x10^{-5}$

TABLE 2.4-16

PUBLIC WATER SUPPLIES IN VICINITY OF THE SITE[1]

Distance miles	Town	Population Served	Average Output mgd	Source of Water
9	Salem, New Jersey	9,000	1.7	About 2/3 of water consumed is surface water, pumped from the Quinton pumping station about 3 miles east of town and 9 miles northeast of the site. Remainder is obtained from four wells, ranging in depth from 80 to 168 feet, located east of Salem.
14	Pennsville, New Jersey	10,500		Four wells ranging in depth from 105 to 240 feet. The wells are probably completed in the Magothy Formation.
17	Penns Grove, New Jersey	8,000		Two wells, 292 and 360 feet deep. The water probably comes from the Potomac Group.

Distance miles	Town	Population Served	Average Output mgd	Source of Water
17	Woodstown, New Jersey	3,000		Eight wells; six are about 100 feet deep and the others are about 300 and 350 feet deep.
22	Elmer, New Jersey	2,500		Three wells; two are 80 feet deep and the third is 500 feet deep. The shallow wells probably tap the Mount Laurel-Wenonah Formation.
16	Bridgeton, New Jersey	22,000		A total of 12 wells, some of which are no longer in use, range in depth from 75 feet to 129 feet. They are completed in the Chansey sand.
11	Smyrna, Delaware		0.27	Two wells, 20 feet and 95 feet deep supply the town. The shallower wells is used for stand-by purposes.

TABLE 2.4-16 (Cont)

Distance miles	Town	Population Served	Average Output mgd	Source of Water
13	Clayton, Delaware	825	1.2	One well, 272 feet deep, is the source of water supply.
10	Middletown, Delaware	2,000	0.2	Three wells, having depths of 100 feet, 200 feet and 500 feet, supply the town.
9	Delaware City, Delaware	1,500		Two wells, one 26 feet in the Wenonah Formation and the other in the Magothy Formation, supply the town.
14	New Castle, Delaware			The town obtains water from a shallow infiltration gallery system located in Pleistocene deposits.

(1) Locations are shown on a map in Figure 2.4-28

TABLE 2.4-17

SECURITY-RELATED INFORMATION
WITHHELD UNDER 10 CFR 2.390

SECURITY-RELATED INFORMATION
WITHHELD UNDER 10 CFR 2.390

SECURITY-RELATED INFORMATION
WITHHELD UNDER 10 CFR 2.390

TABLE 2.4-18

COEFFICIENTS OF PERMEABILITY, TRANSMISSIBILITY, AND STORAGE IN THE RARITAN FORMATION

Owner and Location	County	Range in Coefficient of Transmissibility (gpd/ft)	Range in Aquifer Thickness (ft)	Range in Coefficient of Permeability (gpd/ft)	Range in Coefficient of Storage (dimensionless)	Source of Data
Air Reduction Co, Riverton, NJ	Burlington	150,000	100[(1)]	1,500	$1.5x10^{-4}$	U.S.G.S. unpublished data
California Oil Co, Barber, NJ	Middlesex	4000 to 14000	10 to 21	240 to 660	$8.1x10^{-2}$ to $4.0x10^{-5}$	U.S.G.S. unpublished data
Camden Water Dept, Camden, NJ	Camden	23000 to 79000	19 to 46	680 to 2500	$1.7x10^{-4}$ to $5.6x10^{-4}$	U.S.G.S. and Leggette and Brashears unpublished data
E.I. duPont deNemours, Gibbstown, NJ	Gloucester	47,000	25$\pm$	1480	$1.5x10^{-4}$	U.S.G.S. unpublished data
N.J. Water Co, Haddon Hts, NJ	Camden	124,000	70$\pm$	1800	$1.0x10^{-3}$	U.S.G.S. unpublished data
N.J. Water Co, Stockton Station, Camden, NJ	Camden	53000 to 64000	45 to 50	1060 to 1400	$7.2x10^{-5}$ to $8.6x10^{-5}$	U.S.G.S. unpublished data
N.Y. Shipbuilding Corp, Camden NJ	Camden	62000	24[(1)]	2600[(1)]	$1.2x10^{-8}$	U.S.G.S. unpublished data
E.I. duPont deNemours, Parlin, NJ	Middlesex	50000 to 76000	85	590 and 890	$3.7x10^{-6}$ and $8.6x10^{-5}$	U.S.G.S. unpublished data
Hercules Powder Co, Parlin, NJ	Middlesex	100,000	66	1500	$1.55x10^{-3}$	U.S.G.S. unpublished data
Perth Amboy Water Dept, Old Bridge, NJ	Middlesex	17000 to 67000			$2.4x10^{-3}$ to $5.8x10^{-4}$	U.S.G.S. unpublished data
Texax Co, Westville, NJ	Gloucester	51000 to 68000	40 to 67	1020 to 1400	$1.7x10^{-4}$ to $9.0x10^{-5}$	U.S.G.S. unpublished data
U.S. Navy Yard, Philadelphia, PA	Philadelphia	51000 to 69000	54 to 63	920 to 1200	$2.0x10^{-4}$ to $8.0x10^{-5}$	U.S.G.S. open-file memorandum by Graham and Kammerer

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(1) Aquifer probably not fully penetrated (Reference 2.4-3).

TABLE 2.4-19

SUMMARY OF WATER ANALYSES OF SALEM GENERATING STATION WELLS

Date	pH	Cond(1)	Cl^{-}(2)	F^{-}(2)	S^{-2} (2)	NO_3(3)	Turbidity(3)	TDS(2)
Well 1								
8-6-73	7.20	675	45.8	0.12	-	0.15	2.5 FTU	269
10-23-73	7.95	471	42.5	-	-	-	-	251
10-31-73	7.96	476	41	0.02	-	-	-	201
11-16-73	7.80	386	26.7	0.18	0.1	0.04	12 FTU	170
11-23-73	7.82	428	28.2	0.24	-	-	-	210
12-6-73	7.84	438	26.3	-	-	-	-	254
1-25-74	-	-	34.9	-	0.1	0	10 FTU	-
Well 2								
8-6-73	7.40	590	59.9	0.12	0	0.1	4 FTU	298
9-10-73	7.70	492	40.8	0.16	0.1	0.02	7 FTU	235
10-22-73	7.50	420	42.8	0.18	0	-	10 FTU	244
11-12-73	7.88	374	47.4	0.12	-	-	-	239
12-6-73	7.85	496	42.5	0.18	0.1	0.005	8 FTU	238
1-21-74	-	-	37.1	-	-	-	-	-
Well 3								
8-6-73	7.80	469	27.5	0.15	0.1	0.01	8 FTU	373
10-22-73	7.90	425	25.1	0.18	0	-	18 FTU	205
12-6-73	-	-	22.5	-	-	-	-	-
1-21-74	7.90	426	34.9	0.14	0.1	0	10 FTU	240

(1) Conductivity is μmhos.
(2) Cl^{-}, F^{-}, S^{-2}, NO_3, TDS are in ppm.
(3) Turbidity is in Formazin turbidity units (FTU).

TABLE 2.4-20

WATER ANALYSIS WELL 1[1]

Constituents		PPM as	7-11-77	10-20-77	1-16-78	2-22-78	3-2-78	7-17-78
			Date Sampled					
Cations								
Calcium	(Ca^{++})	$CaCO_2$	85.0	90.0	89.0	92.0	88.0	90.0
Magnesium	(Mg^{++})	$CaCO_2$	69.0	71.0	68.0	68.0	72.0	74.0
Sodium	(Na^{+})	$CaCO_2$	60.0	59.0	64.0	56.0	59.0	63.0
Potassium	(K^{+})	$CaCO_2$	26.0	17.0	16.0	16.0	16.0	16.0
Total Cations		$CaCO_2$	240.0	237.0	237.0	232.0	235.0	243.0
Anions								
Bicarbonate	(HCO_3^{-})	$CaCO_2$	175.0	173.0	168.0	164.0	169.0	164.0
Carbonate	(CO_3^{--})	$CaCO_2$	0.0	0.0	0.0	0.0	0.0	0.0
Hydroxide	(OH^{-})	$CaCO_2$	0.0	0.0	0.0	0.0	0.0	0.0
Sulfate	(SO_4^{--})	$CaCO_2$	5.0	2.0	0.0	0.0	0.0	0.0
Chloride	(Cl^{-})	$CaCO_2$	56.0	61.0	65.0	64.0	65.0	73.0
Phosphate (Total)	(PO_4^{--})	$CaCO_2$	0.8	0.93	1.53	1.3	0.8	0.17
Nitrate	(NO_3^{-})	$CaCO_2$	3.3	0.18	2.8	3.0	0.1	5.8
Total Anions			240.0	237.0	237.0	232.0	235.0	243.0
Total hardness		$CaCO_2$	154.0	161.0	157.0	160.0	160.0	164.0
Methyl orange alkalinity		$CaCO_2$	175.0	173.0	168.0	164.0	169.0	164.0
Phenolphthalein alkalinity		$CaCO_2$	0.0	0.0	0.0	0.0	0.0	0.0
Carbon dioxide, free		CO_2	4.0	7.0	2.0	2.0	5.0	4.0
pH		-	7.8	7.7	8.17	8.0	7.7	7.75
Iron, total		Fe	0.37	0.43	0.28	0.4	0.47	0.51
Manganese		Mn	0.0	0.0	0.0	0.0	0.0	0.0
Silica		SiO_2	13.5	13.9	12.7	13.3	13.1	13.5
Turbidity after shaking			1.3	1.0	0.9	0.7	3.0	2.0
Sediment, color			None	None	None	None	None	None
Sediment, nature			None	None	None	None	None	None
Color			5.0	5.0	5.0	5.0	5.0	5.0
Odor			None	None	Musty	None	None	None
Conductivity, μmhos			430.0			445.0	460.0	450.0

TABLE 2.4-20 (cont)

Constituents		PPM as	10-29-79	10-15-80	1-15-81	4-15-81	10-15-81
			Date Sampled				
Cations							
Calcium	(Ca^{++})	$CaCO_2$	106.0	118.0	108.0	32.0	118.0
Magnesium	(Mg^{++})	$CaCO_2$	81.0	86.0	100.0	34.0	94.0
Sodium	(Na^{+})	$CaCO_2$	80.0[2]	80.0[2]	81.0[2]	48.0[2]	91.0[2]
Potassium	(K^{+})	$CaCO_2$					
Total Cations		$CaCO_2$	267.0	284.0	289.0	114.0	203.0
Anions							
Bicarbonate	(HCO_3^{-})	$CaCO_2$	162.0	160.0	161.0	48.0	163.0
Carbonate	(CO_3^{--})	$CaCO_2$	0.0	0.0	0.0	0.0	0.0
Hydroxide	(OH^{-})	$CaCO_2$	0.0	0.0	0.0	0.0	0.0
Sulfate	(SO_4^{--})	$CaCO_2$	0.0	4.0	5.0	24.0	4.0
Chloride	(Cl^{-})	$CaCO_2$	102.0	117.0	120.0	40.0	134.0
Phosphate (Total)	(PO_4^{--})	$CaCO_2$	0.15	0.15	0.05	0.08	0.12
Nitrate	(NO_3^{-})	$CaCO_2$	3.0	3.2	2.5	1.6	1.8
Total Anions			267.0	284.0	289.0	114.0	303.0
Total hardness		$CaCO_2$	187.0	204.0	208.0	66.0	212.0
Methyl orange alkalinity		$CaCO_2$	162.0	160.0	161.0	48.0	163.0
Phenolphthalein alkalinity		$CaCO_2$	0.0	0.0	0.0	0.0	0.0
Carbon dioxide, free		CO_2	5.0	3.0	4.0	4.0	12.0
pH		-	7.65	7.7	7.75	7.4	7.4
Iron, total		Fe	0.2	0.28	0.03	3.24	0.32
Manganese		Mn	0.0	0.0	0.0	0.075	0.02
Silica		SiO_2	12.7	12.5	12.7	2.7	12.5
Turbidity after shaking			0.7	1.3	0.7	(3)	(3)
Sediment, color			None	None	None	Rust	None
Sediment, nature			None	None	None	Iron	None
Color			5.0	5.0	5.0	5.0	7.0
Odor			None	-	None	None	None
Conductivity, µmhos				490.0	505.0	237.0	530.0

(1) Hardness in grains per U.S. gal. as $CaCO_3$.

(2) Includes any potassium.

(3) Instrument out of order.

TABLE 2.4-21

WATER ANALYSIS WELL 2(1)

Constituents		PPM as	Date Sampled 7-11-77	10-20-77	1-16-78	2-22-78	3-2-78	7-17-78
Cations								
Calcium	(Ca^{++})	$CaCO_2$	135.0	119.0	136.0	151.0	150.0	160.0
Magnesium	(Mg^{++})	$CaCO_2$	113.0	88.0	100.0	113.0	116.0	123.0
Sodium	(Na^{+})	$CaCO_2$	87.0	100.0	95.0	95.0	95.0	101.0
Potassium	(K^{+})	$CaCO_2$	21.0	18.0	21.0	20.0	20.0	20.0
Total Cations		$CaCO_2$	356.0	325.0	352.0	379.0	381.0	404.0
Anions								
Bicarbonate	(HCO_3^{-})	$CaCO_2$	167.0	172.0	166.0	160.0	165.0	159.0
Carbonate	(CO_3^{--})	$CaCO_2$	0.0	0.0	0.0	0.0	0.0	0.0
Hydroxide	(OH^{-})	$CaCO_2$	0.0	0.0	0.0	0.0	0.0	0.0
Sulfate	(SO_4^{--})	$CaCO_2$	5.0	2.0	0.0	0.0	0.0	0.0
Chloride	(Cl^{-})	$CaCO_2$	180.0	146.0	184.0	218.0	215.0	241.0
Phosphate (Total)	(PO_4^{--})	$CaCO_2$	0.7	1.07	1.53	0.7	1.1	0.6
Nitrate	(NO_3^{-})	$CaCO_2$	3.5	3.7	0.0	0.5	0.2	3.7
Total Anions			356.9	325.0	352.0	379.0	381.0	404.0
Total hardness		$CaCO_2$	248.0	207.0	236.0	264.0	266.0	283.0
Methyl orange alkalinity		$CaCO_2$	167.0	172.0	166.0	160.0	165.0	159.0
Phenolphthalein alkalinity		$CaCO_2$	0.0	0.0	0.0	0.0	0.0	0.0
Carbon dioxide, free		CO_2	4.0	7.0	15.0	14.0	6.0	5.0
pH		-	7.7	7.7	7.2	7.25	7.6	7.7
Iron, total		Fe	0.60	0.63	0.04	0.01	0.9	0.72
Manganese		Mn	0.0	0.0	0.0	0.0	0.0	0.0
Silica		SiO_2	13.0	13.1	13.1	12.7	13.1	12.5
Turbidity after shaking			1.2	3.8	0.9	0.3	6.0	3.4
Sediment, color			None	None	Dark	None	None	None
Sediment, nature			None	None	?	None	None	None
Color			5.0	5.0	5.0	7.0	5.0	5.0
Odor			None	None	None	None	None	None
Conductivity, µmhos			670.0			750.0	770.0	760.0

TABLE 2.4-21 (cont)

Constituents		PPM as	10-29-79	7-14-80	10-15-80	1-15-81	4-15-81	10-15-81
			Date Sampled					
Cations								
Calcium	(Ca^{++})	$CaCO_2$	185.0	182.0	190.0	192.0	210.0	208.0
Magnesium	(Mg^{++})	$CaCO_2$	45.0	142.0	146.0	146.0	150.0	157.0
Sodium	(Na^{+})	$CaCO_2$	223.0(2)	128.0(2)	131.0(2)	132.0(2)	130.0(2)	153.0(2)
Potassium	(K^{+})	$CaCO_2$						
Total Cations		$CaCO_2$	453.0	452.0	467.0	470.0	490.0	518.0
Anions								
Bicarbonate	(HCO_3^{-})	$CaCO_2$	162.0	167.0	160.0	150.0	160.0	159.0
Carbonate	(CO_3^{--})	$CaCO_2$	0.0	0.0	0.0	0.0	0.0	0.0
Hydroxide	(OH^{-})	$CaCO_2$	0.0	0.0	0.0	0.0	0.0	0.0
Sulfate	(SO_4^{--})	$CaCO_2$	0.0	4.0	6.0	6.0	7.0	5.0
Chloride	(Cl^{-})	$CaCO_2$	282.0	277.0	297.0	310.0	320.0	353.0
Phosphate (Total)	(PO_4^{--})	$CaCO_2$	0.17	0.1	0.08	0.25	0.08	0.12
Nitrate	(NO_3^{-})	$CaCO_2$	3.0	3.8	3.8	4.0	2.9	1.3
Total Anions			453.0	452.0	467.0	470.0	490.0	518.0
Total hardness		$CaCO_2$	230.0	324.0	336.0	338.0	360.0	365.0
Methyl orange alkalinity		$CaCO_2$	162.0	167.0	160.0	150.0	160.0	159.0
Phenolphthalein alkalinity		$CaCO_2$	0.0	0.0	0.0	0.0	0.0	0.0
Carbon dioxide, free		CO_2	5.0	7.0	4.0	7.0	5.0	12.0
pH		-	7.6	7.6	7.6	7.65	7.85	7.4
Iron, total		Fe	0.47	0.51	0.72	0.26	0.08	0.47
Manganese		Mn	0.0	0.01	0.0	0.0	0.0	0.02
Silica		SiO_2	12.7	10.9	12.7	12.0	12.9	12.5
Turbidity after shaking			2.4	3.8	6.0	3.2	(3)	(3)
Sediment, color			Rust	None	Rust	None	None	None
Sediment, nature			Iron	None	Iron	None	None	None
Color			5.0	5.0	5.0	5.0	5.0	7.0
Odor			None	None	None	None	None	
Conductivity, µmhos				788.0	800.0	880.0	1,000.0	950.0

(1) Hardness in grains per U.S. gal. as $CaCO_3$.

(2) Includes any potassium.

(3) Instrument out of order.

TABLE 2.4-22

WATER ANALYSIS WELL 3[(1)]

Constituents		PPM as	7-11-77	10-20-77	1-16-78	2-22-78	3-2-78	7-17-78
			Date Sampled					
Cations								
Calcium	(Ca^{++})	$CaCO_2$	73.0	78.0	77.0	80.0	75.0	78.0
Magnesium	(Mg^{++})	$CaCO_2$	61.0	60.0	57.0	50.0	62.0	58.0
Sodium	(Na^{+})	$CaCO_2$	64.0	61.0	62.0	61.0	62.0	66.0
Potassium	(K^{+})	$CaCO_2$	27.0	17.0	16.0	16.0	16.0	15.0
Total Cations		$CaCO_2$	225.0	216.0	212.0	207.0	215.0	217.0
Anions								
Bicarbonate	(HCO_3^{-})	$CaCO_2$	117.0	175.0	168.0	163.0	171.0	170.0
Carbonate	(CO_3^{--})	$CaCO_2$	0.0	0.0	0.0	0.0	0.0	0.0
Hydroxide	(OH^{-})	$CaCO_2$	0.0	0.0	0.0	0.0	0.0	0.0
Sulfate	(SO_4^{--})	$CaCO_2$	5.0	0.0	0.0	0.0	0.0	0.0
Chloride	(Cl^{-})	$CaCO_2$	39.0	39.0	41.0	40.0	43.0	45.0
Phosphate (Total)	(PO_4^{--})	$CaCO_2$	0.65	1.53	1.55	0.9	0.85	0.25
Nitrate	(NO_3^{-})	$CaCO_2$	3.0	0.0	1.9	3.4	0.1	2.2
Total Anions			225.0	216.0	212.0	207.0	215.0	217.0
Total hardness		$CaCO_2$	134.0	138.0	134.0	130.0	137.0	170.0
Methyl orange alkalinity		$CaCO_2$	177.0	175.0	168.0	163.0	171.0	136.0
Phenolphthalein alkalinity		$CaCO_2$	0.0	0.0	0.0	0.0	0.0	0.0
Carbon dioxide, free		CO_2	4.0	3.0	7.0	4.0	4.0	1.0
pH		-	7.78	7.9	7.65	7.8	7.65	8.18
Iron, total		Fe	0.32	0.35	0.14	0.34	0.42	0.29
Manganese		Mn	0.0	0.0	0.0	0.0	0.0	0.0
Silica		SiO_2	11.8	13.1	13.1	12.1	12.6	12.1
Turbidity after shaking			1.0	0.5	0.2	0.3	0.7	0.5
Sediment, color			None	None	None	None	None	None
Sediment, nature			None	None	None	None	None	None
Color			5.0	5.0	5.0	5.0	5.0	5.0
Odor			None	None	None	None	None	None
Conductivity, μmhos			390.0			400.0	410.0	395.0

TABLE 2.4-22 (cont)

Constituents		PPM as	Date Sampled				
			10-29-79	10-15-80	1-15-81	4-15-81	10-15-81
Cations							
Calcium	(Ca^{++})	$CaCO_2$	129.0	158.0	172.0	202.0	165.0
Magnesium	(Mg^{++})	$CaCO_2$	99.0	118.0	134.0	154.0	124.0
Sodium	(Na^{+})	$CaCO_2$	82.0[2]	99.0[2]	116.0[2]	133.0[2]	191.0[2]
Potassium	(K^{+})	$CaCO_2$					
Total Cations		$CaCO_2$	310.0	375.0	422.0	489.0	380.0
Anions							
Bicarbonate	(HCO_3^{-})	$CaCO_2$	162.0	159.0	160.0	159.0	170.0
Carbonate	(CO_3^{--})	$CaCO_2$	0.0	0.0	0.0	0.0	0.0
Hydroxide	(OH^{-})	$CaCO_2$	0.0	0.0	0.0	0.0	0.0
Sulfate	(SO_4^{--})	$CaCO_2$	0.0	4.0	5.0	7.0	5.0
Chloride	(Cl^{-})	$CaCO_2$	145.0	204.0	253.0	321.0	302.0
Phosphate (Total)	(PO_4^{--})	$CaCO_2$	0.15	0.15	0.300	0.08	0.12
Nitrate	(NO_3^{-})	$CaCO_2$	2.7	8.2	3.3	2.4	2.7
Total Anions			310.0	375.0	422.0	489.0	480.0
Total hardness		$CaCO_2$	228.0	276.0	306.0	356.0	289.0
Methyl orange alkalinity		$CaCO_2$	162.0	159.0	160.0	159.0	170.0
Phenolphthalein alkalinity		$CaCO_2$	0.0	0.0	0.0	0.0	0.0
Carbon dioxide, free		CO_2	4.0	6.0	4.0	6.0	12.0
pH		-	7.75	7.6	7.7	8.0	7.4
Iron, total		Fe	0.25	0.65	0.55	0.02	0.22
Manganese		Mn	0.0	0.0	0.0	0.0	0.02
Silica		SiO_2	12.4	12.3	12.5	12.9	12.2
Turbidity after shaking			0.7	3.5	3.4	(3)	(3)
Sediment, color			None	Rust	None	None	None
Sediment, nature			None	Iron	None	None	None
Color			5.0	7.0	5.0	5.0	7.0
Odor			None	None	None	None	None
Conductivity, µmhos				665.0	780.0	1000.0	885.0

(1) Hardness in grains per U.S. gal. as $CaCO_3$.

(2) Includes any potassium.

(3) Instrument out of order.

TABLE 2.4-23

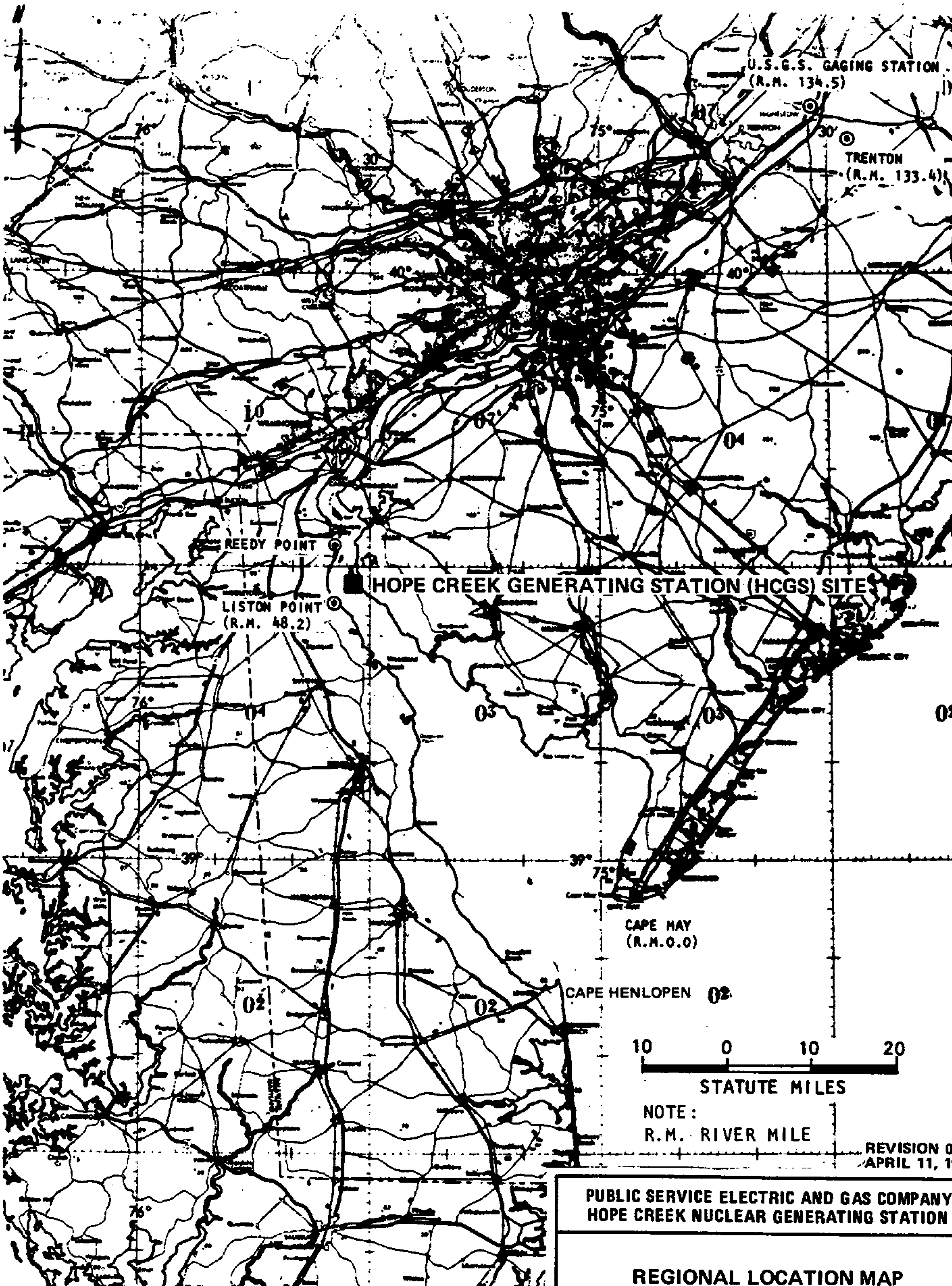
WATER ANALYSIS WELL 5[1]

Constituents		PPM as	7-11-77	10-20-77	1-16-78	7-17-78	10-15-80	1-15-81	4-15-81	10-15-81
			Date Sampled							
Cations										
Calcium	(Ca^{++})	$CaCO_2$	6.0	4.0	6.0	5.0	88.0	203.0	200.0	7.0
Magnesium	(Mg^{++})	$CaCO_2$	0.0	3.0	1.0	1.0	62.0	151.0	156.0	4.0
Sodium	(Na^+)	$CaCO_2$	166.0	169.0	172.0	169.0	140.0[2]	133.0[2]	137.0[2]	189.0[2]
Potassium	(K^+)	$CaCO_2$	15.0	4.0	4.0	4.0				
Total Cations		$CaCO_2$	187.0	180.0	182.0	179.0	290.0	487.0	493.0	200.0
Anions										
Bicarbonate	(HCO_3^-)	$CaCO_2$	155.0	155.0	157.0	153.0	157.0	157.0	160.0	156.0
Carbonate	(CO_3^{--})	$CaCO_2$	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Hydroxide	(OH^-)	$CaCO_2$	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Sulfate	(SO_4^{--})	$CaCO_2$	5.0	0.0	0.0	0.0	6.0	7.0	8.0	5.0
Chloride	(Cl^-)	$CaCO_2$	23.0	23.0	23.0	24.0	124.0	318.0	323.0	37.0
Phosphate (Total)	(PO_4^{--})	$CaCO_2$	2.45	2.45	2.5	1.72	0.93	0.17	0.05	1.65
Nitrate	(NO_3^-)	$CaCO_2$	1.1	0.0	0.0	0.7	2.5	4.8	1.6	0.1
Total Anions			187.0	180.0	182.0	179.0	290.0	487.0	493.0	200.0
Total hardness		$CaCO_2$	6.0	7.0	7.0	6.0	150.0	354.0	356.0	11.0
Methyl orange alkalinity		$CaCO_2$	155.0	155.0	157.0	153.0	157.0	157.0	160.0	156.0
Phenolphthalein alkalinity		$CaCO_2$	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Carbon dioxide, free		CO_2	2.0	3.0	13.0	2.0	3.0	5.0	5.0	6.0
pH		-	7.95	7.9	7.25	8.05	8.0	7.6	7.95	7.7
Iron, total		Fe	0.11	0.13	0.02	0.11	0.49	0.48	0.02	0.22
Manganese		Mn	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.02
Silica		SiO_2	9.7	9.9	10.0	9.0	11.2	12.7	12.9	9.6
Turbidity after shaking			1.4	0.0	0.1	0.5	6.5	4.7	(3)	(3)
Sediment, color			None	None	None	None	None	None	None	None
Sediment, nature			None	None	None	None	None	None	None	None
Color			5.0	5.0	5.0	5.0	5.0	5.0	5.0	7.0
Odor			None	None	None	None	None	None	None	None
Conductivity, µmhos			325.0			320.0	490.0	910.0	1,000.0	333.0

(1) Hardness in grains per U.S. gal. as $CaCO_3$.

(2) Includes any potassium.

(3) Instrument out of order.



REFERENCE:
PSE&G, 1978, POST-EXCAVATION FOUNDATION STUDIES, HOPE CREEK GENERATING STATION, FIGURE 1-1.

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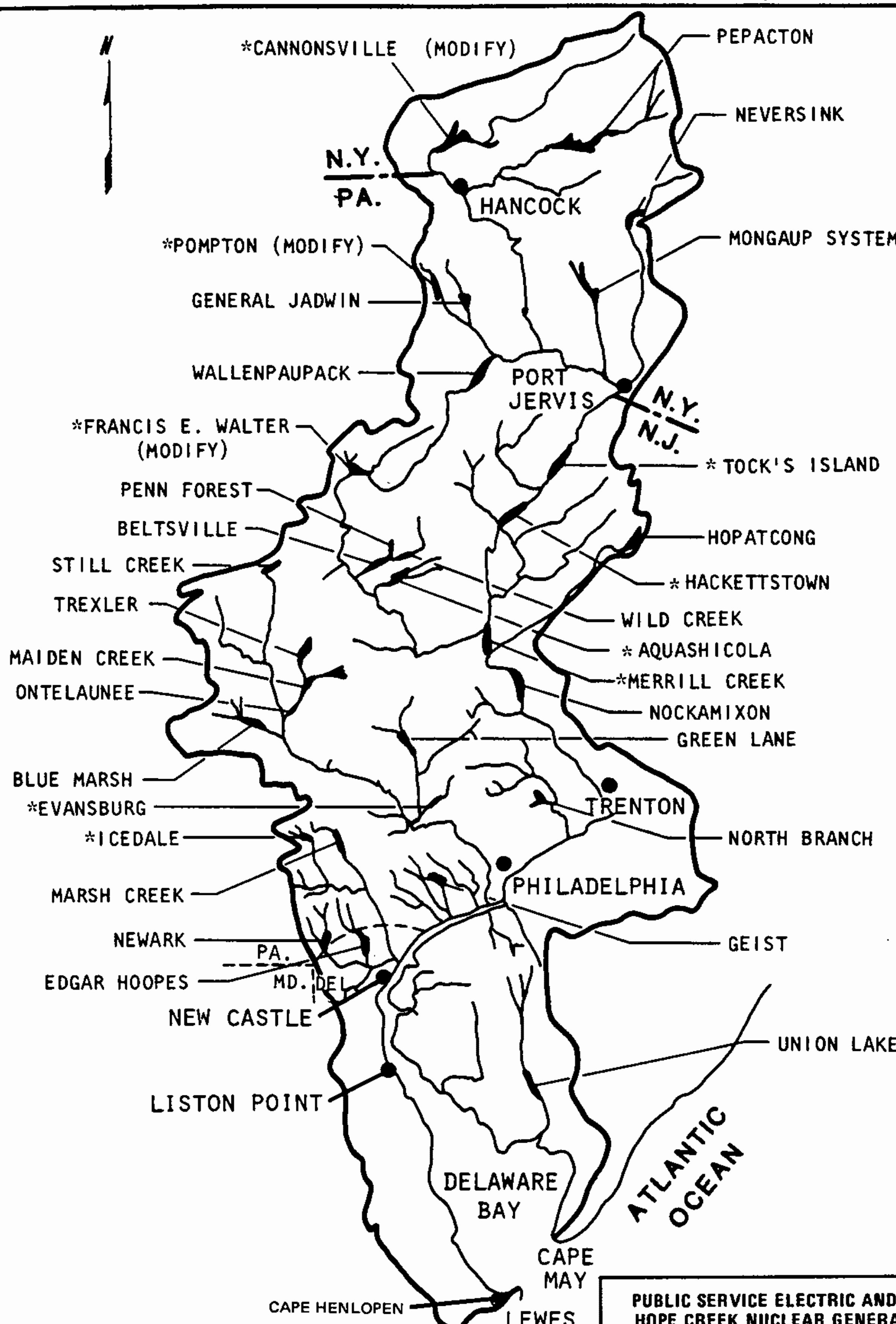
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

REGIONAL LOCATION MAP

UPDATED FSAR

FIGURE 2.4-1



SOURCE:
REF. 2.4-1
* PROPOSED FACILITIES IN COMMISSIONS COMPREHENSIVE PLAN OF DRBC.

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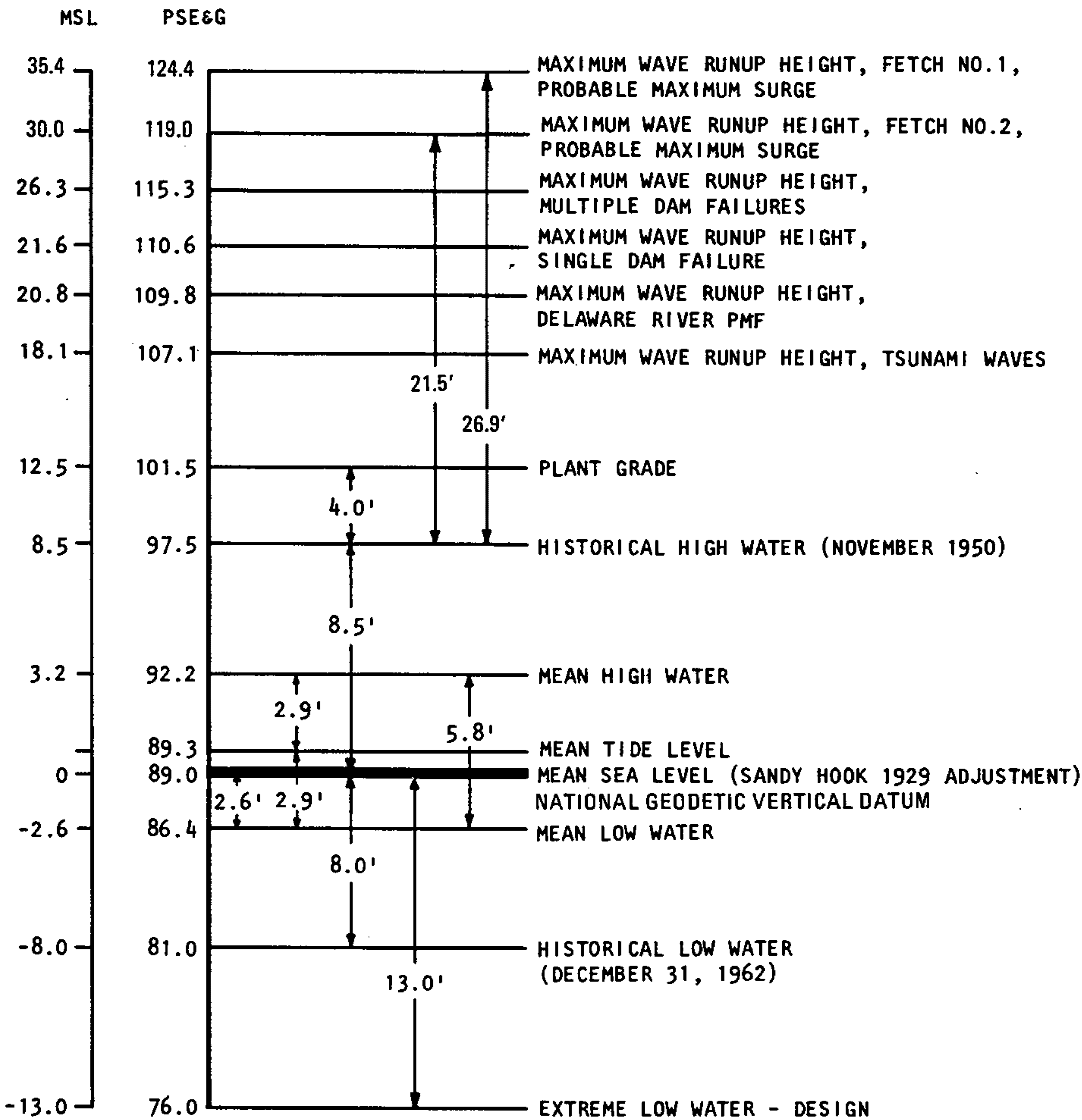
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOCATION OF MAJOR IMPOUNDMENTS IN THE DELAWARE RIVER BASIN

UPDATED FSAR

FIGURE 2.4-2



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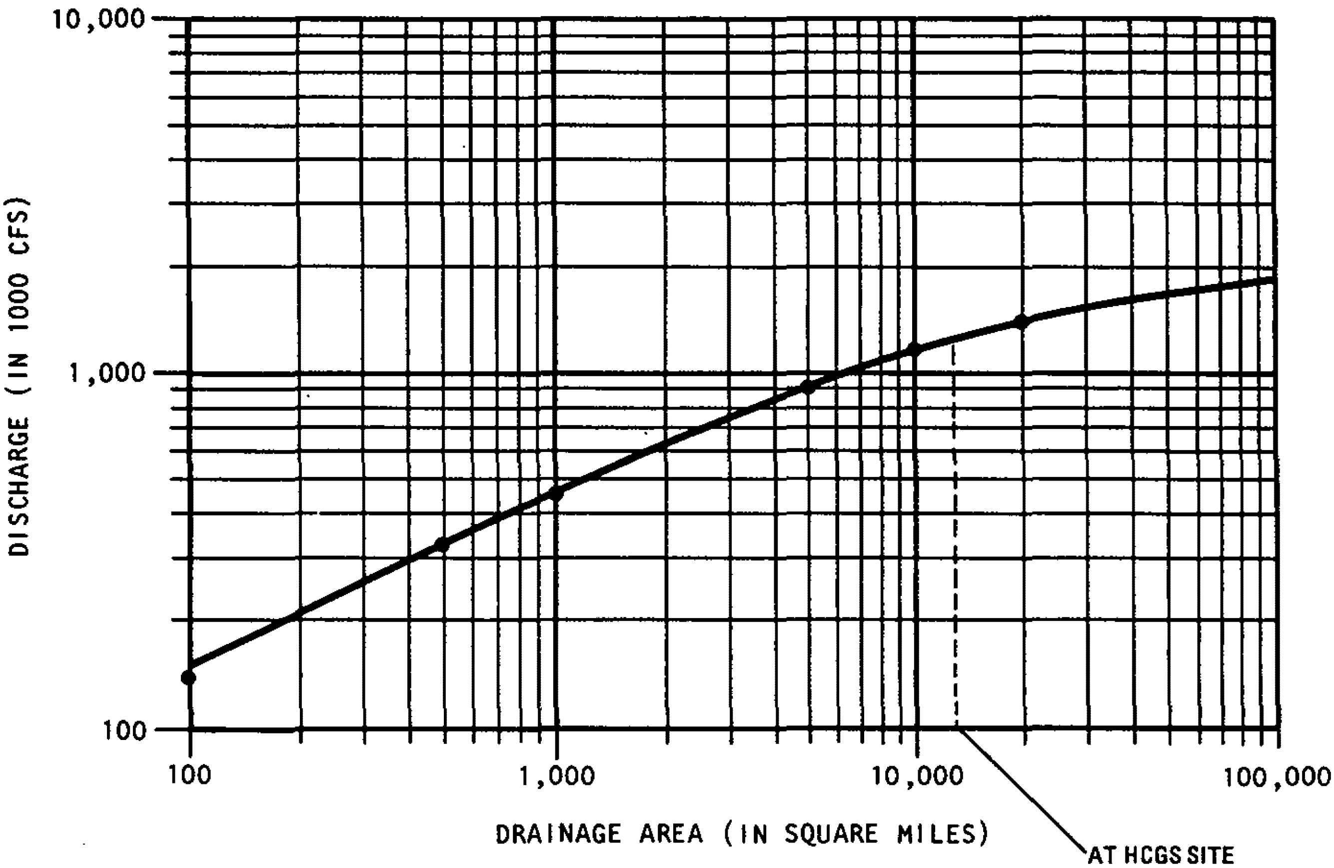
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

DATUM AND WATER LEVEL RELATIONSHIP

UPDATED FSAR

FIGURE 2.4-3



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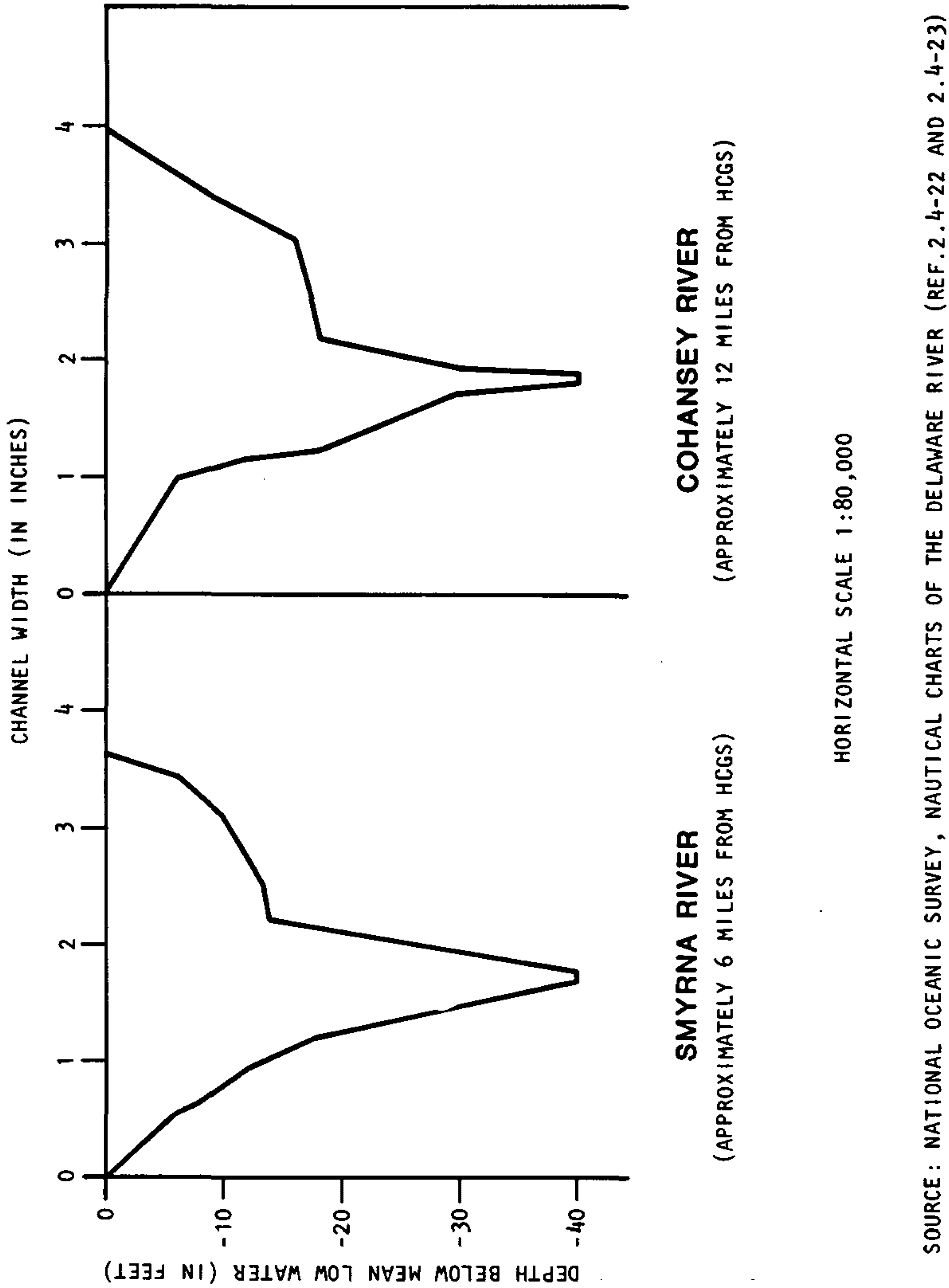
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

PMF PEAK DISCHARGE

UPDATED FSAR

FIGURE 2.4-4



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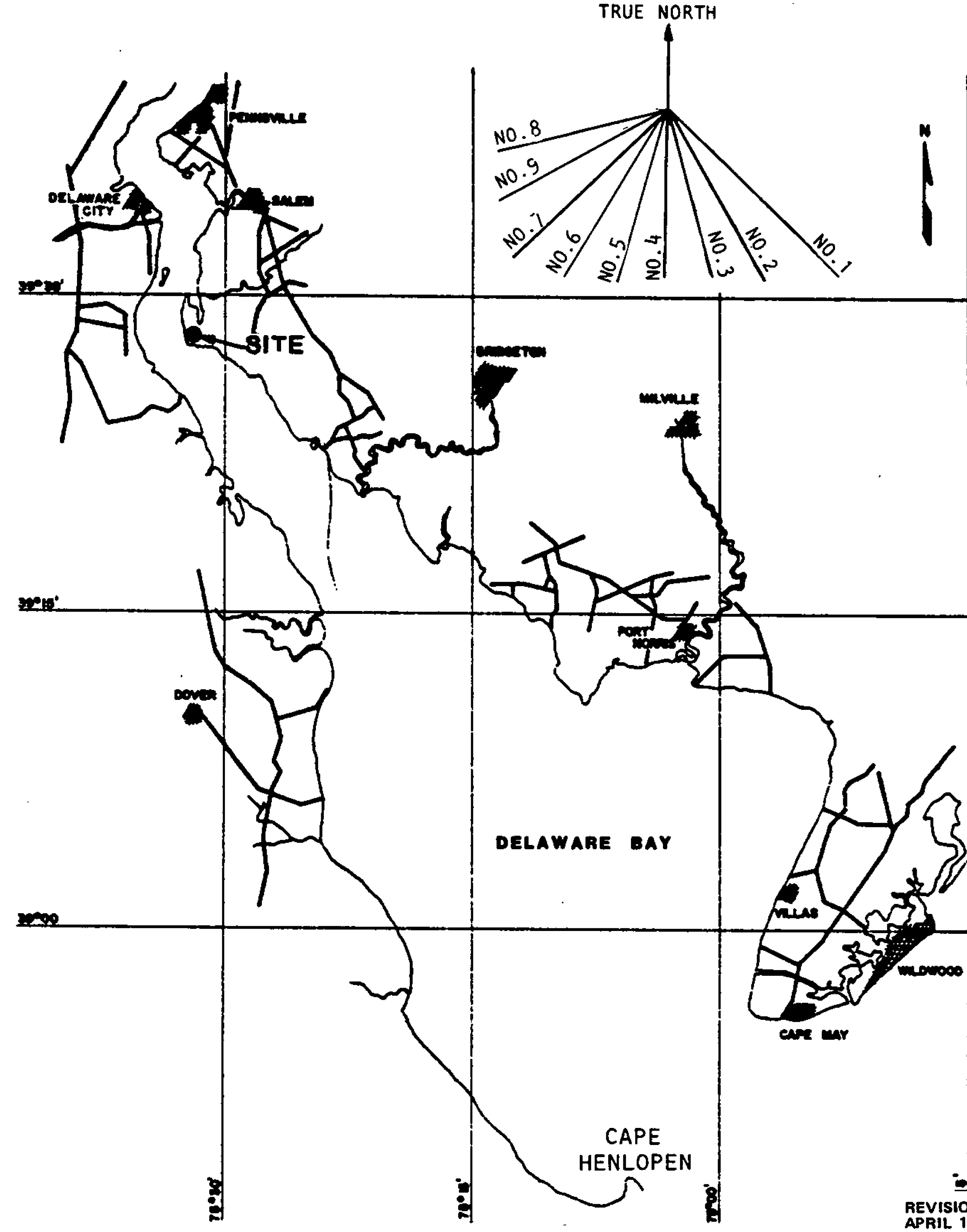
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

RIVER CHANNEL CROSS-SECTIONS FOR PMF WATER LEVEL ESTIMATION

UPDATED FSAR

FIGURE 2.4-5



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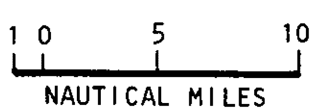
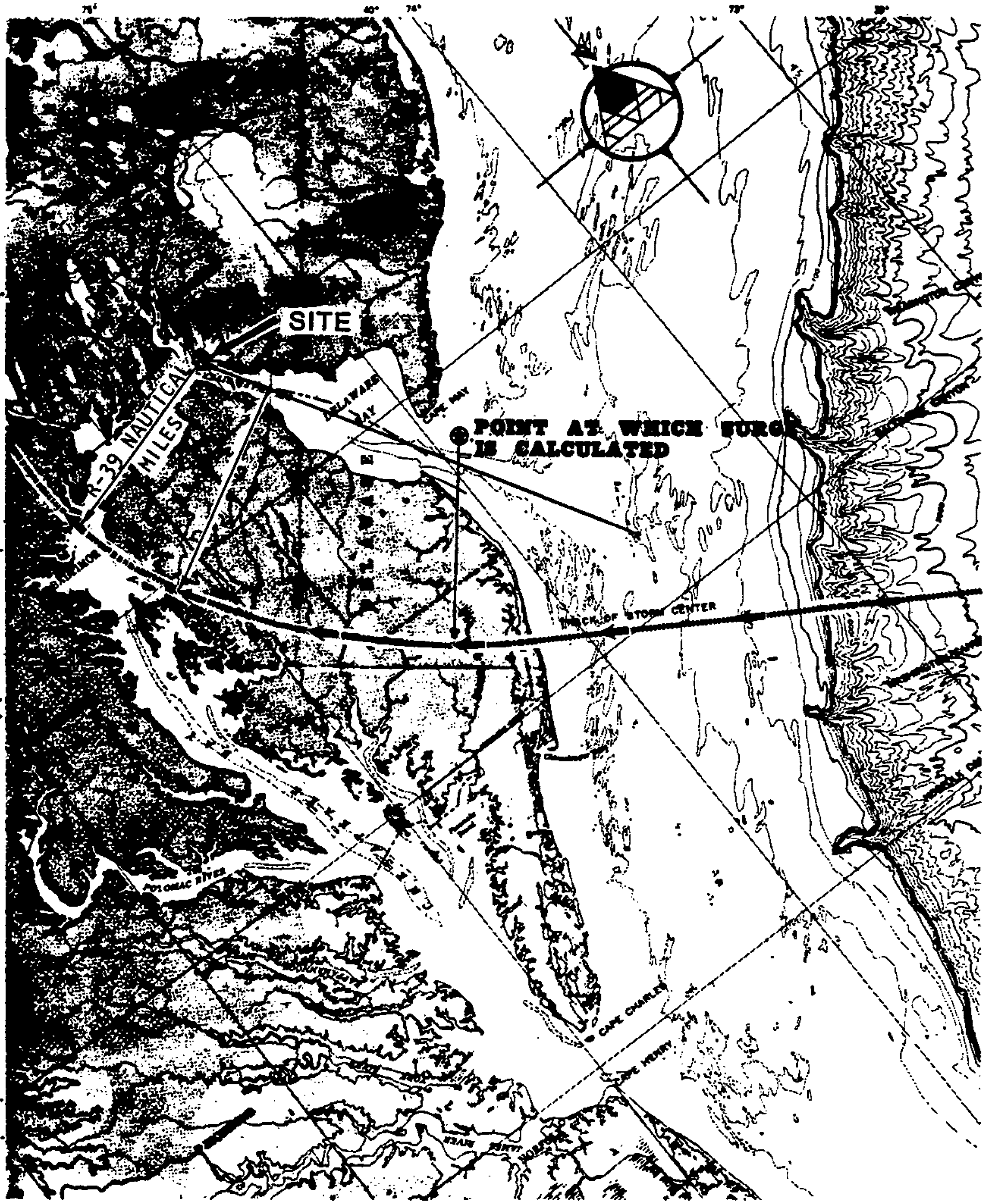


PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

FETCH DIAGRAM FOR COINCIDENT WIND-WAVE ACTIVITY ANALYSIS

UPDATED FSAR

FIGURE 2.4-6



25 0 25 50

STATUTE MILES

REFERENCE:

THIS MAP WAS ENLARGED FROM A PORTION OF USGS "PROFESSIONAL PAPER 529-C, PLATE 2", ENTITLED "MAP SHOWING BOTTOM TOPOGRAPHY OF THE CONTINENTAL MARGIN, NEW JERSEY TO SOUTH CAROLINA" DATED 1967.

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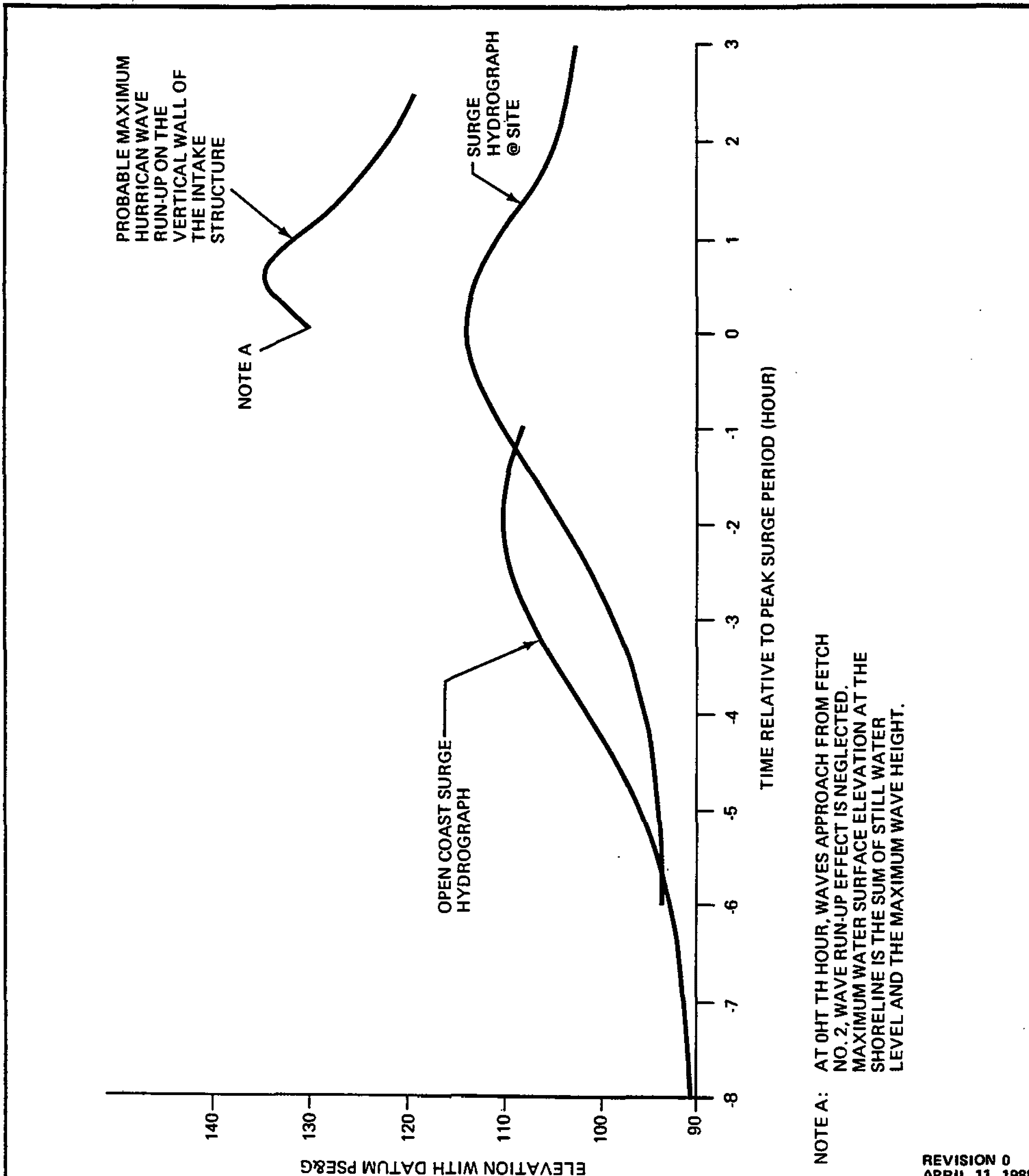
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

CRITICAL PATH OF PMH FOR MAXIMUM WATER LEVEL ESTIMATION

UPDATED FSAR

FIGURE 2.4-7



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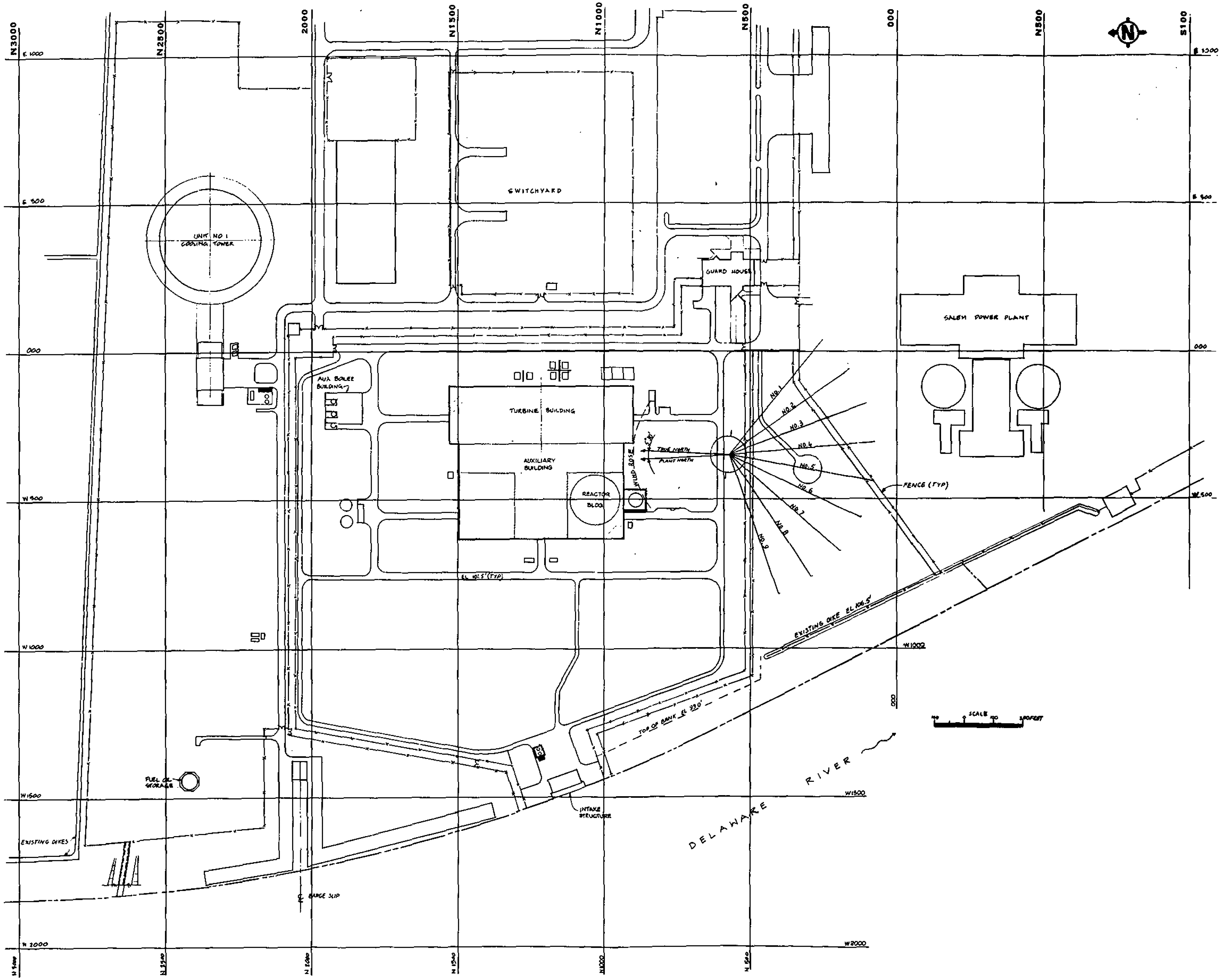
PUBLIC SERVICE ELECTRIC AND GAS COMPANY HOPE CREEK NUCLEAR GENERATING STATION	
COMPUTED SURGE HYDROGRAPH AND RUN-UP HYDROGRAPH	
UPDATED FSAR	FIGURE 2.4-8



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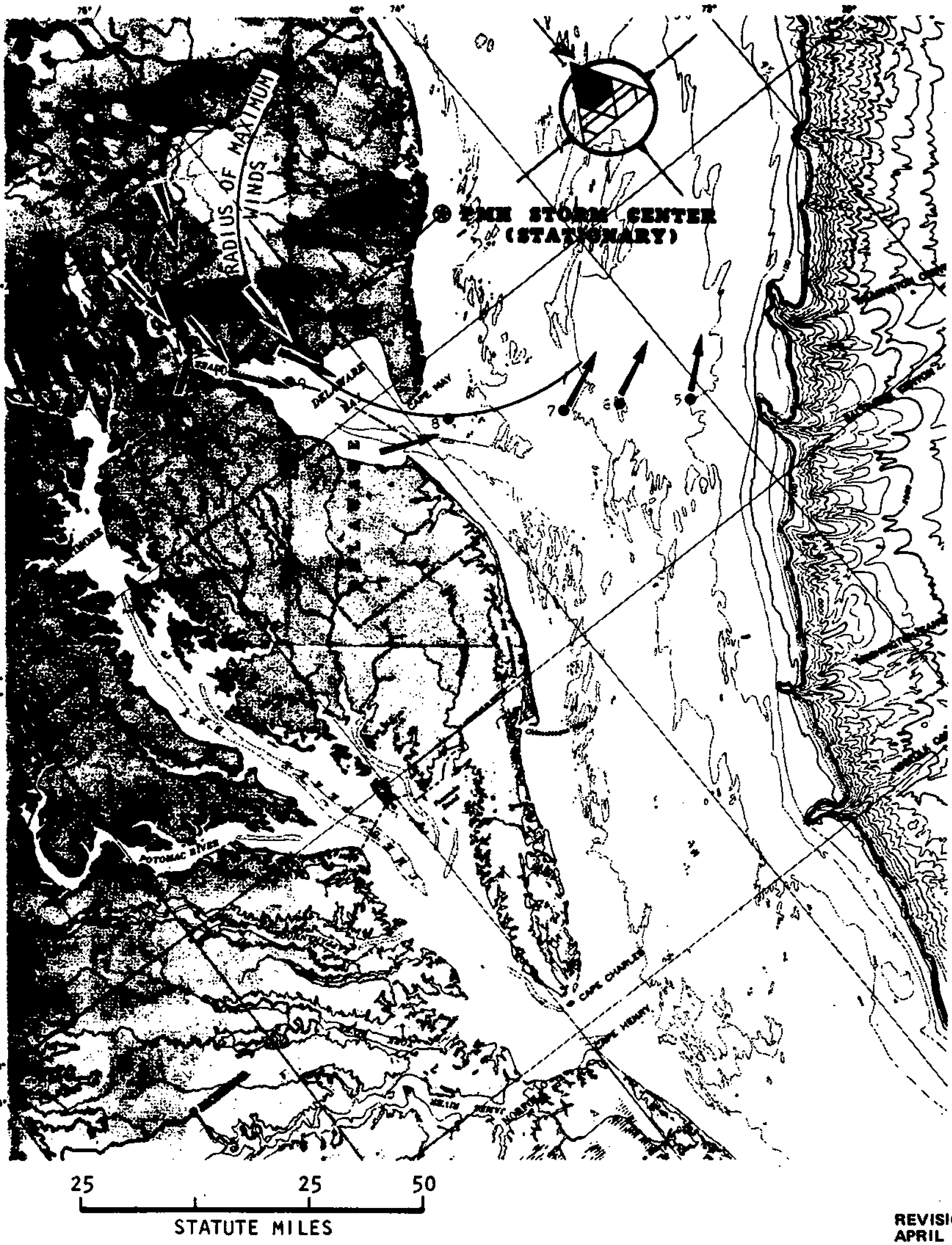
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

HCGS PLOT PLAN

UPDATED FSAR | **FIGURE 2.4-9**



KEY:

● COMPUTED POINTS

ARROW INDICATES WIND DIRECTION

REFERENCE:

THIS MAP WAS ENLARGED FROM A PORTION OF USGS "PROFESSIONAL PAPER 529-C, PLATE 2", ENTITLED "MAP SHOWING BOTTOM TOPOGRAPHY OF THE CONTINENTAL MARGIN, NEW JERSEY TO SOUTH CAROLINA" DATED 1967.

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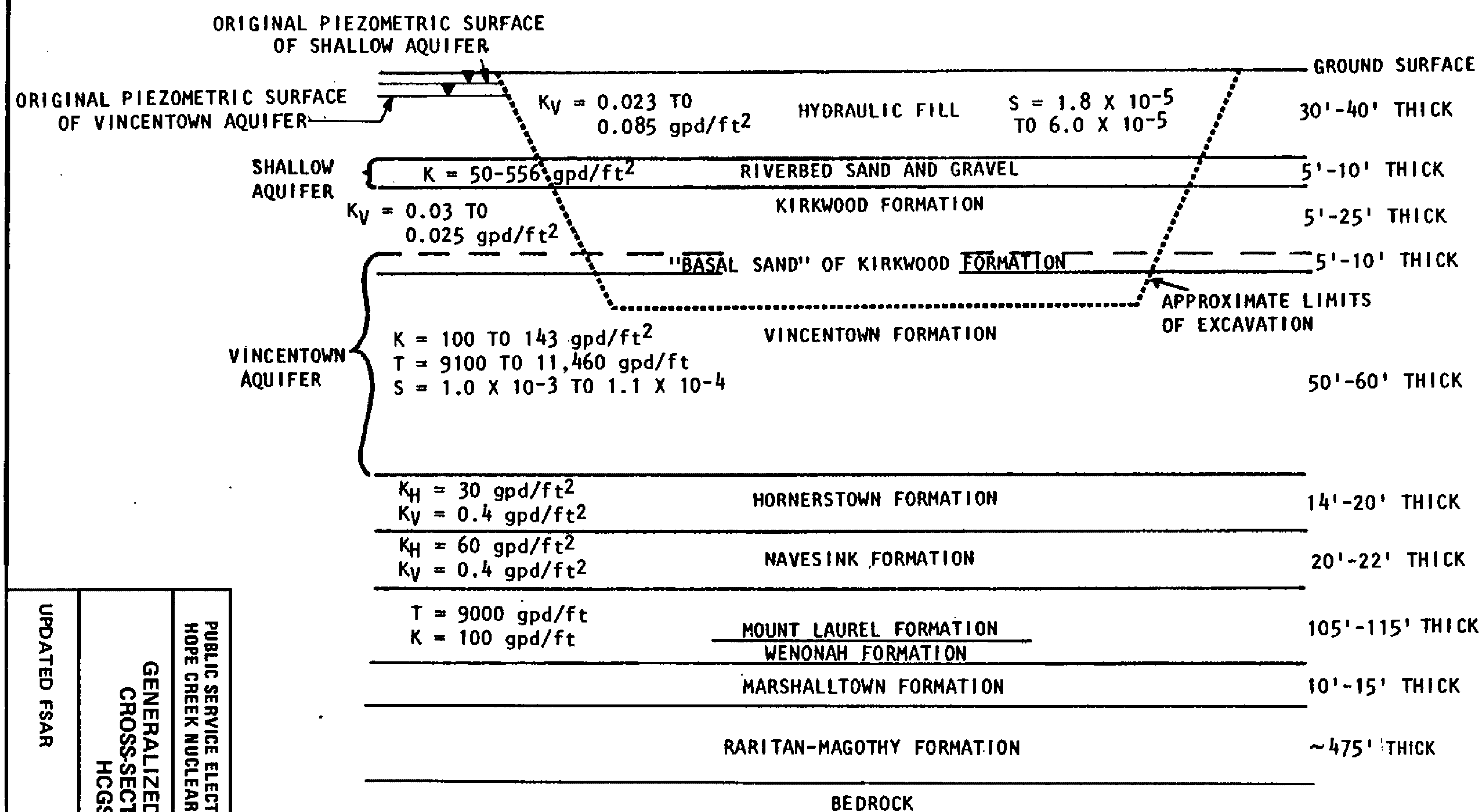
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

CRITICAL PATH OF THE HURRICANE FOR EXTREME LOW WATER ESTIMATION

UPDATED FSAR

FIGURE 2.4-10



NOTES:

1. This figure is not drawn to scale and should not be used for any purpose other than providing a generalized picture of the site stratigraphy.

2. Some deviation in the stratigraphy can be expected.

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

GENERALIZED GEOLOGICAL CROSS-SECTION OF THE HCGS SITE

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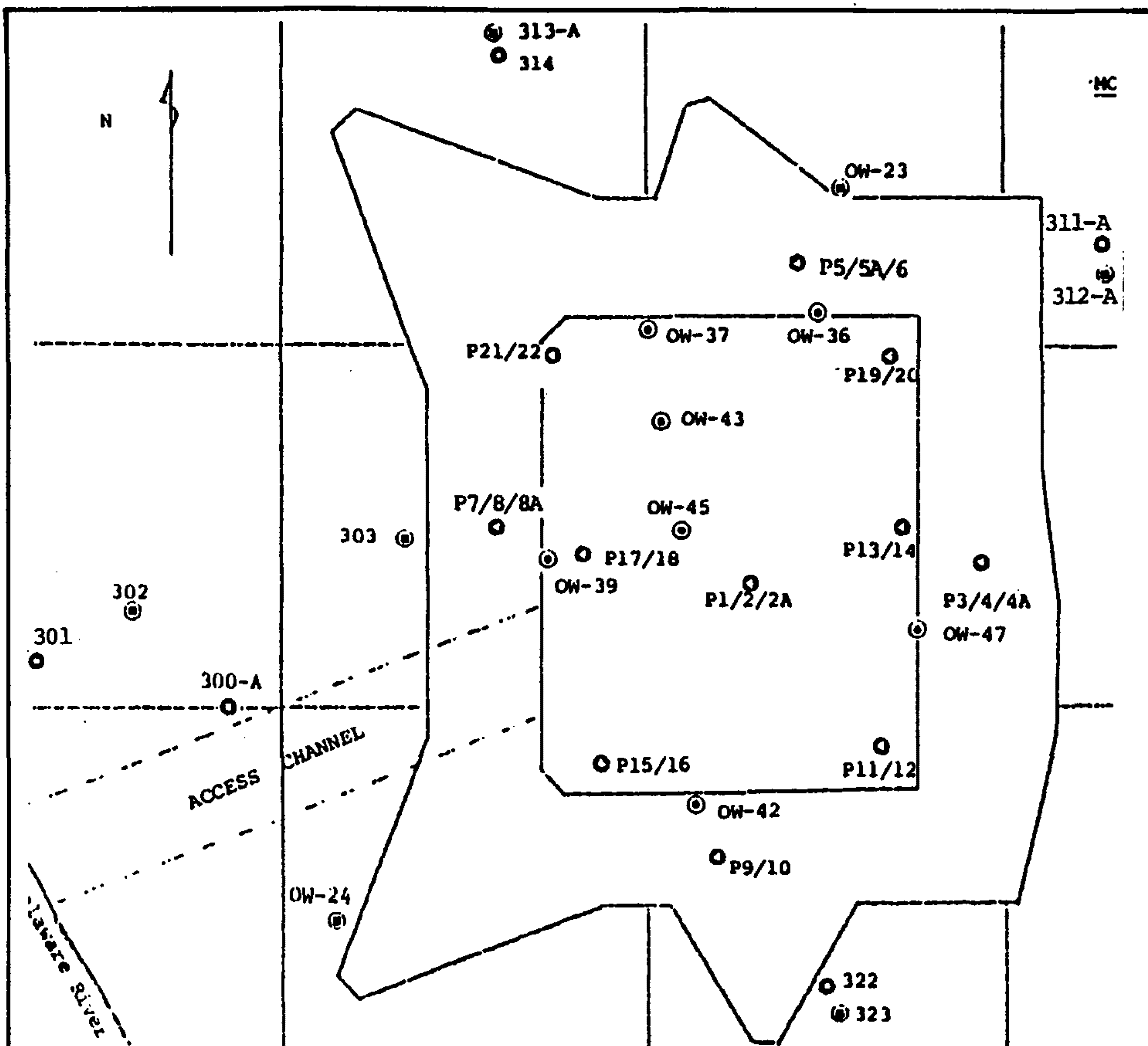
FIGURE 2.4-11



KEY:

- Deep Observation Well
- Shallow Observation Well
- Pressure Cell Piezometer
- Other Observation Well

0 100 200 300

Scale in Feet

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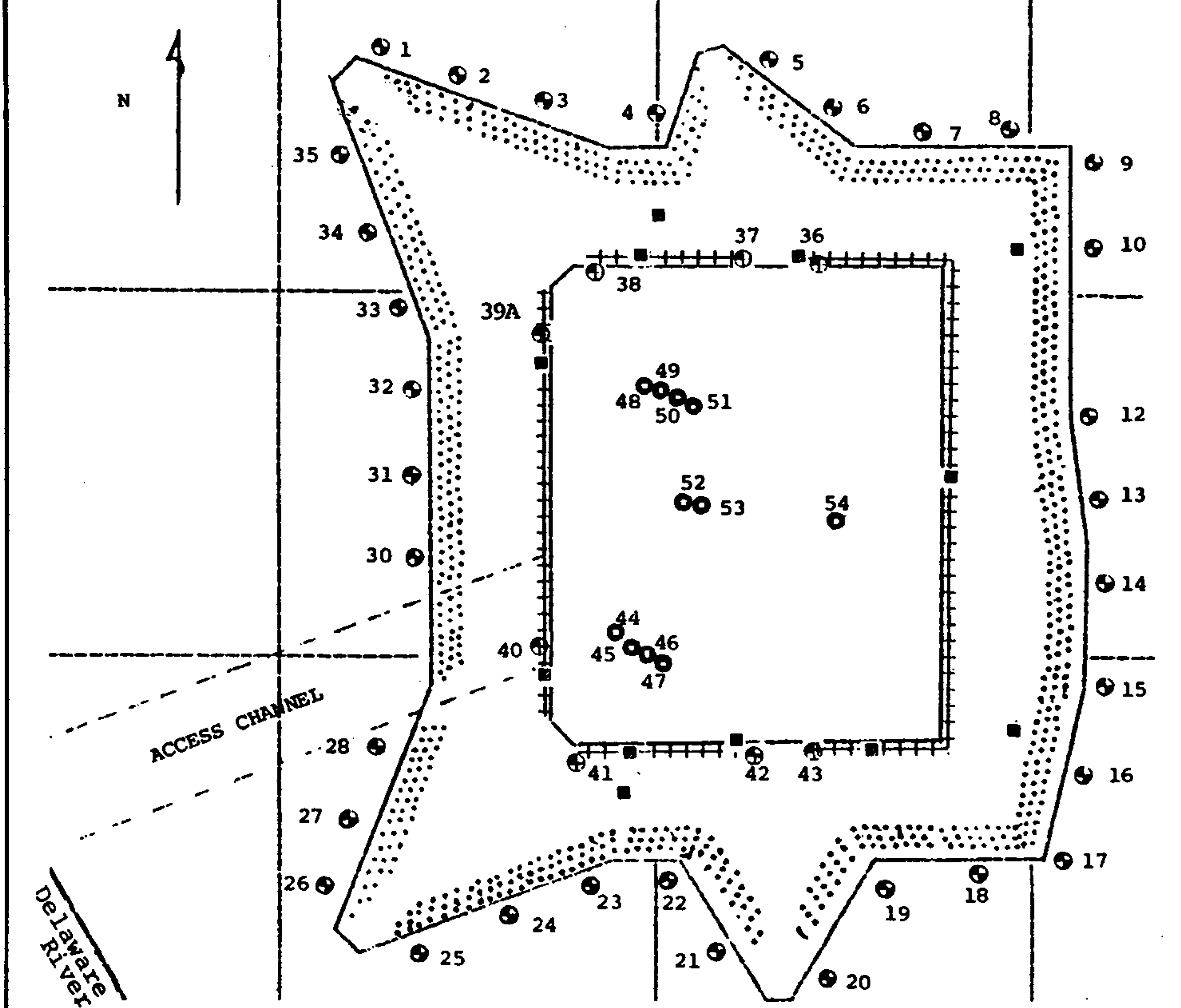
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

SCHEMATIC MONITORING SYSTEM PLAN

UPDATED FSAR

FIGURE 2.4-12



KEY:

Deep Dewatering Well (DDW)

Temporary Deep Dewatering Well

Supplementary Deep Dewatering Well for Pits and Trenches

Sand Drains

Well Point System

Surface Water Sump

0 100 200 300

Scale in Feet

NOTE: DDW 3 was not put into production

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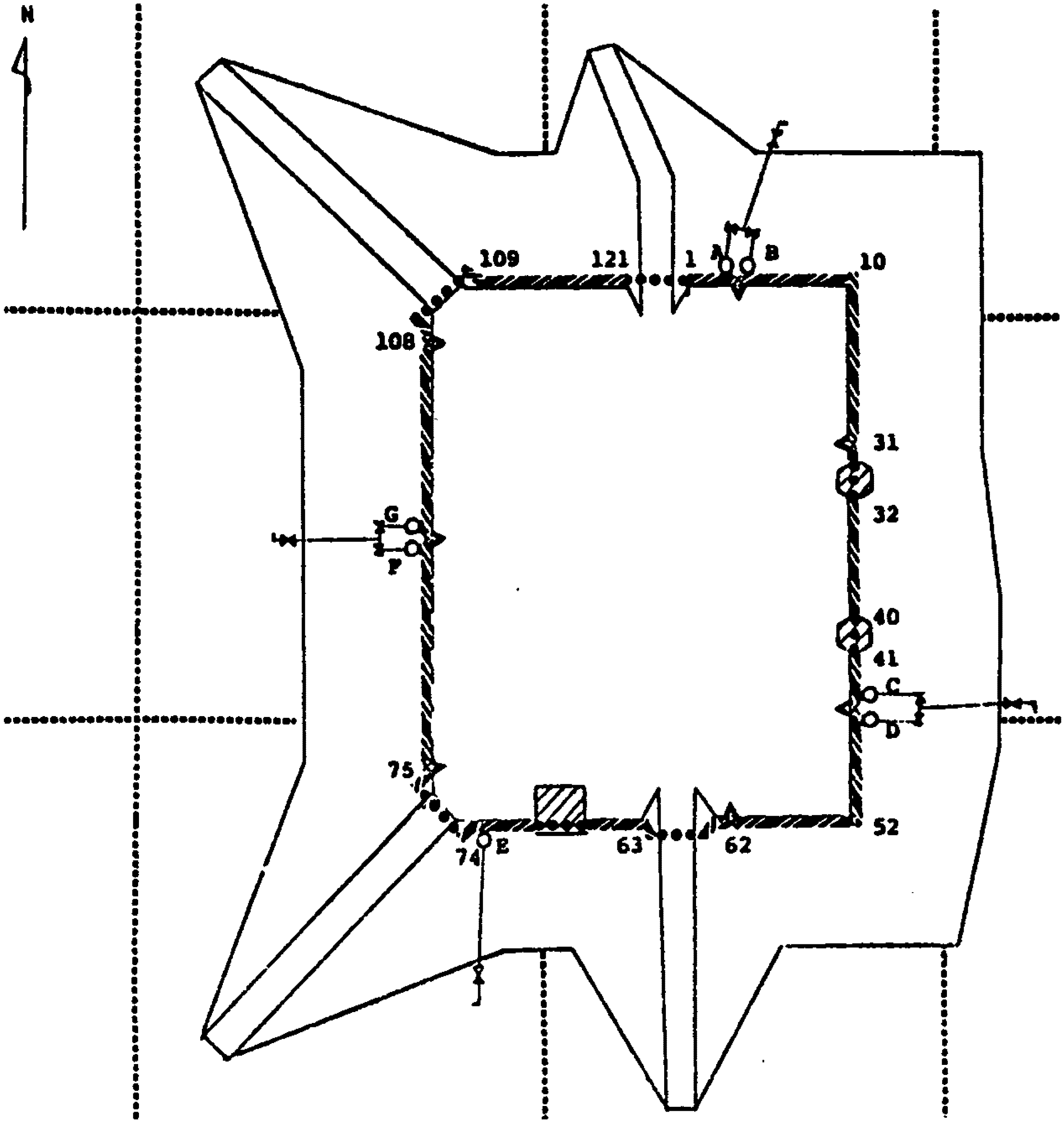
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

SCHEMATIC DEWATERING SYSTEM PLAN

UPDATED FSAR

FIGURE 2.4-13



KEY:

- A / O — Well Point Sumps
- Valve on Discharge Lines
- Discharge Header (10 inch schedule 80 PVC)
- Valve in Discharge Header Used for Segmentation of System
- Discharge Header Passing Through 24 inch steel pipe under ramp or crane foundation pad.
- Crane Foundation Pads over Well Point Header

All 121 well points are located just outside discharge header around the excavation floor.

Well point numbers shown here at key locations only.

0 100 200 300
Scale In Feet

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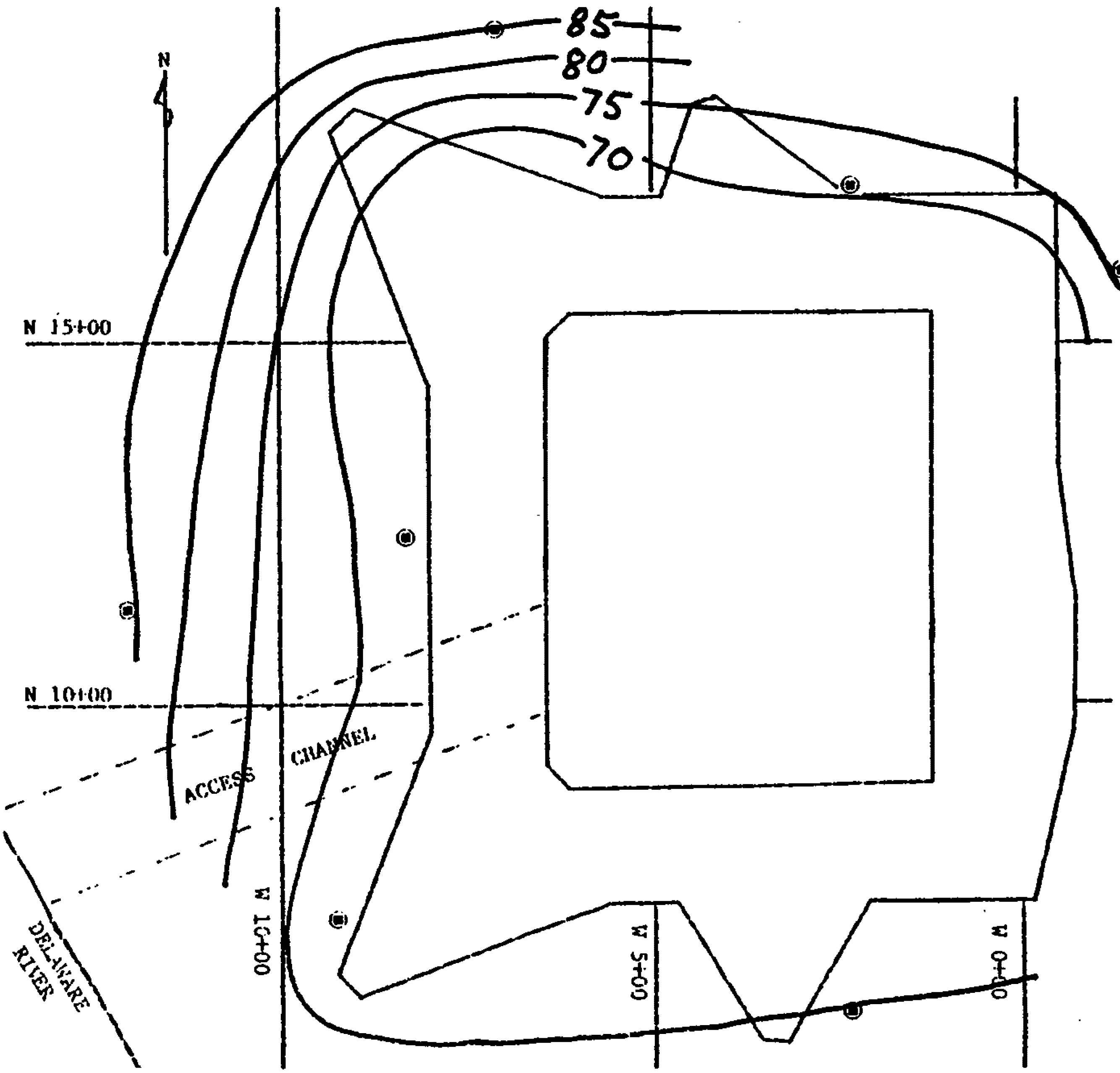
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

SCHEMATIC WELL POINT SYSTEM — SUMPS, DISCHARGE HEADER, DISCHARGE LINES & VALVES

UPDATED FSAR

FIGURE 2.4-14



EXPLANATION:

⊛ Shallow Observation Well

—— 90 ——

Piezometric Surface Contour Line (Elevation in ft. PSD) estimated from water level measurements in the depicted wells

DATE May 17, 1978

TIME ≈1200 hrs. EST

0 100 200 300

SCALE (in feet)

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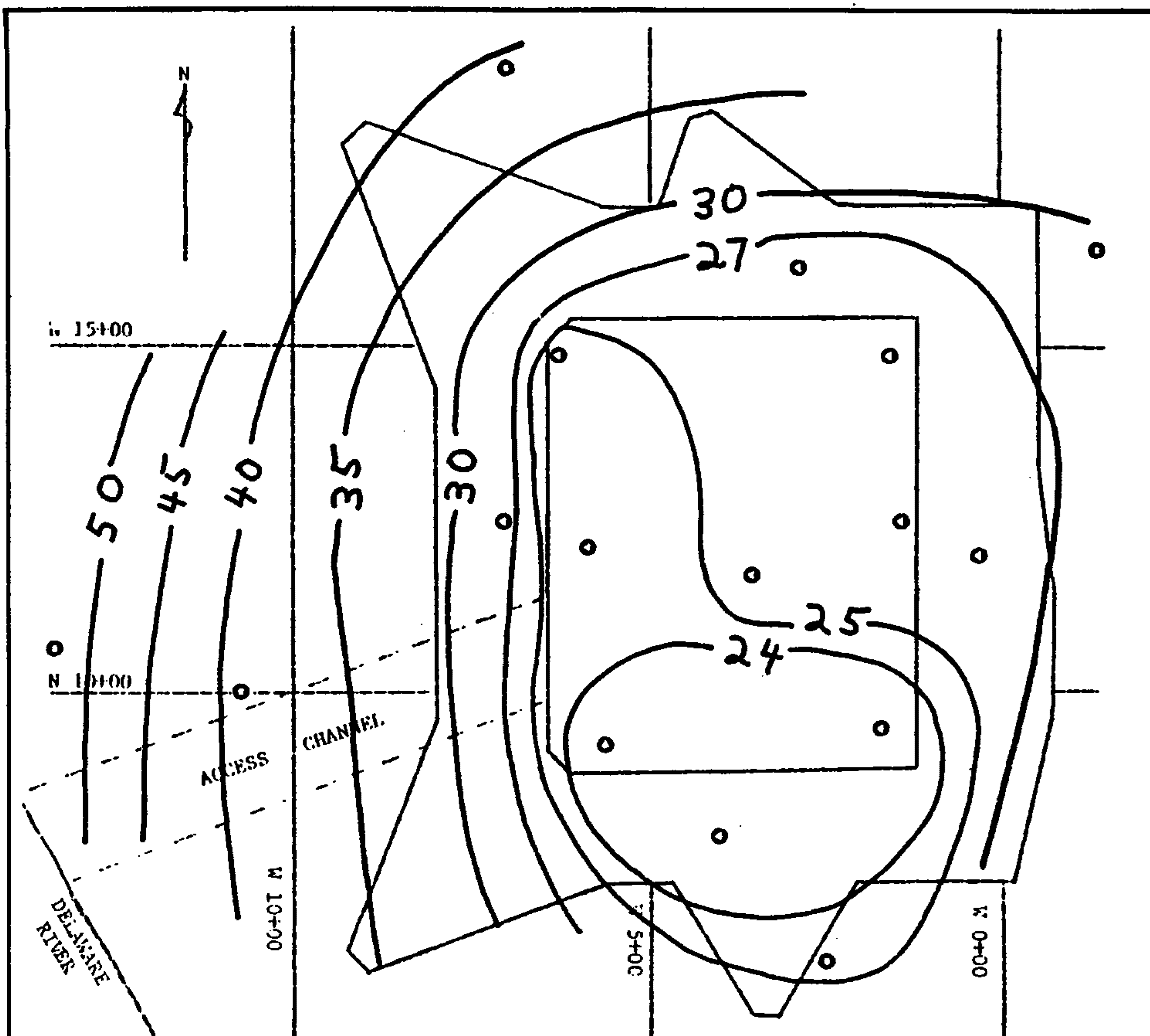
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

SHALLOW AQUIFER
PIEZOMETRIC LEVELS – 5/17/78

UPDATED FSAR

FIGURE 2.4-15



EXPLANATION:

O Deep Observation Well

O Hydrostatic Pressure Cell Piezometer

DATE May 17, 1978

TIME ≈1200 hrs. EST

—— 55 ——

Piezometric Surface Contour Line (Elevation in ft. PSD) estimated from water level measurements in the depicted wells and piezometers.

0 100 200 300

SCALE (in feet)

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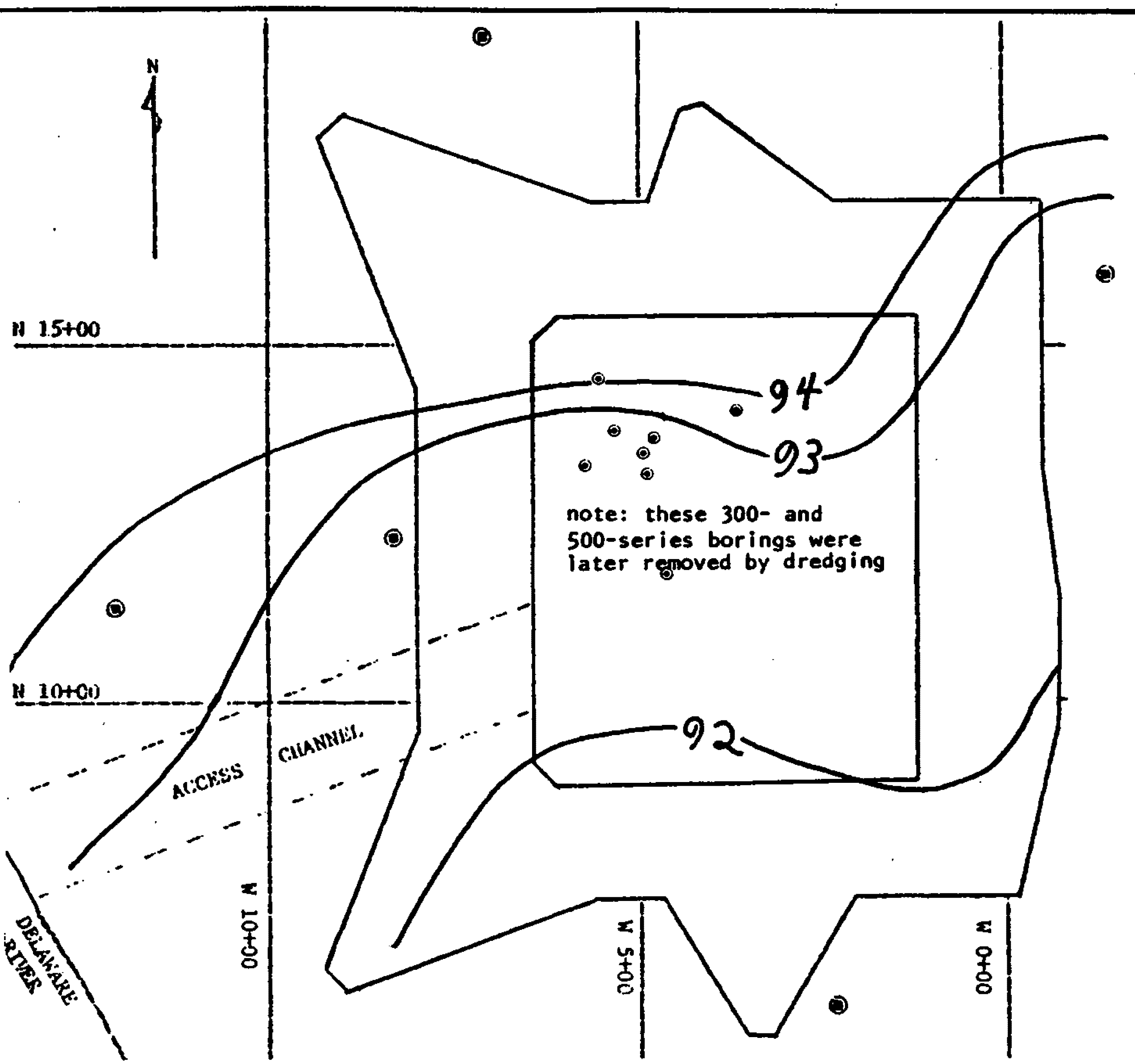
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

VINCENTOWN AQUIFER
PIEZOMETRIC LEVELS – 5/17/78

UPDATED FSAR

FIGURE 2.4-16



EXPLANATION:

◉ Shallow Observation Well

⊙ Other Observation Well

—— 90 —— Piezometric Surface Contour Line (Elevation in ft. PSD) estimated from water level measurements in the depicted wells

0 100 200 300
SCALE
(in feet)

DATE January 16, 1976

TIME ______________ EST

Contours taken from Figure 9 of Stage 1 Report dated June 21, 1976.

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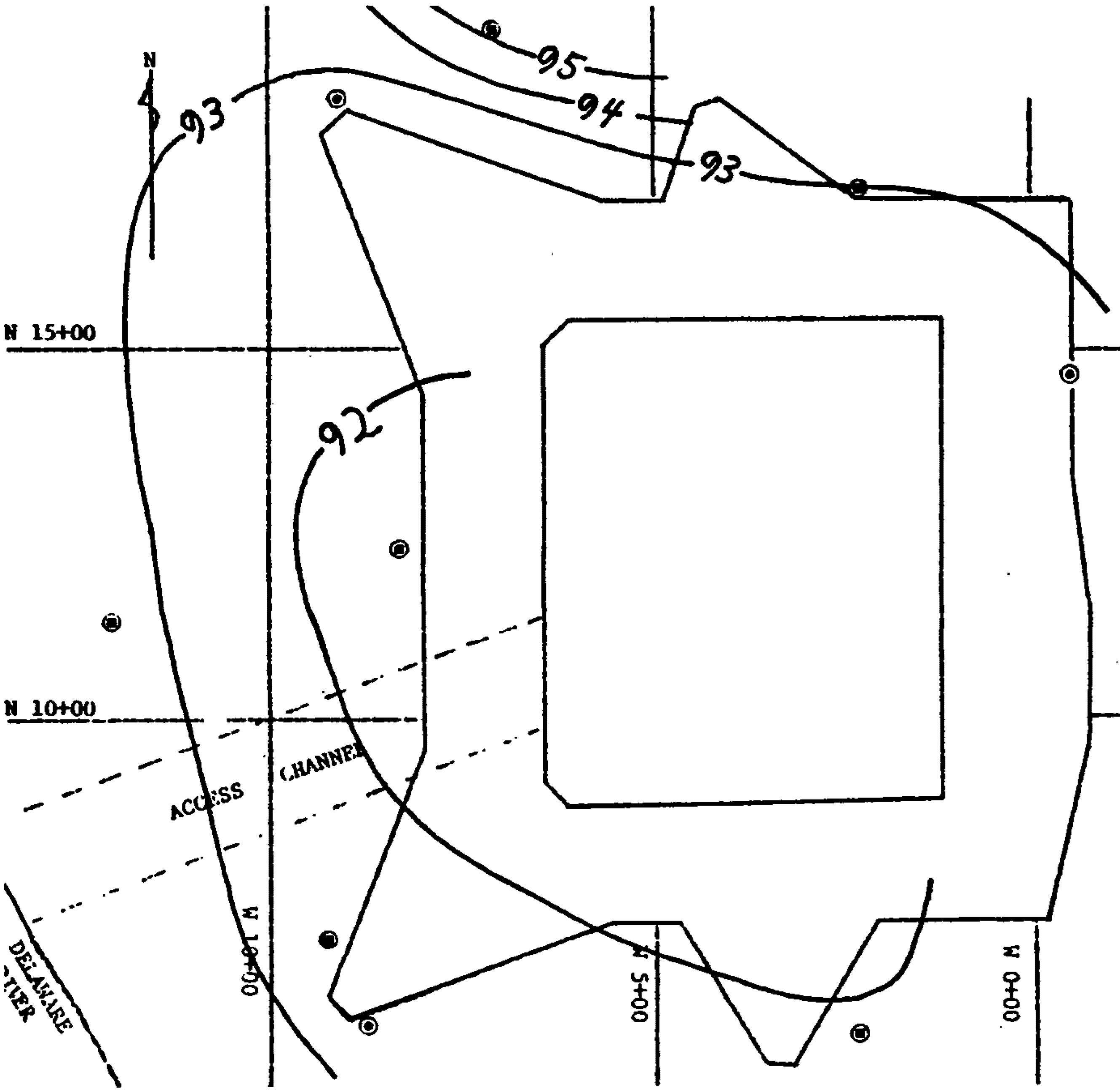
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

SHALLOW AQUIFER
PIEZOMETRIC LEVELS – 1/16/76

UPDATED FSAR

FIGURE 2.4-17



EXPLANATION:

◉ Shallow Observation Well

• Other Observation Well

—90— Piezometric Surface Contour Line (Elevation in ft. PSD) estimated from water level measurements in the depicted wells

DATE August 24, 1976

TIME 0900 hrs. EST

Pool Level Elevation = 93.8 PSD

0 100 200 300

SCALE
(in feet)

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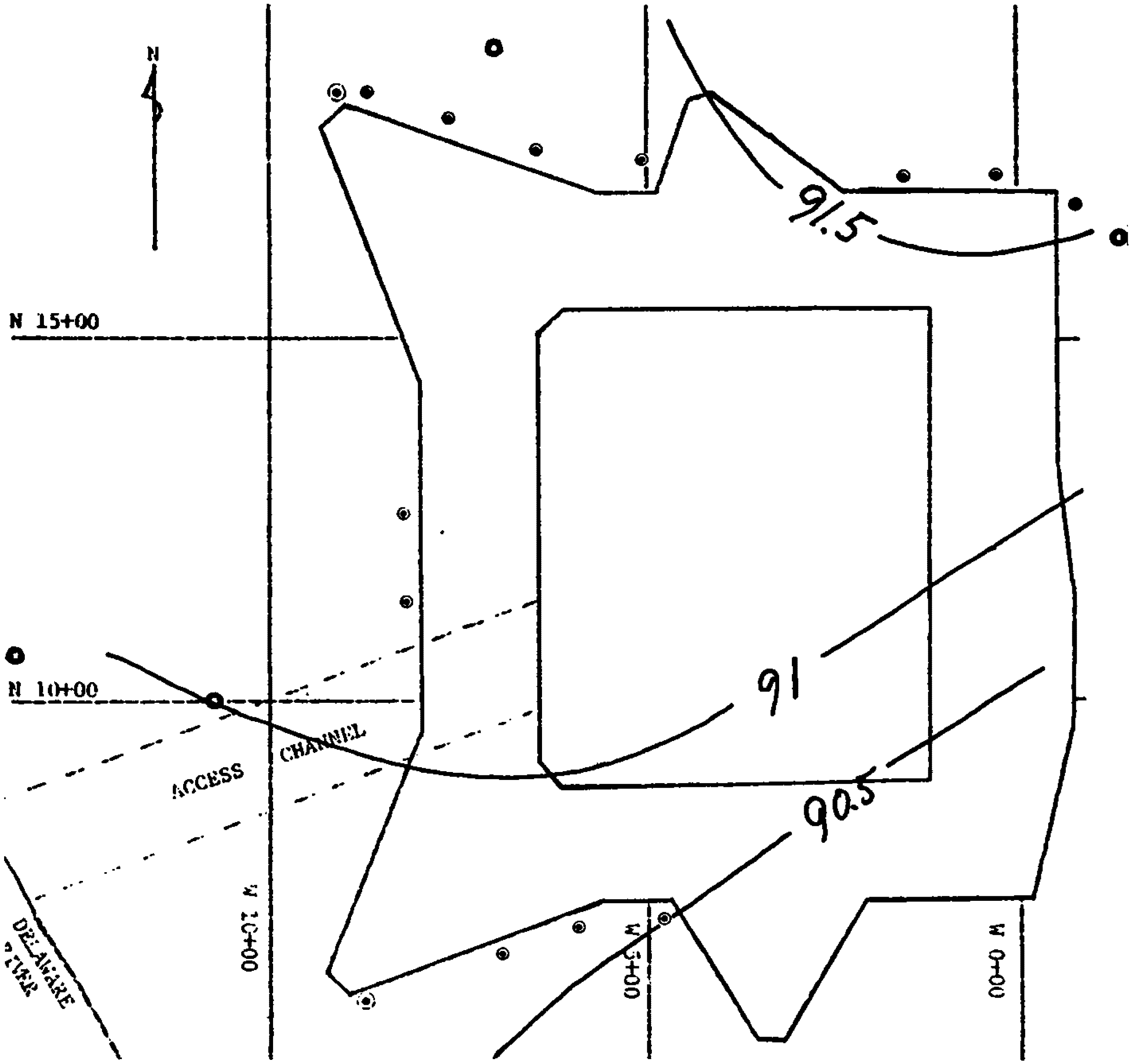
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

SHALLOW AQUIFER
PIEZOMETRIC LEVELS – 8/24/76

UPDATED FSAR

FIGURE 2.4-18



DATE August 24, 1976

TIME 0900 hrs. EST

Pool Level Elevation = 93.8 PSD

EXPLANATION:

- Deep Observation Well
- Hydrostatic Pressure Cell Piezometer
- Point Piezometer
- Other Observation Well

55

Piezometric Surface Contour Line (Elevation in ft. PSD) estimated from water level measurements in the depicted wells and piezometers.

0 100 200 300

SCALE (in feet)

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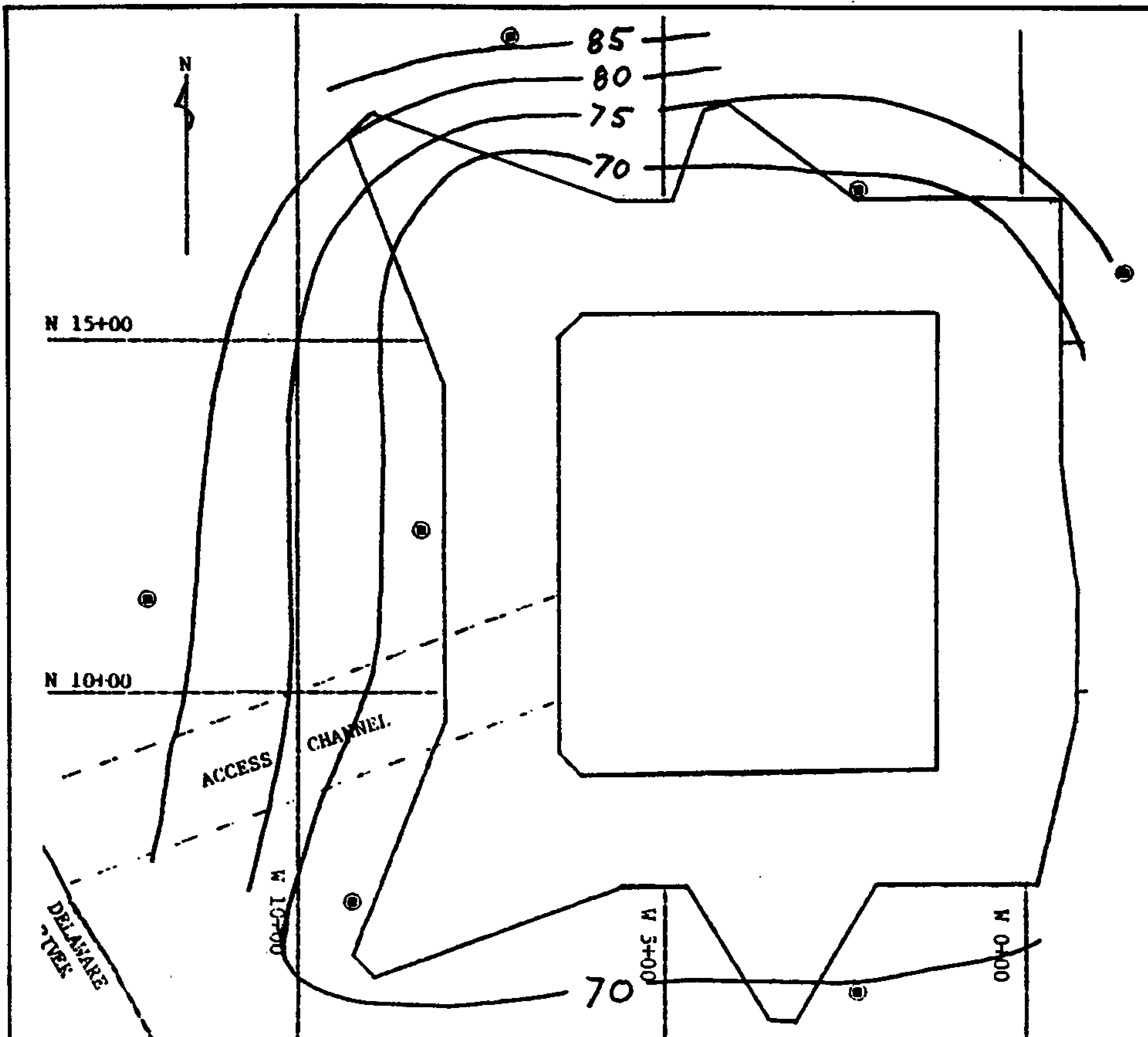
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

VINCENTOWN AQUIFER
PIEZOMETRIC LEVELS – 8/24/76

UPDATED FSAR

FIGURE 2.4-19



EXPLANATION:

◉ Shallow Observation Well

● Other Observation Well

—— 90 ——

Piezometric Surface Contour Line (Elevation in ft. PSD) estimated from water level measurements in the depicted wells

DATE June 21, 1977

TIME 1000 hrs. EST

0 100 200 300

SCALE
(in feet)

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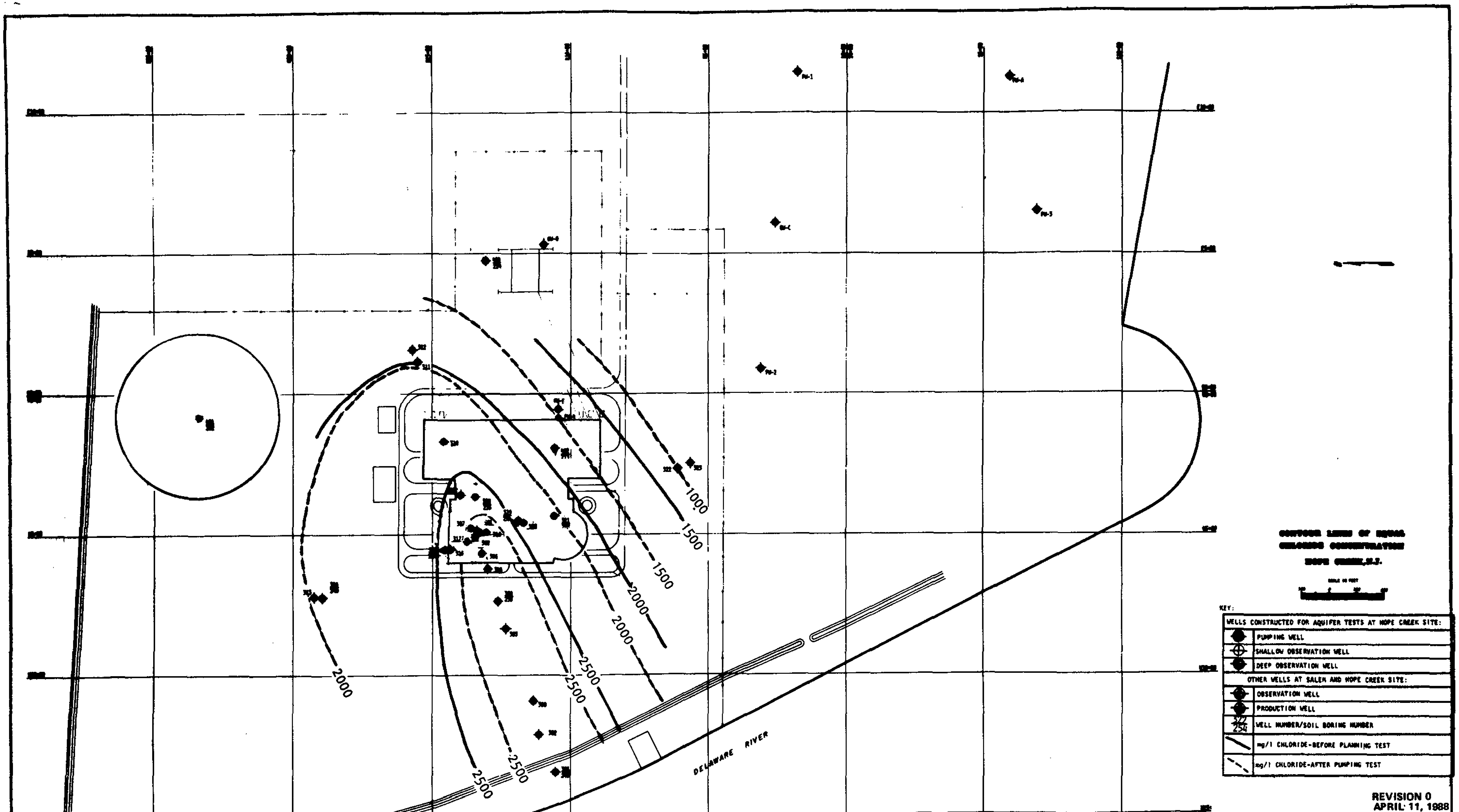
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

SHALLOW AQUIFER
PIEZOMETRIC LEVELS – 6/21/77

UPDATED FSAR

FIGURE 2.4-20



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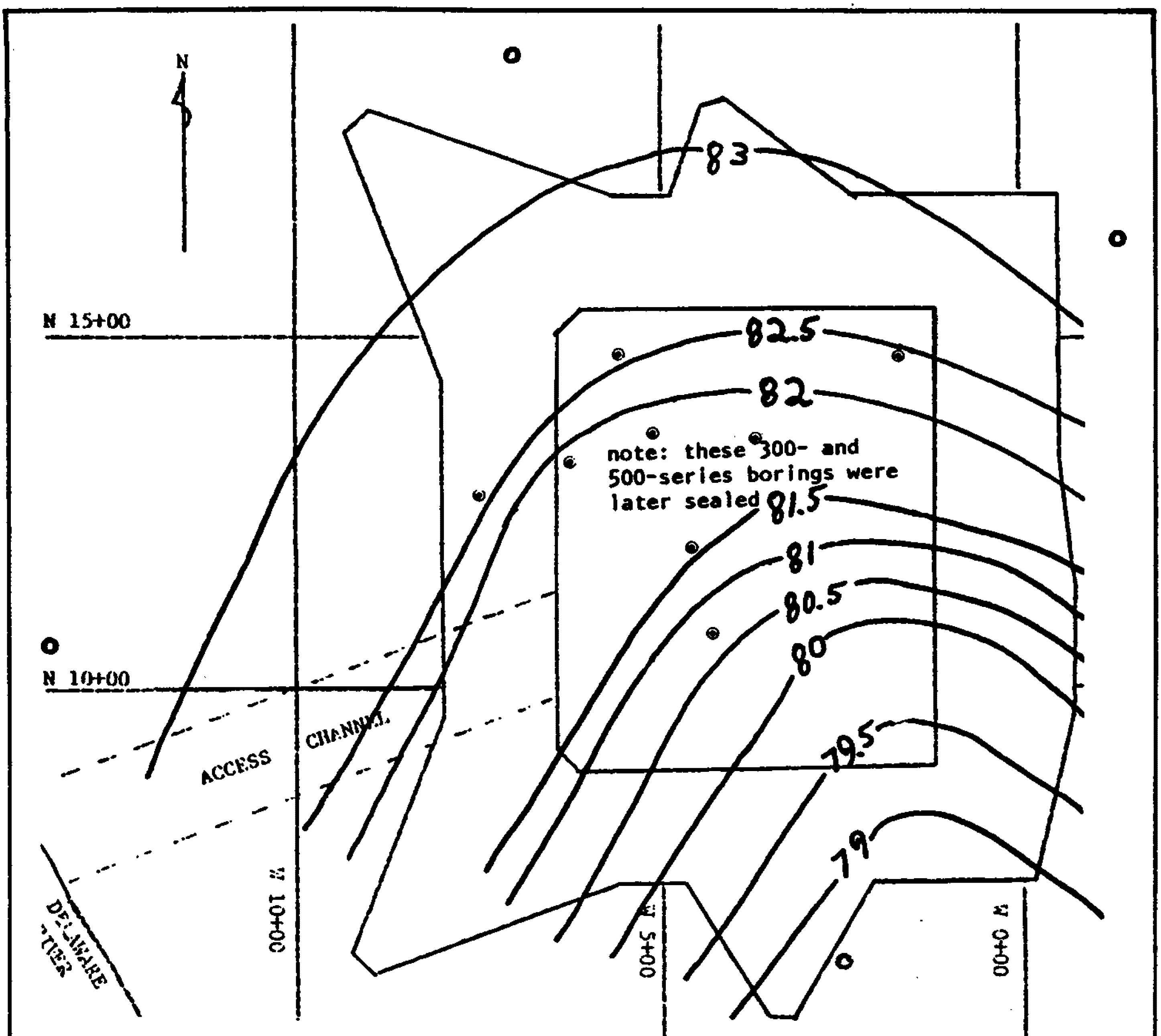
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

GROUNDWATER QUALITY MAP
FOR THE SHALLOW AQUIFER

UPDATED FSAR

FIGURE 2.4-21



EXPLANATION:

- Deep Observation Well
- Hydrostatic Pressure Cell Piezometer
- Point Piezometer
- Other Observation Well

55

Piezometric Surface Contour Line (Elevation in ft. PSD) estimated from water level measurements in the depicted wells and piezometers.

0 100 200 300

SCALE
(in feet)

DATE April 22, 1975

TIME _______________ EST

Contours taken from Figure 14 of Stage 1 Report dated June 21, 1976.

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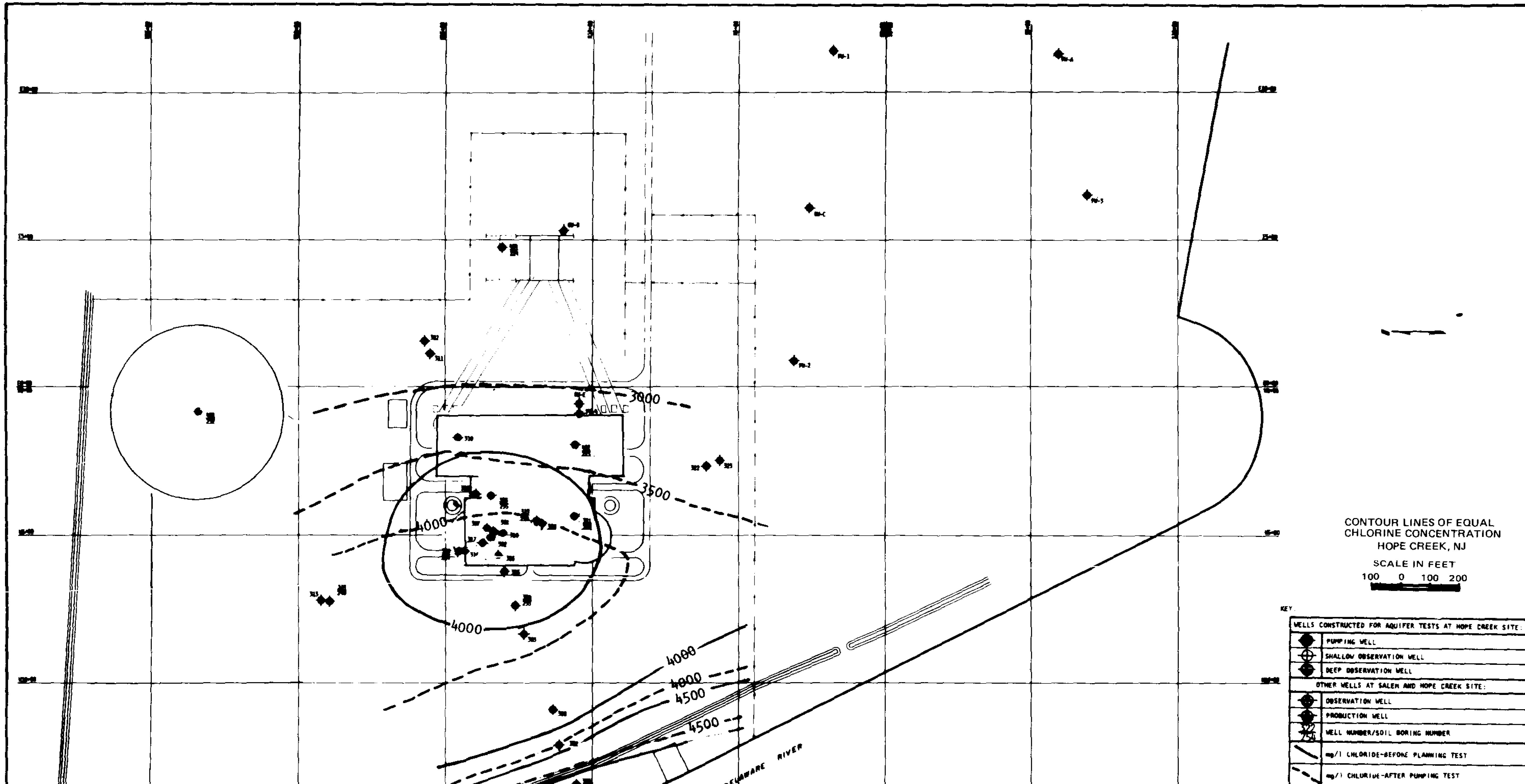
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

VINCENTOWN AQUIFER
PIEZOMETRIC LEVELS – 4/22/75

UPDATED FSAR

FIGURE 2.4-22



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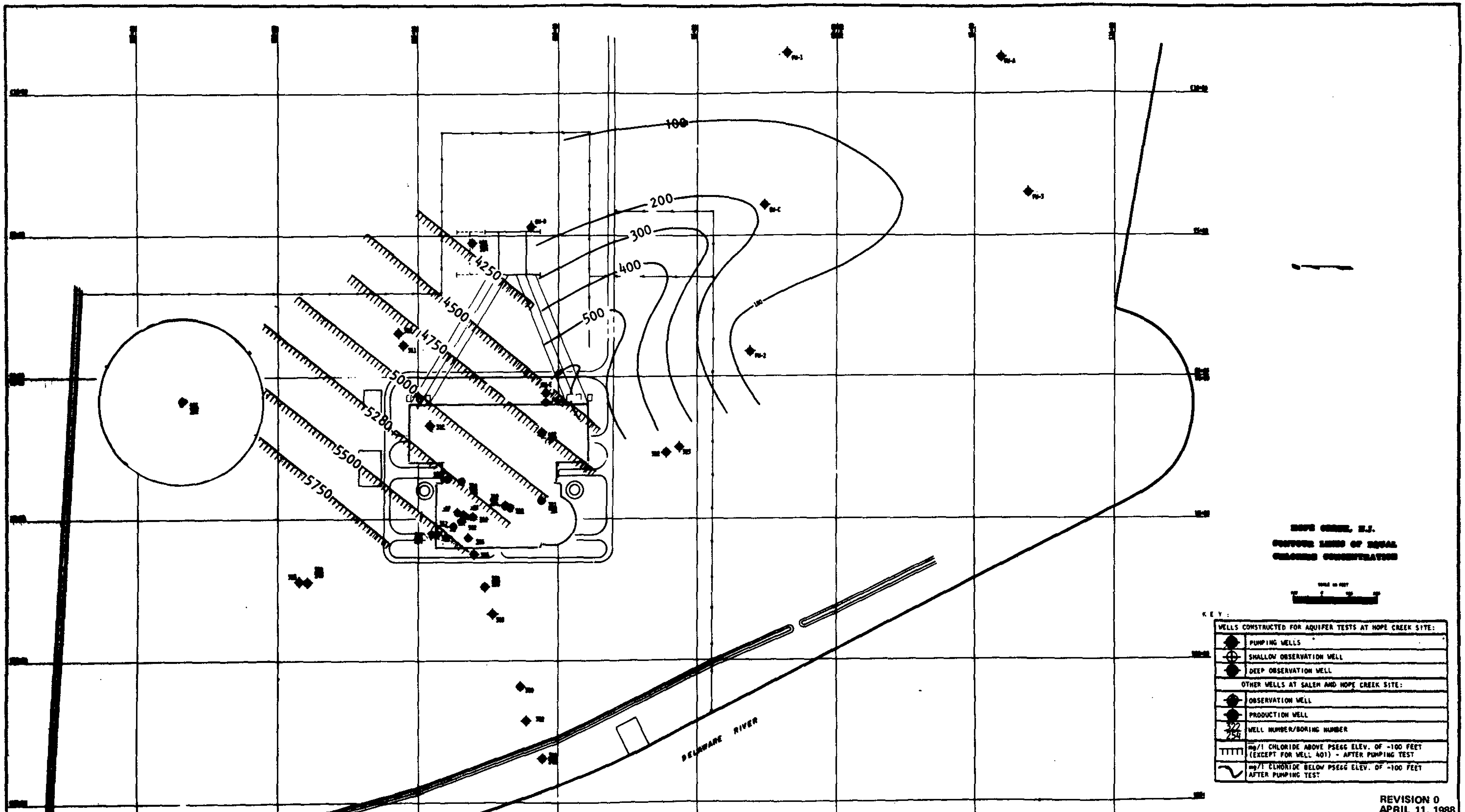
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

GROUNDWATER QUALITY MAP
FOR THE VINCENTOWN AQUIFER

UPDATED FSAR

FIGURE 2.4-23



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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

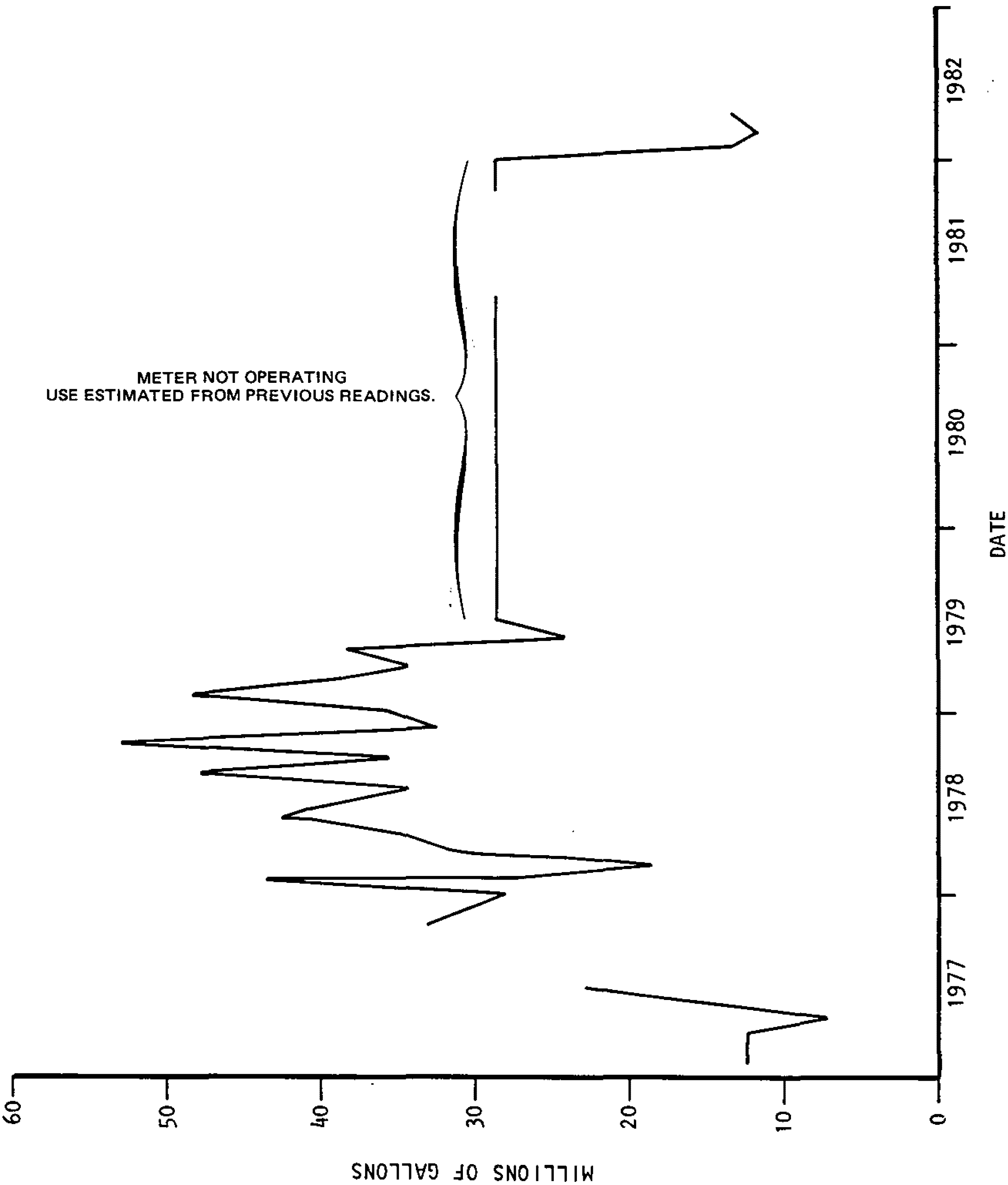
GROUNDWATER QUALITY MAP FOR THE MOUNT LAUREL – WENONAH AQUIFER

UPDATED FSAR

FIGURE 2.4-24

Figure F2.4-25 intentionally deleted.

Refer to Plant Drawing C-5018-0 in DCRMS



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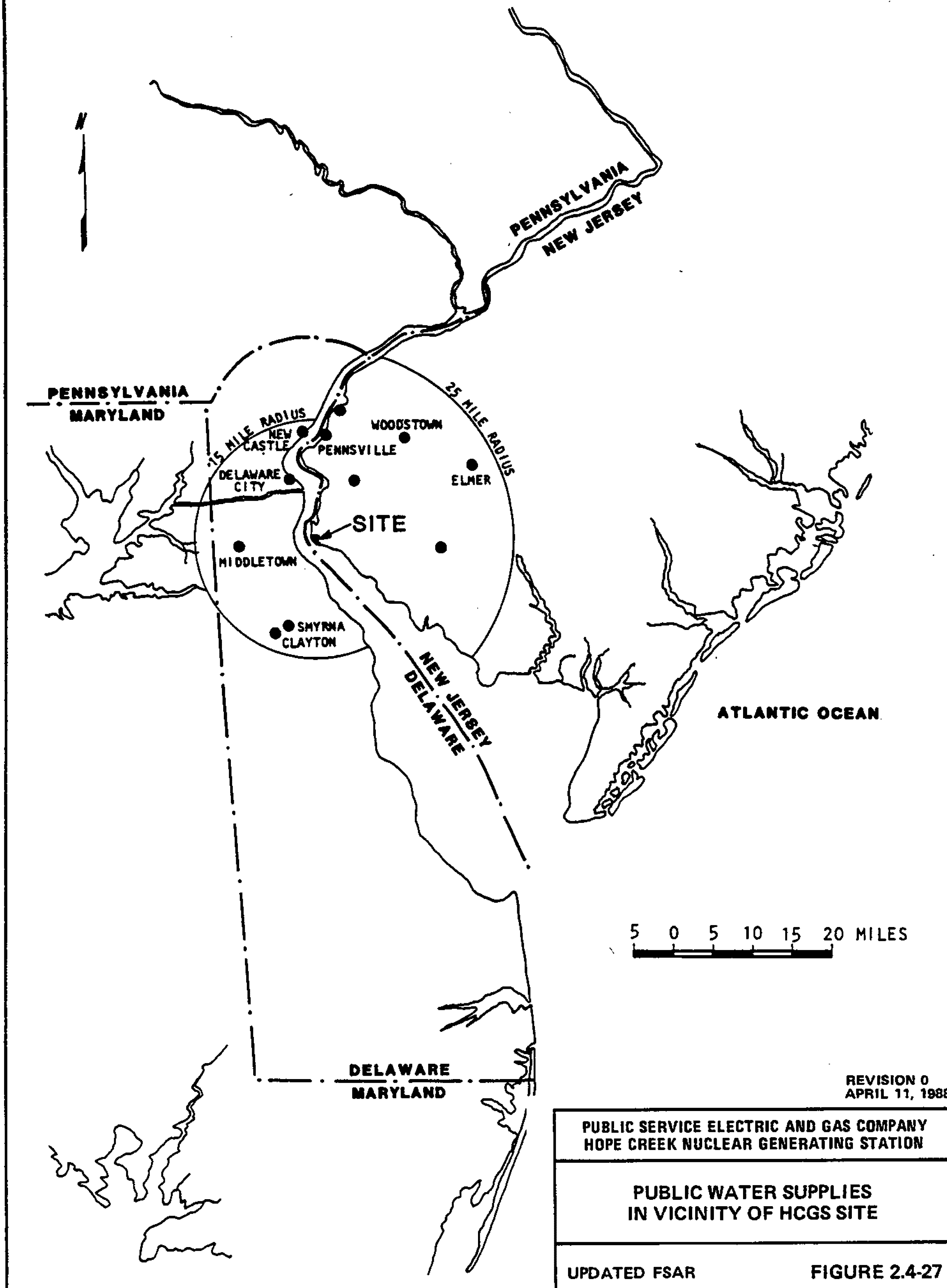
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

HISTORICAL USE OF WELL WATER
BY SNGS

UPDATED FSAR

FIGURE 2.4-26



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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

PUBLIC WATER SUPPLIES IN VICINITY OF HCGS SITE

UPDATED FSAR

FIGURE 2.4-27

SECURITY - RELATED
INFORMATION WITHHELD
UNDER 10 CFR 2.390

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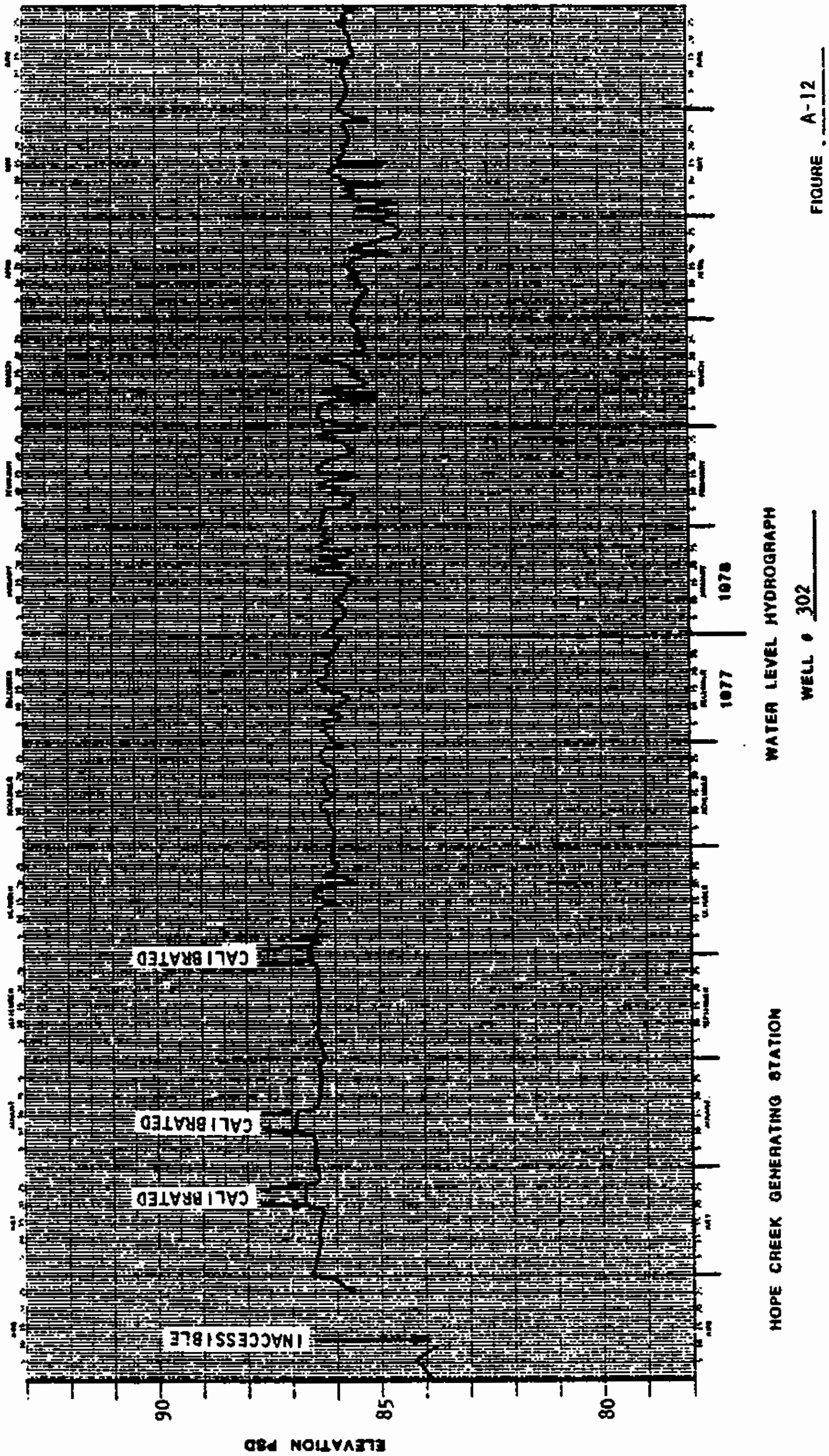
APRIL 11, 1988

PSEG NUCLEAR, L.L.C. HOPE CREEK NUCLEAR GENERATING STATION
MAP OF AREA-KNOWN WATER WELLS IN NEW JERSEY IN VICINITY OF SITE
Updated FSAR Fig. 2.4-28



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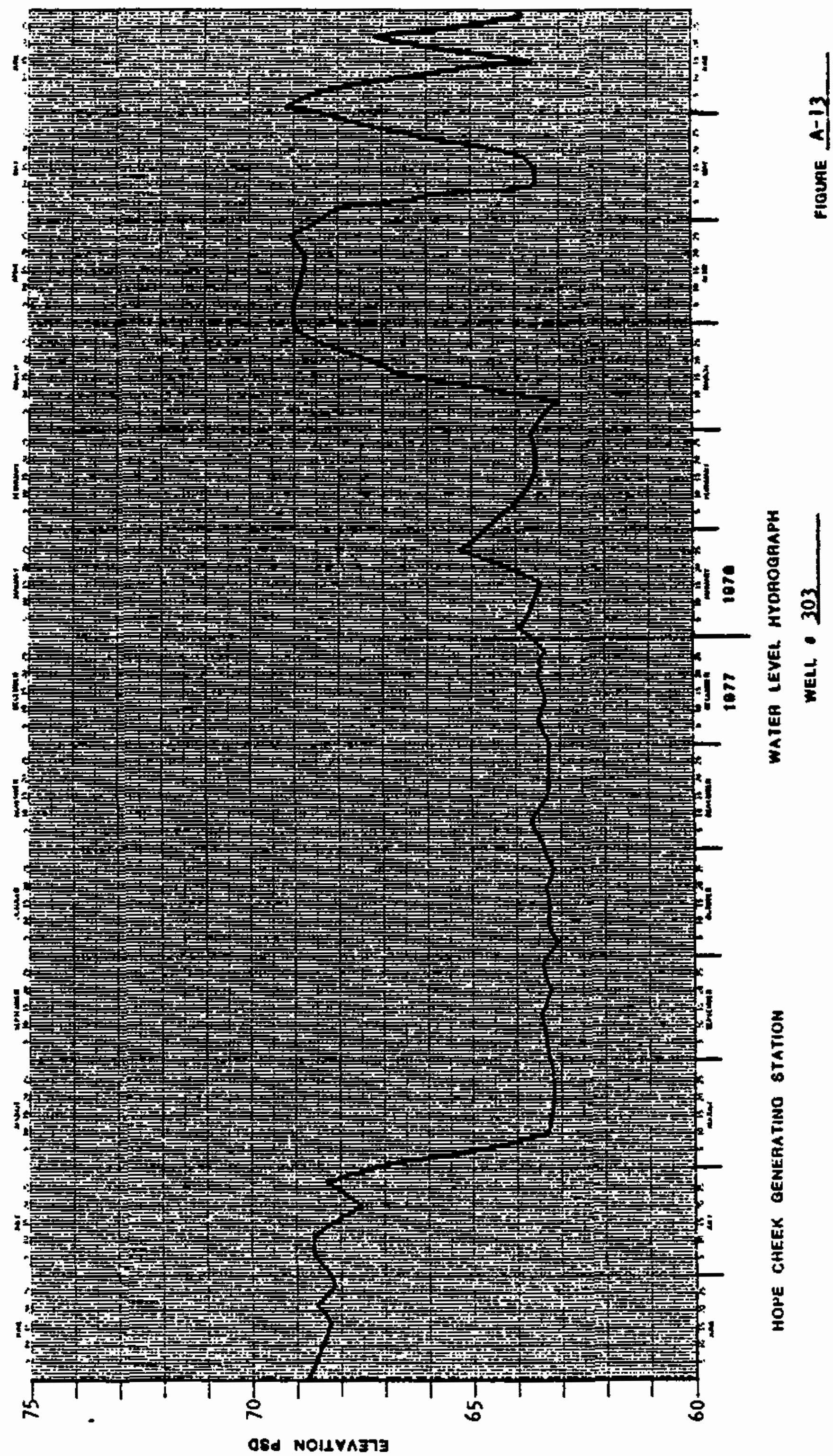
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

WATER WELL HYDROGRAPH
WELL NO. 302

UPDATED FSAR

FIGURE 2.4-29



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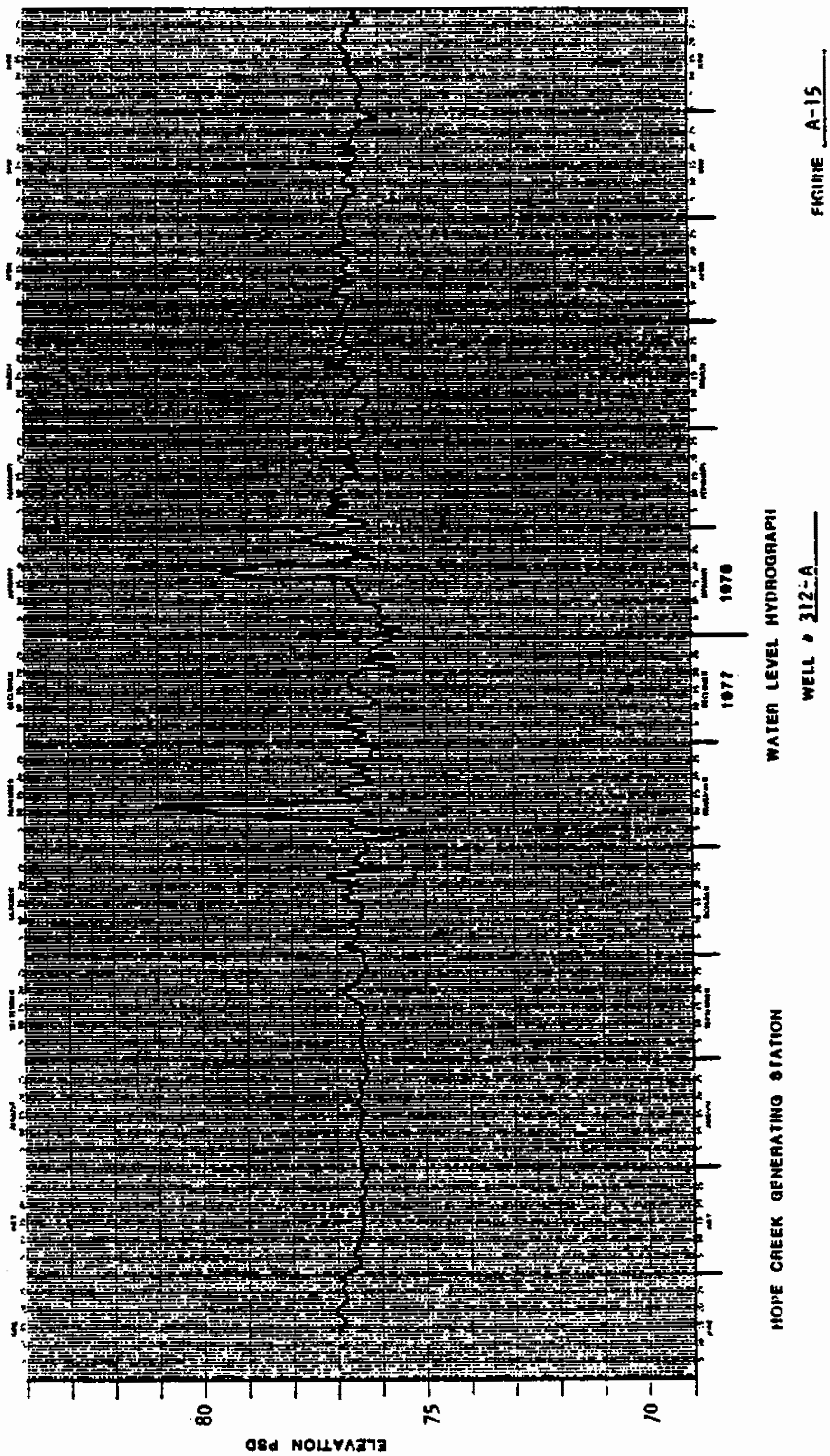
HOPE CREEK NUCLEAR GENERATING STATION

WATER LEVEL HYDROGRAPH
WELL NO. 303

UPDATED FSAR

FIGURE 2.4-30



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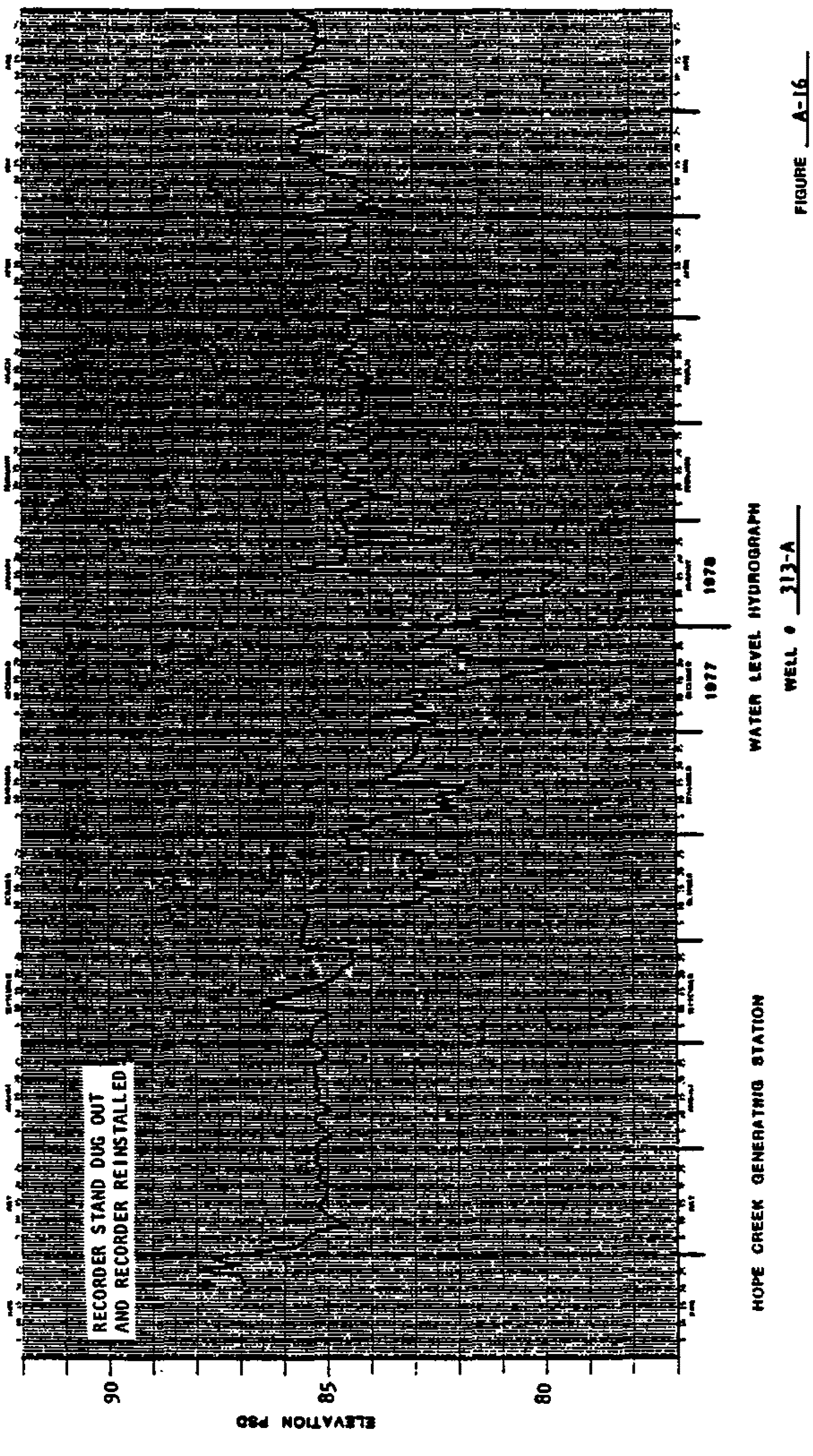
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

WATER LEVEL HYDROGRAPH
WELL NO. 312-A

UPDATED FSAR

FIGURE 2.4-31



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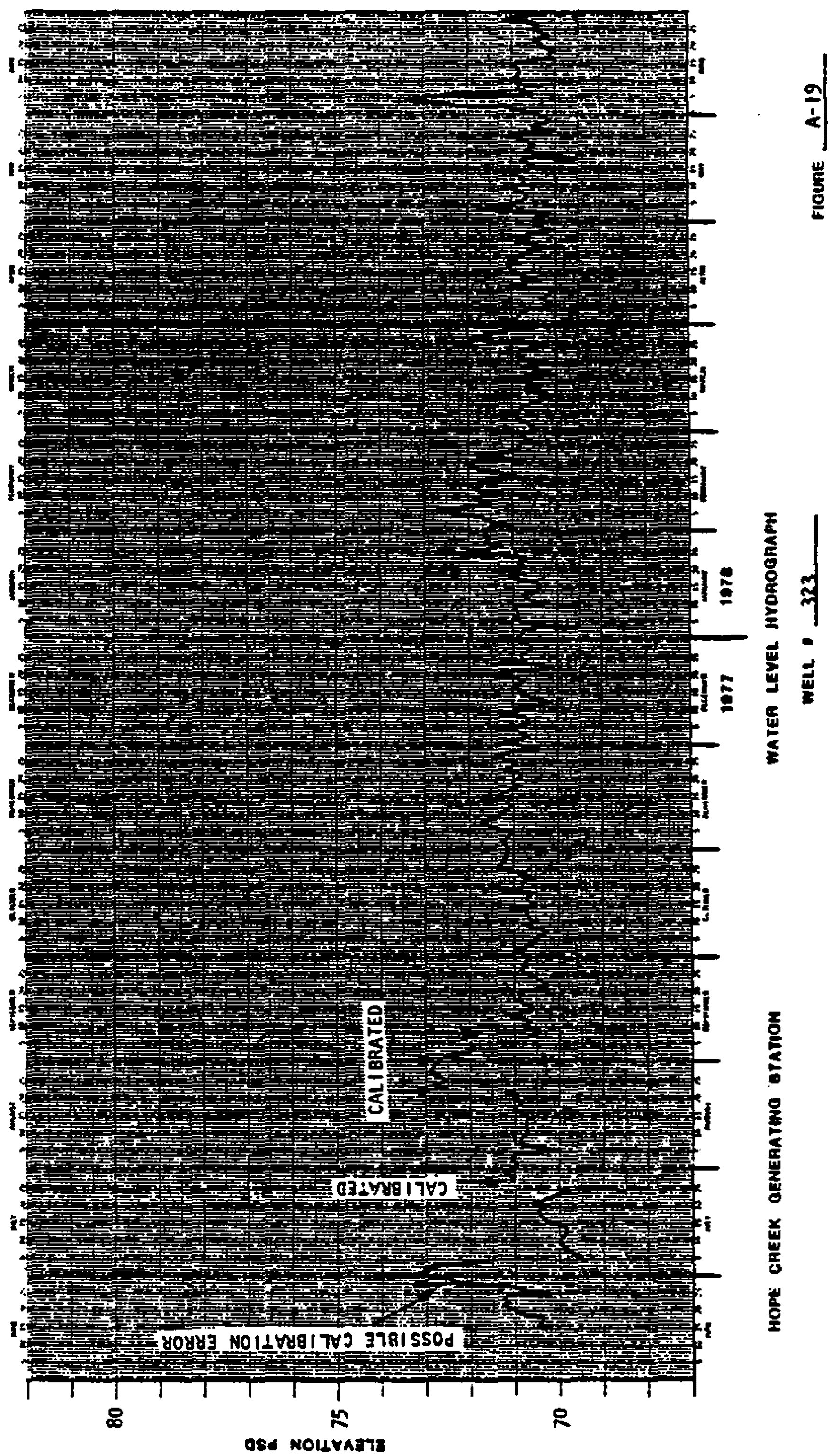
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

WATER LEVEL HYDROGRAPH
WELL NO. 313-A

UPDATED FSAR

FIGURE 2.4-32



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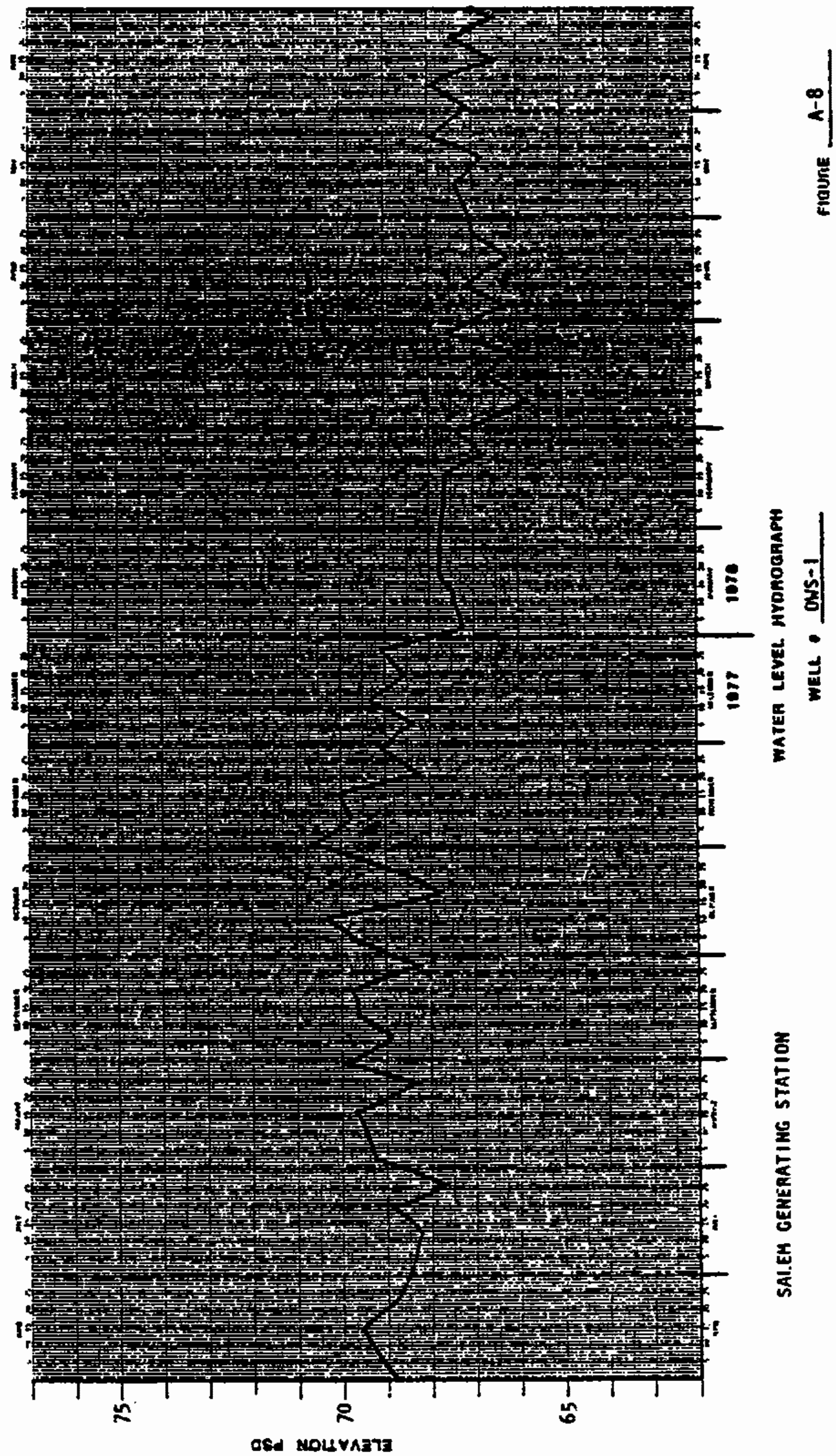
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

WATER LEVEL HYDROGRAPH
WELL NO. 323

UPDATED FSAR

FIGURE 2.4-33



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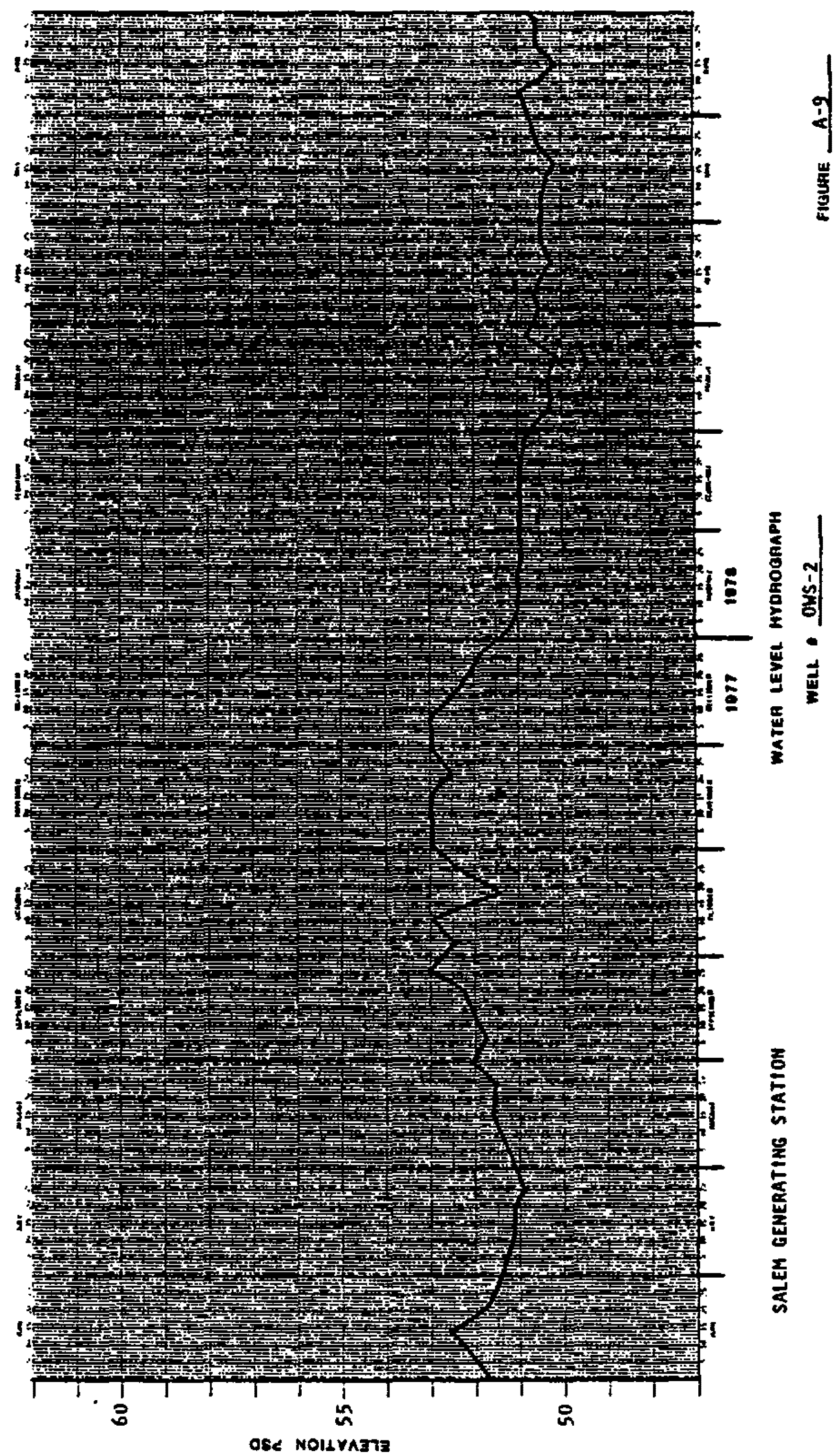
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

WATER LEVEL HYDROGRAPH
WELL NO. OWS-1

UPDATED FSAR

FIGURE 2.4-34



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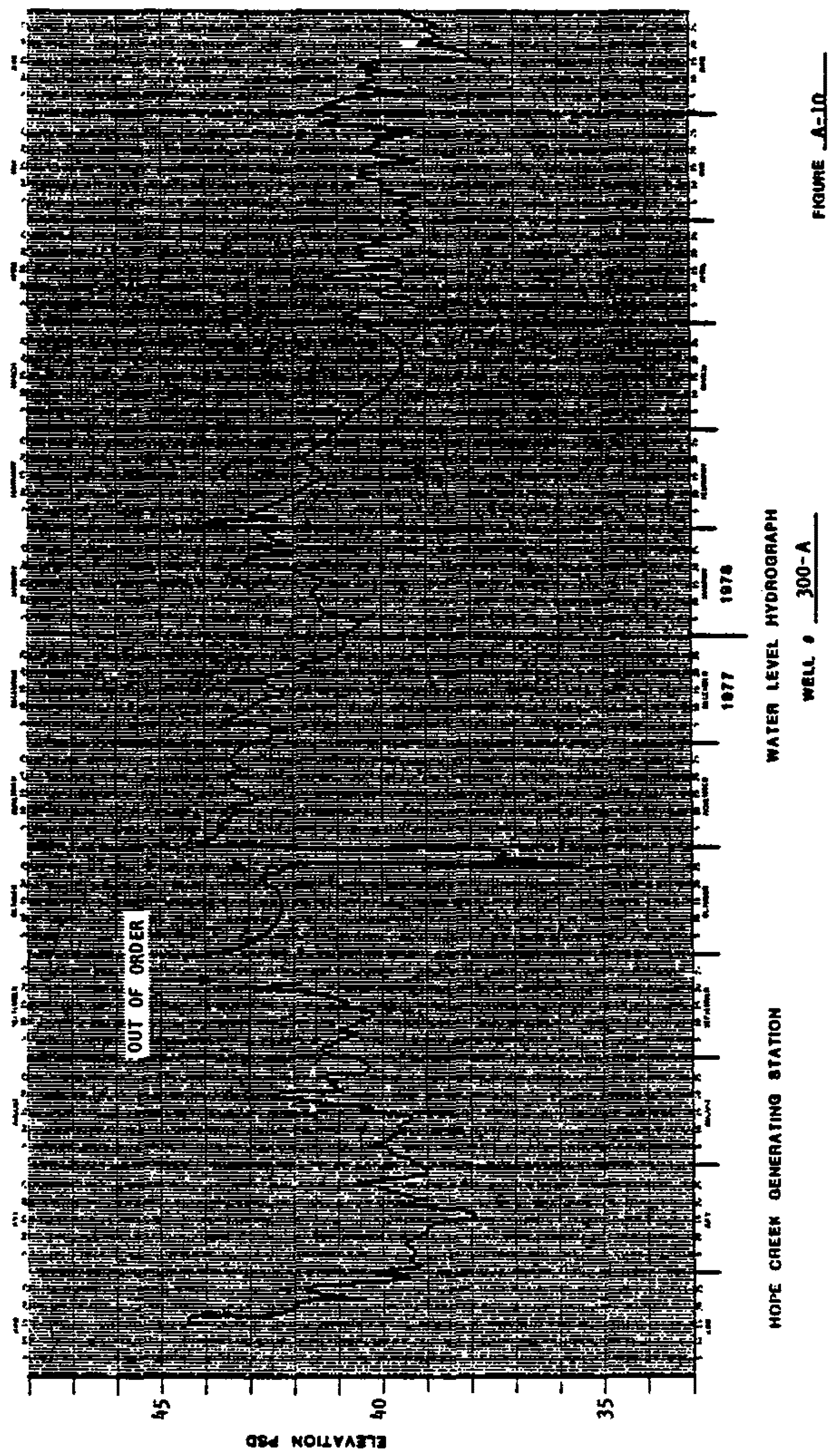
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

WATER LEVEL HYDROGRAPH
WELL NO. OWS-2

UPDATED FSAR

FIGURE 2.4-35



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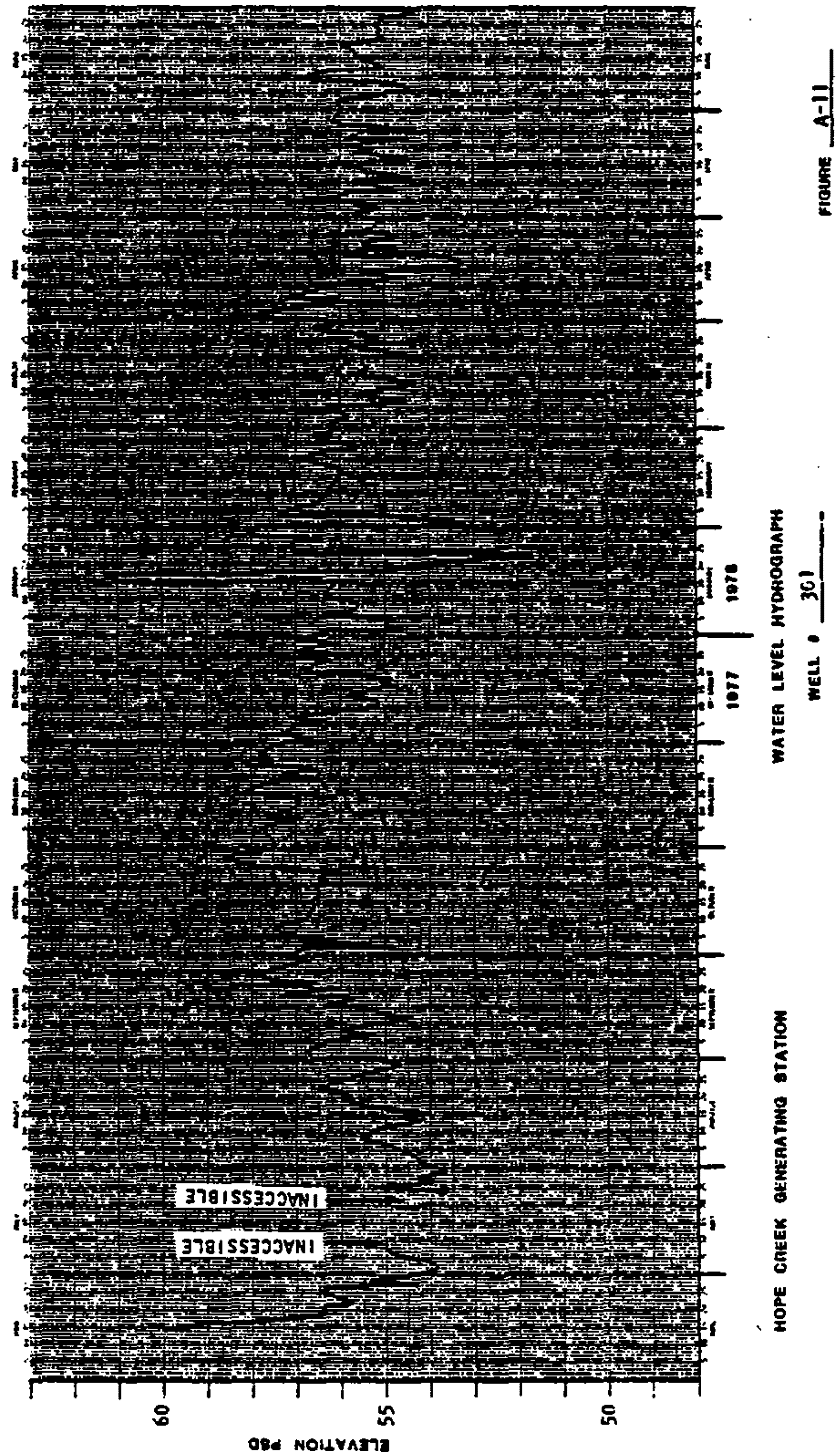
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

WATER LEVEL HYDROGRAPH
WELL NO. 300-A

UPDATED FSAR

FIGURE 2.4-36



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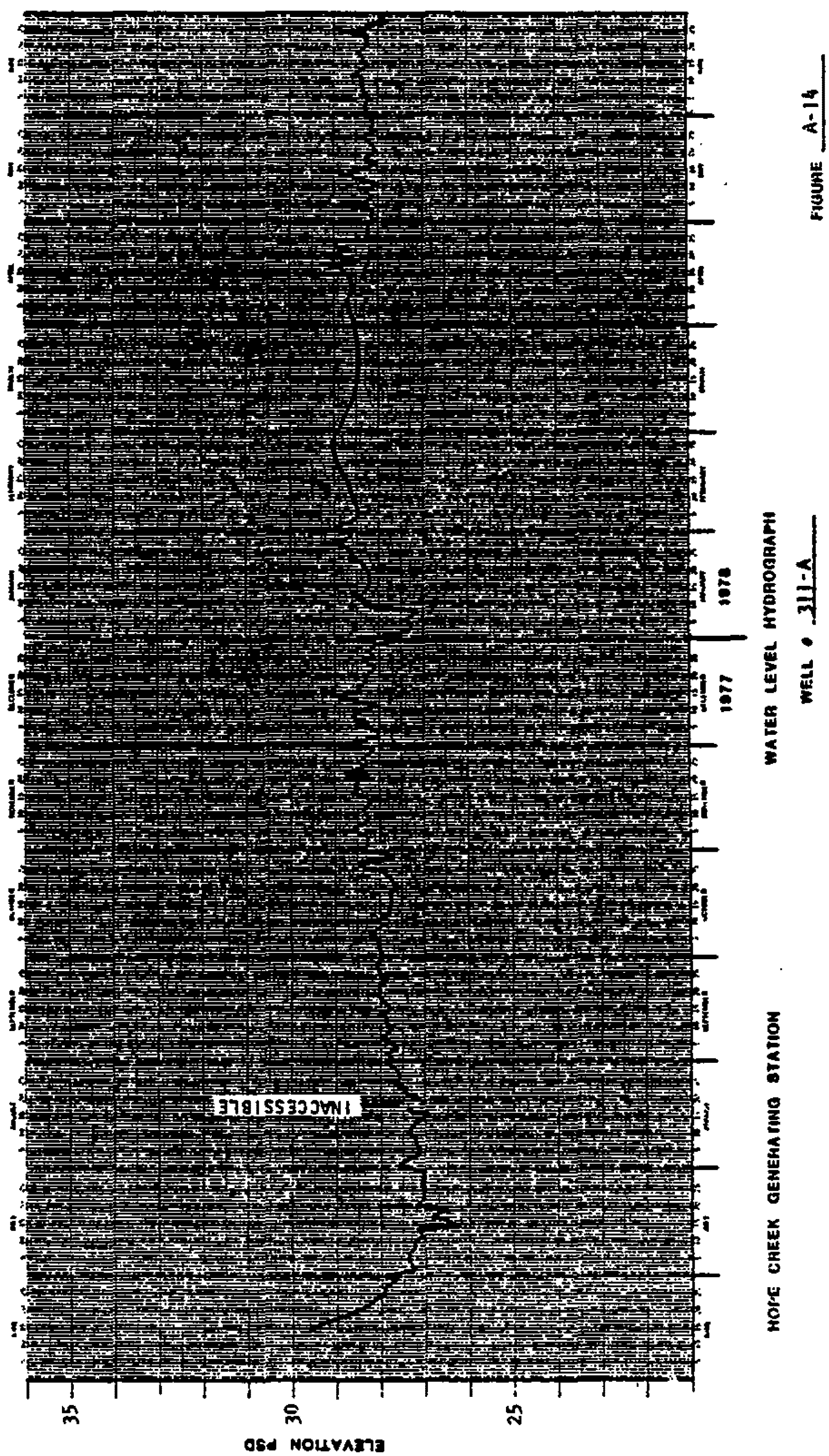
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

WATER LEVEL HYDROGRAPH
WELL NO. 301

UPDATED FSAR

FIGURE 2.4-37



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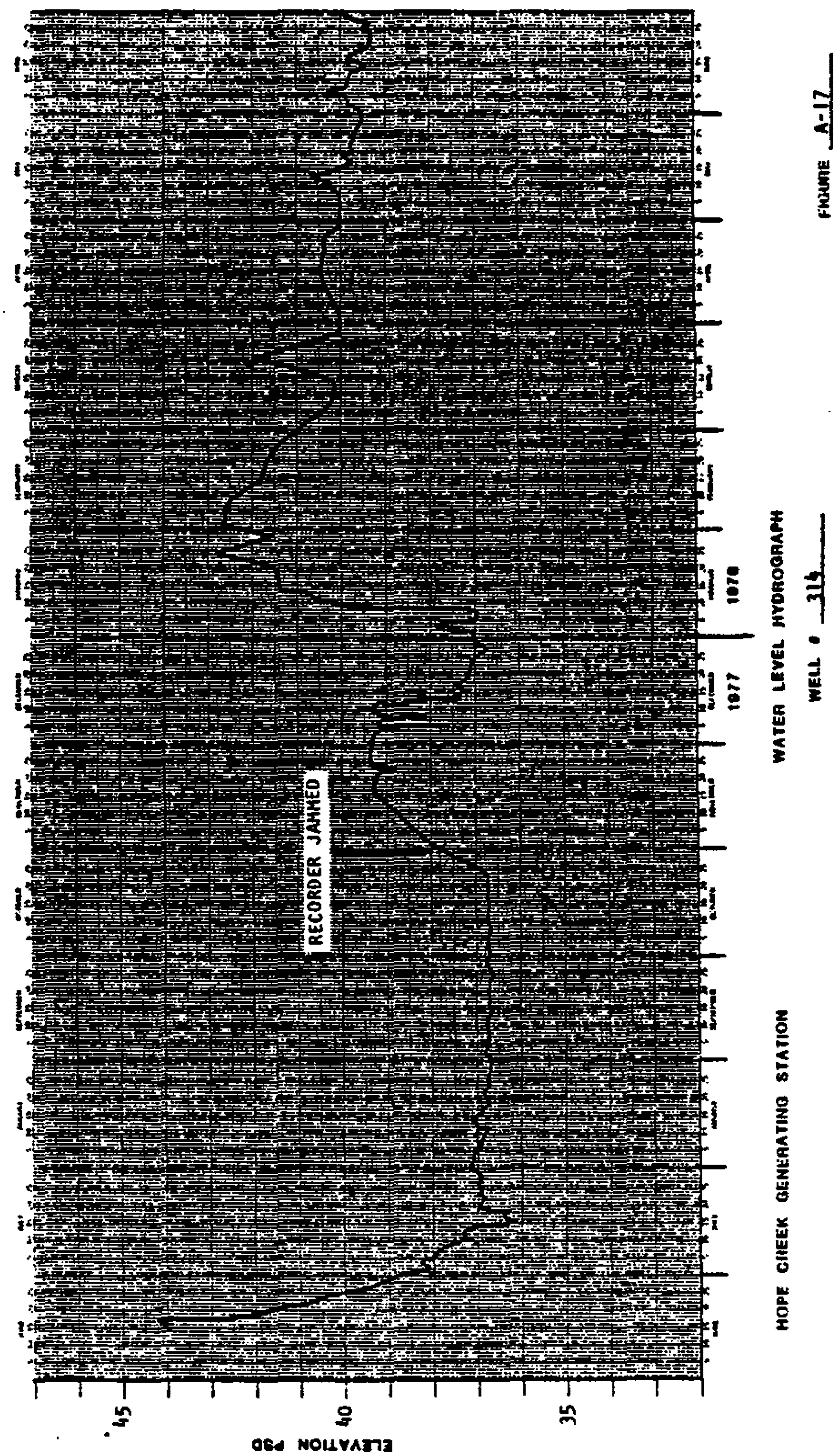
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

WATER LEVEL HYDROGRAPH
WELL NO. 311-A

UPDATED FSAR

FIGURE 2.4-38



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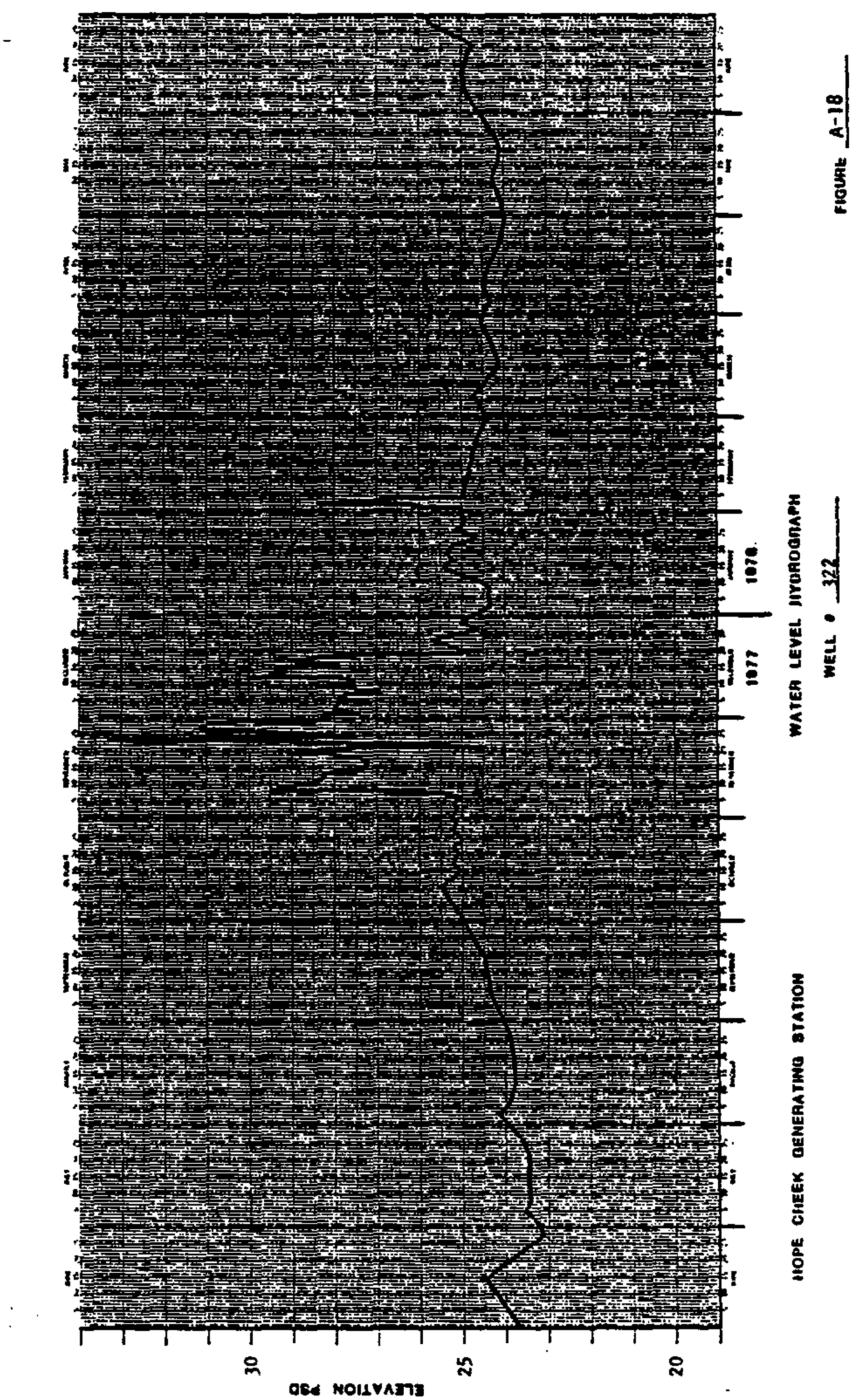
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

WATER LEVEL HYDROGRAPH
WELL NO. 314

UPDATED FSAR

FIGURE 2.4-39



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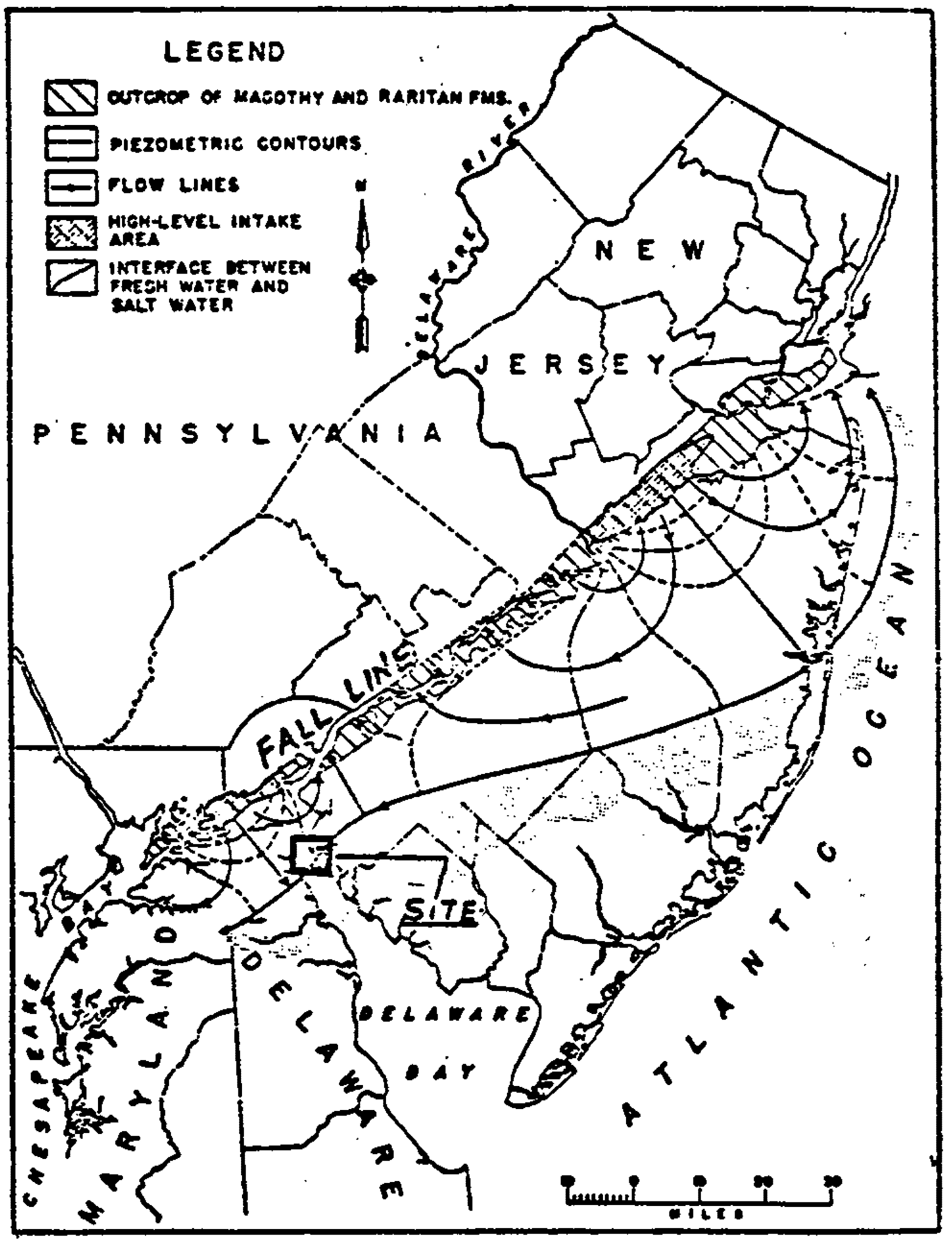
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

WATER LEVEL HYDROGRAPH
WELL NO. 322

UPDATED FSAR

FIGURE 2.4-40



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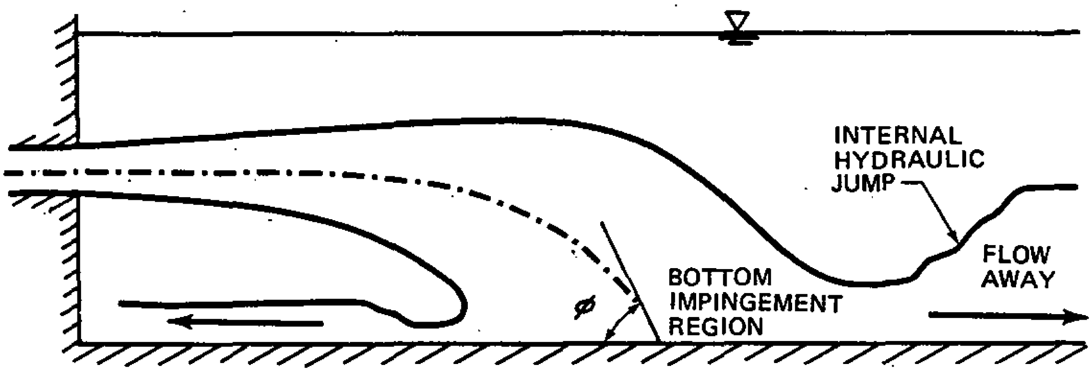
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

REGIONAL MAP – THEORECTICAL FLOW PATTERN, LOCATION OF INTERFACE BETWEEN FRESH WATER & SALT WATER IN RARITAN & MAGOTHY FORMATIONS BEFORE ARTIFICIAL WITHDRAWALS OF WATER

UPDATED FSAR

FIGURE 2.4-41



SOURCE: REF. 2.4-60

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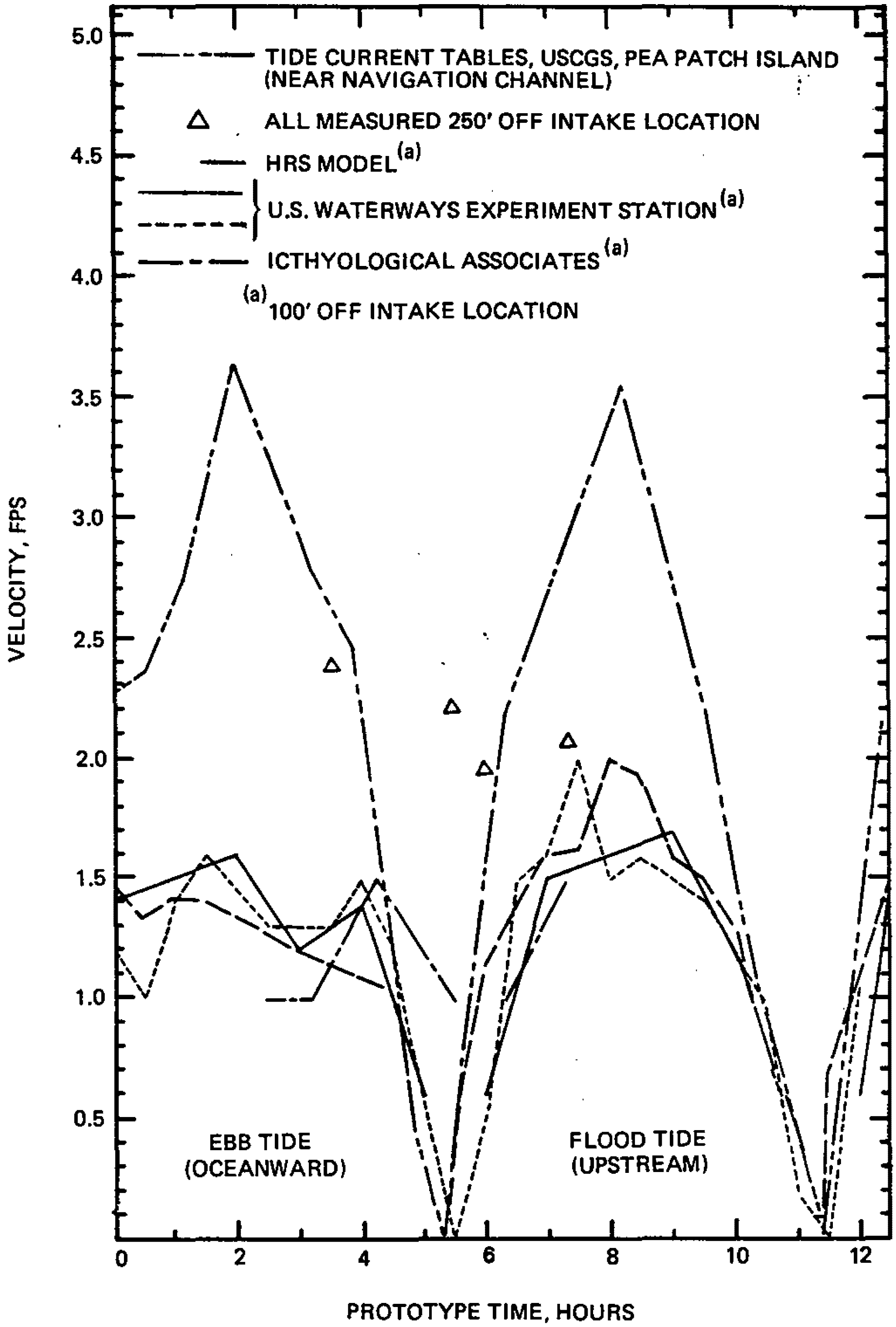
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

JET CONFIGURATION FOR LOW DISCHARGE VELOCITY AND HIGH DENSITY EXCESS OVER AMBIENT

UPDATED FSAR

FIGURE 2.4-42



SOURCE: REF. 2.4.12-3

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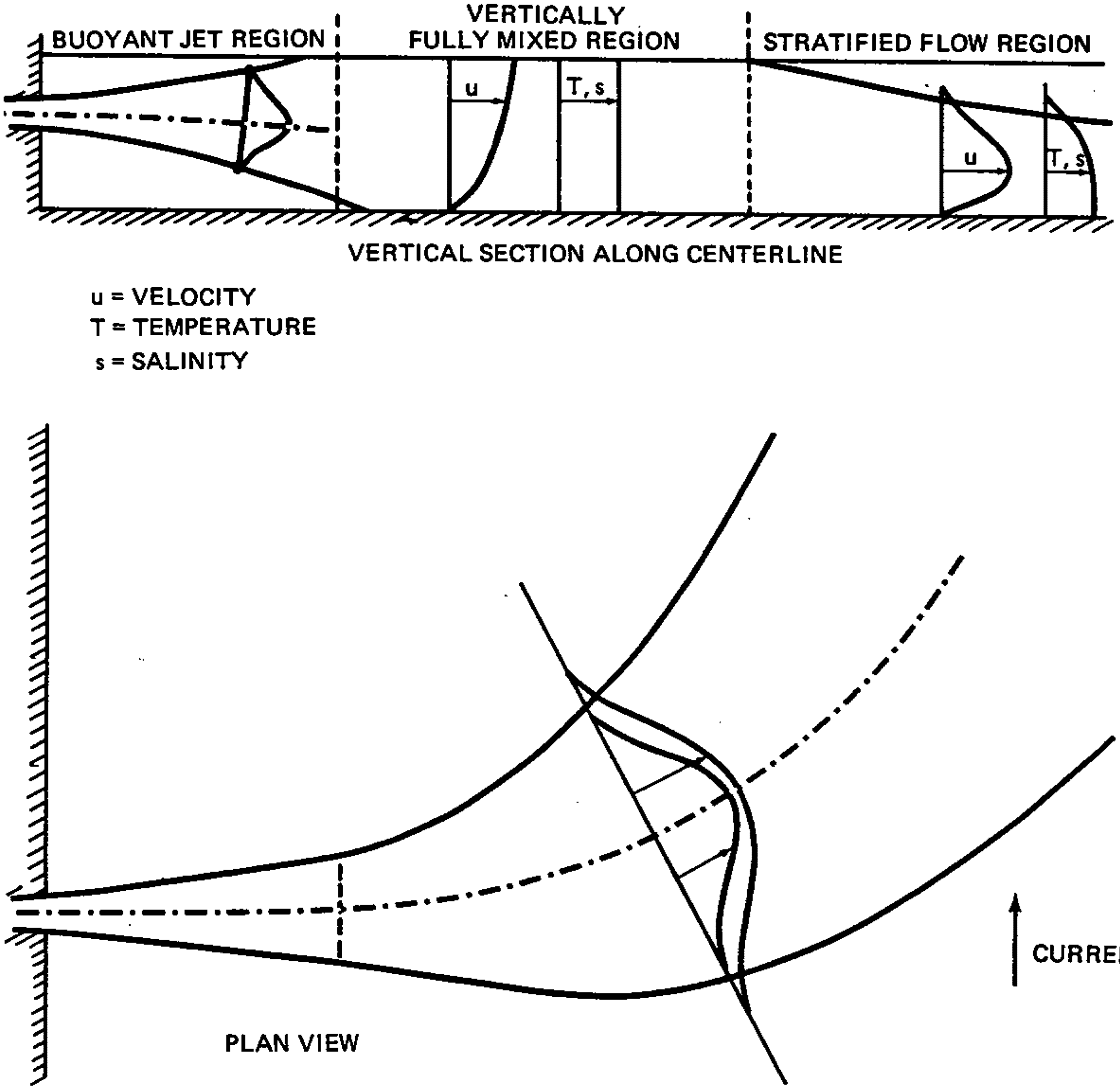
HOPE CREEK NUCLEAR GENERATING STATION

DELAWARE RIVER FLOW VELOCITY

UPDATED FSAR

FIGURE 2.4-43



SOURCE: REF. 2.4-60

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

SCHEMATIC STRUCTURE OF
HOPE CREEK BLOWDOWN DISCHARGE

UPDATED FSAR

FIGURE 2.4-44

2.5 GEOLOGY, SEISMOLOGY, AND GEOTECHNICAL ENGINEERING

In accordance with the criteria provided in Appendix A of 10CFR100, Seismic and Geologic Siting Criteria for Nuclear Power Plants, and NRC Regulatory Guide 1.70, Revision 3, this section of the FSAR describes and evaluates the geologic and seismologic conditions for the region around Hope Creek Generating Station (HCGS). It also evaluates the geotechnical engineering aspects and foundation conditions at the site and describes the foundation design. The information presented in this section was prepared by Dames and Moore and principally consists of summaries of the results of detailed investigations prepared by Dames and Moore that have been described in previously docketed reports. However, certain sections are based on a literature review and have been updated as necessary to include new information reported in the literature.

This section provides the appropriate information to demonstrate that the geological, seismologic, and geotechnical engineering evaluations of the site are of sufficient detail to ensure the safe design and operation of the nuclear power facility. The geology of the site is consistent with that of the surrounding region (Section 2.5.1). There is no indication of faulting or folding in the site area, no evidence was identified that indicated adverse behavior of the surficial subsurface materials during prior earthquakes (Section 2.5.1.2).

As discussed in Section 2.5.2, the HCGS site lies within a region that has experienced relatively few earthquakes. The site has experienced minor ground motion of no more than a few percent of gravity in the past, and may be subjected to minor ground accelerations in the future. The selection of the safe shutdown earthquake as a Modified Mercali Intensity VII shock with an epicenter at the site and a 0.20g horizontal ground acceleration at foundation level is considered conservative in light of tectonic setting of the site area (Section 2.5.2). Based on the data, interpretations, and conclusions contained in Section 2.5.1 and

2.5.2, it is stated that there is no capable fault within 5 miles of the site (Section 2.5.3).

The site is considered suitable for the construction of a nuclear power facility from the standpoint of geotechnical engineering. The characteristics of the foundation materials have been investigated and found to be acceptable, including their suitability for supporting the structures, the depth and configuration of the ground water table, and the characteristic effect of the foundation materials on the migration of radioactive solutions, should such solutions come in contact with them (Section 2.5.4).

Major plant structures are supported on mat foundations. Mat loadings, foundation grades, and ultimate bearing capacities are discussed in Section 2.5.4. Evaluations of the behavior of the sandy soils under earthquake loadings show adequate margin of safety against liquefaction (Section 2.5.4). Settlement computations based on test data show total and differential settlements to be within acceptable limits. Results of the settlement analysis are presented in Section 2.5.4. No evidence exists suggesting that the site could be subject to potential surface or subsurface subsidence.

2.5.1 Basic Geologic and Seismic Information

The basic geologic and seismic information presented in this section is required for the evaluations contained in the sections that follow. The information is subdivided into two categories. One addresses the regional geologic setting (Section 2.5.1.1) and the other presents data specifically concerning the geologic character of the site (Section 2.5.1.2).

2.5.1.1 Regional Geology

This section discusses the physiography, stratigraphy, structural geology, and geologic history of the site region. The information presented is primarily derived from a review of the current published literature. Summaries of these aspects of the regional geology are presented to provide the framework for the evaluation of geologic, seismologic, and man-made hazards in succeeding sections of the FSAR.

2.5.1.1.1 Regional Physiography

The HCGS site is located on Artificial Island, a man-made promontory on the east bank of the Delaware River, approximately 18 miles south of Wilmington, Delaware (Figure 2.5-1). The site is situated within the Atlantic Coastal Plain Physiographic Province (Figure 2.5-2). This province encompasses the area from the Fall Line to the coastline. The Fall Line separates the Coastal Plain from the Piedmont, New England, and Valley and Ridge Physiographic Provinces, which are located successively to the northwest. The coastline is the boundary between the Coastal Plain and the Continental Shelf, Slope, and Rise (Figure 2.5-2).

The Atlantic Coastal Plain Physiographic Province is characterized by low lying, gently rolling terrain developed on unconsolidated sediments of Cretaceous, Tertiary, and Quaternary age. Topographic relief is generally less than 200 feet, and the topographic gradient is usually less than 5 ft/mi. Northeast of the Chesapeake Bay, the Coastal Plain is made up of extensive tracks of nearly level plain, less than 100 feet above sea level. This morphology has resulted from deposition and erosion associated with the rise and fall of Pleistocene sea levels. Southwest of Chesapeake Bay, marine and fluvial terraces developed during the Pliocene and Pleistocene Epochs are common. As a result of the post-Pleistocene sea level rise, the outline of the present day coastline is controlled by the configuration of drowned valleys typified by the deeply recessed Chesapeake and Delaware Bays. Exposed headlands and shorelines have been modified by the development of barrier islands and extensive lagoons.

Contemporary vertical crustal movements also contribute to geomorphologic development of the coastal plain. Analysis of precise leveling data (Reference 2.5-1) indicates that the Atlantic

Coastal Plain is currently tilting downward toward the ocean and to the north. Based on the geomorphic evidence, several analysts (References 2.5-2 and 2.5-3) have indicated that the region of the Chesapeake and Delaware Bays the rate of crustal subsidence in late Pleistocene time ranges from 2 to 10 millimeters/year. It is also suggested that the Piedmont province is experiencing differential uplift relative to the coastal plain at a lesser rate (Reference 2.5-4). Furthermore, the Appalachian Highlands appear to be rising at a rate of as much as 6 mm/yr relative to the Coastal Plain (Reference 2.5-1).

Vertical crustal movements along the eastern margin of North America are interpreted to be epeirogenic in character (Reference 2.5-3). The movements are considered oscillatory which may account for the apparently high rates when considered in the context of long periods of geologic time. However, the effects of these relatively short term oscillatory movements on long term epeirogenic movements is not known (Reference 2.5-3).

To the northwest, Piedmont Physiographic Province is immediately adjacent to the Coastal Plain. This province is an eroded plateau of low relief and rolling topography that slopes gently to the southeast. The Piedmont is subdivided into the Piedmont Complex and Triassic Lowland (Figure 2.5-2). The Piedmont Complex is an upland area underlain by metamorphosed sedimentary and crystalline rocks of Paleozoic and Precambrian age. These lithologies are relatively resistant, and their erosion has resulted in a moderately irregular surface. Topographically higher terrain is underlain by Cambrian quartzites and Precambrian crystalline rocks, while broad valleys are developed on carbonate lithologies. The second subdivision of the Piedmont Province is the less rugged lowland section, referred to as the Triassic Lowland. This physiographic subprovince is located northwest of the Piedmont Complex and consists of irregular shaped and fault controlled basins filled with sedimentary and igneous rocks of Triassic and Early Jurassic age. Valleys are developed on sandstone and shale strata and trend northeast-southwest parallel to the strike of the bedrock formations. Higher and more rugged terrain is underlain by intrusive and extrusive rocks consisting predominantly of diabase and basalt.

The western margin of Piedmont Province is marked by a northeast trending ridge system comprised of the Reading Prong and the New Jersey Highlands. Both of these areas are local subdivisions of the New England Physiographic Province (Figure 2.5-2). These highlands consist of a relatively narrow band of mostly Precambrian crystalline rocks comprising an area of rugged topographic relief. This ridge system separates the Piedmont Province from the Valley and Ridge Physiographic Province.

The Valley and Ridge Physiographic Province is underlain by a folded sequence of Paleozoic age sedimentary rocks. The northeast trending folds which characterize the province were formed during the most intense stages of the Appalachian orogeny. The province consists of mountainous terrain with moderate to severe topographic relief. Topographic relief is generally controlled by the relative resistances of the underlying strata.

The Appalachian Plateau Physiographic Province is situated northwest of the Valley and Ridge Province (Figure 2.5-2). This province consists of a deeply dissected sequence of relatively underformed Paleozoic age strata.

The Atlantic Continental Margin east of the Coastal Plain is comprised of three discrete physiographic subprovinces: the continental shelf, the continental slope, and the continental rise (Figure 2.5-2). The continental shelf is the submerged continuation of the Atlantic Coastal Plain and extends from the shoreline to the continental slope. The shelf is characterized by a small gradient, averaging less than 5 ft/mi, and many shallow water features that are relicts of lower sea levels. Off the New Jersey coast, the continental shelf varies in width from 70 to 80 miles. The 100 meter bathymetric contour effectively separates the shelf into two geomorphic regions. Southeast of this dividing line, bathymetric contours essentially follow the morphology of the shelf edge, while to the northwest, the majority of the shelf is characterized by linear, low relief features more closely aligned with the present shoreline (Reference 2.5-5). Beneath the continental shelf off New Jersey there is a large basement structural feature referred to as the Baltimore Canyon Trough (Figure 2.5-2).

The most striking features of the continental shelf in the region are the large submarine canyons that incise its seaward margin. Off the New Jersey coast, the most prominent are the Hudson, Wilmington, and Baltimore Canyons (Figure 2.5-2). The canyons cut the continental shelf in steep, winding, V-shaped gorges. They range in width from 1 to more than 10 miles and extend far down the continental slope. Only the Hudson Canyon crosses the entire continental shelf and is connected with a present day drainage system. Bottom contours at depths of less than 300 feet indicated that other rivers may have once connected with other submarine canyons (Reference 2.5-5). It is suggested that these canyons were eroded when the shelf was exposed during the Pleistocene glacial stages and that they were modified or eliminated by encroaching seas during the interglacial stages (Reference 2.5-5).

The continental slope is the topographic boundary between the continental shelf and the continental rise (Figure 2.5-2). It rises steeply, from about -6,500 foot depth at the top of the continental rise to approximately -500 to -600 foot depth at the continental shelf edge (Reference 2.5-5). Slope gradients range between 200 and 300 ft/mi.

The continental rise is a broad, gently sloping surface between the continental slope and the abyssal depths of the Atlantic Ocean. Gradients on the continental rise are generally less than 550 ft/mi (Reference 2.5-5). Surface relief on the rise is minimal, with the occasional exception of submarine canyons and associated fan like deposits that extend downward from the adjacent continental slope.

2.5.1.1.2 Regional Stratigraphy

The stratigraphic sequence of the Atlantic continental margin in site region consists of an eastward thickening wedge of sedimentary strata unconformably deposited on a seaward sloping basement complex surface. In the Baltimore Canyon Trough (Figure 2.5-2), these strata range from early Mesozoic to late Cenozoic and are as much as 18 kilometers thick (Reference 2.5-6). Formations generally thicken southeast of their outcrop on the Coastal Plain. The general dip of the strata is to the southeast and varies from as much as 120 ft/mi at the base of the Upper Cretaceous section to as little as 10 ft/mi for the Upper Tertiary units.

The stratigraphic record of the sediments in the site region reflects a wide range of depositional environments as well as episodes of erosion, which correspond to changes in eustatic sea level and the effects of regional tectonic movements. The strata are characterized by significant lateral and vertical variations in lithofacies that reflect the varying depositional regimes. Marine, marginal marine, and terrestrial depositional sequences are recognized. Regional stratigraphic relationships are shown on Figure 2.5-3, and a cross section illustrating the configuration of the upper Coastal Plain sediments is presented on Figure 2.5-4.

The regional stratigraphy is discussed in three parts which correspond to fundamental stratigraphic domains as follows:

1. The basement complex, which underlies the entire continental margin

2. The coastal plain stratigraphic sequence, which corresponds to the emerged portion of the continental shelf

3. The continental shelf stratigraphic sequence, which consists of strata deposited in the Baltimore Canyon Trough.

2.5.1.1.2.1 Basement Complex

Information concerning the stratigraphy of the basement complex is primarily derived from its exposure in the Piedmont Province. The closest approach of this province to the site is approximately 18 miles to the northwest near Newark, Delaware. Lithologies composing the basement complex are subdivided into two groups. The first group consists of Precambrian and early Paleozoic metamorphic and igneous crystalline rocks. The second group is composed of early Mesozoic terrestrial sedimentary strata and mafic igneous rocks. In a strict sense, the latter group probably should not be categorized as basement rocks. However, they are discussed in this section for two reasons. First, the early Mesozoic group comprises a lithotectonic assemblage closely related to basement structure. Secondly, this assemblage occurs both beneath the Coastal Plain and to the west of the Piedmont Province. The distribution of the basement complex lithologic groups is shown on the regional geologic map (Figure 2.5-5).

West of the Fall Line, the Precambrian and early Paleozoic lithologies include basic igneous intrusives, as well as various granites, gneisses, and schists. The complex stratigraphic relationship between these metamorphic and igneous crystalline rocks is not well known. In the Wilmington and Newark, Delaware, areas these rocks are typified by the Wissahickon Formation, the Wilmington Complex, and the Port Deposit Granodiorite.

Little direct information is available on the character of the basement rocks underlying the sediments of the Coastal Plain. It is generally interpreted that they are a complex of Precambrian and early Paleozoic lithologies similar to the rocks exposed in the Piedmont Province. This interpretation is supported by geophysical data and boring data that indicate the presence of such diverse rock types as schist, gneiss, granite, and mafic igneous rocks at depth (Reference 2.5-5).

The second group of rocks that compose part of the basement complex consists of terrigenous conglomerates, sandstones and shales, intruded by sills and dikes, and intercalated with volcanic rocks. These lithologies are Triassic to Early Jurassic in age and fill irregularly shaped fault bounded basins within the Piedmont Province. The regional trend of the basins is parallel to the northeast-southwest alignment of geologic structure in the Appalachians (Figure 2.5-5). Several of the Triassic basins have been traced in varying degrees from where they are exposed in portions of the Piedmont Province to beneath the Coastal Plain sediments. Other Triassic basins are considered to underlie extensive areas of Atlantic Coastal Plain (Reference 2.5-7), and have been interpreted to underlie younger sediments within the Baltimore Canyon Trough (Reference 2.5-8).

The basement surface is exposed west of the Fall Line and dips to depths approximately 6000 feet beneath the coastline at Cape May (Reference 2.5-9). The gradient of the basement surface ranges from 40 ft/mi near the Fall Zone to as much as 550 ft/mi beneath the coast. The general configuration of the basement surface is presented on the regional geologic cross section (Figure 2.5-6).

2.5.1.1.2.2 Atlantic Coastal Plain

Mesozoic and Cenozoic stratigraphic units have been mapped on the surface and in the subsurface of the Atlantic Coastal Plain. Many of the units are thin and discontinuous, but others are remarkably persistent. The lower section of the stratigraphic sequence consists of terrestrial sediments that are principally Jurassic (References 2.5-10 and 2.5-11) and Early Cretaceous in age (Figure 2.5-3). These nonmarine lithofacies are often difficult to differentiate from each other and vary greatly in composition and thickness. Overlying this section is a sequence of well defined marine stratigraphic units, primarily Late Cretaceous and Tertiary in age. Downdip, these strata thicken markedly and the corresponding lithofacies represent increasingly offshore depositional environments. Some exceptions to this pattern occur in

Upper Cretaceous beds, which thin downdip and wedge out in such a manner that fewer units of this age occur in the southeastern portion of the Coastal Plain in New Jersey and Delaware. The marine stratigraphic limits are as follows:

1. Pre-Late Cretaceous Strata - The basement complex of the New Jersey Coastal Plain is nonconformably overlain by both Jurassic and Early Cretaceous terrestrial sediments (Reference 2.5-10) (Figures 2.5-3 and 2.5-6). In areas near the Fall Line, the basement rocks are directly overlain by the Early Cretacous strata. However, farther to the east, Jurassic sediments are deposited on the crystalline rocks. The Jurassic sediments are nonmarine in origin and consist of coarse sandstones and red and green shales (Reference 2.5-10). The Lower Cretaceous strata comprise the Potomac Group (Figure 2.5-3). In New Jersey, formal subdivisions of this group are not recognized. However, within deeper portions of the Salisbury embayment, it is subdivided into the Patuxent, Arundel, and Patapsco Formations. These strata are almost entirely fluvial sediments consisting of interbedded sandstones, siltstones, and silty claystones and are as much as 3500 feet thick beneath Cape May, New Jersey (Reference 2.5-11).

2. Late Cretaceous Strata

   a. Raritan Formation - The Late Cretaceous strata represent both nonmarine and marine strata. The oldest Upper Cretaceous stratigraphic unit is the Raritan Formation (Reference 2.5-10). This unit is principally nonmarine in origin and mainly consists of lenticularly intercalated sand and clay. Some geologists have regarded the Raritan Formation as part of the Potomac Group (Reference 2.5-12). Although the Raritan is primarily nonmarine, the Woodbridge Clay Member of this unit contains marine fossils that reflect the first marine deposition within the Cretaceous sediments of the coastal plain (Reference 2.5-10).

b. Magothy Formation - The Magothy Formation unconformably overlies the Raritan Formation throughout the Coastal Plain (Figure 2.5-3). It consists of intercalated carbonaceous silty clay and light colored sand. This unit is interpreted to be a coastal deposit, reflecting the Late Cretaceous marine transgression (Reference 2.5-10). The thickness of the Magothy Formation varies considerably; however, in the southwestern New Jersey Coastal Plain it is about 50 feet thick (Reference 2.5-13).

c. Matawan Group - The Upper Cretaceous Matawan Group disconformably overlies the Magothy Formation. This group is composed of five stratigraphic units: the Merchantville, the Woodbury, the Englishtown, the Marshalltown, and the Wenonah Formations (Figure 2.5-3). These formations are all marine in origin, and represent deposition during the Late Cretaceous marine transgression. Changes in the lithologic character of these units records sedimentation during transgressive and regressive depositional regimes (Figure 2.5-3) in response to cyclic variations in relative sea level (Reference 2.5-10).

The Merchantville Formation is the basal stratigraphic unit of the Matawan Group and is the oldest glauconitic unit of the Coastal Plain (Reference 2.5-13). Along strike, the Merchantville varies in composition but predominantly consists of interbedded massive marine glauconitic sands, and thin bedded micaceous carbonaceous rich clayey silts.

Siderite concretions and carbonized wood characterize the formation's basal contact (Reference 2.5-13). This massive shelf unit was deposited during a transgressive marine phase (Reference 2.5-10), and its maximum thickness is about 60 feet (Reference 2.5-14).

The Woodbury Formation conformably overlies the Merchantville Formation. The Woodbury unit consists of dark gray, massive clayey silt with localized thin lenses of glauconite and in general contains much more glauconite than the Merchantville. The lithologic character of this unit suggests that it marks the start of a phase of marine regression (Reference 2.5-10). The Woodbury is 50 feet thick in west-central New Jersey and pinches out in the southwestern part of the state (Reference 2.5-13).

Overlying the Woodbury Formation is the Englishtown Formation, which represents both coastal and shoreface depositional environments (References 2.5-10 and 2.5-13). It mainly consists of sand but contains small amounts of mica, glauconite, and lignite with some local areas of iron-oxide cementation. It is interpreted that this unit represents the culmination of the regressive marine phase that began with the deposition of the Woodbury Formation (Reference 2.5-10). The formation is 100 to 140 feet thick in the northern part of the New Jersey Coastal Plain but thins to approximately 40 feet to the southwest (Reference 2.5-14).

The Marshalltown Formation (Figure 2.5-3) consists of massive extensively burrowed beds of greenish black sandy clay and glauconitic sand. It is generally 10 to 15 feet thick in most parts of the New Jersey Coastal Plain (Reference 2.5-14). It is a mid shelf marine deposit and marks another marine transgressive cycle (Reference 2.5-10).

The Wenonah Formation is the youngest formation of the Matawan Group and conformably overlies the Marshalltown Formation. The Wenonah Formation was deposited in an inner shelf environment and consists of a dark gray, poorly sorted, micaeous, silty, quartz sand. Glauconite is locally abundant in the lowermost portion of the unit (Reference 2.5-13). The lithologic character of this unit suggests that it represents a regression of sea level relative to that required for the deposition of the Marshalltown Formation. The formation has a maximum thickness of 60 feet in west-central New Jersey (Reference 2.5-14) and thins along the strike of the Coastal Plain units.

d. Monmouth Group - The Monmouth Group conformably overlies the Wenonah Formation and is subdivided into five stratigraphic units: The Mount Laurel, Navesink, Red Bank, New Egypt, and Tinton Formation. These units are entirely marine in origin. Their lithologic variations reflect minor transgressive and regressive phases within the overall marine transgressive trend of the Late Cretaceous (Reference 2.5-10).

The Mount Laurel Formation (Figure 2.5-3) is the oldest unit of the Monmouth Group. It consists of three nearshore facies: a basal thinly interbedded dark clay and light sand facies; an intermediate massive sand bed facies; and an upper, thin pebbly sand facies (Reference 2.5-13). The uppermost part of the Mount Laurel Formation is bioturbated with glauconite infilling worm burrows. This, in conjunction with the lithologic character of the upper facies, suggests that the uppermost beds may represent a lag deposit related to the overlying Navesink Formation. The Mount Laurel Formation is a regressive unit reflecting a further reduction of sea level from the level associated with deposition of the Wenonah Formation (Reference 2.5-10). The thickness of the Mount Laurel Formation varies from 20 to 70 feet (Reference 2.5-14). This unit thins downdip toward the southeast and merges with its offshore counterpart (Reference 2.5-10), the Wenonah Formation (Figure 2.5-3).

The Navesink Formation conformably overlies the Mount Laural Formation and consists of massive, dark greenish gray, clayey glauconitic sand. Basal strata are characterized by shell beds, whereas the upper strata are characterized by an increasing content of limey clay. This unit represents a transgressive phase of sedimentation and reflects mid shelf depositional conditions (Reference 2.5-10). The Navesink Formation is commonly about 30-feet thick (Reference 2.5-13). However, these strata are only recognized in the New Jersey Coastal Plain, apparently thinning out along strike to the southwest (References 2.5-10, 2.5-14).

Overlying the Navesink Formation is the Red Bank Formation, which is subdivided into three members: an upper quartz sand, a lower silt, and a lowermost glauconitic sand (Reference 2.5-14). The members represent inner shelf deposition and reflect marine regression following deposition of the Navesink Formation (Reference 2.5-10). The thickness of the formation may be as much as 150 feet in the northeastern portion of the New Jersey Coastal Plain, but, it thins out downdip and to the southwest (Reference 2.5-14).

The New Egypt Formation consists of a clayey glauconitic facies of the Red Bank Formation, representing deposition in a deeper water environment. It appears to be a shelf facies deposited marginally to both the Red Bank and Tinton Formations (Reference 2.5-10). Stratigraphically, it is their equivalent and in places is underlain by the Navesink Formation and overlain by the Hornerstown Formation.

The Tinton Formation is the youngest stratigraphic unit in the Monmouth Group, the uppermost Cretaceous sedimentary unit in New Jersey. It consists of massive, dark gray, glauconitic quartz sand. Typically, the Tinton Formation is cemented by finely crystalline siderite, making a resistant surface that marks the contact with the overlaying Tertiary sediments (Reference 2.5-15). This stratigraphic unit represents further deposition during the marine regression that began during deposition of the Red Bank Formation (Reference 2.5-10). The Tinton Formation is very limited in its extent, is confined to the northeastern part of the New Jersey Coastal Plain, and only attains a maximum thickness of 25 feet (Reference 2.5-13).

3. Tertiary Strata - The Tertiary strata of the Coastal Plain consist primarily of marine sediments. The depositional relationship between the Tertiary strata and the underlying Late Cretaceous units has been interpreted both as an angular unconformity and as a facies relationship (Reference 2.5-10). In general, the Tertiary units represent relatively minor marine transgressions onto the Coastal Plain, when compared with the Late Cretaceous submergence.

a. Rancocas Group - The Paleocene Rancocas Group represents the oldest Tertiary strata in the region (Figure 2.5-3). This group consists of two stratigraphic units, the Hornerstown and Vincentown Formations. In New Jersey, the Hornerstown Formation disconformably overlies successively older units from northeast to southwest. In the north, it overlaps the Tinton Formation and in the south, the Navesink Formation. It consists of dark green, almost pure beds of glauconite. The Honerstown Formation represents inner to middle shelf deposition and is transgressive with regard to the underlying units (Reference 2.5-10).

   Conformably overlying the Hornerstown Formation is the Vincentown Formation, the upper formation of the Rancocas Group. In New Jersey, the lithology of this unit is quite variable and ranges from a massive quartz sand in the northeast to a glauconitic quartz sand in the southwest. The sediments of the Vincentown reflect marine inner to mid shelf sedimentation (Reference 2.5-10), and represent a regressive sequence relative to the underlying Hornerstown Formation. The Vincentown underlies nearly all of the New Jersey Coastal Plain and may be up to 100 feet thick; however, it thins rapidly downdip where it merges with a siltier facies (Reference 2.5-10).

b. Manasquan Formation - The Eocene Manasquan Formation disconformably overlies the Vincentown Formation. It consists of two members. The lower Farmingdale Member is a dark glauconitic coarse quartz sand, and the Upper Deal Member is a greenish white, slightly glauconitic clayey fine sand (Reference 2.5-10). This formation represents mid shelf to upper slope deposition and is transgressive relative to the underlying Vincentown Formation (Reference 2.5-10). The thickness of the Manasquan Formation is not precisely known; however, it is generally about 40 feet thick (Reference 2.5-13).

c. Shark River Formation - The Eocene Shark River Formation overlies the Manasquan Formation in gradational contact. It consists of glauconitic sand and light silty clay (Reference 2.5-10). This unit was deposited under inner shelf conditions. Hence, it is a regressive sequence relative to the Manasquan Formation.

d. Piney Point Formation - An extensive erosional unconformity is developed on the Eocene sediments of the Coastal Plain (Figure 2.5-3). The Oligocene Piney Point Formation (Reference 2.5-10) was deposited on this surface. This formation consists of glauconitic silt and coarse quartz and glauconite sand, and contains weathered Eocene sedimentary clasts and reworked fossil fragments. Deposition of the Piney Point Formation represents a marine transgression that followed Late Eocene to Early Oligocene erosion of the Coastal Plain. Within the New Jersey Coastal Plain, this unit is as much as 400 feet thick (Reference 2.5-10).

e. Kirkwood Formation - The Miocene Kirkwood Formation conformably overlies the Piney Point Formation where the latter is present. Elsewhere, it lies on a surface of low relief eroded on Eocene to Late Cretaceous formations. The Kirkwood Formation is composed of mainly thick bedded, fine grained, micaceous quartz sand (Reference 2.5-13). Thin beds of sandy gravel and cross bedded sands occur along the inner edge of the outcrop belt. In some areas, the top of the Kirkwood is marked by a fossiliferous silty, clayey sand known as the Shiloh Marl. The Kirkwood was deposited in a nearshore and inner shelf environment and represents marine regression after deposition of the Piney Point Formation. The Kirkwood reflects progradation of the continental shelf during the Miocene Epoch (Reference 2.5-10) and is the uppermost marine unit in the Coastal Plain stratigraphic sequence. The thickness of the formation ranges from less than 100 feet to as much as 1000 feet (Reference 2.5-13).

f. Cohansey Formation - Above the Kirkwood Formation and outcropping over large areas of southern New Jersey is the Cohansey Sand, a terrestrial Late Miocene or Pliocene Formation (Reference 2.5-13). The Cohansey Sand rests unconformably on the Kirkwood and older formations. It is composed of light colored, medium to coarse grained quartz sand with lenses of clay. Locally, the lenses of clay are as much as 25 feet thick. The Cohansey ranges in thickness from 100 feet to 240 feet and dips southeast at about 11 ft/mi (Reference 2.5-15).

g. Beacon Hill Formation - The youngest Tertiary unit in southern New Jersey is the Beacon Hill Gravel. Occurring at scattered locations, it caps broad hills and ridges and is thought to represent erosional remnants of Pliocene strata. The formation consists of heavily weathered sand, gravel, and silt with some clay, locally cemented with iron oxide, and is terrestrial in origin.

4. Quaternary Strata - The Quaternary deposits of the New Jersey Coastal Plain can be separated into three major units: the Bridgeton-Pennsauken Complex, the Cape May Formation, and undifferentiated Holocene strata. The terrigenous Quaternary sediments overlie Tertiary strata

as valley fill, caps on upland ridges and hills, and as a relatively thin blanket in the coastal areas. The Quaternary strata are generally not thicker than 50 feet (Reference 2.5-15).

The basal Bridgeton Formation and the younger Pennsauken Formation consist of gravel, sand, and silt that were apparently deposited in broad interstream areas by the ancestral Delaware River and its tributaries. Although these two formations are similar in composition, the Bridgeton is characteristically more weathered.

The youngest and most extensive Pleistocene unit recognized in New Jersey is the Cape May Formation. These sediments were deposited during a higher stand of sea level associated with the last (Sangamonian) interglacial period. The formation occurs as a coast parallel, wedge shaped mass including blanket like sheets and channel fill. Much of the Cape May Formation is comprised of fluvial sand and gravel. In Cape May County, it consists of estuarine and marine sands, silts, and clays.

Undifferentiated Holocene sediments unconformably overlie the older sediments of southern New Jersey. These strata consist of limited terrestrial deposits in stream valleys and marshes. More extensive Holocene strata are the marine and marginal marine sediments of the coastal areas. The Holocene sediments consist of clay, silt, and organic material accumulating in lagoons and along rivers and the sand of the barrier systems that line most of the New Jersey and northern Delaware coastline. The beach and dune sand consists of loose reworked glacial sediment that has been deposited by the present high energy coastal environment. Holocene sediments generally do not exceed 30 feet in thickness.

2.5.1.1.2.3 Atlantic Continental Shelf and Slope

Most recent information concerning the stratigraphy of the Atlantic continental shelf and slope in the site region has been derived from work performed in association with petroleum exploration. This exploration has principally consisted of conducting numerous high resolution seismic surveys and drilling deep stratigraphic test wells. The results of these programs have significantly altered the previous interpretations concerning the subsurface conditions of the continental shelf and slope. These data have greatly enhanced the current understanding of the geologic setting of this area.

More than 20 years ago, regional seismic refraction surveys revealed the presence of a major structural basin, referred to as the Baltimore Canyon Trough (Figure 2.5-2) beneath the continental shelf (Reference 2.5-5). More recent seismic exploration (Reference 2.5-8) has shown that the thickness of sediments within the basin is much greater than originally interpreted. Current estimates of the thickness of Jurassic and younger sediments within the trough are approximately 15 kilometers (Reference 2.5-8). Hence, the basin is considered to be the controlling factor for deposition along the eastern North American continental margin at least since the Jurassic Period.

The configuration of the basin and the sediments within the trough are shown on Figure 2.5-6. Several generalizations can be made concerning the stratigraphy of the continental shelf and slope. The sediments that are present on the Coastal Plain also extend out onto the continental shelf (Reference 2.5-9). Time equivalent stratigraphic units become more marine in the deeper parts of the central portion of the Coastal Plain. Jurassic sediments are known to be present only east of the central portion of the Coastal Plain (Reference 2.5-11). On the continental shelf, stratigraphic units usually thicken in a seaward direction. This thickening is particularly evident for the Jurassic and Early Cretaceous units (Figure 2.5-6).

It is interpreted (Reference 2.5-11) that Jurassic sediments of the continental shelf unconformably overlie both continental and oceanic basement (Figure 2.5-6). The oceanic crust is inferred to be basaltic in composition and to have been emplaced during the opening of the Atlantic Ocean. The continental crust is interpreted to be composed of late Precambrian and Paleozoic age metamorphic rocks, as well as Triassic redbeds and igneous rocks that correspond to the basement complex underlying the Coastal Plain stratigraphic sequence (Section 2.5.1.1.2.1).

The Jurassic sediments predominantly consist of nonmarine sandstone and shale with beds of coal and lignite (Reference 2.5-11). In the eastern portions of the trough (Figure 2.5-6), these lithologies grade into a sequence of evaporites, dolomites, and limestones (Reference 2.5-8). These sediments reflect the terrestrial depositional environment in the bulk of the basin with shallow marine conditions pertaining in its eastern areas, during Jurassic time. Jurassic sediments attain a maximum thickness of 10 to 12 kilometers beneath the outer continental shelf (Reference 2.5-8).

The Lower Cretaceous strata in the trough are similar in character to the units assigned to the Potomac Group in the Coastal Plain (Reference 2.5-9). They consist of nonmarine sandstone and shale containing lignite and coal beds (Reference 2.5-9) and reflect continental sedimentation within the basin. However, beneath the continental slope, these strata also contain limestones and dolomites reflecting shallow marine carbonate deposition along the Early Cretaceous shelf edge (Reference 2.5-10).

The Late Cretaceous sediments consist predominantly of marine shales and mudstones deposited in an outer shelf environment (Reference 2.5-11). These sediments represent deeper water facies of the Coastal Plain units such as the Magothy Formation, the Matawan Group, and the Monmouth Group (Reference 2.5-9). The character of the Upper Cretaceous strata reflects the Late Cretaceous marine transgression of the continental margin. The maximum thickness of the Cretaceous stratigraphic sequence in this area is approximately 3 kilometers (Reference 2.5-9).

The Tertiary sediments of the Continental Shelf consist of a 1 to 2 kilometer thick sequence of marine sand, shale, mudstone, and limestone. The depositional environment of these strata ranges from outer shelf to marginal marine conditions (Reference 2.5-11). They reflect more distant offshore facies of the Tertiary Coastal Plain strata. Within this stratigraphic sequence, two prominent unconformities are present (Reference 2.5-9). The lower unconformity is located at the Cretaceous/Tertiary boundary and, in some locations, Paleocene sediments are absent (Reference 2.5-11). The upper unconformity is located at the Eocene/Oligocene boundary and, in some locations, the Oligocene section is also absent (References 2.5-9 and 2.5-11). These unconformities reflect a relative lowering in sea level during Tertiary time.

Quaternary sediment of the continental shelf consists predominantly of nonmarine and nearshore marine Pleistocene sediments up to 250 meters thick (Reference 2.5-9). These strata consist of gravel, sand, and silty clay. They reflect complex stratigraphic relationships induced by Pleistocene sea level changes (Reference 2.5-9). These sediments are overlain by a relatively thin veneer of Holocene strata.

2.5.1.1.3 Regional Structural Geology

The purpose of this section is to discuss the geologic structural framework in the site region. To provide a framework for the discussion of regional structural elements relevant to the safety-related aspects of the site, the large scale tectonic elements in the region require identification. However, there is no specific and widely accepted terminology that is used to identify broad geotectonic domains. In part, this results from the presentation in the literature of ideas derived from different perspectives. Also, this situation results from the fact that the application of geotectonic terminology to a particular geologic region is dependent on the period of time discussed. The boundaries for structural provinces discussed in this section are established from the literature. When a widely used term was not available to identify a particular area, a descriptive term was developed for the purposes of this discussion.

HCGS is situated within the west-central area of the emerged portion of the eastern North American continental margin. In terms of its present tectonic setting, this continental margin has been characterized as Atlantic type (Reference 2.5-16). The margin extends westward from a prominent magnetic anomaly, referred to as the east coast magnetic anomaly (Figure 2.5-7) to the Fall Line. Many authors regard this magnetic anomaly as an interface between the continental and oceanic crusts (References 2.5-17, 2.5-18, and 2.5-19).

In terms of overall structure, a characteristic feature of the present continental margin is the progressive increase in crustal thickness away from the ocean basin (Figure 2.5-6). Depths to the Moho discontinuity are interpreted on the basis of seismic refraction and gravity measurements to be 12 to 15 kilometers under the continental rise, about 20 kilometers under the slope, and 30 to 35 kilometers under the continental shelf and interior (References 2.5-17 and 2.5-8). The crustal thinning may account for the large scale subsidence of the continental margin during the Mesozoic and Cenozoic Eras (Reference 2.5-20). Numerous mechanisms have been proposed for the restructuring of the crust along the continental margin (References 2.5-17 and 2.5-20).

On the basis of current understanding of the evolution of eastern North America, it is inferred that the arrangement of the lithologies composing the crust of continental margin and adjacent parts of the continental interior was produced in several stages. For the purpose of this discussion, the crust has been divided into three distinctive tectonic assemblages, characterized by the type and age of the rocks within them, as well as by the mode of their structural development. These assemblages are:

1. A pre-Mesozoic assemblage consisting of crystalline, metamorphic, and sedimentary rocks and associated structures, formed prior to the Mesozoic Era in response to divergent and convergent interaction along the eastern North American plate boundary.

2. An early and middle Mesozoic assemblage consisting of sedimentary and igneous rocks deposited and deformed during the early stages of the latest divergent interaction between the North American and African plates.

3. A late Mesozoic and Cenozoic assemblage consisting of a wedge of unconsolidated and semiconsolidated sediments deposited on the continental margin during the advance stages of divergent interaction and deformed by structures of a similar age.

2.5.1.1.3.1 Pre-Mesozoic Tectonic Assemblage

The bulk of the continental margin and the adjacent parts of the continental interior is composed of metamorphic, igneous, and sedimentary rocks and structures related to the pre-Mesozoic tectonic assemblage. These rocks and structures were formed during three distinct tectonic episodes: the Grenville orogenic cycle, the early Paleozoic crustal divergence associated with formation of the proto Atlantic Ocean (Reference 2.5-21 and 2.5-22), and the middle to late Paleozoic continental convergence resulting in the formation of the Appalachian orogen.

It is possible to distinguish three structural provinces within the Appalachian orogenic belt (References 2.5-21, 2.5-23, and 2.5-24) that represent subdivisions of the pre-Mesozoic tectonic assemblage. These provinces are the Avalon Platform, the Paleozoic mobile belt, and the ancient North American craton. The locations of the structural provinces are shown on Figure 2.5-8.

1. Avalon Platform - The Avalon Platform is situated between the western flank of the Meguma Geosyncline (Reference 2.5-23) and the eastern flank of the Paleozoic mobile belt (Figure 2.5-8). The western edge of the Avalon Platform is marked by a pronounced gravity and magnetic anomaly (Reference 2.5-18). The areal extent of this province is well defined in New England and Maritime Canada; however, south of Long Island Sound, its limits are not clearly established. It is possible that the western boundary of this province extends eastward and intersects the east coast magnetic anomaly (Figure 2.5-8) at a relatively acute angle. On the other hand, the boundary may extend further south, in which case portions of the Avalon Platform compose the easternmost part of the continental crust in the Mid Atlantic Region.

   The Avalon Platform is comprised of continental crust, which is not older than late Precambrian (650 million years) (Reference 2.5-24). An important aspect of this province is that it behaved as a platform or micro continent during the closing of the proto Atlantic ocean (Reference 2.5-23). The crust of this platform includes highly deformed late Precambrian carbonate and detrital strata interbedded with volcanic rocks. These lithologies are intruded and slightly metamorphosed by Eocambrian (560 million years) granites (Reference 2.5-23). The entire sequence is intruded by Ordovician to Devonian age plutons ranging in composition from dioritic to gabbroic. The emplacement of these intrusions occurred during the early to middle stages of continental convergence. They represent the remnants of an ancient island arc. The plate tectonic role of the Avalon Platform during the early to middle Paleozoic was not dissimilar from the present day island arc of Japan.

2. Paleozoic Mobile Belt - The boundaries of the Paleozoic mobile belt (Figure 2.5-8) are the western limit of the

Avalon Platform or the east coast magnetic anomaly and the ancient continental margin as defined by others (References 2.5-21 and 2.5-22). This structural province generally corresponds to the ophiolite volcanic terrain as discussed by others (Reference 2.5-21).

The ancient continental margin is the easternmost limit of occurrence of the continental crust of Grenvillian age and is marked by several significant geologic and geophysical features (Reference 2.5-21). These are the main Appalachian gravity high (Reference 2.5-18), a change in crustal seismic refraction velocities (Reference 2.5-23), and a change in structural style and metamorphic facies. The mobile belt probably extends from New England southward underneath the coastal plain along the entire length of the Appalachian orogen. Hence, in the site area the crystalline basement belongs to the Paleozoic mobile belt structural province.

The aspects of the Paleozoic mobile belt that distinguish it from the surrounding provinces are its age and tectonic character. The crust within this structural province is composed of rocks which are younger than either Grenvillian or Avalonian. These rocks consist of two assemblages. One includes early to middle Paleozoic eugeosynclinal sediments (Reference 2.5-24). The other is comprised of an extensive system of plutons, the composition of which varies from mafic to felsic. The sediments are metamorphosed to varying degrees and are intensely deformed. Two metamorphic events related to the Taconic and Acadian orogenies are evident in these rocks. From the standpoint of plate tectonics, this province may represent a broad suture zone along which two different continental fragments were welded together with the destruction of the proto Atlantic Ocean, (Reference 2.5-21).

3. Ancient North American Craton

The ancient North American craton is that part of the continent situated west of the aforementioned ancient continental margin, Figure 2.5-8. This structural province includes parts of the western Piedmont, the western part of the New England Upland, and the Valley and Ridge as well as the Appalachian Plateau Physiographic Provinces.

For the purpose of this discussion, the ancient North American craton is divided into three subprovinces. From northwest to southeast they are: the Appalachian Basin, the Appalachian Highlands anticlinoria, and the ancient cratonic margin. The crust composing the North American craton consists of two broad rock assemblages. One assemblage consists of rocks that were formed and deformed during the Grenville orogenic cycle (1 billion years before present). The other assemblage is composed of rocks, the origin of which is related to the opening and closing of the proto Atlantic during late Precambrian and Paleozoic time (Reference 2.5-22).

Near the eastern edge of the ancient craton, the Grenvillian rocks crop out to form the Appalachian Highlands anticlinoria (Figure 2.5-8). These Grenvillian rocks comprise the Blue Ridge Anticlinorium, Reading Prong, Berkshire Mountains, and Green Mountains (Reference 2.5-25). West of the anticlinoria is the elongate Appalachian Basin, the floor of which is formed by Grenvillian rocks. The basin floor is greatly depressed (40,000 feet) in the east and gradually rises toward the west. The Appalachian Basin (Figure 2.5-8) contains a thick sequence of clastic and carbonate sedimentary strata. These strata range in age from Early Cambrian to Carboniferous. They form a southeastward thickening wedge, reflecting the asymmetry of the basin.

Near the interface with the Appalachian Highlands anticlinoria, these strata are greatly deformed by tight folds and extensive thrust faults as in the Valley and Ridge. This deformation diminishes progressively to the west, and the strata in the western part of the basin are only slightly deformed as in the Appalachian Plateau.

East of the Appalachian Highlands anticlinorium is the ancient cratonic margin. The deeper crust in this area is most probably formed by Grenvillian age rocks, which only crop out in isolated exposures (Figure 2.5-8) within a northeast trending belt located immediately west of the ancient continental margin (References 2.5-21 and 2.5-25). The belt is defined by a series of migmatitic gneiss domes such as the Sauratown Mountain Anticlinorium, the Baltimore Gneiss Domes, and possibly the Chester Dome of Vermont (Reference 2.5-22).

The Grenvillian basement within the ancient cratonic margin is overlain by a sequence of late Precambrian metamorphosed clastic sediments and associated mafic intrusive and extrusive rocks. Rocks of this sequence comprise the Ashe, Lynchburg, Catoctin, and Yonkers lithologic units (Reference 2.5-22). This sequence is commonly regarded to have been deposited in the trough of the late Precambrian rift system related to the early stages of the opening of the proto Atlantic Ocean (References 2.5-21 and 2.5-22).

The ancient cratonic margin also contains various metamorphosed early and middle Paleozoic rocks that overlie the Precambrian metasediments and metavolcanics. These rocks were formed during the advanced divergent stage of the proto Atlantic Ocean and continued to be deposited throughout the long period of continental convergence. The divergent stage rocks are represented by remnants of the obducted ophiolite sequence and the continental terrace wedge deposited on the late Precambrian early Paleozoic continental shelf (Reference 2.5-21). Presently, the remnants of the continental terrace strata include quartzites, slates, and marbles.

Within the ancient cratonic margin, there are also rocks of eugeosynclinal character formed during the convergent stage. These are predominantly pelitic rocks containing mafic and felsic intrusions as well as volcanic debris. All of the rocks within the ancient cratonic margin are very highly deformed during the Paleozoic orogenic cycles. This deformation includes polyphase folding and faulting, as well as many grades of metamorphic recrystallization.

In terms of plate tectonic setting, the ancient North American craton can be subdivided into two belts. The first belt consists of the arc trench gap and the volcanic arc, as defined by others (Reference 2.5-26). This belt roughly corresponds to the Appalachian Highlands anticlinoria, ancient cratonic margin, and Paleozoic mobile belt. The second belt consists of the marginal basin (Reference 2.5-26), which in this case is represented by the Appalachian Basin.

4. Pre-Mesozoic Structures

Numerous geologic structures were developed during the evolution of the Pre-Mesozoic tectonic assemblage. These structures are usually categorized according to physiographic provinces (Figure 2.5-2), and in the site region these areas correspond closely to the above outlined structural provinces (Figure 2.5-8). Pre-Mesozoic structures comprise the majority of prominent folds and faults in the Piedmont, Blue Ridge, and Valley and Ridge provinces, northwest of the site. Recent geophysical studies (Reference 2.5-27) have been interpreted to indicate that the Precambrian and Paleozoic rocks of the Blue Ridge and Piedmont provinces are allochthonous. It is suggested that the thrust sheet may be up to 15 kilometers thick and the amount of westward transport is in hundreds of kilometers (Reference 2.5-27).

Precambrian and early Paleozoic rocks of the Piedmont Province are strongly metamorphosed and exhibit tightly folded structures superimposed on broader synclinoria and anticlinoria. The majority of the faults that displace Precambrian and Paleozoic strata are thrust faults that generally exhibit displacement toward the northwest. These faults do not involve Mesozoic or Cenozoic strata and are usually considered to be healed. To the northwest of the site, some of the major thrust faults are transected by Triassic diabase dikes that show no displacement. Similar relationships of Triassic dikes crossing Paleozoic faults are found in the Reading Prong area, northwest of the site. The last movement along these faults probably occurred over 200 million years ago, and certainly no later than 140 million years ago, based on the absence of displacement of the Triassic diabase dikes (Reference 2.5-28).

The regional synclinal anticlinal structures in the Piedmont (Reference 2.5-29) are the Virgilina Synclinorium and James River Synclinorium of Virginia and the Baltimore-Washington Anticlinorium of Maryland (Figure 2.5-8). The synclinoria of Virginia are generally recumbent, dipping steeply to the southeast. The Baltimore-Washington Anticlinorium extends northeastward about 50 miles in a gentle arc from Washington, D.C., to a point north of Baltimore. Along its axial zone are numerous mantled domes with cores of the Precambrian Baltimore Gneiss. Other prominent fold structures of the Piedmont are the Peach Bottom Fold, the Tuquan Arch, the Sherwill Anticline, and the Arvonia Syncline (Figure 2.5-8).

The Martic Line is located approximately 50 miles northwest of the site (Figure 2.5-8). It was defined by early researchers as a major thrust fault. However, recent reevaluation of the Glenarm Series stratigraphic sequence (Reference 2.5-30) and lack of field evidence of faulting (Reference 2.5-31) discount the presence of the Martic Line as a long continuous thrust fault. Any movement along these inferred faults probably occurred prior to regional metamorphism because foliation planes are similar on both sides of the line. No post-Triassic deformation is recognized along the Martic Line. Triassic diabase dikes, intruded across the line, have not been offset.

The east to northeast trending Cream Valley-Huntingdon Valley fault is located approximately 30 to 40 miles north of the site. Drag folds suggest a complex history of movement along what now appears to be a southward dipping high angle reverse fault (Reference 2.5-32). The fault is mapped for some 45 miles along trace in Pennsylvania but is concealed by Cretaceous and Tertiary strata of the Coastal Plain in New Jersey. Simple strike projection indicates that if the fault continued beneath the Coastal Plain it would extent to the vicinity of Raritan Bay, New Jersey, approximately 100 miles northeast of the site.

The Delaware Geological Survey has interpreted remote sensing data and possible geomorphic phenomena to infer the existence of at least three areas of possible faulting in northern Delaware. None of the three features is considered a capable fault (Reference 2.5-29).

The Reading Prong (Figure 2.5-8) is a major structural and physiographic feature that extends from beyond the Hudson River in New York southwest to Reading, Pennsylvania. It consists of mountainous ridges underlain by Precambrian igneous and metamorphic rocks, and intermontane valleys underlain by early Paleozoic carbonate and pelitic rocks. Within the New Jersey Highlands, a part of the Reading Prong, the rocks are divisible into several northeast trending fault blocks. Data from magnetic and gravity surveys in Pennsylvania indicate that the Precambrian may be rootless, and it has been postulated that the Prong is a complicated nappe structure (Reference 2.5-33). However, others maintain that there is complete lack of evidence for the nappe interpretation in New Jersey (Reference 2.5-34).

The South Mountain Anticlinorium and the Blue Ridge Anticlinorium are major structural features of the Blue Ridge Physiographic Province (Reference 2.5-35). The South Mountain Anticlinorium in northcentral Virginia and Maryland is on its east limb upright and consists of gently dipping sediments, whereas the west limb is overturned and nearly vertical. In general, the anticlinorium plunges to the northeast. Within the anticlinorium, complex smaller folds and faults occur. The Blue Ridge Anticlinorium along the Potomac River is divided by a narrow tight syncline (Reference 2.5-36). On the east flank, the metamorphosed cover rocks show abundant but small isoclinal folds, and the units appear to succeed each other fairly regularly except where they are cut by Triassic normal faults. The west flank, on the other hand, shows a wide variety of large scale structures. In some areas, one finds rocks ranging from "basement" to Middle Ordovician standing vertically; but in other areas the contact with the Valley and Ridge is marked by a major thrust fault.

The Valley and Ridge Provinces consist of an approximately 50 mile wide zone of tightly folded and faulted Paleozoic rock. The structure is dominated by recumbent and upright doubly plunging folds involving 30,000 to 40,000 feet of sedimentary rocks. The folds and associated east-dipping thrust faults strike northeast with the exception of local, easterly strike deflection near Harrisburg, Pennsylvania. Appalachian folding and faulting terminated about 230 million years ago. Subsequent major structural activity appears to be limited to that which may have accompanied the emplacement of isolated dikes during Triassic time. Major structures of the Valley and Ridge within 200 miles of the HCGS site are the North Mountain Thrust Fault and the Massanutten Syncline (Figure 2.5-8).

Farther to the west, the Paleozoic sedimentary rocks are not extensively folded and faulted. This marks the westward termination of the Valley and Ridge and the beginning of the Appalachian (Allegheny) Plateau.

2.5.1.1.3.2 Early and Middle Mesozoic Tectonic Assemblage

The separation of the North American and African plates approximately 180 million years ago led to the development of two successive series of fault bounded basins along the continental margin. The first series is Triassic in age while the later basins are Jurassic. The spatial distribution of these basins is shown on Figure 2.5-9. The older series of basins are situated in the vicinity of the ancient continental margin (Figure 2.5-8). One remarkable aspect of these basins is that their boundaries are often near this ancient margin. This suggests that this fundamental crustal boundary maintained its significance beyond the period of Paleozoic orogenesis.

During development of the Triassic basins, sedimentation occurred within a northeast trending zone of rift valleys more than 70 miles wide which extends from the Carolinas to the Bay of Fundy (Figure 2.5-9). These basins are filled with a thick sequence of terrigenous clastic sediments (redbeds) and contain intrusive and extrusive rocks of tholeiitic composition. These rocks are Triassic to Early Jurassic in age. Remnants of these basins are exposed in the Piedmont province and are also buried beneath the Coastal Plain sediments (Figure 2.5-9), (References 2.5-7 and 2.5-23). Sediments within the basins are deformed by structures indicative of vertical crustal movements that were apparently postdated by translational movements (Reference 2.5-23).

Within the site region, the Triassic basins are represented by the Connecticut, Newark, Gettysburg, and Culpeper Basins (Figure 2.5-9). The principal structures associated with the development of these basins consist of faults with large vertical displacements such as the Ramapo, Flemington-Furlong, Hopewell, and Chalfont Faults (Figure 2.5-9).

Following the initial rifting, which led to the establishment of the present edge of the continental crust (Reference 2.5-18), the continental margin entered a new tectonic regime. This regime was characterized by progressive and localized crustal thinning which provoked large scale subsidence, resulting in the accumulation of thick sedimentary sequences on the pre-Jurassic basement surface (Figure 2.5-9). This process produced the depressed continental margin with a series of elongated basins or depocenters near its edge (Reference 2.5-17). In places, these marginal basins are 14 to 15 kilometers deep (Reference 2.5-8). As shown on Figure 2.5-9, there is not one continuous basin along the entire margin but a series of individual elongated depocenters, including the Georges Bank Basin, the Baltimore Canyon Trough, and the Blake Plateau Basin. It has been interpreted that major block faulting accompanied development of these basins and that translational movement resulted in additional normal faulting along hinge zones (Reference 2.5-17).

The basins are filled with sequences of indurated Jurassic sediments (Figure 2.5-6) as much as 10 to 12 kilometers thick (Reference 2.5-8). These sedimentary sequences occur above an unconformity developed on the crystalline and Triassic basement complex (Section 2.5.1.1.2.1). The strata were deposited in both terrestrial and shallow marine environments (Reference 2.5-8). They include detrital sediments, coal, carbonates, and evaporites. Locally the sediments contain salt diapirs and mafic intrusions (Reference 2.5-8). Based on the sedimentary record, nearly 70 percent of the subsidence of the continental margin occurred during the Jurassic period. It was followed by slower subsidence during the late Mesozoic and Cenozoic Eras.

2.5.1.1.3.3 Late Mesozoic and Cenozoic Tectonic Assemblage

During the advanced stages of the continental separation, a characteristic wedge of unconsolidated sediments was deposited. This deposition resulted from relatively uniform asymmetric subsidence rather than from the further development of the previous isolated depocenters. Subsidence proceeded episodically as indicated by several unconformities within the stratigraphic section (Section 2.5.1.1.2). Intervals of fast subsidence (greater than 50 to 100 meters/million years) alternated with intervals of slower subsidence and even modest uplift (Reference 2.5-7).

The strata of the Coastal Plain define an eastward dipping homocline that extends from the Fall Line over the continental shelf and slope. This homocline lies unconformably on the basement complex or over the basin sediments of Jurassic age (Figure 2.5-6).

Studies (References 2.5-37 and 2.5-38) of the configuration of the basement surface revealed that it defines a characteristic pattern. The pattern consists of a system of embayments and arches, the axes of which are approximately normal to the strike of the homocline. In the site region, these structures are the Salisbury and Raritan Embayments and the New Jersey uplift (Figure 2.5-9). The outcrop pattern of the strata that form the homocline in New Jersey suggest that the strike of the Cretaceous and Tertiary layers vary somewhat. It appears that the changes in strike are compatible with the basement configuration (Reference 2.5-39). It also appears that individual strata comprising the homocline thicken in the direction of basement depressions and that older formations generally dip more steeply than successively younger units. This suggests that the sedimentation and the development of the basement arches and embayments were contemporaneous.

In terms of plate tectonic setting, the present Coastal Plain homocline reflects the relatively stable stage in the development of a continental terrace wedge. This structure has resulted from the general subsidence along the continental margin that has accompanied the continuous plate divergence since Mesozoic time. The structure and sediment assemblage along the present continental margin has been compared with the late Precambrian early Paleozoic margin of eastern North America (Reference 2.5-21).

Spoljaric (Reference 2.5-145) presented several previously undescribed lineaments on Landsat imagery and identified some of them as faults or possible faults. In the same interpretation (Reference 2.5-145) he assigned a fault origin to a few previously described lineaments and in some instances extended these to within smaller radii of the site. Spoljaric's features with implications for the site, for reasons of their length, map position, trend, or interpretation are shown on Figure 2.5-10a and are referenced to Figure S2.5-1 of the Summit PSAR Supplement (Reference 2.5-143).

"Possible Fault" 17 apparently corresponds to lineament 2 on the Summit figure; lineament 2 was described and evaluated in the Summit PSAR Supplement (Reference 2.5-143) The feature terminates southeastward at a greater distance from the HCGS site (10 mile) than from the Summit site and while it may be projected to intersect the HCGS 5-mile radius, it has been interpreted as terminating at regional lineament 5.

"Fault" 16 corresponds in its mid portion to lineament C of the Summit figure. Lineament C, its coincidence with a component of the Newark Fault System, and its relationship with lineament No. 2 were addressed in the Summit PSAR Supplement. "Fault" 16 also may be projected to within five miles of the HCGS site but it too has been interpreted as terminating at regional lineament 5.

According to Spoljaric's (Reference 2.5-145) imagery analysis, both of the features described above converge upon and terminate in the immediate area of the Red Lion Vibroseis survey, and presumably have been interpreted as abutting the structure itself. However, existing maps of basement structure, including Spoljaric's (Reference 2.5-140) interpretation, fail to reflect the presence of either feature at this locality.

"Fault" 14 may be considered the northeastward projection of either lineament 10 or 10a as they appear on the Summit figure. The two lineaments most likely represent separate perceptions of the same feature, but in neither case was that feature seen to extend into

Delaware. Spoljaric's projection of this lineament northeastward presumably is based upon re-examination of the synoptic images previously employed to define lineaments 10 and 10a, but no geologic information is presented in correlation of its designation as a fault. Therefore, although the feature is shown to intersect the HCGS 5 mile radius, it should be regarded as no more than a lineament of unknown origin, and its designation as a fault no more than speculation.

"Possible Fault" 20 corresponds to the Delaware segment of lineament 7 on the Summit figure. Lineament 7 also had been previously defined by both Spoljaric (Reference 2.5-144) and by Dames & Moore during the Perryman studies (Reference 2.5-142). According to its geometry and extent, the lineament has always been a possible fault but its lack of documentation indicates that it be regarded strictly as a lineament of unknown origin. Spoljaric's projection of this lineament northward into New Jersey and assignment of the 1973 earthquake to it are according to his own presentation, highly speculative.

"Possible Fault" 12, if projected northeastward, would pass the site on the southeast at a distance of about five miles. It too, however, is apparently no more than a lineament of unknown geologic significance.

"Fault" 11 and "Fault" 13 are intersecting lineaments neither of which, when projected, pass nearer than ten miles to the HCGS site. Evidence is presented in the form of Cross Section A-B (Spoljaric, Reference 2.5-144) to indicate fault displacement of Middle Miocene strata and suggest that Late Miocene strata were not involved. Given this interpretation, and the orientation of the structures relative to the site, little significance can or should be assigned to them other than their relationship to the structural fabric and geologic history of the region.

2.5.1.1.3.4 Late Mesozoic and Cenozoic Structures

Several types of deformational features have been found recently within the Coastal Plain sediments. These structures include both high angle faults and folds. Most of these structures are Late Cretaceous and Tertiary in age. However, in a few isolated cases there are indications of displacements as young as Quaternary.

In most cases these structures are quite small. There are documented examples where the dimensions of the structures are in kilometers. The principal style of faulting is reverse (Reference 2.5-7), and the faults appear to be propagated reusing older discontinuities. They have been recognized predominantly in the vicinity of the Fall Line. This may be a result of the thin cover as well as the large number of detailed geologic investigations conducted in the area. The overlying unconsolidated sediments provide excellent stratigraphic control to identify the offsets. Where there is thick sedimentary cover, discrete faulting apparently does not extend to the surface. Instead, the structures seem to degenerate upward into broad monoclinal folds which can be difficult to recognize.

Minor deformation has also been detected in the submerged coastal plain sediments. The principal source of information from the continental shelf and slope is multichannel seismic reflection profiles performed for petroleum exploration. Small faults with displacements on the order of 10 to 20 meters have been identified within Upper Cretaceous and Tertiary strata (Reference 2.5-8). These faults are generally considered to be the result of minor adjustments of the underlying crustal blocks.

A compilation of the Late Cretaceous and Cenozoic structures observed within the coastal plain sediments is shown on Figure 2.5-10. This map shows only those structures of relatively large dimensions. Other smaller scale structures within the region are known, and are discussed by others (References 2.5-40 and 2.5-41).

Perhaps the best documented structure is the Stafford Fault Zone. This structure is located along the Fall Line near Fredricksburg, Virginia. In this area, surface mapping and subsurface exploration with borings revealed the existence of several northeast trending high angle reverse faults (Reference 2.5-42). It has been recognized that the fault zone is nearly 35 kilometers long and displaces the basement unconformity 15 to 100 meters, down to the east. Other investigators have concluded that these faults have not experienced any perceptible movement during the last 500,000 years (Reference 2.5-43).

Seismic surveys performed on the Coastal Plain in Maryland revealed the presence of the Brandywine Fault Zone (Reference 2.5-29). This structure consists of two en-echelon high angle reverse faults. These northeastward trending faults displace the basement unconformity 30 and 50 to 60 meters, down to the west, respectively. Above the basement surface, this structure degenerates into a monoclinal fold. The youngest strata affected by the folding are Late Miocene (Reference 2.5-29).

In the area east of Trenton, New Jersey, the presence of two elongate folds has been established with surface mapping and the analysis of subsurface data (Reference 2.5-44). Formations as young as Miocene seem to be affected. These folds are elliptical in plan view with their long axis oriented northeast. Analysis of the configuration of the Englishtown Formation indicated that the northern fold has a maximum vertical closure of about 20 feet, and horizontal closures equal to about 1.6 and 4.5 miles. The vertical closure of the southern fold is approximately 40 feet, and the horizontal closures are 1.8 and 4.0 miles.

An investigation of the Wilmington Canyon with seismic reflection techniques revealed the presence of a northeast trending fault (Reference 2.5-45). The strata on the southeastern side are displaced by approximately 60 meters. This structure has been interpreted as a gravity controlled feature.

High resolution seismic profiles obtained near the edge of the New Jersey continental shelf show evidence of possible post-Pleistocene faulting in shallow sediments (Reference 2.5-46). Apparent normal faults with throws of approximately 1.5 meters displace sediments to within 7 meters of the seafloor. Displacements of as much as 90 meters are apparent at depths of 5 to 6 kilometers. These faults appear to be overlain by sediments of relatively recent age. They are en-echelon faults 1 to 2 kilometers apart striking N70E and dipping north. The data suggest that the strata are upthrown on the southside.

Seismic surveys performed in the New York Bight Area, revealed the presence of a fault, referred to as the New York Bight Fault (Reference 2.5-47). This fault trends north-northeast and extends a minimum distance of 30 kilometers. It clearly displaces the Upper Cretaceous strata and may also penetrate Lower Tertiary and Quaternary sediments. The basement offset is 85 meters, down to the west. Upper Cretaceous rocks are displaced 45 meters and inferred displacement of Quaternary strata is about 10 meters. These displacements diminish toward the terminae of the fault.

Evidence of faulting was found in the Long Island continental shelf (Reference 2.5-48). These faults were interpreted to strike northwest with the upthrown block on the seaward side. The inferred age of these structures is at least post-Late Cretaceous.

A large normal fault was detected from continuous seismic profiles on the continental shelf off Rhode Island (Reference 2.5-49). This fault, referred to as the New Shoreham Fault strikes north-south and dips to the east. It has been traced for a distance of 60 kilometers, and displaces the basement as well as Upper Cretaceous, Eocene, and possibly younger strata.

In addition to the structures described above, several similar structures are located outside the site region in the Carolina Coastal Plain. The Belair Fault Zone occurs near the Fall Line (Reference 2.5-50) and a series of en-echelon faults are mapped in the general area of Charleston, South Carolina. These latter faults are known as the Cooke Fault and the Helena Banks Fault.

2.5.1.1.4 Geologic History

The fundamental structures resulting from the tectonic events comprising the site region geologic history are described in Section 2.5.1.1.3. The following discussion is a historical summary of the events that produced the structural framework of the site region.

The results of studies of geologic structure, petrographic, and faunal provinces, when considered in the context of current plate tectonic theory, indicate there are three major stages in the evolution of the region (References 2.5-21, 2.5-22, 2.5-23, 2.5-24, and 2.5-25). These are: initial crustal divergence in late Precambrian to early Paleozoic time, crustal convergence in Ordovician to Carboniferous time, and renewed crustal divergence in Mesozoic time. Rocks comprising the crystalline basement of the site region were formed during the first two stages. The deposition of the rocks of Triassic and Jurassic age, and the seaward thickening wedge of the Coastal Plain sediments are products of the third stage.

The tectonic framework of the Appalachians and the adjacent continental margin appears to have been established in late Precambrian time (Reference 2.5-21). At this time, plate divergence along the eastern margin of the ancient North American continent resulted in formation of a symmetric series of northeast trending depositional troughs on the flanks of the cratons. After a significant period of plate divergence, subsequent plate convergence further enhanced the previously established tectonic framework. This plate convergence resulted in development of the Appalachian orogen, including the fold and thrust belt as well as magmatic and metamorphic belts. Later, elements of these ancient cratonic margins representing the most fundamental and profound structures served as loci for renewed divergent plate movement.

2.5.1.1.4.1 Initial Divergence

Initial crustal divergence occurred in late Precambrian time following the completion of the Grenvillian orogenic cycle. This process initially caused the separation of the North American and African plates and finally resulted in the formation of the proto Atlantic Ocean (References 2.5-21 and 2.5-22). In conjunction with the initial rifting phase, an eastward thickening wedge of clastic sediments consisting of graywackes, arkoses, and shales, interbedded with volcanic rocks, were deposited unconformably on the Grenvillian basement in deep water-filled basins within the ancient continental margin (Reference 2.5-21). These rocks are presently exposed on the eastern side of the Blue Ridge Anticlinorium (Reference 2.5-22). West of the site, they are represented by the Ashe, Lynchburg, and Catoctin Formations (Reference 2.5-22). Radiometric age determinations also suggest that the Yonkers Gneiss of the Manhattan Prong and the Dry Hill Gneiss (Pelham dome of western Connecticut) are northern equivalents of these rocks (Reference 2.5-51).

As rifting progressed, the proto Atlantic Ocean opened and the previous system of isolated rift basins was superceded by a long depositional trough underlain in part by oceanic crust. This trough was located primarily between the ancient margin of eastern North America (Figure 2.5-8), and the ancient western margin of the Avalon platform (References 2.5-21 and 2.5-24). The later phase of the crustal divergence is marked by two rock assemblages; one represents the sediments deposited along continental margin and in the oceanic trough; the other represents oceanic crust that was emplaced during the plate divergence.

The sediments composed a continental terrace wedge developed as a great carbonate bank over the stabilized ancient continental margin, and a Middle to Late Cambrian transgressive basal clastic sequence over the Grenvillian basement across the craton (Reference 2.5-21). Remnants of these sediments are found within the Piedmont-New England and Valley and Ridge Physiographic provinces, which correspond to rocks of Cambrian and Early Ordovician age (Reference 2.5-22). Within the site region, these remnants are represented by various lithologic units such as the Chilhowee Group, the Hardyston sandstone, Kittatinny Limestone, the Poghquag Quartzite, and Inwood marble.

Remnants of the oceanic crust comprise an ophiolite sequence and are known only from the northeastern portion of the orogen south of Logan's line and possibly as the Baltimore-State Line gabbro-peridotite complex (Reference 2.5-22). Otherwise, it appears that the oceanic crust was largely consumed by subduction during the subsequent convergent stage.

2.5.1.1.4.2 Convergent Stage

The convergent stage of the site region geologic history is essentially the history of the closing of the proto Atlantic Ocean. It began in Ordovician time with the onset of the Taconic orogeny. The earliest phases of this stage are evidenced by a pre Middle Ordovician unconformity (Reference 2.5-52) which is thought to reflect events terminating the episode of crustal extension (Reference 2.5-22). This was followed by the influx of detrital sediments (flysch) over the previous carbonate bank along the cratonic margin. At the height of the Taconic orogeny, ophiolitic rocks (presumably oceanic crust) were obducted from the eugeosynclinal and the miogeosynclinal (continental terrace), and detrital and carbonate sediments were thrust onto the craton (References 2.5-21 and 2.5-24).

The close of the Taconic orogeny marked the destruction of the ancient continental margin and the development of the mature arc trench subduction system. Taylor and Toksoz (Reference 2.5-53) regard the Taconic phase as the collision between the North American continent and an island arc that lay between the two continents. The subsequent Acadian orogeny resulted from continent to continent collision and produced additional crustal shortening, magmatism, and metamorphism (Reference 2.5-24). It also resulted in final closure of the already contracted proto Atlantic Ocean (Reference 2.5-53). Apparently, the culmination of the closing of the proto Atlantic Ocean in the southern Appalachians occurred later, in the Carboniferous time (Reference 2.5-27). It appears that during the Carboniferous period convergence was occurring in the southern portion of the Appalachians (Allegheny orogeny), whereas in the north, it was a period of translational movements (Reference 2.5-53). In the southern Appalachians the Allegheny orogeny has been interpreted to be the result of ultimate convergence of the North American and African continents, and full demise of the proto Atlantic (Reference 2.5-53).

The development of the crystalline basement of the Appalachian mobile belt (Figure 2.5-8) underlying portions of the New England and Piedmont Physiographic Provinces as well as the present Atlantic Continental Margin in the site region are attributed entirely to the convergent stage. This development has included geosynclinal sedimentation, magnetic activity, and repeated regional metamorphism.

2.5.1.1.4.3 Final Divergence

The development of the present Atlantic Continental Margin was initiated in Early Mesozoic time. The process was caused by divergence interaction between the North American and African plates, and ultimately lead to the opening of the Atlantic Ocean. This process of plate divergence is the youngest regionally recognizable diastrophism and is characterized by vertical crustal movements resulting in faulting along pre-existing planes of crustal weakness, continental and marine sedimentation, and extrusive and intrusive igneous activity. Based on available geologic evidence, it is possible to subdivide the history of the development of the continental margin into Middle to Late Triassic, Jurassic, and Cretaceous phases.

The oldest phase of the development of the continental margin occurred in Middle to Late Triassic time. In general, this phase was characterized by the development of a series of northeast trending rift valleys, more than 70 miles wide, from the Carolinas to the Bay of Fundy (Figure 2.5-9). A thick series of continental strata were deposited within the rift basins and several periods of diabase extrusion and intrusion accompanied the sedimentation (Reference 2.5-24). Deep borings and seismic surveys indicate that several basins (Figure 2.5-9) filled with Triassic rock overlying the Precambrian Paleozoic rocks, are present beneath the Coastal Plain (Reference 2.5-7). The shapes of the individual basins and groups of basins, as well as the positions of the diabase sills within them, are strongly concordant to the structural grain of the pre-existing orogenic belt. This suggests that the development of the rift system utilized the old structural framework.

The second, Jurassic phase of development is characterized by large scale subsidence and the accumulation of thick sedimentary sequences in depocenters located seaward of the previously formed basins (Figure 2.5-9). This phase is equivalent to the early opening of the North Atlantic Ocean (Reference 2.5-23). These depocenters continued for the most part to receive continental sediments throughout Jurassic time. The subsidence produced series of elongated basins, which in the area of the continental slope and rise are as deep as 10-12 kilometers (References 2.5-7 and 2.5-17). These basins are filled with upper Mesozoic and Cenozoic sediments that overlie a major unconformity cut across the crystalline basement rocks and early Mesozoic red beds. This sedimentary cover thins westward and wedges out at the inner edge of the Coastal Plain, or the Fall Line.

As the opening progressed, a narrow and shallow early Atlantic sea transgressed into the area of the outer Baltimore Canyon Trough. During Early Jurassic time, this led to the deposition of a thick evaporite sequence which grades up into carbonate strata (Reference 2.5-11). The carbonate strata resulted from bank and reef growth along the Jurassic outer shelf edge (Reference 2.5-10). Fluvial sediments were deposited over the Triassic rifted basins landward of the carbonates. During Jurassic to Early Cretaceous time, as the ocean continued to widen, the carbonate complex prograded seaward over the oceanic basement (Reference 2.5-10). In the Coastal Plain area, extensions of the Baltimore Canyon Trough developed as the Salisbury and Raritan embayments, which received thick sequences of Jurassic and Early Cretaceous fluvial sediments (Reference 2.5-10). In Early Cretaceous time shallow marine incursions began to extend landward of the carbonate complex (Reference 2.5-10) marking initiation of the third phase of development of the continental margin.

This phase of development is characterized by continuous but decreased crustal subsidence and related deposition of detritic materials forming the wedge of unconsolidated Coastal Plain sediments. Analysis of drill hole data from the offshore wells indicate that the marginal subsidence proceeded episodically. In the past 100 million years intervals of slower subsidence (greater than 50-100 meters/million years) alternated with intervals of slower subsidence or even modest uplift (Reference 2.5-7). Presence of arches and embayments that have a structural relief ranging from 1 to 2 kilometers indicate that subsidence was nonuniform, affecting some places more than others.

Subsidence and related sediment accumulation generally decreased through time and were largely completed by Middle Tertiary Time. In places minor crystal deformation continued as indicated by Pliocene and Quarternary shorelines that are slightly warped (Reference 2.5-54 and 2.5-55). The subsidence, during the Cenozoic Era, was accompanied by faulting of modest proportions. The principal style of deformation known to have occurred along the continental margin is reverse faulting (Reference 2.5-7). Recently performed investigations, offshore as well as onshore, revealed several examples of such faults (Section 2.5.1.1.3).

The time span represented by the final divergence, especially the third phase, represents only the past 100 million years of Earth's history. One current viewpoint that is gaining in acceptance is that the relative elevation of the continents with respect to sea level has remained essentially constant during the past 2,500 million years (Wise, 1974, Reference 2.5-147). On a globe whose tectonic processes call for creation and consummation of crust and water, tectonic adjustments are necessary to prevent the disappearance of continents by erosion or of the oceans by continental accretion. There have been limited deviations from the constant continent-ocean volume which have spanned 75 to 100 million years, equivalent to the span of the third phase of continental divergence. During this most recent span there has been a general trend of uplife of the continental interior and subsidence of continental margin. Changes in the rates of one relative to the other appear to be in concert. A number of mechanisms have been proposed to explain how it occurs, and a lengthy treatment of each is beyond the scope of this discussion. Examples of mechanisms proposed are:

- thermal contraction

- sea level fluctuation

- geoidal variation

- phase changes in the crust or asthemosphere

- changes in stress regime along the continental margin

- amplification due to isostatically induced loading.

The upper crust has become segmented by an abundance of northeast trending high-angle discontinuities that are relics of past tectonism. At geological rates of strain, the strength of the crust must be very weak, and reduced further by the predominant northeast-southwest "planar" anisotropy. Locally, rapid changes in crustal movement, sedimentation, and subsidence has occurred and concentrations of distortional strain energy develop. Hence, there must be failure of the high-angle discontinuities to resist shear slip. Depending on the stress regime, they will slip in a normal, reverse, or strike-slip manner (or a combination thereof).

Mid-ocean ridge spreading forces and tractions at the base of the crust impart high sub-horizontal stresses in the crust (Reference 2.5-68). Zoback and Zoback 1980 (Reference 2.5-148) have shown that the orientation of the tensor varies, although the maximum stress is generally sub-horizontal where measured or interpreted from focal mechanisms solutions. Statistically, therefore, reverse (thrust) fault-slip is expected as the dominant mode of seismic energy release in the site region. This is not to say, however, that normal- or strike-slip motion cannot occur (for example, the June 1973 Maine-Quebec border earthquake -- Aggarwal and Sykes, 1981, (Reference 2.5-1). Nevertheless, the record of instrumentally determined earthquakes indicates that reverse faulting is dominant. This is not expected to change during the next century.

2.5.1.2 Site Geology

This section presents a summary of the geologic conditions at the HCGS site. It provides information concerning the physiography, stratigraphy, structure, geologic history, and engineering geology which specifically pertains to the site. The information summarized in this section is derived from the results of several geologic and geotechnical investigations conducted at the site using subsurface exploration with borings, geophysical surveys, and detailed mapping of excavations. Detailed discussions of these investigations have been presented in a series of reports previously submitted to the USNRC (References 2.5-56, 2.5-57, 2.5-58, and 2.5-59).

The mean sea level according to the National Geodetic Vertical Datum (NGVD) corresponds to Elevation 89.0 feet of the PSE&G Plant Datum.

2.5.1.2.1 Physiography

The HCGS site is located in Salem County, New Jersey, approximately 18 miles south of Wilmington, Delaware (Figure 2.5-1). It is situated on Artificial Island along the east short of the Delaware River Estuary. The extent and elevation of original river bar which occupied this location was increased by the emplacement of hydraulic fill derived from the adjacent river channel.

The site is located within the Atlantic Coastal Plain Physiographic Province (Figure 2.5-2). It is situated approximately 18 miles southeast of the Fall Line which separates it from the Piedmont Physiographic Province. A summary of the regional physiography is presented in Section 2.5.1.1.1.

The morphology of the site is typical of locations within the Coastal Plain (see Figure 2.5-11a). The Delaware River Estuary borders the site to the west and south. Most areas immediately north and east of the site consist of marine tidal marshes dissected by shallow stream channels. Surface drainage at the site is poor because of its low topographic relief and the relatively impermeable soils immediately below the surface.

The northern end of the island is covered with marsh grass in many places. A low-lying dike, constructed from soil fill, runs parallel to the shoreline on the south and west. The top of this dike ranges in elevation from about 16 to 19 feet above mean sea level (MSL).

The site surface is relatively flat and generally ranges in elevation between 6 to 30 feet above MSL. The highest terrain is in the area north of the reactor. This area forms a nearly rectangular bench about 6 to 10 feet higher in elevation than the central area where the Reactor and Turbine Building are situated. A broad, very shallow trough extends southward from the cooling tower location and then continues westward from the turbine building to the western edge of the site. This depression is a minor erosional feature which drains the higher elevations in the northern portion of the site.

2.5.1.2.2 Stratigraphy

The HCGS site is underlain by a thick sequence of terrestrial and marine strata deposited on a pre-Cretaceous basement surface (Figure 2.5-11a). This stratigraphic sequence is about 1500 feet thick, and the sediments vary from Early Cretaceous to Holocene in age. The principal plant structures are founded on the Vincentown Formation of Tertiary age. A complete discussion of the regional Coastal Plain stratigraphy is presented in Section 2.5.1.1.2. This discussion will predominantly pertain to the upper 450 feet of the stratigraphic sequence for which site specific information was obtained. Figures 2.5-11 through 2.5-15 illustrate the sequence and configuration of the stratigraphic units at the site. All elevations specified in the text and on the figures are related to the PSE&G datum established at the site. Elevation 100 is equivalent to 11 feet above MSL.

2.5.1.2.2.1 Pre-Upper Cretaceous Strata

The lowermost sedimentary units underlying the site consist of Early Cretaceous strata which are non-marine in origin and unconformably overlie the basement rocks. The Early Cretaceous sediments are represented by a series of stratigraphic units referred to as the Potomac Group. This stratigraphic group consists of discontinuous beds of sand, clay, and silt, deposited as continental fluvial and deltaic detrital sediments. The clastic wedge of sediments is estimated to be about 1200-200 feet thick beneath the HCGS site (Reference 2.5-57). Potomac Group sediments were not penetrated by borings at the site with the exception of Figure 2.5-11a. Section 2.5.1.1.2 provides further discussion of the sedimentological characteristics of this group.

2.5.1.2.2.2 Upper Cretaceous Strata

The Upper Cretaceous strata consist of a sequence of formations which record the change from non-marine to marine deposition. The marine formations of the sequence reflect deposition during three marine transgressions and two regressions. These stratigraphic units are the Raritan and Magothy Formations, as well as the formations of the Matawan and Monmouth Groups (Figure 2.5-11).

1. Raritan and Magothy Formations

   The Raritan Formation is predominantly of non-marine origin and consists mostly of light colored interbedded sand and clay. This formation was not penetrated by borings at the site. Hence, the detailed description of this stratigraphic unit is presented in Section 2.5.1.1.2.

   The Magothy Formation (Figure 2.5-11) disconformably overlies the Raritan Formation and is differentiated on the basis of increasing clay content. The lithologic character of the Magothy Formation indicates that it is a coastal deposit, representing the initial marine transgression in Late Cretaceous time (Reference 2.5-10). Only the deepest boring (201) at the site terminated in the Magothy Formation at a depth of 451 feet (Elevation -351 feet). Only one sample consisting of white, coarse to fine silt was obtained from the top of this formation. The geophysical log of this borehole indicates that the upper surface of the Magothy Formation is at elevation -316 feet (Figure 2.5-12).

2. The Matawan Group

   The Matawan Group is represented by the Merchantville, Woodbury, Englishtown, Marshalltown, and Wenonah

Formations (Figure 2.5-11a). With the exception of the Englishtown Formation, these stratigraphic units are massive shelf and inner shelf sediments accumulated during both transgressive and regressive phases of marine deposition. The Englishtown Formation consists of near-shore coastal sediments deposited during the regressive phase of the Late Cretaceous marine incursion.

The Merchantville Formation disconformably overlies the Magothy Formation and is the oldest glauconitic unit in the New Jersey Coastal Plain (Reference 2.5-13). The Merchantville sediments represent massive shelf deposits of a marine transgression. Two borings (201 and 206) penetrated the Merchantville Formation at the site (Figure 2.5-12). The thickness is approximately 28 feet. The elevation of its upper surface is approximately -290 feet and slopes to the south between the two borings at approximately 90 feet per mile. One sample was obtained from each boring at Elevation -300 feet, near the top of the formation. In these samples, the sediment consisted of dark, brown to black silt with a trace of clay. An abrupt change to dark green clay occurred at Elevation -301.

The Woodbury Formation conformably overlies the Merchantville Formation and indistinguishably grades upward into the Englishtown Formation (Figures 2.5-12). These strata are marine regressive inner shelf and nearshore sediments, respectively. Borings 201 and 206 encountered the top of the Englishtown Formation at Elevations -212 and -218 feet, respectively. The combined thickness of the units was approximately 75 feet in both borings. The upper contact slopes approximately 85 feet per mile southward between the two borings. At the site, the thickness of this combined unit suggests that it consists mostly of a finer grained facies of the

Englishtown Formation, which is not easily distinguishable from the clayey silts of the underlying Woodbury Formation (Reference 2.5-57). Samples obtained from these sediments contained gray to black clay, silty clay, and clayey silt, with trace quantities of very fine, uniformly disseminated mica. Most samples taken near the top of the Englishtown Formation contained fine, thin, calcareous shell fragments with fine sand partings.

The Marshalltown Formation conformably overlies the Englishtown Formation (Figure 2.5-12). Typically, the Marshalltown Formation is conformably overlain by the Wenonah Formation. Both units are comprised of sediments indicative of a shelf depositional environment. However, the lower unit was deposited during a transgressive phase and the Wenonah Formation represents a marine regression. The lithologic similarities of the Marshalltown and Wenonah Formations at the site make differentiation of these two units difficult (Reference 2.5-57).

Examination of samples, and interpretation of geophysical logs (Figure 2.5-12) indicate that Borings 201 and 206 penetrated the Wenonah Formation at Elevation -174 feet. The upper contact of this lithologic unit is essentially horizontal between the two borings. The Marshalltown-Wenonah Unit is about 40 feet thick. Owens and Minard (Reference 2.5-14) state that the Marshalltown Formation is consistently 10 to 15 feet thick throughout the New Jersey Coastal Plain; therefore, it is interpreted that the Wenonah Formation comprises the upper 25 to 30 feet of this lithologic unit at the site (Reference 2.5-57). Samples from within this unit consisted of gray clayey silt and fine sand with trace quantities of glauconite and mica.

3. Monmouth Group

At the site only the two oldest formations of the Monmouth Group are present (Reference 2.5-57). These are the Mount Laurel and Navesink Formations (Figures 2.5-11a and 2.5-12). The Mount Laurel Sand was deposited in a relatively nearshore environment during a marine regression while the overlying Navesink Formation was deposited in the deeper water shelf environment during a successive transgressional phase. The Redbank, New Egypt, and Tinton Formations, which compose the upper strata of the Monmouth Group elsewhere on the Coastal Plain (Section 2.5.1.1.2), are not present at the site (Reference 2.5-57).

The Mount Laurel Formation was encountered in a large number of borings at the site (Figures 2.5-13, 2.5-14, and 2.5-15). Only borings 201 and 206 penetrated the entire formation. In these borings, the formation varied from 109 to 104 feet thick. The top of the Mount Laurel Formation is generally encountered between Elevations -64 and -76 feet (Figure 2.5-13). The upper surface of the Mount Laurel Formation locally slopes toward the southeast, between 25 and 35 feet per mile.

Within the upper 10 to 15 feet of the Mount Laurel Formation, the sand fraction consists of equal quantities of fine to coarse, subangular to subrounded quartz sand and glauconite sand. The silt fraction makes up about 35 to 50 percent of the entire sediment with only trace quantities of clay and few shell fragments. A zone between Elevations -80 and -90 feet usually contains semi-consolidated layers or nodules. Several of these nodules consist of highly weathered, cemented shell fragments. The sand silt ratio at Elevation -90 feet is similar to the top of the formation, but the percentage of quartz in the sand fraction increases to 70 percent with glauconite decreasing to 30 percent. Shell fragments remain in trace quantities. The quartz to glauconite ratio continues to change with depth. At Elevation -100 feet this ratio is 90 to 10 percent with glauconite decreasing rapidly to trace quantities below Elevation -110 feet. Below Elevation -110 feet, the silt content decreases abruptly and cleaner sands with finer grain size begin to appear.

Although it has not been recognized in surface outcrops (Figure 2.5-11a), the Navesink Formation conformably overlies the Mount Laurel Sands in the subsurface (Figure 2.5-12 and 2.5-13) and is readily identified by its unique mineralogy (Reference 2.5-57). The thickness of this formation ranges from 20 to 22 feet at the site. Its upper contact was penetrated by borings at approximately Elevation -50 feet, and generally slopes toward the southeast at approximately 30 feet per mile. The sand fraction in the upper strata of the Navesink Formation contains about 90 percent subrounded to round glauconite and 10 percent fine to medium, subangular quartz sand. Many borings encountered numerous pelecypod fragments near the upper contact. The base of the Navesink Formation is characterized by a 4 to 6 foot thick zone where the glauconite content comprises more than 95 percent of the sand size fraction.

2.5.1.2.2.3 Tertiary Strata

The Tertiary strata at the site consist of the Hornerstown, Vincentown, and Kirkwood Formations (Figure 2.5-11a). The Hornerstown and Vincentown stratigraphic units consist predominantly of glauconite sand deposited in an inner and mid-shelf marine environment. The Hornerstown Formation appears to be transgressive in character, while the Vincentown was deposited during a marine regression (Reference 2.5-10). These units are unconformably overlain by the nearshore marine regressive sand and clay of the Kirkwood Formation (Figure 2.5-11a).

In the subsurface, the Hornerstown Formation unconformably overlies the Navesink Sands. The thickness of this formation, below the site, ranges between 14 to 20 feet and is generally encountered at Elevation -25 feet (Figures 2.5-13, 2.5-14, and 2.5-15). The Hornerstown Formation consists of fine to medium sand and 25 to 40 percent silt. The sand fraction in the upper portion of the formation is predominantly fine to medium, subangular to subrounded quartz, with 15 to 30 percent fine to medium, subround and botryoidal glauconite. Fine shell fragments comprise approximately 5 percent of the sand. The relative percentage of quartz to glauconite changes with depth. Quartz sand grains decrease to 30 percent, whereas, the glauconite content correspondingly increases to approximately 60 to 70 percent of the sand fraction. Shell content remains in trace quantities; however, the basal sands of the Hornerstown Formation are usually characterized by large, angular pelecypod fragments, up to one inch long. The upper contact of the Hornerstown Formation was determined by noting an increase in silt, a rapid increase in size and quantity of glauconite, and a corresponding decrease in shell fragments (Reference 2.5-57). The above characteristics vary somewhat between borings due to the gradational nature of the Vincentown Hornerstown contact.

The Vincentown Formation conformably overlies the Hornerstown Sands (Figures 2.5-11a, 2.5-13, 2.5-14, and 2.5-15). Because of the erosional relief on its upper surface, both the thickness of the Vincentown Formation and the elevation of its upper contact is somewhat variable. Boring 206 penetrated 57 feet of the Vincentown sands, and 73 feet of this unit was present at Boring 216. The elevation of the upper surface of this unit ranges between 26 feet and 57 feet across the site (Reference 2.5-57).

The Vincentown Formation is a relatively homogeneous stratigraphic unit predominantly composed of greenish gray, fine to medium grained sand above Elevation +10 feet and fine sand below Elevation +10 feet. The sand fraction of the sediments is composed of calcium carbonate fragments (including shells, bryozoa, echinoid spines and foraminifera), detrital quartz, and glauconite. The ratio of detrital quartz to shell fragments is not predictable at any given depth. Glauconite usually represents less than 10 percent of the sand fraction, but it can range to nearly 20 percent in concentrated zones. Both the Hornerstown and Vincentown Sands contain numerous thin hard layers which resist penetration by sampling spoons due to cementation of the sand grains by calcium carbonate. Detailed petrographic studies were conducted to investigate the character of cementation (Reference 2.5-57, 2.5-58, and 2.5-59). The cemented layers usually range in thickness between several inches and one foot. The upper 30 to 40 feet of the Vincentown Formation generally consists of quite strongly cemented rock lenses interspersed with partially cemented silty sands, while the lower portion of the Formation is more variable, ranging from slightly cemented sands to rock (Reference 2.5-56). Single point electrical resistance logs (Figures 2.5-13 and 2.5-14) clearly define the elevations of these cemented zones and also show that resistance is lower in the weathered zones due to lack of cementation.

The Vincentown Formation is the foundation stratum for HCGS (Section 2.5.4). The composition of this sandstone unit is calcareous, glauconitic, variably cemented fine to medium grained quartz sand. Although the sandstone is not divisible into refined stratigraphic units. Three lithologically distinct layers were exposed to the walls of the foundation excavations (Reference 2.5-59). These layers are from lowermost to uppermost:

1. Layer 1 - grayish green, calcareous, glauconitic sandstone

2. Layer 2 - oxidized yellow to reddish brown weathered sand and sandstone

3. Layer 3 - unweathered dark green, clayey, glauconitic sand.

A plot plan of the main excavation is shown on Figure 2.5-16. Figures 2.5-17 and 2.5-18 are geologic profiles of the slopes of the walls of the main excavation. The locations of profiles are shown on the plot plan (Figure 2.5-16).

Layers 2 and 3 were not approved as suitable foundation material (Section 2.5.4). Where they occurred, these layers were removed during excavation to the final foundation surface. The character of the individual layers is discussed below.

Layer 1, the deepest layer of the Vincentown exposed in the excavations is a variably cemented, light grayish green calcareous glauconitic sandstone (Figures 2.5-17 and 2.5-18). This layer extends below the floor of the excavation and forms the foundation for the power block structure of the facility. Layer 1 is composed of three lithologies: a quartz sand, a calcareous sandstone, and a pure limestone (Reference 2.5-59). The engineering descriptions (Section 2.5.4) of slightly, moderately, and highly cemented, correspond to these lithologies.

Petrographic analyses were performed on samples of the Vincentown Formation to determine the degree of cementation, which appears to vary, throughout the unit. The analyses were conducted as part of the post excavation foundation studies (Reference 2.5-59) and reported the following conclusions:

1. The mineralogy and texture of the sand-grain fraction is uniform and distinct bedding structures do not occur within the Vincentown in the site area.

2. The entire formation has been subjected to some degree of cementation.

3. Local recrystallization, precipitation, and solution of calcium carbonate has resulted in a sandstone which varies in degree of cementation in both lateral and vertical directions.

4. The effects of cementation are not always readily apparent in hand specimens. Some samples visually classified as uncemented in the field were found to be partially cemented in the laboratory.

5. Evidence indicates that significant alteration of the Vincentown occurred prior to deposition of the overlying strata.

Layer 2 of the Vincentown Formation occurs locally above the light green foundation strata and consists of a distinctive oxidized silty quartz sand. This layer is mottled yellow brown to reddish brown and is partially cemented in iron-oxide and/or calcite cement. The oxidized sediment appears less calcareous and glauconitic than the underlying non-oxidized portion of the Vincentown Formation.

The contact between the oxidized sediment (Layer 2) and the non-oxidized strata (Layer 1) is a distinct but highly irregular surface. This surface appears to be a geochemical "front" whose position is a product of post-depositional chemical and groundwater conditions rather than any depositional stratigraphic control (Reference 2.5-59). The nature of the contact suggests that the oxidized sediments are weathered equivalents of the underlying non-oxidized strata.

Layer 3 interpreted to be part of the Vincentown Formation (Reference 2.5-59), is a dark green, clayey, highly glauconitic quartz sand. It commonly occurs in the excavations in two modes consisting of a 0.5 to 5 foot thick layer over most of the upper surface of the Vincentown Formation, and localized pods of glauconite rich sands which appear to fill small channels or surface depressions within the upper part of the formation. In some instances, the margins of the pods and the lower edges of the capping glauconitic sand layer, appear to be rimmed with and partially cemented by a reddish-brown iron-oxide zone.

The origin of Layer 3 is not certain, and it has been interpreted to have resulted from erosional reworking of the upper Vincentown Formation prior to formation of the underlying highly oxidized facies (Reference 2.5-59). Alternatively the clayey glauconitic sand (Layer 3) overlying the weathered Layer 2 of the Vincentown Formation could be interpreted as a thin layer of the Manasquan Formation. The lithology of Layer 3 is similar to that provided for the Manasquan Formation (Reference 2.5-13). Furthermore, Layer 3 occurs above what appears to be an erosional surface developed on the Vincentown Formation. The Manasquan Formation is similarly described as disconformably overlying the Vincentown unit.

The Kirkwood Formation unconformably overlies the glauconitic sand unit (Figure 2.5-11a). These sediments vary considerably in thickness because of the irregular configuration of the upper and lower Kirkwood contacts (Figures 2.5-17 and 2.5-18). The upper surface of the Kirkwood Formation is usually encountered between elevations 56 and 64 feet. Based on information from borings and the excavation mapping, the Kirkwood strata are subdivided into two lithologically distinct subunits consisting of a 2 to 6 foot thick basal sand layer overlain by a thicker gray micaceous silty organic clay. The Kirkwood Formation identity of the two subunits is based upon their similarity to lithologies described within the Kirkwood unit stratigraphic position, and results of micropaleontologic analysis (Reference 2.5-59).

The basal sand unit is composed of numerous gradationally intercalated beds of silt, sand, gravel, and cobbles. The overlying organic clay layer is composed of a lower woody peat and a peaty and clayey silt that grades upward into a silty organic clay. Radiocarbon analysis in both the wood and peat samples, from the lower organic clay unit, indicates that their age is greater than 37,000 years before present (Reference 2.5-59).

The contact between the organic clay and basal sand appears to be both unconformable and conformable in the excavation (Figures 2.5-17 and 2.5-18). The nature of the contact is interpreted to indicate that the two subunits are of similar age, but represent deposition from two laterally varying, adjacent but sharply different, sedimentary environments (Reference 2.5-59). An initial fluvial environment appears to have been slowly replaced by a fresh-water marsh environment.

2.5.1.2.2.4 Quaternary Strata

The Kirkwood Formation is overlain by three lithologic units of Quaternary age. The lowest of these is a non-organic clay unit which directly overlies the organic clay of the Kirkwood Formation (Figures 2.5-17 and 2.5-18). The non-organic clay unit was distinguished from the Kirkwood clay on the basis of its texture, composition, and micropaleontologic content (Reference 2.5-59). The non-organic clay unit varies in thickness from approximately 5 to 20 feet. In general, it is thickest in the southern portion of the excavation and thins to the north. The Quarternary age of this unit is based on mineralogic and micropaleontologic analyses (Reference 2.5-59).

The non-organic clay unit is overlain by coarse sands and gravels that formerly comprised the bed of the Delaware River. The thickness of these strata vary across the site from 2 to 12 feet and they consist of gray, fine to coarse, subround to round quartz sand and fine to coarse, subround to round gravel with a trace of mica and glauconite. The uppermost and very recent unit consists of hydraulic fill, composed of irregular discontinuous lenses of gray, micaceous, clayey silt, fine silty sands, and organic clays varying from 30 to 45 feet in thickness.

2.5.1.2.3 Structural Geology

The structural geology of the HCGS site has been investigated using an extensive boring program, seismic refraction surveys, and detailed examination and mapping of the foundation excavations. The results of these investigations are presented in detail in previously docketed reports (References 2.5-57, 2.5-58, and 2.5-59). The regional geologic structure is summarized in Section 2.5.1.1.3. This section consists of a summary of the conclusions based on the results of earlier site investigation.

It is concluded from the site geological studies that the structure of the sedimentary strata below the site is limited to a sequence of uniformly dipping strata. No folds, shear zones, abrupt changes in stratigraphic elevations, or anomalous sequences of strata were detected (Reference 2.5-57). The structure of the strata at the site conforms with the regional character of the Atlantic Coastal Plain. It consists of a gently southeastward dipping homocline composed of a southeastward thickening wedge of sedimentary strata deposited on the pre-Cretaceous age basement complex that lies approximately 1500 feet below the ground surface (Figures 2.5-11a, Reference 2.5-60).

Specific information regarding the geologic structure at the site is limited to the upper 450 feet of strata (Reference 2.5-57). Therefore, the structure of deeper geologic elements can only be inferred on the basis of regional geologic data (Section 2.5.1.13).

The slope of the basement surface is variable along the continental shelf; however, below the site it appears to slope uniformly to the southeast at approximately 75 to 100 feet per mile (Figures 2.5-6 and 2.5-11a). Overlying this surface is a thick sequence of post-Jurassic terrestrial and marine sediments which strike northeast and dip gently to the southeast. The dip of these strata appears to decrease up through the sequence as a result of the eastward thickening of the stratigraphic section.

Continuity of the stratigraphic units within 200 feet of the surface was established with information obtained by subsurface exploration with borings and confirmed using geophysical techniques (Reference 2.5-57). The configurations of these strata are shown on Figures 2.5-13 through 2.5-15. Beneath the unconformity developed on the top of the Vincentown Formation, the strata maintain relatively uniform stratigraphic thicknesses and dip to the southeast at approximately 30 feet per mile. The conformable contact between the Mount Laurel Formation and the overlying Navesink Formation is a particularly distinct horizon beneath the site. This contact clearly represents a uniform surface below the site that exhibits minor undulations but lacks any discontinuities or abrupt changes in elevations.

The unconformable contact between the basal sands of the Kirkwood Formation and the underlying Vincentown Formation is undulating displaying as much as 20 feet of relief (Figures 2.5-14 and 2.5-15). Subaerial exposure and fluvial erosion of the upper Vincentown unit is evident from the character of samples from borings and the examination of exposures in foundation excavations. Figure 2.5-19 is a subsurface contour map of the top of the Vincentown Formation across a large part of the site. All the relief on this surface is attributed to erosional processes (References 2.5-57 and 2.5-59). The thickness of the Kirkwood Formation changes considerably at the site, yet, these variations directly correspond to the relief observed on the underlying Vincentown unit (Reference 2.5-57).

Three stratigraphic contacts were mapped in detail as part of the examination of the foundation excavations. The three horizons are the upper and lower contacts of the Kirkwood Formation basal sand and the top of the Kirkwood organic clay (Section 2.5.1.2.2). The configurations of these horizons are illustrated on the profiles of the walls of the site excavations (Figures 2.5-17 and 2.5-18).

Generalized structure contour maps of the top of the Vincentown and top of the Kirkwood basal sand unit within the main excavation (Figure 2.5-16) are shown on Figures 2.5-20 and 2.5-21. The structure maps are contoured using detailed topographic data from the main excavation and supplemented with information from borings within the excavated portion of the site.

Across the site in general, the top of the Vincentown Formation represents a surface of subaerial exposure and terrestrial erosion as indicated by its relief (Figure 2.5-19) and the zone of oxidation. The elevation of the contact ranges from approximately 51 feet in the northeast area of the excavation to 31 feet in the southwest area (Figure 2.5-20). Several small channels, or shallow depressions, contribute to the undulatory character of this surface (References 2.5-57 and 2.5-59).

There is no definite explanation for the very strong NNW oriented grain on the surface of the Vincentown formation and the contact between the two horizons within the Kirkwood formation (shown on Figures 2.5-19, 2.5-20, and 2.5-21, respectively). The contour intervals in these figures are either 1 foot or 2 feet. Thus, the control provided by these data is good and the NNW trend may be considered accurate. The relief illustrated may be more interpretive. The origin of this NNW trend may be interpreted in several ways. These possibilities are discussed below.

The top of the Vincentown formation (base of the Kirkwood formation) is an unconformable surface which is erosional in origin. Perhaps the NNW-oriented grain results from variable erosion effects, influenced by surface drainage, currents, or other factors. The Kirkwood sediments were deposited on an irregular surface, with their configuration reflecting that of the unconformity.

It is noteworthy that the structure contours illustrated in Figures 2.5-19, 2.5-20, and 2.5-21 are approximately parallel with the long axis of Artificial Island which was created by hydraulic dredging. Perhaps there is a resistant layer along the river bed which influenced the direction of the dredging. This direction may further reflect the paleogeographic character of the Vincentown formation. However, this possibility is speculative.

Possible support for the importance of the paleography of the Vincentown Formation may be found in Reference 2.5-153. It has been shown that the buried, ancestral Delaware River channel does not coincide with the present Delaware River valley. The NNW grain observed in the Kirkwood and the top of the Vincentown could be controlled by the infilling of a former channel of the present Delaware River of Tertiary age. Again, there is no conclusive evidence to support or disprove this interpretation.

The NNW trend of the Vincentown surface and the contact between the horizons within the Kirkwood may be the result of the respective thickness variations of the formations as well as the different locations of the source terrains for each. The Vincentown Formation thickens to the northeast and southeast. The Kirkwood Formation thickens to the southwest and southeast, Reference 2.5-39. Figure 2.5-63 illustrates generalized isopach form lines for the New Jersey Coastal Plain for the top of the Kirkwood and Vincentown formations. There is a strong NNW orientation in the contours of the top of the Vincentown formation. There are areas within the surface of the Kirkwood formation which also illustrate this NNW trend although the evidence is more clearly observed in the Vincentown formation.

It is tempting to suggest that the orientation of the Kirkwood and Vincentown formation might be structurally controlled by the South New Jersey Uplift. Artificial Island is located on the southwest flank of the South New Jersey Uplift (Reference 2.5-13.). The trend of which is approximately north to northwest. However, no data are available covering a sufficient area to strongly support this hypothesis. In fact, isopachs of Cretaceous and Cenozoic formations (Reference 2.5-5) do not seem to reflect the basement relief at all.

In summary, there is no clear explanation for the cause of the NNW orientation of the top of the Vincentown and Kirkwood formations. This orientation is thought to be real -- resulting from one or more causes including erosional processes; the infilling of a buried river valley; the variation in thicknesses in the Kirkwood and

Vincentown formations; formation provenances; or structural control by the basement.

In the northeast corner of the excavation (Figure 2.5-17, Station W01+90) the slope of the upper contact of the Vincentown is interrupted by a minor, but abrupt, relief feature. Reference 2.5-59 (Appendix B, page B-23) states,

> "Careful examination, by extensive trenching <u>at and around this feature</u>, indicates that it is of an erosional origin. The overlying lower clay/basal sand contact. . . and intermediate bedding planes cross this feature with no evidence of disruption".

Moreover, photographs of this feature have been reviewed. The oxidized upper Vincentown sands are distinctively weathered. This can explain the undulatory surface. No evidence of fluidization of the sand or silt was ever observed. Moreover, the unit that truncates the sand layers is itself a very fine sand (Figure 2.5-17), and an excessive fluid pressure gradient probably would not be expected to develop across this horizon.

The overlying upper contact of the basal sand, as well as intermediate bedding planes, cross this feature with no evidence of disruption.

The top of the Kirkwood basal sand unit in the northeast and eastern portion of the excavation suggests that this contact represents an erosional unconformity. However, this was not consistently observed and in the southwest portion of the excavations, the contact is more gradational and apparently conformable (Reference 2.5-59). In places, a transition facies, between the lower organic clay unit and the underlying basal sands, indicates continued deposition throughout a gradual change or shift in depositional environments.

The upper contact of the Kirkwood basal sand undulates gently across the site with a maximum topographic relief of 11 feet

(Figure 2.5-21). The trace of this contact appears almost flat, lying to gently south dipping along the east and west walls (Figure 2.5-18). Elsewhere, the contact trace gently undulates and the depressions are filled with dark gray silty peat (Figure 2.5-17), and an excessive fluid pressure gradient probably would not be expected to develop across this horizon.

The uppermost stratigraphic contact mapped at the site separates the Quaternary non-organic clay unit from the underlying Kirkwood Formation organic clay unit. This contact is commonly marked by a persistent thin sandy gravel layer. The contact appears to be a relatively conformable surface that represents a brief hiatus between varying depositional environments (Reference 2.5-59). The trace of this contact on the east and west excavation slopes has a gently apparent dip to the south (Figure 2.5-18). It merges with the underlying upper contact of the basal sand in the northeast portion of the excavation where the lower clay unit appears to pinch out (Figure 2.5-17).

The detailed mapping of the aforementioned stratigraphic contacts preclude the presence of faults, shear zones or folds within the Tertiary and Quaternary sediments exposed in the site excavations. Several other kinds of smaller structures were observed in the excavations (Reference 2.5-59). Of these, clay seams were the only features not obviously related to construction activities. Their occurrence at shallow depth with respect to the dredge-cut surface, as well as their general morphology, suggest that the seams are filled tensional fractures created by the excavation procedures (Reference 2.5-59). The common technique of undercutting the dredge banks to induce large scale slumping, as part of dredging, could well create such tensional fractures in small local areas. These seams were all located within the topographically higher oxidized Vincentown Formation in the northeast corner of the excavation. No exposures of similar features were observed within the approved foundation strata or in the overlying strata.

2.5.1.2.4 Geologic History

The geologic history of the HCGS site closely parallels the history of the region as presented in Section 2.5.1.1.4. Hence, only the history of the site pertaining to the development of the Coastal Plain sediments will be discussed in this section. The post-Paleozoic history of the site was predominated by crustal divergence and associated vertical crustal movements. The early Mesozoic stage consisted of initial development of the continental margin and accumulation of a thick sequence of terrestrial sediments. The following stage was characterized by continued crustal divergence and subsidence reflected in the sequence of marine strata present at the site.

The development of the present Atlantic continental margin began in early Mesozoic time, approximately 220 million years ago with an episode of rifting which preceded the initial opening of the Atlantic Ocean. Rifting continued until possibly as late as Early Jurassic time (190 million years ago) and lead to the development of numerous isolated fault bounded basins along the margin. These basins were filled with continental sediments as well as extrusive and intrusive basaltic rocks, characteristic of crustal extension. The Newark-Gettysburg Triassic Basin, located northwest of the site (Figure 2.5-5) is one of these basins.

During the Jurassic period, initial formation of the Atlantic Ocean was occurring to the east of the site, as indicated by evaporite and carbonate rocks of that age (Reference 2.5-10). However, near the site Jurassic sediments are apparently not present, (Section 2.5.1.2.2) suggesting that erosion of the basement surface predominated throughout this period.

The site area underwent relative subsidence from the Early Cretaceous epoch into the beginning of Late Cretaceous time (140 to 90 million years ago). This facilitated the accumulation of the thick sequence of terrestrial sediments transported from a source to the west. These sediments are present at depths greater than 500 feet beneath the site and consist of approximately 1000 feet of fluvial deposits categorized as the Potomac Group and Raritan Formation. The disconformity developed on top of the Raritan Formation indicates that a relatively short period of erosion was followed by the deposition of the fluvial sediments.

Throughout nearly all of Late Cretaceous time (90 to 65 million years ago) further subsidence resulted in marine transgression and deposition of strata indicative of nearshore to midshelf environment. The Magothy Formation is a reflection of the initial marine transgression. Late Cretaceous units overlying the Magothy Formation (Section 2.5.1.2.2) also record smaller scale fluctuations in the rate of subsidence or sea level change. The majority of these stratigraphic units reflect inner to mid shelf conditions, whereas the Englishtown and Mount Laurel units record minor regressive phases suggestive of a decrease in the rate of subsidence. During Late Cretaceous time sedimentation briefly ceased in the site area, as evidenced by the disconformity above the Navesink Formation as well as by the absence of the uppermost Cretaceous Redbank and Tinton Formations at the site.

The Paleocene units at the site (Section 2.5.1.2.2) record another marine transgression, followed by subaerial exposure of the site as reflected in the erosion and oxidation of the upper portion of the Vincentown Formation. No record of sedimentation is present at the site for the period of time from the Early Eocene epoch (approximately 50 million years ago) to the late Miocene epoch (approximately 10 million years ago). This may represent the waning of the relatively rapid subsidence which persisted throughout Late Cretaceous and Early Tertiary time.

The Kirkwood Formation appears to represent a brief marine transgression during Late Miocene time. No large degree of subsidence can be interpreted from the presence of this unit because of the general lack of marine fossils within it. On the basis of the site sedimentary record it appears that since Late Miocene time the relative position of the site with respect to present sea level has remained relatively constant.

2.5.1.2.5 Site Engineering Geology

The engineering geology of the HCGS site has been investigated by extensive boring and laboratory testing programs, seismic refraction surveys, and detailed examination and mapping of the foundation excavations. The results of these investigations are summarized in Sections 2.4, 2.5.1.2, and 2.5.4. The detailed data and results of the site geotechnical investigations have been reported in previously docketed reports (References 2.5-56, 2.5-57, 2.5-58, and 2.5-59).

The geologic conditions underlying the Seismic Category I structures of the site are described in Sections 2.5.1.2.2 and 2.5.1.2.3 and are shown on the site cross sections (Figures 2.5-13, 2.5-14, and 2.5-15). The static and dynamic engineering properties of the site foundation strata are considered suitable for construction of the power facility as discussed in Section 2.5.4.

As discussed in Section 2.5.1.2.3, examination of excavation walls and samples from boreholes revealed no evidence which indicated adverse effects on the foundation soils from prior earthquakes. The seismology of the site is discussed in Section 2.5.2. The dynamic behavior of the site foundation materials during an earthquake is described in Section 2.5.4.

The results of thorough examination and mapping of the site excavations are summarized in Section 2.5.1.2.3. No deformational zones such as shears, joints, fractures, and folds, nor combinations of these features were identified at the site. As a result of these investigations it is concluded that there are no geologic deformational zones within the excavations which might have an adverse impact on the site foundations.

Zones of alteration and weathering were detected in the Vincentown Formation at the site. The lower portion of the Vincentown has undergone various degrees of cementation by normal diagenetic processes. Cementation of the Vincentown Formation has been investigated by petrologic analysis (Reference 2.5-59). In addition it was found to be suitable for foundation purposes on the basis of extensive testing programs. The results of these investigations are summarized in Sections 2.5.1.2.2 and 2.5.4. The upper portion of the Vincentown Formation displayed a zone of weathering of variable thickness (Section 2.5.1.2.2). This weathered zone was not considered suitable for the site foundations (Sections 2.5.4) and was removed in the area of the main excavation (Reference 2.5-59).

At the site no soils were detected that might be unstable because of their mineralogy or unstable physical and chemical properties. Section 2.5.1.2 describes the site stratigraphy and Section 2.5.4 provides information concerning the static and dynamic properties of the site foundation materials. The analyses of slope stability and liquefaction are presented in Section 2.5.4. This section concludes that there are no adverse safety-related soil interactions.

The conclusions of regional and site geologic and geotechnical investigations (Sections 2.5.1 and 2.5.4) indicate that no adverse conditions are expected to affect the site as a result of landsliding, subsidence, or karst conditions. Human activities in the area pertaining to withdrawal or addition of subsurface fluids or mineral extraction are also not anticipated to result in any adverse effects on the site. A summary of the hydrologic and groundwater conditions pertaining to the site safety are presented in Section 2.4. Past and present dredging activities are discussed in Section 2.5.4. There were no adverse effects resulting from these activities.

2.5.1.3 SRP Rule Review

SRP Acceptance Criteria 2.5.1.2.4.C requires that the engineering significance of unrelieved residual stresses in bedrock be provided.

Subsection 2.5.1.2, Site Geology, contains all of the required general site information required in the acceptance criteria with two exceptions. The information regarding dynamic behavior during prior earthquakes is cross-referenced to Sections 2.5.2 and 2.5.4. Because of the soil like properties of the bedrock formations at the site, unrelieved residual stresses are not treated.

2.5.2 Vibratory Ground Motion

This section provides a discussion and analysis of the seismotectonic characteristics of the HCGS site and the surrounding region. The purpose of this section is to develop appropriate seismic design criteria for major structures at the HCGS site in conformance with USNRC Regulatory Guide 1.70 (Revision 3); 10CFR Part100, Appendix A; and the Standard Review Plan for Section 2.5.2 (NUREG-0800).

A description of the results of the geologic investigation which provided the background information for this section are presented in detail in Section 2.5.1.

2.5.2.1 Historical Seismicity

The station is situated in a region which has experienced only a moderate amount of earthquake activity in the past. Earthquake occurrences in this region have been reported in historical records and newspapers since the early 18th century. Table 2.5-1 lists all earthquakes of Modified Mercalli Intensity IV or magnitudes greater than 3.0 within 200 miles of the HCGS site and all seismic events of any size within 50 miles of the site. Figure 2.5-22 shows significant seismic activity and tectonic provinces within the site region. Figure 2.5-23 shows all known seismic activity occurring within 50 miles of the site. None of these seismic events is of major or catastrophic proportion, although several of the events caused some minor structural damage.

The Newark-Wilmington, Delaware, area has been associated with minor earthquakes on February 28, 1973, and October 9, 1981. The maximum epicentral intensity of the 1973 event was a MMI (Modified Mercalli Intensity) VI. Figure 2.5-24 presents the isoseismal map for this earthquake. The epicenter of this event was located approximately 14 miles northwest of the HCGS site. It was barely perceptible at the site. Slight damage was reported from Northeast and Perryville, Maryland; Harrisonville, Laurel Springs, Palmyra, and Penns Grove, New Jersey; and New London, Norristown, Thornton, and Wallingford, Pennsylvania. The most significant earthquake of the region occurred near Wilmington, Delaware on October 9, 1871. This earthquake was located approximately 15 miles north of the HCGS site and reached an intensity of VII, causing damage at Wilmington, Newport, New Castle, and Oxford, Delaware. The maximum intensity experienced at the site area from this earthquake was probably no greater than VI.

The largest earthquake in the Coastal Plain of New Jersey occurred on June 1, 1927, with an epicentral intensity of MMI VII. This event, located in the vicinity of Asbury Park, was felt over an area of 3,000 square miles. Ground motion at the site from this event would have been, at most, only barely perceptible.

Several other reported shocks of MM Intensity V have occurred in the Coastal Plain of New Jersey in the vicinity of Salem County, some 17 miles south of the site. The most significant shock occurred on November 15, 1939, and was felt over a relatively wide area of some 6,000 square miles with almost equal intensity distribution in the epicentral area (Salem and Gloucester counties).

On August 23, 1938, four shocks of MM Intensities V, VI, V, and IV occurred in central and southern New Jersey. The largest shock was felt from northern New Jersey to Wilmington, Delaware.

On September 1, 1895, an event of Intensity VI near Philadelphia, about 47 miles from the site, was felt from Sandy Hook, New Jersey to Brooklyn, New York to Darby, Pennsylvania, and Wilmington, Delaware. Another shock of Intensity VI occurred on March 23, 1957, in the same general vicinity. These shocks were not reported felt in the vicinity of the HCGS site.

Reported earthquakes in the vicinity of the Ramapo Fault in northern New Jersey occurred in 1783, 1943, 1947, 1951, 1962, 1975, and 1976. The largest of these, the earthquake of 1783, was felt with a maximum MM Intensity VI and occurred at a distance of about 7.5 miles northwest of the Ramapo Fault. The shock of 1943, which was reported by one person from memory about 30 years after the event, was probably no greater than MM Intensity IV. The earthquakes which may be related to the Ramapo fault system are of small magnitude. The HCGS site is about 85 miles from the nearest approach to the Ramapo fault.

Two MM Intensity VII events occurred near New York City (1737 and 1884), approximately 125 miles northeast of the site. The 1884 shock affected an area extending from Portsmouth, New Hampshire, to Burlington, Vermont, southwest to Binghamton, New York, Williamsport, Pennsylvania, southeast to Baltimore, Maryland, and Atlantic City, New Jersey.

The closest major earthquake (MM Intensity VIII) to the site occurred in Giles County, Virginia on May 3, 1897, some 340 miles southwest of the site.

The largest event to have occurred on the East Coast was a MM Intensity X event in 1886 near Charleston, South Carolina, approximately 520 miles south of the site. The intensity felt at the HCGS site was probably no greater than MM Intensity IV.

2.5.2.2 Geologic and Tectonic Characteristics of Site and Region

2.5.2.2.1 Tectonic Provinces

The area within a 200 mi radius of the HCGS site includes parts of five tectonic provinces (Figure 2.5-22). The provinces are, from west to east: Fold and Thrust Belt, Blue Ridge-Highlands, Piedmont, Central New England, and Coastal Plain.

The tectonic province concept used to define these provinces is based on an evolutionary model of the Appalachian Orogen (Reference 2.5-61). This concept was used in this study to provide the province boundaries of significance to the HCGS site.

A detailed description of the regional geology, structural development of the Appalachian Orogen, and more recent Cenozoic faults is found in Section 2.5.1.1.

2.5.2.2.2 Tectonic Differentiation of the Appalachian Orogen

Considering the tectonic evolution of the Appalachian Orogen, as displayed on Figure 2.5-22, the Orogon can be subsided into two fundamental areas: the craton, or that part affected only by convergent diastrophism, and the mobile belt which are those parts of the crust affected by initial divergent, convergent, translational and final divergent diastrophisms. The mobile belt, as defined in this section, is situated east of the great anticlinoria cored by Grenvillian rocks and thus includes the Appalachian eugeosyncline and the quasi cratonic margins. The western edge of the mobile belt parallels and lies just to the west of the eastern edge of what was the North American continent during Cambro-Orodovician time, as defined by Rodgers (Reference 2.5-62 and Figure 2.5-8).

2.5.2.2.3 Tectonic Differentiation of the Craton

The cratonic portion of the Appalachian Highlands is underlain by 1000 million year old crystalline rocks which were deformed during the Grenvillian orogenic cycle. On the eastern edge of the craton, these rocks crop out at the surface as great anticlinoria. West of these elevated anticlinoria, lies an elongated, downwarped segment of the continental crust that forms the asymmetric Appalachian basin. The floor of this basin is formed of Grenvillian rocks that are greatly depressed in the east (up to 40,000 feet below sea level) and gradually rise toward the west. The basin is filled with largely unmetamorphosed sedimentary rocks (both clastic and carbonate), ranging in age from Early Cambrian to Carboniferous. These rocks form a sedimentary wedge, thickening to the southeast, that reflects the asymmetry of the basin floor.

Blue Ridge-Highlands Tectonic Province

The eastern portion of the craton, termed here the Blue Ridge-Highlands, constitutes a tectonic province and is characterized by Grenvillian rocks deformed during the Paleozoic convergence stage (dermal or thick skinned deformation).

Characteristically, the terrain is mountainous and exhibits exposure of some of the oldest rocks in the eastern United States (1,000-1,100 million years old). Earthquakes no greater than Intensity VI are characteristic of this tectonic regime, and none have been related to specific structures.

Fold and Thurst Belt Tectonic Province

The Fold and Thrust Belt tectonic province is characterized by tightly folded and thrust faulted Paleozoic sediments developed as flysch, or molasse. The northwestern boundary of this province generally marks a transition between gently folded rocks on the northwest (Stable Interior) and intensely folded and faulted rocks on the southeast, thus marking the western limit of Paleozoic thrusting (Reference 2.5-63).

The largest earthquake which has been recorded in the Fold and Thrust Belt tectonic province was the Giles County, Virginia, Intensity VIII shock of 1897, approximately 340 miles from the site. This earthquake has recently been identified as occurring within a seismogenic zone (Reference 2.5-64). Other earthquakes in this province are widely scattered, with none exceeding Intensity VI in the area of the site, and none correlatable to specific structures.

The recorded Intensity VII event at Wilkes-Barre, Pennsylvania, has been related to a mine collapse.

The tectonic differentiation of the cratonic portion of the Appalachian orogeny largely follows the tectonic subdivisions proposed by Rodgers, (Reference 2.5-36), with only a modification of the eastern boundary of the Blue Ridge-Highlands tectonic province.

This tectonic subdivision is also similar to that recognized by the regulatory agencies (Reference 2.5-65), with the Stable Interior tectonic province being an equivalent to Hadley and Devine's Appalachian Plateau, and the Blue Ridge-Highlands and the Fold and Thrust Belt tectonic provinces constituting Hadley and Devine' Blue Ridge and Valley and Ridge provinces, respectively.

2.5.2.2.4 Tectonic Differentiation of the Mobile Belt

The mobile portion of the northern Appalachian orogen within the region includes the eastern cratonic margin, which is partly underlain by continental crust of predominantly Grenvillian age, and party by thick, dense, presumably mafic crust (Reference 2.5-21).

The eastern cratonic margin is bounded on the western side by the Blue Ridge-Highlands tectonic province and on its eastern side by the easternmost extent of Grenvillian basement. This eastern boundary is interpreted principally from a line of gneiss domes of one billion year old continental crust including the Pine Mountain Belt, the Sauratown Mountains anticlinorium, the Baltimore Gneiss domes, and possibly the Chester dome of Vermont. This boundary corresponds to the eastern limit of the ancient continental margin of North America (References 2.5-21 and 2.5-22) (Section 2.5.1.1). It also coincides with several significant geological and geophysical changes (Reference 2.5-21). First, it parallels the main gravity high of the Appalachian (Reference 2.5-18). Second, it is marked by contrasting seismic refraction profiles that reflect deep crustal contrast. Finally, it is a zone of faulting, contrasting structural style, and contrasting metamorphic facies.

To the east of this boundary, the portion of the Appalachian Mobile Belt underlain by thick, dense, presumably mafic crust is a remnant of the final convergent stage of diastrophism of the proto Atlantic.

A detailed description of the structural evaluation of the Appalachian Mobile belt is found in Section 2.5.1.1.

The mobile belt can be subdivided into the Piedmont tectonic province (including Triassic-Jurassic Basins), the Central New England province, and finally the Coastal Plain province:

1. Piedmont Tectonic Province

   The Piedmont Tectonic Province is characterized by a eugeosynclinal assemblage over an older clastic assemblage, which is characterized in this region by a northeast-southwest trending belt of Precambrian to early Paleozoic schists, gneisses, slate, metaconglomerates, and some igneous intrusions. These rocks are interrelated in an complex manner by faulting and folding.

   Within the Piedmont Tectonic Province are a series of down faulted basins containing a miogeosynclinal assemblage overlapping an older clastic assemblage. Triassic Basins of the Newark Group are characteristic of this assemblage and are found all along the continental margin between offshore Maine and northern Florida. Triassic rocks have been encountered in borings all along the continental margin beneath large portions of the Coastal Plain province.

   These basins were formed during Triassic-Jurassic time as down faulted and folded elongate graben structures. Non-marine arkosic sediments and intercalated lava flows filled these basins as they were down faulted and filled. At the close of the period, the processes of erosion and basin development continued to modify the topography of the eastern section to form the base for deposition of Coastal Plain sediments.

The Triassic Basins intrusions of Juro-Triassic age are cut and displaced indicating a post-Juro-Triassic age for some of the faulting. Similar intrusions in the Piedmont are not disrupted or offset in this manner.

No earthquakes larger than Intensity VII have been recorded in this province, and none of the historical shocks can be satisfactorily related to specific structures. The Piedmont province is, in general, apparently the most seismically active portion of the area within 200 miles of the site. Concentrations of moderate events are apparent in the New York City area and in the Central Virginia seismic zone near Charlottesville, as described by Bollinger (Reference 2.5-66). Both of these zones are characterized by low to moderate seismic activity. Seismicity elsewhere in the province is relatively rare and apparently random.

2. Coastal Plain Tectonic Province

The Coastal Plain tectonic province is characterized by the development of a miogeosynclinal wedge during the advanced phases of the final crustal divergence. In the region south and east of the site, this province is characterized by a stratigraphic sequence of interbedded sands, gravels, clays, and silty sands of both marine and continental origin. These materials were deposited on the downwarped basement complex from Early Cretaceous to Quaternary time. The strata crop out at the Fall Zone and form a wedge shaped mass that thickens to the southeast. The average dip of these strata varies from 75 feet per mile within the Cretaceous sediments to approximately 10 feet per mile in the upper Tertiary formations.

The Salisbury Embayment is a structural low in the basement rocks between Newport News, Virginia and Atlantic City, New Jersey. The Embayment is marked by a deep accumulation of Mesozoic and Cenozoic sediments, which approach a thickness of 3,500 to 7,500 feet at the Maryland coastline. The feature is fairly prominent in the basement rocks but loses form in the younger sedimentary sequences suggesting that it is predominantly a pre-Tertiary feature. As described by Wentworth and Mergner-Keefer (Reference 2.5-7) part of the development of the Atlantic Continental Margin has been accompanied by the periodic reactivation of high angle faults and attendant release of stored strain. Figure 2.5-10 shows those structures within 200 miles of the HCGS site that have documented Cenozoic displacements. The significance of these structures to the HCGS site cannot be measured accurately in terms of the seismicity which the structures are capable of producing. The correlation of seismicity with geologic structure in the eastern U.S. is not high, especially in the Atlantic Coastal Margin. In the absence of recorded seismicity associated with these structures, stresses in the vicinity of these structures (orientation and magnitude), rock properties, failure criteria, etc. all that can be said relative to their significance to the HCGS site is that the structures reflect a structural style compatible with the development of the Atlantic Continental Margin since at least the Cretaceous period. While earthquakes are known to occur in the Atlantic Coastal Margin, the suggestion by Wentworth and Mergner-Keefer (Reference 2.5-7) that movements on these high angle faults have been occurring at decreasing rates since the Cretaceous implies, therefore, that the probability of a large ($>m_b = 7.0$) earthquake occurring on any one specific structure is very small.

The Coastal Plain underwent regional epeirogenic movements from Pliocene to Quaternary time that lifted a portion of the continental terrace above sea level (Reference 2.5-30).

Continuing work by the U.S. Geological Survey and private agencies (primarily in search of petroleum resources in the outer continental margin) have shown that the Coastal Plain is continuing to undergo tectonism. Wentworth and Mergner-Keefer (Reference 2.5-7) suggest that compression across the Atlantic continental margin has been driving northeast trending high angle reverse faults at decreasing rates since at least early Cretaceous. They further suggest that this driving force can be responsible for earthquakes, including the 1886 Charleston Intensity X event, occurring in the eastern United States. Hutchinson and Grow (Reference 2.5-47) recently described a fault, with possible Quaternary displacement, which occurs in upper Tertiary sediments some 20 miles east of Sandy Hook, New Jersey. A detailed discussion of Cenozoic tectonism in the continental margin can be found in Section 2.5.1.

The significant seismic activity in the Coastal Plain includes the 1886 Intensity X event at Charleston, South Carolina, and, for the sake of conservatism, the 1873 Wilmington, Delaware event of Intensity VII.

2.5.2.2.5 Alternative Tectonic Models

The tectonic model used to construct tectonic provinces for this study is based on an evolutionary model of the Appalachian Orogen and essentially parallels the early work of Rogers (Reference 2.5-62). Hadley and Devine (Reference 2.5-65) have arrived at essentially the same provinces, and their use in this study would provide the same seismic design basis.

The tectonic provinces used in the HCGS FSAR represent provinces based on an evolutionary model of the Appalachian Orogen. In this model, the 1982 Miramichi Earthquake sequence would have been correlated to the Central New England Tectonic Province (Reference 2.5-60, Figure 1). The Central New England Tectonic Province is characterized by thick, dense probably mafic crust overlain by eugeosynclinal sediments. These Paleozoic sediments are further characterized by intense deformation and large areas in which Acadian metamorphism overprints Taconic recrystallization. In contrast, the Inner Piedmont tectonic province represents a portion of the eastern Cratonic margin which is cored by Grenvillian basement rocks and is the eastern limit of the ancient continental margin. It is a zone of faulting, contrasting structural styles and metamorphic facies.

Additionally, the tectonic provinces proposed for the HCGS site are identical to those used in the Indian Point show cause proceeding, and were adjudged to be representative of tectonic provinces for the eastern United States according to Appendix A to 10CFR100 (ALAB 372, September 1977).

Consequently, the 1982 Miramichi Earthquake (m_b = 5.7) would be the largest event unassociated with geologic structure (notwithstanding the discussion above) and as such could be translocated to the closest approach of the Central New England Tectonic Province to the HCGS site, or approximately 140 miles (see Figure 2.5-22). The existing SSE design consideration, that of the equivalent of the 1871 Wilmington MMI = VII earthquake occurring at the site, would not be superceded. As discussed in Section 2.5.2.6, we have developed a site specific spectrum for soil sites for a range of earthquakes m_b=5.3 $\pm$0.5 recorded at distances less than 20 kilometers.

With respect to the HCGS site, it is considered that the tectonic model selected is conservative and allows appropriate selection of a seismic design basis for the facility.

2.5.2.3 Correlation of Earthquake Activity with Geologic Structure or Tectonic Provinces

Only a few of the historical earthquakes in the northeastern United States can be satisfactorily related to the specific structure at this time. Therefore, a consideration of the significant events which could influence the seismic design for the HCGS site will rely, for the most part, on an approach based on the tectonic settings discussed above. To augment the tectonic province approach, the concept of the seismic zones within the provinces, as discussed by Bollinger (Reference 2.5-66 and 2.5-64) and Hadley and Devine (Reference 2.5-65), will be addressed.

Those events that constitute the largest earthquakes of record in the Eastern United States, and that embrace all significant considerations for the safe shutdown earthquake for the site, are:

1. The large events (maximum historical Intensity IX) such as those which occurred in 1635, 1734, and 1870 in the lower St. Lawrence Valley and Ottawa-Bonnechere Graben area.

2. The large events such as those (maximum historical MM Intensity VIII in 1755) which occurred in the Cape Ann, Massachusetts, area.

3. The Intensity VII Attica shock (1929) in western New York State.

4. The Intensity X Charleston, South Carolina, earthquake (1886) in the Coastal Plain.

5. The intensity VII-VIII Giles Co., Virginia, earthquake of 1897.

6. The Intensity VII events such as those shocks which have been recorded at the local site area in and around New York City; Wilmington, Delaware; Asbury Park, New Jersey; and Lake George, New York.

Seismic zones within the provinces are:

1. St Lawrence Valley

   The St. Lawrence Valley and the Ottawa-Bonnechere Graben area are contained in the Ottawa Basin tectonic province (Reference 2.5-67). Earthquakes as large as Intensity IX are reported in this region. The structural interpretation shows that this area is the extension of a transverse trough and mobile zone into the stable interior (Reference 2.5-67).

   Because of the obvious historical confinement of seismic activity to this region marked by an interplate weakness, recurrence of such large shocks are expected to remain in that area, and thus are not translatable to the site. The closest approach of the Ottawa Basin Province to the site is 320 miles.

2. Boston-Cape Ann

   The large, (Maximum Intensity VIII), events in the Boston-Cape Ann area were historically associated with the Boston-Ottawa trend of earthquake activity (Reference 2.5-68), or the White Mountain-Monteregian Intrusives (Reference 2.5-69) which included the Ottawa-Bonnechere Graben area. However, a reevaluation (Reference 2.5-67) has resulted in the identification of tectonic regimes which separate the former "Boston-Ottawa trend," into specific tectonic provinces. On the basis of this, the Cape Ann Intensity VIII event, being the largest event to have occurred in the Avalon Platform province (Reference 2.5-67) would be restricted to a distance from the site of no less than 225 miles. Moreover, according to Ballard and Uchupi (Reference 2.5-23), it is possible that the significant Boston-Cape Ann seismic activity is associated with the faulted northwestern boundary of the Avalon Platform.

For these reasons it is not deemed necessary to translate this activity (Maximum Intensity VIII) out of the Avalon Platform.

3. Western New York

The shock of 1929 near Attica, New York, is anomalous with respect to the exceedingly sparse seismicity of this portion of the Stable Interior. It does mark, however, a noted concentration of earthquakes which are spatially related to the well recognized geologic structure of the immediate area, the Clarendon-Linden Fault. It is generally accepted that any recurrence of a similar event would be confined to the Attica area (Reference 2.5-67). Therefore, the postulation of a recurrence of this shock at the closest approach of the Stable Interior to the site (160 miles) is not warranted. A recurrence of the largest event at any location along the Clarendon-Linden Structure, the closest approach to the site being 300 miles, would result in only minimal ground motion at the site.

4. Charleston, South Carolina

The largest event to occur in the eastern United States is the event of approximately Intensity X, at Charleston, South Carolina, in 1886.

The concentration of seismic activity (over 400 events) in the immediate vicinity of Charleston is unique to the Atlantic Coastal Plain; moreover, such a confined density of epicenters is unmatched anywhere in the central and eastern United States, with the possible exception of the New Madrid, Missouri, region. On the strength of this areal distribution alone, it would be concluded that a specific tectonic anomaly is responsible for this localized activity.

In recent years, the U.S. Geological Survey and other investigators have been concentrating on understanding the geologic and tectonic framework of the Charleston, South Carolina, region. A detailed review of the literature on this subject reveals that, in spite of the tremendous amount of work accomplished in the past decade, the current range of views is very broad and at times conflicting (Reference 2.5-70).

There are three principal mechanisms which have been proposed to explain the occurrence of the 1886 Charleston earthquake:

a. Decollement reactivation

b. Stress amplification around the margins of mafic plutons

c. Reactivation of steep basement faults.

All of these proposed mechanisms have pro and con arguments, but the last proposed mechanism, reactivation of steep basement faults, would be considered the most likely, because of more convincing geologic and seismologic arguments, for instance, widespread late Cretaceous-Early Cenozoic reverse faults exist along the inner margin of the Atlantic Coastal Plain, showing consistent north-easterly orientations and dip slip displacements. In addition seismicity, both historical and instrumental, supports a reverse slip faulting mechanism (Figure 2.5-25) with northeasterly trending geologic structures.

At the current state of knowledge, it would appear that the present day reactions of the earth's crust (the dense historical seismic activity restricted to the Charleston-Summerville zone) may be the most diagnostic constraint in assessing the distribution of significant events in the immediate future. This review has revealed no significant evidence that indicates that an event of this intensity should be transferred to the HCGS site from Charleston. The spectrum of interpretations is so broad that for any given hypothesis, arguments can be made as to why such an event would occur again only within the Charleston area. These arguments, of course, would be entirely speculative because the data at hand are possibly misleading as to their true meaning regarding the origin or cause of the 1886 event. Consequently, there is no justification based upon the review presented herein, for overturning the commonly accepted conclusion of restricting the locality of a recurrent event of similar intensity as the 1886 Charleston earthquake to the same meisoseismal area.

5. Giles Count, Virginia (Southern Appalachian Seismic Zone)

The Giles County, Virginia, earthquake of 1897 is the largest shock to have occurred in the southern Appalachian region. It is listed (Reference 2.5-71) as Intensity VIII, and occurred in the Southern Appalachian Seismic Zone near its intersection with the Central Virgina Seismic Zone (Reference 2.5-66), more than 340 miles from the site. This intersection is marked by a definite break in the continuity of the activity of the northeast trending Southern Appalachian Seismic Zone and lies well to the south of an area of apparent differentiation of the system of tectonic stresses along the Appalachians called the Central Appalachian Salient in southern Pennsylvania.

Bollinger (Reference 2.5-64) recently reported on the results of a detailed investigation of the Giles County, Virginia, area. Utilizing Joint Hypocenter Determination techniques, and the results of microearthquake monitoring, a seismogenic zone some 25 miles, 3 to 15.5 miles in depth and 6.2 miles wide has been delineated near Pearisburg, Virginia. The trend of the zone is N36°E, or somewhat oblique to the structural fabric of the Appalachians. Bollinger also suggests that there were too few Intensity VIII locations to justify a separate Intensity VIII contour. Consequently, he recognizes the Giles County 1897 shock as MM Intensity VII-VIII.

Because the Giles County 1897 shock is associated with a seismogenic zone, it is not considered a random event in the fold and thrust belt tectonic province.

6. Local Site Area

Finally, consideration must be given to the likelihood of Intensity VII events which are known to occur within the Piedmont and Coastal Plain provinces. None of these shocks has been assigned to a specific structure within these provinces at this time and, therefore, these shocks should be considered as random events, capable of occurring adjacent to the site.

The region surrounding Wilmington, Delaware has been the locus of a moderate level of sporadic seismicity in recorded history (Sbar, et al, Reference 2.5-138). As described in Section 2.5.2.1, the largest events that have been recorded were the October 9, 1871 earthquake of Modified Mercalli Intensity (MMI) VII and the February 28, 1973, event of MMI VI. Additionally, during the time period from July 1971 to February 1972, an apparent intensification of minor (less than MM Intensity II) earthquake activity took place primarily in the area between Newark, Delaware and southwestern Wilmington. Subsequent paragraphs provided details regarding the seismicity surrounding the Wilmington, Delaware area:

a. Wilmington Earthquake of October 9, 1871: This shock is the largest earthquake in historic time, originating in or near the Piedmont, and because of its close proximity to the site area is considered to be the most significant to the HCGS seismic design analysis. Accurate location of its epicenter is difficult because of limited available information. Based on damage reports and intensities felt, the epicenter has been located near Wilmington, Delaware, whereas the shock was felt from near Chester, Pennsylvania on the north, to Middletown, Delaware on the south and from Salem, New Jersey on the east to Oxford, Pennsylvania on the west. The initial shock was followed by a much smaller shock just after midnight on October 10th. A contemporary newspaper account indicates that the initial shock was felt at Wilmington "with great distinctness." Buildings were shaken severely and a number of chimneys were damaged in the surrounding towns of Oxford, Pennsylvania, and New Castle and Newport, Delaware. An interesting aspect of this earthquake is the fact that it was accompanied by a very loud sound, as of an explosion. This loud noise, in face, led to the belief that the shock was caused by an explosion, probably at the powder mill of E.I. DuPont de Neumours Company, near Wilmington. This possibility was carefully investigated at the time and it was concluded that the shock was a legitimate earthquake.

b. Delaware River Earthquake of March 25, 1879, near Wilmington: This earthquake is also considered to be significant in that it occurred close to the epicenter of the 1871 shock. The shock was felt most strongly on the east side of the Delaware River, with felt reports indicating an area of some 600 square miles being affected between Chester, Pennsylvania and Salem, New Jersey. No damage was reported to have occurred from this event.

c. The Salem County, New Jersey Shock of November 14, 1930: This Intensity V event caused considerable alarm but little or no damage. The shock was felt in a rather widely distributed area of some 6,000 square miles from Trenton, New Jersey on the east to Baltimore on the west and Philadelphia on the north to Cape May on the south. The intensities at the hardest hit areas were almost, but not quite, damaging. At Deepwater, New Jersey, light objects were reported to have been turned over. In Philadelphia, some 20 miles to the northwest, dishes rattled shook and windows rattled pronouncedly. Two minor after shocks were recorded on seismographs at the Franklin Institute in Philadelphia at 10:32 and 11:30 p.m. the same night. Many persons believed the earthquake at first to have been an explosion at one of the many powder mills in the area.

d. The February 10, 1972 Earthquake near Newark and Wilmington, Delaware and the 1971-1972 Sequence of Microearthquakes: The incidents leading up to the recording of the February 10, 1972 earthquake apparently started on July 14, 1971 when residents of southwestern Wilmington reported a series of "booms". Many public officials, fearing sewer or natural gas leaks, became concerned. However, no trace of gas or damage was found. Again, hundreds of reports were received on December 29, 1971 and January 2, 6, 22, and 23, 1972, from the same area of southwestern Wilmington. The cause for these events was not determined. On January 27, 1972, instrumentation was installed by the Seismological Investigation Group of NOAA and Lamont-Doherty Geophysical Observatory to monitor microseismic tremors, believing these as possible explanations to the reports of the preceding months. On February 10, 1972, an earthquake of Magnitude (M_L) 2 was recorded. The epicenter could not be tied down exactly due to the "tightness" of the seismic network and the frequency of the seismic waves generated; however, NOAA concluded that the epicenter was probably located along the Fall Zone between Newark and Wilmington (Jordan, et al, Reference 2.5-137). NOAA also postulated a southwesterly migration of seismic activity from their monitoring program.

e. The February 28, 1973 Earthquake near Wilmington, Delaware: The event that occurred at 08:21 GMT on February 28, 1973, near Wilmington, Delaware was the largest event to have occurred in the region since the October 9, 1871 earthquake. The magnitude (M) was 3.8 at a depth of 14 kilometers and a maximum intensity of MMI = VI was felt near Clayton, Delaware. The total felt area for the shock was 15,000 km^2. The epicenter was located by NOAA (now U.S.G.S.) about 12 kilometers east of Wilmington in or near the Delaware River about 22 kilometers north of the site. Figure 2.5-24 shows the isoseismal map for this event.

In the several days following the main shock, a series of aftershocks, some of which were heard and felt, occurred some 8 kilometers northwest of the main shock epicenter (Woodruff, et al, Reference 2.5-141). Because of the distribution of portable monitoring stations, there was a bias of locational accuracy in a northeast-southwest direction. It was suspected (Sbar, et al, Reference 2.5-138) that the epicenter for the main shock was no more than a few kilometers from the aftershcok zone. Depths for the aftershocks ranged from 5 to 8.4 kilometers (Sbar, et al, Reference 2.5-138).

A composite focal mechanism solution for the main shock and five aftershocks indicated dip slip displacement on a nearly vertical (82°) plane striking N28°E, with the southeast side down. This nodal plane was chosen by Sbar, et al, Reference 2.5-138) because of agreement with local geologic conditions.

f. Possible caused for the Seismicity near Wilmington: Several hypotheses have been advanced by workers studying the possible relationship between recorded seismicity and geologic structure in the Newark-Wilmington, Delaware area. They include:

(1) Spoljaric, Reference 2.5-139, discusses the association of the 1971-1973 seismic activity and the October 9, 1871 MMI VII earthquake with a northeast trending strike slip fault between Newark and Wilmington, Delaware. Subsequent analysis failed to support this hypothesis, other than possible geomorphic evidence of stream offsets in Brandywine Creek.

(2) Sbar, et al, Reference 2.5-138, selected the nearly vertical nodal plane striking N28°E from the composite focal mechanism of the February 28, 1973 earthquake and subsequent aftershocks as being the causative fault plane for the earthquake in 1973. Their selection of this nodal plane was made on the basis of the

dip slip motion, southeast side down being consistent with uplift of the Piedmont and subsidence of the Coastal Plain. Sbar, et al, further suggest that motion on a vertical plane would not be expected in a compressional tectonic stress regime, but would be indicative of extensional or vertical tectonics conducive to graben formation. They note the similarity of the strike of the nodal plane to that of a small graben 30 kilometers to the southwest described by Spoljaric, Reference 2.5-140. They further note that the trend of the Delaware River between the graben and the epicenter of the February 28, 1973 event is nearly the same as that of the fault plane. Sbar, et al, conclude that their observations suggest that there may be a number of parallel faults in northern Delaware along which seismicity could occur and that more accurate locations of seismic events and extensive mapping of faults would be required to verify the hypothesis.

As shown on Figure 2.5-10, there are numerous indications of Cenozoic structures that occur near the fall zone in the eastern U.S. Some of these structures have seimsicity occurring in the general region around them while others do not. As discussed in Sections 2.5.2.3, 2.5.2.2.4, and 2.5.1.1.3.3 regarding the probability of renewed movement on high angle faults in the eastern U.S. and associated large earthquakes, faults in the Newark-Wilmington Delaware area would have no greater potential for being the locus of damaging seismic activity than others in the eastern seaboard.

Thus, it is considered that the seismic design basis for the Hope Creek site (a MMI VII earthquake, similar to the October 9, 1871 event at Wilmington, with an associated ground motion of 0.2g) is adequately conservative for the level of knowledge available regarding the occurrence and possible causal mechanism of earthquakes in the eastern U.S.

7. Regional focal mechanism solutions

As an indication of the earthquake generating stress field of the northeastern United States, focal mechanism for small, shallow earthquakes have been plotted on Figure 2.5-25.

Although the observations are still few in number, a fairly consistent picture of the earthquake generating stress field appears. Regularities observed are too remarkable to be entirely random and therefore are considered to be real.

High angle reverse or thrust faulting is the dominant mechanism shown by fault-plane solutions. Strike-slip and normal faulting are also involved but to a lesser degree.

The hypothesis correlating modern eastern seismicity with reverse movement on ancient northeast trending high angle faults has gained favor as structures consistent with this hypothesis and with the prevailing stress field have been discovered and documented. Nonetheless, well documented structures exhibiting reverse displacement are still very few. Consideration of steeply dipping faults of similar trend for which reverse displacement has been indirectly determined or merely inferred considerably increases the number of structures in this category, but the fact remains that known structures in the most seismically active areas (in numbers of events recorded) - southern New York-northern New Jersey, and South Carolina - have not been unequivocally described as reverse faults on the basis of geologic evidence. Conversely, known reverse faults in Maryland and Virginia (Stafford-Brandywine) and Georgia (Belair) have not been described as seismically active structures historically.

As to the 1886 Charleston earthquake and its correlation with inferred northeast trending structures it appears equally as likely that the event can be assigned to a structural component of the northwest trending South Carolina-Georgia Seismic Zone, and that, given the present stress field, displacement on that structure will be determined to have had a considerable transform (strike slip) component. Wentworth and Mergner-Keefer (Reference 2.5-7), however, contend that reverse faulting is amply demonstrated by earthquake focal mechanism solutions gathered from New England to the Charleston area, as also shown on Figure 2.5-25. Thus, given the number of high angle faults in the eastern U.S., compared to the number of possible high angle faults in the vicinity of the site, it is considered extremely improbable that seismic activity would occur on one of these faults (see also Section 2.5.1.1.3.3).

This pattern indicates that a regional and possibly continuous compressive stress field is prevalent in the upper levels of the lithosphere of northeastern North America. The most consistent observation regarding this stress field is that the largest and intermediate stress vectors are approximately horizontal.

The least principal stress tends to be vertical. The maximum compressive stress is generally oriented east-west approximately at right angles to the prominent structural grain of the Appalachian Orogen.

This stress field is locally perturbed in a manner suggestive of the rotation of the stress ellipsoid about the axis of the intermediate principal stress. Consequently, this secondary stress field is one in which the least principal stress tends to be near horizontal and the major principal stress near vertical. In some cases, the rotation does not occur about any of the principal stress axes, resulting in major principal stress vectors that strike north-south or northeast.

It is possible that these various perturbations are initiated at the transition zone between different stress fields, compressional and extensional, which might be the result of variable rates of vertical crustal movements.

2.5.2.4 Maximum Earthquake Potential

A review of the significant earthquakes in the eastern United States, based on the association with tectonic provinces, (or in two cases, Attica, New York, and Giles County, Virginia, discrete structure) discussed herein, results in following listing of possible candidates for the maximum design events. The site intensity listed is based on conservative estimates of attenuation (Reference 2.5-72) for central and eastern events, as presented on Figure 2.5-26.

Candidate Event	Epicentral Location	Maximum Epicentral Intensity	Proximity to Site (mi)	Site Intensity
Oct 20, 1870	St. Lawrence	IX	650	Less than IV
Nov 15, 1755	Cape Ann, Massachusetts	VIII	225	Less than V

Candidate Event	Epicentral Location	Maximum Epicentral Intensity	Proximity to Site (mi)	Site Intensity
Aug 12, 1929	Attica, New York	VII	460	III-IV
Aug 31, 1886	Charleston, South Carolina	X	500	IV-V (Reported)
May 31, 1897	Giles County, Virginia	VII-VIII	340	III
Oct 9, 1871	Wilmington, Delaware	VII	Adjacent to Site	VII

From inspection of the foregoing list, it can be concluded that:

1. The recurrence of major events (Intensity VIII or greater) can be confined to structures or tectonic provinces which occur no closer than 225 miles from the HCGS site.

2. Conservative, empirical, attenuation characteristics would confine the ground motion experienced at the site caused by these major events, to Intensity V or less.

Return periods for these larger events are unknown, but they are thought to be at least an order of magnitude greater than the design life of the HCGS facility.

In the interest of a conservative, deterministic appraisal of the design earthquake for the HCGS site, the safe shutdown earthquake is specified as an Intensity VII event occurring near the site. Such an event would supersede the site intensity generated by any considerations of greater events as given above, and is preferred because of the apparently random occurrence of Intensity VII events in, or near the boundary of, the Piedmont province.

2.5.2.4.1 The 1982 Miramichi Earthquake Sequence and its Impact on the Hope Creek Generating Station Site

Much attention has been directed to the extensive research underway regarding the Central New Brunswick earthquake sequence of 1982 and its tectonic significance - both in terms of intraplate tectonics and seismicity in general and its significance to licensing activities for nuclear power plants in the eastern United States. A discussion is provided below regarding the 1982 New Brunswick earthquake sequence, its occurrence in geologic terrain similar to the New England-Piedmont Tectonic Province, and its potential impact on HCGS.

During a recent field trip (September 26, 27, 1983) to examine rock outcrops in the epicentral region of the 1982 New Brunswick earthquake, much of the discussion held concerned the possible relationship of a N5°E fracture dipping 40°W (that exhibited 25 millimeters of reverse slip throw at the surface) with the Miramichi earthquake or one of its principal aftershocks. The fracture was discovered during geologic reconnaissance in the epicentral region after the main shock (January 9, 1982) occurred. As discussed in Wetmiller, et al, Reference 2.5-135, none of the geologic structure mapped in the region around the epicentral zone could be related spatially with the main shock or aftershock sequences or to the focal mechanism solutions constructed. The only geologic features with which there was any correlation to the trends indicated by the focal mechanisms are north-south trending joint sets, similar to the one mentioned previously along which 25 millimeters of reverse slip surface offset was documented.

Closer inspection of the N5°E, 40°W fracture, which transects about 8 to 10 meters of massive dioritic granite of the North Pole Pluton at the outcrop, indicated that while the edges of the fracture were sharp (suggesting post-glacial movement, in the absence of rounded edges) the displacement could have been related to passive response of the bedrock mass to the intense ground shaking during the earthquake (main shock or principal aftershocks). Evidence supporting this was gleaned from inspection of the large outcrop area which showed other blocks of diorite where till had been squeezed into both horizontal and vertical open joints in the bedrock mass, and the blocks had subsequently been rotated in place (along horizontal sheeting or exfoliation joints).

Evidence that supports the association of the 1982 Miramichi Earthquake with tectonic and in-situ stress conditions specific to the Central New Brunswick region, on the other hand, is also available. The most compelling evidence is represented on Figure 2.5-26a which shows the cumulative seismic activity in the epicentral region from January through April, 1982. An east-west section through the cluster of activity strongly support a set of north-south trending conjugate shears as being the locus of the main shock and aftershock activity of the 1982 Miramichi earthquake sequence. Wetmiller, et al, Reference 2.5-135, have suggested that the main shock occurred on the north trending westward dipping conjugate shear (eastern part of the seismically active zone on Figure 2.5-28A). Subsequent principal aftershocks were associated with this trend as well as the eastward dipping conjugate shear (January 11 m = 5.4 aftershock; western part of seismically active zone). Wetmiller, et al, Reference 2.5-135, have concluded that the "prime feature of the aftershock on January 11. This feature controlled the distribution and average mechanism of the smaller aftershocks."

All of the focal mechanism and composite focal mechanism solution for the 1982 Miramichi earthquake sequence are indicative of north trending reverse faulting on steeply dipping (to the east or west) fault planes. They are consistent with the distribution of seismic activity and Wetmiller, et al, Reference 2.5-135, interpretation that the earthquake sequence has occurred on north trending east and west dipping conjugate shears.

Additionally, in-situ stresses in bedrock in the epicentral region are apparently quite high, even close to or at the surface of the earth. This is seen from the distribution of many of the aftershocks which occurred in the upper 1 kilometer of the earth's crust. During the recent field trip to the epicentral region (specifically the outcrop or North Pole dioritic granite near Indian Lake where the north trending westerly dipping fracture with 25 millimeters of surface offset was observed) a pop-up occurred during the 24 hour period that personnel from the New Brunswick Division of Natural Resources, Earth Physics Branch (Ottawa) and visiting scientists from the United States examined the outcrop in detail. The pop-up occurred within a small mass of dioritic granite that had been separated partially from the surrounding bedrock mass along the till filled exfoliation (horizontal) joint, such that the small rock mass essentially "bridged" two larger parts of the rock mass at the outcrop. The pop-up created an arch like appearance with a fracture at its crown that had the same trend (N5°E) as the nearby fracture initially suspected to having surface displacement from the earthquake. The rock mass pulled away from the till, infilling the exfoliation joint, such that a two or three inch gap was visible. Additional pop-ups have also been documented on this outcrop since the end of September, 1983 (J. Wallach, Reference 2.5-136).

In summary, there is evidence, both pro and con regarding the association of the 1982 Miramichi Earthquake sequence with specific geologic structure, in the meaning of Appendix A to 10CFR100. This evidence is:

1. the north trending fracture observed in outcrop of North Pole dioritic granite with 25 millimeters of reverse slip displacement was more likely associated with passive response of the bedrock mass to strong shaking during the 1982 Miramichi Earthquake rather than occurring as a direct result of surface faulting,

2. detailed coverage of aftershock activity subsequent to the January 9, 1982 main shock has defined as very well constrained conjugate shear fracture system along which the main shock and principal aftershocks occurred. Focal

mechanisms constructed for the principal aftershocks and smaller aftershocks are consistent with the interpretation,

3. Bedrock stresses are at very high levels in the epicentral region, even close to the surface. This is supported by the number of very shallow (1 kilometer) aftershocks in the epicentral region and the occurrence of pop-ups on the outcrop surface. One pop-up has an axial fracture that parallels the trend of focal mechanism nodal planes,

4. The 1982 Miramichi earthquake sequence was not out of character with the region's previous earthquake history where four moderate earthquakes had occurred with magnitudes between 4 and 5.5 (Wetmiller, et al, Reference 2.5-135).

On balance, it would seem that the 1982 Miramichi earthquake sequence occurred in a region where high bedrock stresses coupled with favorably oriented joint or fracture systems in massive dioritic granite bedrock were and are presently operative. There are no known comparable situations in the HCGS area. Therefore, it is considered overly conservative to translocate the 1982 Miramichi earthquake to the HCGS site for purposes of evaluating the seismic design of the HCGS facility.

2.5.2.4.1.1 <u>Comparison of Seismic Activity Rates and Recurrence Models for HCGS and Miramichi, New Brunswick</u>

The seismic activity rates and recurrence models for several alternative sized regions surrounding the HCGS site and the epicentral region of the January 1982 Miramichi, New Brunswick earthquake have been compared. This comparison was done in the following manner.

1. Earthquakes listed in Table 2.5-1 for which no magnitudes were available (i.e., historical events prior to about 1950) were

converted from intensity to magnitude using Nuttli and Herrmann's (1978) relationship (Reference 2.5-152)

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$$m_b = 1.75 + 0.5\ I_o \qquad (1)$$

2. Other seismicity data (i.e., events listed in Earth Physics Branch Files or North Eastern U.S. Seismic Network Bulletins) where magnitude scales other than m_b were used to characterize seismic events, were considered to be numerically equivalent for purposes of this analysis.

3. A recurrence model was constructed for both the HCGS region and the area surrounding the New Brunswick magnitude 5.7 event of the form:

   $$\text{Log } N\ (\geq M) = a - bM \qquad (2)$$

   and normalized to $10^4 mi^2$ for direct comparison of earthquake density in a $5^o \times 5^o$ area centered about both the HCGS site and the New Brunswick event (see Figure 2.5-62 and Table 2.5-22).

4. A qualitative comparison of a $1^o \times 1^o$ area about both areas was also made to reflect the relative numbers of instrumentally recorded earthquakes of varying magnitudes over equivalent recording periods at both sites (see Tables 2.5-23 and 2.5-24).

A $5^o \times 5^o$ area surrounding the HCGS site and the Miramichi, New Brunswick magnitude 5.7 earthquake epicenter was selected as being broad enough to represent seismicity on a regional level. Events $\geq M_L = 4.0$ were compiled from Table 2.5-1 for HCGS and from data files of the Earth Physics Branch, Dominion Observatory, (see Table 2.5-21) for the Miramichi region (Reference 2.5-149), not including the January 9, 1982 Miramichi event or its aftershocks. Table 2.5-22 provides a summary of the recurrence parameters for both areas and the recurrence curves are shown on Figure 2.5-62.

The fact that the population density in central New Brunswick is low and that earthquakes have only been routinely recorded in that region for the last decade or so may reflect an even greater difference in seismicity between Miramichi and the HCGS site.

As a further comparison between the two areas, a 1° x 1° area, centered on the HCGS site and the Miramichi 1982 epicenter, was selected and earthquakes occurring within both the areas were determined. As earthquakes have been instrumentally recorded since about 1930 in eastern Canada, this date was used as the low cut off in both data sets available. Table 2.5-23 shows the comparison; for the HCGS area, there have been a total of 8 events since 1930 - 2 events between magnitudes 2.0-2.9; 5 events between magnitude 3.0-3.9; and one event between magnitude 4.0 and 4.4. The 1° x 1° region surrounding the 1982 Miramichi epicenter, on the other hand, shows a greater number of events (25) than HCGS, not including the 1982 Magnitude 5.7 event and aftershocks. There have been recorded (since 1930) 14 events of magnitude 2.0-2.9 (Reference 2.5-151); 4 between magnitudes 3.0-3.4; 5 events between 3.5-3.9; 1 earthquake between magnitude 4.0 and 4.4; and 1 event between magnitude 4.5 and 4.9. Table 2.5-24 lists those events used in this comparison for both areas.

It appears that the HCGS site is within an area of significantly lower seismicity than for the Miramichi, New Brunswick Region.

2.5.2.5 Seismic Wave Transmission Characteristics of the Site

The static and dynamic properties of the subsurface materials at the site are presented in Section 2.5.4. The analyses presented in this referenced section are based on characteristic ground motion and significant frequencies generated by the maximum potential earthquake described above.

2.5.2.6 Safe Shutdown Earthquake (SSE)

As a result of the derivations discussed previously, a SSE of Intensity VII is the maximum intensity considered, consistent with tectonic models presented for the site.

Correlations between MM Intensity and peak ground acceleration make use of currently available data. The largest portion of the data has been collected from seismic instrumentation in the western United States (References 2.5-73, 2.5-74, and 2.5-75). As more of these strong motion data have been obtained two effects have been observed. The first is the firmer definition of the mean value associating intensity with acceleration. The second is a normal attribute of almost all data sets. Namely the data set becomes larger the probability that a value larger or smaller than the bounds of the existing set will occur only decreases slightly. Of more importance has been the larger number of ground motions recorded at close epicentral distance where near field phenomena predominate. Care must be taken when extrapolating observations at one intensity level to possible observations at another level.

Observations indicated that the mean value of peak horizontal accelerations is approximately 13 percent of gravity for recording sites where Intensity VII damage was sustained. This mean value, or trend of the mean as used by the NRC, is derived from recordings taken on various types of foundation media, from alluvial valley fill to competent hardrock. Trifunac and Brady (Reference 2.5-73) also notes that there is no significant difference (considering the scatter) between the peaks of acceleration recorded on different geologic materials.

By virtue of the dependence of the correlations utilized above on peak ground motions, the relationships are considered conservative, because the level of sustained or effective acceleration significant to structural design will be of a somewhat lower level. However, on the basis of the relationships discussed above, it is recommended that the design acceleration for the HCGS site be considered as 20 percent of gravity at foundation level, resulting from the occurrence of the SSE adjacent to the site. This value is considered conservative, as it is equivalent to ground motion of the mean plus one standard deviation for recording sites where MMI VII damage was sustained (Reference 2.5-73).

There are no strong motion records of comparable earthquakes recorded in the eastern United States. Thus, the spectral characteristics of eastern events must be assumed on the basis of other information. Events in eastern North America may be interpreted as intraplate events, seismograms of which are impulsive and richer in high frequencies, as opposed to the events at plate margins (Reference 2.5-68). The greater high frequency content and large felt areas of eastern events may be explained by variations in attenuation (Reference 2.5-76), although the presence of in-situ compressive stresses suggests that source parameters may differ for eastern events; that is, stress drops may be higher (Reference - 2.5-68). However, the response spectra shown in Figures 2.5-27 and 2.5-28 should adequately envelope the effective accelerations generated by the SSE near the site, or the recurrence of larger, more distant events, such as the shocks at Charleston, South Carolina, and in the St. Lawrence River Valley. These spectra are developed in compliance with the guidelines set forth in Regulatory Guide 1.60, as revised. The duration of strong motion would not be expected to exceed 5 seconds (References 2.5-77 and 2.5-78) and, in all probability, would be less. However, in the seismic response analyses for the service water intake structure and appurtenances, a time history of 0.2g with 12 seconds strong ground motion and 24 seconds total duration was used (Reference 2.5-79). Duration of motion from a larger, more distant event, such as the Charleston, South Carolina, event of Intensity X, would be relatively longer than that from the design event but, the low accelerations attributed to long period motion from a distant, large event would be adequately enveloped by the response spectra.

A set of response spectra for soil sites where records were obtained was examined by Lawrence Livermore Laboratories during their studies for the Wolf Creek site. This set included 30 records obtained for events whose magnitudes ranged between 4.9 and 5.6 and whose distance from the source ranged between 6 and 22 kilometers. From the study of these 30 records they prepared 50th percentile and 84th percentile response spectra. These were compared with similar results obtained for a smaller set of spectra obtained on rock sites. This comparison is shown on Figure 2.5-28a. The 50th percentile values compare in a way that is expected from strong motion observations. The rock sites show a higher response at high frequencies or short periods and the soil sites show a higher response at lower frequencies. The greater scatter amongst the soil site spectra can be seen in the comparison of the 84th percentile curves where the soil site spectrum envelopes the rock site spectrum at all frequencies.

The position of the NRC staff has been to consider the 84th percentile spectrum. The 84th percentile is shown together with the standard RG spectrum anchored to 0.2g which was used as the HCGS SSE on Figure 2.5-28b. Also shown on Figure 2.5-28b is the 50th percentile spectrum for soil sites. It is readily apparent that the SSE spectrum envelopes the 84th percentile soil spectrum except at a period of 0.04 seconds (25 Hz). This exceedance is noted. The soil sites spectra set includes records from very short distances and it is believed that they contribute to the greater scatter between the soil spectra than the rock spectra. The 6 percent exceedance of the value at a period of 0.04 seconds is very small compared to the factor of 2.3 difference between the 50th and 84th percentile spectra at the same period. For this reason we believe that SSE spectrum anchored to 0.2g used for HCGS site remains appropriate.

2.5.2.7 Operating Basis Earthquake (OBE)

In accordance with Appendix A to 10CFR100, the OBE is herein defined as an earthquake which would produce a horizontal acceleration at the site of 0.10g (one-half the SSE). In order to assess the OBE risk over the life of the plant, an existing probabilistic analysis has been made. A standard computer program (Reference 2.5-80) was used to calculate the probability that various levels of acceleration will be exceeded annually at the HCGS site; specifically, that ground motion of 0.10g associated with the OBE. The program uses the theory of seismic risk analysis developed by Cornell (References 2.5-81 and 2.5-82), Merz and Cornell (Reference 2.5-83), and McGuire (Reference 2.5-80). The source areas used are defined on Figure 2.5-22. The historical earthquake data set includes all events of Intensity V or greater which have occurred within the tectonic sources listed on Table 2.5-1. An eastern attenuation function was used (Reference 2.5-84). McGuire's program yielded a 643 year return period for ground accelerations of 10 percent of gravity (an annual exceedance probability of 0.2 percent),

The OBE level of 0.10g is adequately conservative and is used in the response spectra shown on Figures 2.5-29 and 2.5-30.

2.5.2.8 SRP Rule Reviews

2.5.2.8.1 Acceptance Criterion 2.5.2.5

SRP Acceptance Criteria 2.5.2.5 requires that in meeting the requirements of Reference 3, this subsection is accepted when the seismic wave transmission characteristics (amplification or deamplification) of the materials overlying bedrock at the site are described as a function of the significant frequencies. The following material properties should be determined for each stratum under the site: seismic compressional and shear wave velocities, bulk densities, soil index properties and classification, shear modulus and damping variations with strain level, and water table elevation and its variation. In each case, methods used to determine the properties should be described or a cross-reference should be given indicating where in the SAR the description is provided. For each set of conditions describing the occurrence of the maximum potential earthquake, determined in Section 2.5.2.4, the type of seismic wave producing the maximum ground motion and the significant frequencies must be determined. For each set of conditions an analysis should be performed to determine the effects of transmission in the site material for the identified seismic wave types in the significant frequency bands.

Where horizontal shear waves produce the maximum ground motion, an analysis similar to that of Schnabel et al. (Reference 24 of SRP 2.5.2) is appropriate. Where compressional or surface waves produce the maximum ground motion, other methods of analysis (References 25, and 26 of SRP 2.5.2) may be more appropriate. However, since the techniques are still in the developmental state-of-the-art stage and no generally agreed on procedures can be promulgated at this time, the staff must use discretion in reviewing any method of analysis. The site amplification determined in this way should be compared with characteristics of site amplification in the epicentral area of the historical earthquake used as a basis for each maximum potential earthquake. If detailed soils investigations have been made in the epicentral area, the amplification analysis should be based on these. Because detailed geologic investigations are generally not available for the epicentral areas of historical earthquakes, several factors should be considered in assessing amplification effects there, including: regional geology and soil conditions, earthquake isoseismal maps, and descriptions of earthquake effects.

Material properties such as compressional and shear wave velocities bulk densities; soil index properties and classification; shear modulus and damping variations with strain level; water table elevation and its variation with time are defined and described in Section 2.5.4. Because most earthquakes in the eastern United States are historical in nature and not recorded on strong motion instruments, the required information is rarely available.

2.5.2.8.2 Acceptance Criterion 2.5.2.6

1. SRP Acceptance Criteria 2.5.2.6 requires that the amplitude and variation of acceleration at the ground surface, the effective frequency range, and the duration corresponding to each maximum potential earthquake must be identified. The acceleration is to be expressed as a fraction of the acceleration of gravity (g). Where the earthquake has been associated with a specific geologic structure, the acceleration should be determined using a relation between acceleration, magnitude or fault length and distance from the fault (cf. References 20 and 23 of SRP 2.5.2). Where the earthquake has been associated with a tectonic province, the acceleration should be determined

using appropriate relations between acceleration, intensity, epicentral intensity, and distance (e.g., References 27, 28, 29, 32, and 42 of SRP 2.5.2).

Since there are no discernible physical characteristics of the competent founding bedrock beneath the plant that would contribute to amplification or wave guide effects to seismic energy, the general attenuation and acceleration/Intensity relationships are considered to adequately reflect the wave transmission characteristics and resultant ground motion estimates at the site (Section 2.5.2.4). Earthquakes associated with geologic structure (Section 2.5.2.3.2) and earthquakes associated with Tectonic Provinces (Section 2.5.2.3.3) were evaluated using the mean of the relationship between Intensity and acceleration described by Trifunac and Brady (Reference 31 in NUREG-0800 Section 2.5.2) as to their ground motion effects at the site.

2. Available ground motion time histories for earthquakes of comparable values of magnitude, epicentral distance, acceleration level, and site conditions should be presented (Reference 40 of SRP 2.5.2). The spectral content for each potential maximum earthquake should be described; it should be based on consideration of the available ground motion time histories and regional characteristics of seismic wave transmission. The dominant frequency associated with the peak acceleration should be determined either from analysis of ground motion time histories or by inference from descriptions of earthquake phenomenology, damage reports, and regional characteristics of seismic wave transmission.

As described in response to Criteria for Subsection 2.5.2.5, the vast majority of eastern United States earthquakes are reported in Modified Mercalli Intensity units, rarely with accompanying strong motion values. Consequently, conversion to "equivalent" magnitude values and comparison to a western U.S. data set where epicentral distance, acceleration level and site conditions are available is considered unwarranted.

2.5.3 Surface Faulting

No occurrences of surface faulting are known to be located at or within 5 miles of the Hope Creek Generating Station site.

Subsurface exploration and detailed mapping of excavations were conducted at the site to detect any evidence or surface faulting. The results of these investigations presented in previously docketed reports (References 2.5-57 and 2.5-59) are summarized in Section 2.5.1.2. The conclusions reached as a result of these investigations provide that there is no evidence of surface faulting at the site.

2.5.3.1 Geologic Conditions at the Site

The relationship of the regional and site geologic conditions to the safety-related foundations for the HCGS are addressed in Section 2.5-57 and 2.5-59). A summary of the site specific lithologic, stratigraphic, and subsurface structural conditions is presented in Section 2.5.1.2.

2.5.3.2 Evidence of Fault Offset

There is no evidence of fault offset of the sedimentary strata comprising the Coastal Plain stratigraphic sequence at or near the site. Analysis of marker horizons using information from subsurface exploration (Section 2.5.1.2, Reference 2.5-57) indicates that there is no fault offset of the principal stratigraphic contacts within the Late Cretaceous and Early Tertiary strata, which underlie the site. Furthermore, no evidence of fault offset within the sediments was detected during detailed mapping of the foundation excavations (Section 2.5.1.2.3, Reference 2.5-59).

2.5.3.3 Earthquakes Associated with Capable Faults

The discussion of the historical seismicity of the region around the HCGS site is presented in Section 2.5.2.1. No earthquakes have been associated with faults within 5 miles of the site (Section 2.5.2.3).

2.5.3.4 Investigation of Capable Faults

There is no surface or near surface faulting within 5 miles of the site. Hence, the investigations outlined in Appendix A of 10CFR100 are not applicable to the HCGS site.

2.5.3.5 Correlation of Epicenters with Capable Faults

No earthquakes are associated with faults within 5 miles of the site (Section 2.5.3.3).

2.5.3.6 Description of Capable Faults

No known capable faults occur within 5 miles of the site (Section 2.5.3.4). Hence, this topic requires no discussion.

2.5.3.7 Zone Requiring Detailed Faulting Investigation

There are no surface or near surface faults known within 5 miles of the site (Section 2.5.3.4).

2.5.3.8 Results of Faulting Investigation

There are no surface or near surface faults known within 5 miles of the site (Section 2.5.3.4).

2.5.4 Stability of Subsurface Materials and Foundations

2.5.4.1 Geologic Features

The geologic features of the site are discussed in detail in Section 2.5.1.2, and summarized below as they relate to engineering properties. A comprehensive field investigation program, including borings, trenches, and geophysical surveys, was undertaken to determine the subsurface features at the site and their relevance to site stability. A detailed description of the field operations performed at the site is presented in Section 2.5.4.3.

The site lies within the Atlantic Coastal Plain physiographic province. The unconsolidated sediments of the Coastal Plain comprise a stratigraphic sequence of interbedded sands, silts, and clays, which generally dip toward the southeast. The surface of the crystalline bedrock is between 1500 and 2000 feet below the site.

The Coastal Plain sediments, which overlie the basement rocks, consist of a non-marine sequence of Lower Cretaceous strata and Upper Cretaceous-Tertiary marine deposits. Three formations of Tertiary age are present at the site. The lower Tertiary Hornerstown Formation lies unconformably on the Cretaceous age Navesink Formation and consists of olive green to green quartz and glauconite sand. This formation grades into the overlying Vincentown sands which consist of gray green fine shell and quartz sand. The western half of the site contains an elongated shallow trough in the surface of the Vincentown, which may have been scoured to its present depth by streams which drained into the retreating sea.

The basal sands and overlying clays of the Miocene Kirkwood Formation lie on a thick sequence of the calcareous Vincentown sands. The top of the Kirkwood clays is covered with Pleistocene sand and gravel which formerly comprised the bed of the Delaware River Estuary. Hydraulic fill was subsequently placed over these deposits and now forms the surface of the artificial island. The Vincentown and overlying strata, which are the formations most directly affecting site stability, are described in brief below and in detail in References 2.5-29 and 2.5-57. These near surface strata are represented in site subsurface sections, on Figure 2.5-32. Locations of the sections are shown on Figure 2.5-31.

The hydraulic fill extends to a depth of about 30 feet below the present ground surface. This fill deposit is of man-made origin, deposited on the site as a result of channel maintenance in nearby areas. It is similar in gradation and texture to soils of recent deposits underlying the fill. The fill is composed of irregular and sometimes discontinuous layers of micaceous silty clays to clayey silts to silty sands, with organics in some places. Between depths of 10 and 15 feet, there are sporadic 2 to 5 foot thick sand lenses throughout the site. These lenses are composed of fine to medium sand, usually with less than 15 percent silt. The hydraulic fill is generally loose or soft in consistency, and the fine grained portions are highly plastic. Blow counts are low, generally between 2 and 10 blows per foot, with a maximum of 15 blows per foot.

Below the hydraulic fill is a gray sandy and gravelly material, which formerly comprised the bed of the Delaware River. This layer varies in thickness from 2 to 8 feet and is composed of fine to coarse sand, little fine to coarse gravel, and a trace of silt. The fines content is less than 20 percent and D_{50} (grain size which is larger than 50 percent of particles in sample) varies from 0.2 to 0.75 millimeters. The blow counts from standard penetration tests generally vary from 20 to 85 blows per foot. Occasionally, there are isolated blow counts between 5 to 20 blows per foot. These occur in areas to be excavated and are therefore not of concern in foundation design.

Below the old river bottom sand, the soils of the Kirkwood Formation extend to depths varying from 50 to 75 feet below the ground surface. The Kirkwood Formation consists of the medium stiff to stiff clayey soils ranging from high plastic organic clays near the surface of this formation to silty clays and clayey silts. The layer averages about 20 feet thick, generally grading slightly thicker to the south and west. At the northern end of the site, the clays of the Kirkwood Formation grade to clayey fine sand.

The basal sands of the Kirkwood Formation are approximately 2 to 6 feet thick. They are reddish brown micaceous fine to medium sands with a silt content of about 30 percent in the upper portion of the layer, generally grading coarse with depth, and become gravelly at the bottom of the layer. Blow counts from standard penetration tests vary generally between 20 to 70 blows per foot. The D_{50} size varies from approximately 0.07 to 0.3 millimeters.

Below the reddish brown basal sands, the greenish gray silty sands of the Vincentown Formation were typically found at a depth of about 65 feet (Elevation +35 feet). However, the elevation of the top of the Vincentown Formation ranges between about Elevation +23 feet to a high of Elevation +50 feet. Beyond the main station area, to the northeast and east, the surface of the formation is relatively higher. These higher areas are usually characterized by the soils having a bright yellow brown color and about 30 percent fines with a trace of shells. This color is most likely a result of oxidation during exposure to air. Typically, the upper portion of this zone was found to be less dense than the lower zone. Deeper oxidized zones and shallow unweathered zones were noted, indicating that the process was not a function of depth. In general, the Vincentown sands range from silty sands in the upper zone to poorly graded sands in the middle to silty sands near the bottom of the formation. Standard penetration test blow counts vary over a wide range, due to the variable cementation, from isolated values as low as 16 blows per foot up to sampler refusal. Blow counts are generally above 30.

Varying degrees of calcite cementation occur throughout the Vincentown Formation. A petrographic study was performed to evaluate the nature, degree, and distribution of the cementation (Reference 2.5-58). Results of study indicate that the effects of cementation are not always readily apparent to the touch; some samples classified visually as uncemented are found to contain calcite cement when examined in thin sections. The cementation varies to a small scale (within a thin section or sample), but also on a larger scale over the entire stratigraphic interval of the Vincentown Formation at HCGS. The upper portion of the formation, extending from about Elevation +30 feet down to elevation zero (sometimes to -10 feet), has been cemented to a greater degree than that portion of the formation below elevation zero. Although completely cemented (rock) lenses are present throughout the formation beneath the site, the intervening less cemented lenses between Elevations +30 feet and zero generally contain significantly higher percentages of calcite cement than do many of those lenses below elevation zero.

The effects of four main diagenetic processes are apparent in the Vincentown soils. They consist of subaerial weathering, solution of calcium carbonate, precipation of calcite cement, and compaction of the Vincentown sediments. Solution and cementation are the most significant processes to have affected the Vincentown sediments since their deposition approximately 60 million years ago. Evidence points to the termination of significant alteration of the Vincentown sediments at some time during or slightly before Miocene time, about 25 million years ago. The rock lenses appear to represent the final stage of the cementation process, while the less cemented samples represent intermediate stages in incomplete cycles of cementation.

The conclusion of the petrographic study is that the Vincentown formation is a relatively strong and rigid structural unit whose behavior under static and dynamic loads cannot be evaluated by traditional methods of soil mechanics because of its cemented nature. This cementation is a major consideration in the analysis of stability, discussed in Sections 2.5.4.8 and 2.5.4.10.

The soils below the Vincentown formation extending to the depths investigated are generally silty sands with variable amounts of silt. These soils are very dense, and therefore are considered essentially incompressible under the small stresses to be induced by the proposed construction.

No areas of actual or potential subsidence, uplift, or collapse were observed at the site.

2.5.4.2 Properties of Subsurface Materials

This section presents the procedures and results of laboratory testing programs performed to assess the engineering properties of the subsurface materials. The tests were performed on representative soils samples recovered during the test boring programs, which are described in Section 2.5.4.3. The results are discussed in the following sections and summarized in tables and figures referenced therein.

2.5.4.2.1 Classification and Index Properties

2.5.4.2.1.1 Particle Size Analyses

Over three hundred and forty particle size analyses were carried out on representative samples recovered during the investigative work for the site. Many of the analyses were performed on the Vincentown sands, for which the results are summarized in Figure 2.5-36. The samples tested generally represented the uncemented or relatively poorly cemented portions of the formation. This was taken into account in making correlations of index properties to other properties of the soil. Additional analyses were carried out on the hydraulic fill, river bottom sands, and basal sands; the results are summarized in Figures 2.5-33, 34, and 35 respectively. The testing was performed in accordance with ASTM Test Designation D-422 (Reference 2.5-85). The results served to aid in classification and correlation of the soils and to identify the parameters D_{10}, D_{50}, and D_{60} for use in the stability analyses (Section 2.5.4.8).

2.5.4.2.1.2 Atterberg Limit Determinations

Atterberg limits tests were performed to evaluate the plasticity characteristics of the cohesive soils. The tests were done in accordance with ASTM Test Designations D-423 and D-424

(Reference 2.5-85). Representative results for samples of hydraulic fill, Kirwood clays, and silty Vincentown sands are presented in Tables 2.5-2, 4, and 6 respectively.

2.5.4.2.1.3 Moisture and Density Determinations

Moisture content determinations were made in accordance with ASTM Test Designation D-2216 (Reference 2.5-85), and densities were calculated according to Designation D-2937. Representative results for each soil startum are included in Tables 2.5-2 through 2.5-6. The results were used primarily to evaluate total and effective stresses in the formations for use in stability analysis.

2.5.4.2.1.4 Specific Gravity Tests

Specific gravity tests were performed on representative samples of site soils in accordance with the procedure of ASTM Test Designation D-854 (Reference 2.5-85). Results were applied in classification and correlation of soil types. Values obtained ranged from 2.50 to 2.73. Representative results are shown in Table 2.5-2 through 2.5-6.

2.5.4.2.2 Consolidation Characteristics

Fifty consolidation tests were performed on representative undisturbed samples of soil to evaluate their compressibility characteristics. The samples tested were confined laterally and incrementally subjected to increasing vertical loads and the resulting deformations measured. In some cases samples were unloaded incrementally and then reloaded to evaluate the recompression characteristics. Index property determinations were carried out in conjunction with each test. All testing was in accordance with ASTM Test Designation D-2435. Representative test results are presented in Table 2.5-7, indicating compression ratios ranging from 0.11 to 0.28 in the Kirkwood clays and 0.17 and 0.24 in the fill soils. Time readings were also taken in conjunction with each test in order to evaluate coefficient of consolidation c. This parameter is used to estimate the time required for consolidation of a soil stratum. Typical values of c are presented in Table 2.5-8.

Reference 2.5-59 shows the percent of Vincentown sands, finer than the #200 sieve, versus depth for a majority of the samples obtained during various investigations. In general, the spread of the data indicates that typically the Vincentown sands contain 10 to 30 percent fines. The referenced figure also indicates that the variability is greater in the vertical direction than in the horizontal direction. Therefore, the effect of the variability of fines in the horizontal direction of differential settlement will be much less than that indicated by the overall variability.

It is important to note that, in general, greater percentage of fines in the Vincentown sands relates to a higher cementation and stiffer material. This can be seen in the referenced figure which shows that almost all the samples from the post-excavation studies have much less fines than the overall average of fines in the Vincentown sands. The block samples in the post-excavation studies were obtained only in the less-cemented Vincentown sands.

The actual differential settlement of a structure depends on the rigidity of the structural mats as well as the variability of the foundation materials. The thick concrete mats of the Category I structures will reduce the differential settlements indicated by the variability of the foundation materials alone.

Data from pre and post-construction settlement monitoring of Category I structures is provided in Section 2.5.4.60. These data indicate that the total settlements of all the structures under significant loads, and the variability of the measured settlements within a structure (and from one structure to another) are both small. Since the magnitude of the total settlements is small, the differential settlements are expected to be smaller.

2.5.4.2.3 Static Strength

2.5.4.2.3.1 Unconfined Compression Tests

Unconfined compression tests were carried out on representative samples of cohesive soil to evaluate undrained shear strength. A load deflection curve was plotted for each test, and the strength of the soil was defined as the peak shear strength or the shear strength at 15 percent strain, whichever occurred first. Determinations of natural moisture content and dry density were made in conjunction with the tests. The testing procedure was in conformance with ASTM Test Designation D-2166 (Reference 2.5-85). The results are summarized in Table 2.5-9. Results indicate that the Kirkwood clay is medium stiff to stiff and the cohesive hydraulic fill is soft to medium stiff.

2.5.4.2.3.2 Triaxial Compression Tests

Unconsolidated undrained and consolidated undrained triaxial compression tests were performed on selected undisturbed soil samples recovered from several soil strata at the site. These tests provided information on the stress strain behavior as well as the undrained shear strength of the various layers.

Twenty eight unconsolidated undrained tests were performed on samples recovered from the hydraulic fill and Kirkwood clay. The results of the tests are summarized in Table 2.5-9. These results and those from the unconfined compression tests indicate undrained shear strengths in the range of 1100 and 2200 pounds per square foot and 150 to 1100 for the Kirkwood clay and cohesive hydraulic fill, respectively.

One hundred twenty one consolidated undrained tests were carried out on samples recovered from the hydraulic fill, river bottom gravels, Kirkwood clays and the Vincentown, Hornerstown, and Navesink Formations. One hundred fifteen samples were consolidated isotropically and the remainder ansiotropically under K_0 conditions.

Subsequent to consolidation 113 samples were tested in compression and five samples were tested in extension. For each test axial load, deflection, and pore pressure were recorded and a load deflection curve produced. All tests were run in accordance with ASTM Test Designation D-2850, and References 2.5-86 and 2.5-87. Representative results of the consolidated undrained tests are presented in Table 2.5-10 and Figures 2.5-37 through 2.5-40.

2.5.4.2.4 Dynamic Properties

2.5.4.2.4.1 Resonant Column Tests

Resonant column (dynamic torsional shear) tests were performed on selected undisturbed samples to evaluate modulus of rigidity and damping. Ten samples of Kirkwood clay and hydraulic fill and 22 samples of Vincentown, Hornerstown, and river bottom sands were tested. References 2.5-88 through 2.5-92 represent the published literature on resonant column testing procedures. All tests were performed according with the methods and procedures described in Reference 2.5-133. The referenced manual has been revised subsequent to the publication date without any significant changes in procedures in this chapter. Shear moduli and damping ratios obtained from the tests are summarized in Tables 2.5-11 and 2.5-12.

2.5.4.2.4.2 Dynamic Triaxial Tests

The dynamic behavior of the various soil strata encountered at the site was evaluated by dynamic triaxial testing of representative undisturbed soil samples.

2.5.4.2.4.2.1 Dynamic Strain Controlled Cyclic Triaxial Tests

Dynamic strain controlled tests were performed on undisturbed samples of Kirkwood clay, Vincentown sands, Hornerstown, and basal sands to further evaluate shear modulus and damping. Samples were tested by applying a sinusoidally varying stress. The amplitude of the resulting deformation was controlled to correspond to predetermined levels of axial strain. Each specimen was tested for shear strains varying from about 0.05 percent to approximately 1 percent. Results of the tests are presented in Tables 2.5-13 and 2.5-14. A plot of the normalized modulus AK = $G/(\overline{\sigma c})$ versus the shear strain for sands and a plot of modulus G versus the shear strain for clay are presented in Figures 2.5-41 and 43, respectively. Variation of damping with shear strain is graphically presented in Figures 2.5-42 and 2.5-44. These plots include representative data from resonant column tests and dynamic strain controlled cyclic triaxial tests.

All dynamic strain controlled cyclic triaxial tests were performed according to the methods and procedures described in Reference 2.5-133. The referenced manual has been revised subsequent to the publication date without any significant changes in procedures in this chapter.

A frequency of 1 hertz was used for all the tests. Consolidation was allowed in all the tests.

The design curves for the clay from fill and the Kirkwood clay were derived based on geophysical survey data at low shear strain levels (Table 2.5-15), and resonant column and strain controlled cyclic triaxial test data at relatively higher shear strain levels (Tables 2.5-12 and 2.5-14).

Laboratory tests such as the resonant column and the cyclic triaxial tests, were used to evaluate the shear modulus of the Kirkwood clay at the higher strain levels. Sample disturbance due to the sampling and testing process tends to underestimate the value of shear modulus. A greater reliance was placed on the values of shear modulus that were determined by geophysical surveys which were very low strain levels.

The data presented on Figure 2.5-43 were utilized in the site response analyses (for liquefaction assessment) and in the soil structure interaction analyses. These analyses were performed with parametric studies to incorporate the range of soil modulus properties measured or estimated. The dynamic shear modulus of the Kirkwood clay at low shear strain levels was varied between 1,500 kips per square foot (ksf) and 8,000 ksf.

Envelope values of the response were than considered for further safety evaluation.

The design curve for the Kirkwood clay shown in Figure 2.5-43 provides a conservative liquefaction assessment. It should also be noted that the difference in the design values and the laboratory values of modulus is much less at seismic induced shear strain levels (greater than 0.1 percent) than the difference at low shear strain levels (10^{-4} percent).

2.5.4.2.4.2.2 Cyclic Static Triaxial Tests

Three cyclic static compression tests were carried out at various confining pressures on undisturbed samples. Test data for a sample of the Vincentown formation is presented in Figure 2.5-45, which shows a typical stress strain curve for three cycles of loading and unloading. The variation of Poisson's ratio as a function of axial strain for the first three cycles of loading is also presented.

2.5.4.2.4.2.3 Dynamic Stress-Controlled Cyclic Triaxial Tests

Eighty four stress controlled cyclic triaxial tests were carried out on undisturbed and reconstituted samples. The samples were first consolidated and then a sinusoidally varying deviator stress was applied. Representative results for Vincentown, river bottom and basal sands are summarized graphically and tabularly and presented in Figures 2.5-46, 2.5-47, and 2.5-48. These show the plot of stress ratio versus number of cycles to cause ~2.5 percent axial strain for undisturbed and reconstituted samples of the Vincentown and undisturbed samples of river bottom and basal sands. These results indicate in the relatively weakly cemented undisturbed Vincentown samples obtainable for testing, that cementation significantly influences the strength of the soil. From these tests the lower bound strength was evaluated for use in assessing liquefaction potential.

All dynamic stress controlled cyclic triaxial tests were performed according to the methods and procedures described in Reference 2.5-134. The referenced manual has been revised subsequent to the publication date without any significant changes in procedures in this chapter.

2.5.4.3 Exploration

Prior to commencement of field investigations, an extensive regional geological survey was carried out at the site incorporating a literature survey and a review of data from the adjacent SNGS site (Reference 2.5-29). Field explorations were undertaken to evaluate the site specific conditions and to obtain samples of the subsurface materials for laboratory testing of engineering properties.

Site drilling began in December 1973 and continued through May 1974, for a total of 76 boreholes (Reference 2.5-57). Geophysical surveys (described in Section 2.5.4.4) were performed at that time. In October 1974 an additional 25 boreholes were drilled in the reactor building area in order to further study the Vincentown Formation (Reference 2.5-58). The objective of these borings was to assure that representative samples were obtained from all areas of the Vincentown; this was accomplished by continuous sampling with a 3-inch Denison sampler or Christensen D-3 split tube core barrel and by sampling from a secondary boring 5 feet away when 100 percent recovery was not achieved in the primary boring.

In January 1977, geologic mapping of the exposed faces of the main excavation was initiated and coordinated with the continuing excavation activities. Three trenches and seven test pits were excavated below the surface of the Vincentown Formation level and mapped for significant geologic features. In addition, numerous construction related excavations, such as those for crane foundations and power block sump pits, were mapped as they were exposed. Twenty three large diameter (~30 inches) borings were also performed to explore a relatively uncemented area in the northwestern quadrant of the excavation (Reference 2.5-59).

In August 1978, 21 additional borings were drilled to obtain supplemental field and laboratory data for the soils in areas to the north and west of the main excavation (Reference 2.5-97). In addition to these exploration programs, several smaller scale supplementary investigations were performed:

1. November 1974, four borings to evaluate the strength of the hydraulic fill for the purpose of slope stability evaluation (Reference 2.5-94).

2. December 1975, 10 borings to evaluate the soils along the service water supply lines (Reference 2.5-96).

3. April 1979, two borings to evaluate soil properties for pile design (Reference 2.5-98).

4. November 1981, 15 borings to define the elevation of the unweathered Vincentown sand in the vicinity of the service water intake structure (Reference 2.5-99).

The locations of borings drilled for the different phases of field explorations are shown on Figure 2.5-31. Summary logs of the borings are presented on Figure 2.5-50. Soils are classified in accordance with Unified Soil Classification System, presented on Figure 2.5-49.

The ground water monitoring and excavation dewatering programs are described in detail in Section 2.4.13 and referenced in Section 2.5.4.6.

2.5.4.4 Geophysical Surveys

The following geophysical surveys were performed during 1967 and 1974 at the Hope Creek site:

1. A seismic refraction survey to evaluate the compressional wave velocities of materials at the site and to differential the velocity layering of those materials

2. An uphole compressional and shear wave survey to confirm the compressional wave velocities determined from the seismic refraction survey and to investigate shear wave velocities of subsurface materials in the vicinity of the main power block area

3. A seismic wave survey consisting of a combined surface wave study and long distance in line downhole shear wave study to determine the types and characteristics of the various wave trains generated by small explosive sources at the site and a supplementary short near surface shear wave and surface wave study performed in conjunction with the seismic wave survey

4. Micromotion measurements of ambient motions at the site to determine the characteristics of ground motion initiated by background noise.

The locations at which the geophysical surveys were performed are shown Figure 2.5-31.

2.5.4.4.1 Seismic Refraction Survey

A seismic refraction survey was conducted at the site along two perpendicular lines at the locations shown on Figure 2.5-31. The survey was performed using an SIE RS-44 24-trace refraction recording system with an SIE R-6 recording oscillograph. Explosive charges of 3-1/2 to 18 pounds were detonated in drilled shot holes at depths of 8 to 35 feet to provide the energy source for the survey. The seismic energy produced by the explosive charges was detected using 14 hertz vertical geophones spaced at 50 foot intervals along the survey lines.

Apparent compressional wave velocities were evaluated from the seismic refraction data by plotting the first arrival times of the seismic energy at each geophone location and calculating the inverse slope of best fit line segments drawn through the time distance data. The time distance plots and interpreted subsurface sections are shown on Figures 2.5-51 and 52.

The refraction data indicated that the site has three velocity layers with nearly horizontal interfaces; the three distinct layers correspond well to the hydraulic fill, kirkwood clay, and the Vincentown formation. The uppermost layer extends to approximately 30 feet below the ground surface and has a compressional velocity in the range between 1,750 and 2,900 feet per second. The second layer extends from about 30 feet to about 60 feet below the ground surface and has a compressional velocity of about 3,900 to 4,600 feet per second. The third layer extends from a depth of 60 feet to the total effective depth of investigation achieved in the survey. The term "effective depth of investigation" is discussed in Section 2.5.4.4.6. The compressional velocity of this third layer is approximately 6,800 feet per second. There is no appreciable additional increase of velocity with depth within the effective depth of investigation.

2.5.4.4.2 Uphole Compressional and Shear Wave Survey

An uphole compressional and shear wave survey was performed using Boring 201 in the main power block area. The survey was performed using an SIE RS-44 refraction recording system with an SIE R-6 recording oscillograph. The source of seismic energy for the survey was impacts made by a sledgehammer at the ground surface adjacent to the boring. Impacts were made in both vertical and horizontal directions against a wooden block placed in a shallow trench.

Seismic waves were detected using both an uphole cable with 12 piezoelectric transducers spaced at 25 foot intervals along the cable and a 3 component, 7.5 hertz borehole geophone.

Recordings were obtained to a depth of 425 feet using the 3 component geophone. Vertical and horizontal hammer blows were recorded at 25 foot depth intervals. Recordings were obtained to a depth of 276 feet using the 12 transducer cable. Vertical and horizontal hammer blows were recorded at 5 foot depth intervals.

Figure 2.5-53 presents the time depth data obtained from the uphole compressional wave survey. The data presented are those obtained using the 3-component geophone as that data was of generally better record quality than the data obtained using the 12 transducer cable. The best fit line segments drawn through these data and the resultant compressional wave velocities and layer thicknesses are essentially the same as those determined by the refraction survey.

A series of high amplitude secondary arrivals were noted on the recordings from this survey at and below a depth of 125 feet. These were interpreted as tube wave arrivals. No shear wave arrivals were positively identified on the recordings.

2.5.4.4.3 Seismic Wave Survey

A combined surface wave survey and long distance in line shear wave survey was performed along Seismic Refraction Line 1 in the vicinity of Boring 201. Surface wave recordings were obtained using a Sprengnether VS-1200-4 engineering seismograph with four 3 component, 2 hertz seismometers, and an Electro-Tech Labs DSW-100 recording oscillograph. Downhole shear wave recordings were obtained using an SIE RS-44 refraction recording system with a 7.5 hertz, 3 component borehole geophone, or a 12 piezoelectric transducer uphole geophone cable and an SIE R-6 recording oscillograph.

The energy source for the seismic wave survey consisted of explosive charges detonated at shot point locations along Seismic Refraction Line 1 at distances of 955, and 1,580 feet from Boring 201.

Surface wave recordings were obtained for particle displacement, velocity, and acceleration at various gain levels. Downhole shear wave recordings were made at both low- and high-gain levels.

A supplementary near surface shear and surface wave survey was performed along a short segment of Seismic Refraction Line 2. Recording of vertical, transverse and radial sledgehammer impacts against a wooden block were obtained using 12 vertical element geophones input to the SIE RS-44 system and four 3 component seismometers with the Sprengnether VS-1200-4 system.

The seismic wave survey provided the most definitive information obtained during the site geophysical surveys on shear wave velocities as well as on surface wave types and velocities.

The surface waves observed at the site show low velocities, ranging from approximately 500 to 200 feet per second. The frequency range of these waves is from about 5 hertz to less than 2 hertz. The corresponding wave lengths range from about 250 feet to about 40 feet. The type of motion is generally of the M type Rayleigh waves with some significant transverse motions occurring with the shorter wave length portion of the very long complicated wave train system. Normal dispersion is indicated from the wavelength and approximate phase velocity relationship. A dispersion curve was not constructed for these surface waves due to the complex nature of the wave train and the quantity of records required to do so.

Records of surface waves were obtained in surveys performed in 1967 and 1974. In the 1974 records, the relative amplitudes between the surface wave and the preceding direct and multiply refracted wave trains are much lower than on the 1967 records, an observation attributed to a change in ground surface material. During the construction of SNGS, 3 to 5 feet of gravel and compacted sandy clay fill was placed over the ground surface in the area where the surface wave survey was run. This uppermost relatively stiff layer, above the water table, is considered the reason for the differences in relative amplitudes of the surface waves between the two surveys.

The shortest wave length velocity for the surface waves of about 200 to 280 feet per second (measured in the 1967 survey) and the addition of significant transverse motion to the surface wave train in this interval, indicates that the shear wave velocity of the materials above 30 feet is approximated by this phase velocity of about 250 feet per second, as was stated in the PSAR for the SNGS site.

The wave trains arriving ahead of the surface waves on the seismic wave records, both on the VA-1200-4 system and the downhole geophones, consist of direct and refracted body waves of several types including apparent compressional (P) to shear (S) conversions. The analysis of these wave train systems consisted of constructing an interpretative model of the site which incorporated all known velocities (P to S), all known layering, and the locations of all of the surface wave, downhole geophones, and shot points. Appropriate ray path analysis was then based on this model and used to compute the arrival times at the various geophones, taking into account the apparent surface velocities and the type of motions recorded at the geophones (i.e., relative angles of incidence of the waves on the geophones as determined from the 3 components). The computed times from the model were then compared to the recorded times and adjustments were made, if necessary, to the respective compressional or shear velocities of each layer to obtain a positive time match between the computed arrival time and the recorded arrival time.

The major arrival paths for the various wave trains appear to be the PPP, the PPS and the SSS refraction trains to the surface geophones and the PP and SS refractions to the geophones in Boring 201, below the top of the third layer. The resultant computed velocities are given in Table 2.5-15. This table presents a comparison of the velocities obtained from various survey methods, and representative values of Poisson's ratios.

2.5.4.4.4 Micromotion Measurements

Measurements of the ambient ground motions at the site were obtained using a Sprengnether VS-1200 Engineering Seismograph with a 3 component (vertical, horizontal, and radial) seismometer. The ambient background motion at the site gave a baseline reference from which to interpret the geophysical data obtained at the site. The high gain ambient vibration records show two predominant frequencies, at 60 hertz and at between 10 and 11 hertz. The former is spurious and is primarily carried by cross feeding from underground electrical utilities into the recording system, and the latter is from vibrations carried by heavy equipment operating in the construction area of the adjacent SNGS plant.

2.5.4.4.5 Summary of Results

A summary of the compressional and shear wave velocities determined by the geophysical studies at the HCGS site is presented in Table 2.5-15.

The shear wave velocity of 1850 FPS for Kirkwood clay was computed from measured values of compressional wave velocities in the Kirkwood clay in the field. The relationship between compressional and shear wave velocities is as indicated below:

$$\frac{VP^2}{VS} = \frac{2(1-\mu)}{1-2\mu}$$

Compressional wave velocities are more readily and reliably obtained in the field from geophysical surveys. Compressional wave velocities were measured at SNGS during 1967 and at the HCGS site in 1974 using data from a seismic refraction survey, an uphole compressional wave survey, and a seismic wave survey.

2.5.4.4.6 Depth of Investigation

The maximum effective depth of investigation achieved during the surveys was at a depth of 418 feet in Boring 201, from a shot at 30 feet depth at a distance of 1,580 feet from the boring location. The computed arrival time of the compressional wave at the boring corresponded to a compressional velocity of 6,800 fps in layer three.

Therefore, no refractor with a significantly higher velocity lies within the effective depth of investigation at this site.

In this case the effective depth of investigation is not a single value, but a continuous function of the contrast between the layer three velocity and all higher velocities which could form a refractory horizon at some depth below the top of layer three. Figure 2.5-54 is an approximation of the effective depth of investigation for the geometric configuration of the geophone in Boring 201 and the shot point. This approximation assumes that the refractor is flat; and reasonable assumption because the known geology indicates a small dip from the boring toward the shotpoint. If the potential refractor is indeed flat, or nearly so, then the graph in Figure 2.5-54 is a good indicator of the depths above which the refractors would actually be seen, if they existed at the site with the corresponding velocity. For example, if a flat refractor with a velocity of 9000 feet per second does exist at the site, then it must be deeper than about 600 feet, because if it were shallower than this it would have been seen by an early arrival time on the geophone at 418 feet in Boring 201. The depth of investigation for very small velocity contrasts (i.e., a flat refractor of 7200 feet per second) is 500 feet, while for a velocity contrast indicative of basement the depth of investigation is about 800 feet.

2.5.4.5 <u>Excavations and Backfill</u>

2.5.4.5.1 Main Power Block Excavation

The location of the main excavation for the power block is shown on Figure 2.5-16. The excavation was extended to a depth of 70 feet (elevation +30 feet) and had a plan area of about 500 by 650 feet. Temporary side slopes were constructed with gradients of 2 horizontal to 1 vertical to a depth of 37 feet (Elevation +63 feet) and 1 horizontal to 1 vertical to a depth of 70 feet (Elevation +30 feet). There is a 25-foot berm at 37 feet (Elevation +63 feet). The stability of these slopes is discussed in Section 2.5.5.

During construction, the groundwater level was maintained about 3 feet below the excavated surface. It is very probable that the upper 3 feet of the Vincentown sands did not dry out because:

1. The upper 3 feet are expected to be within the zone of capillary action.

2. The excavated surface was exposed mostly during the wet/cold season.

3. Due to the expedient construction schedule, most surfaces were covered by fill/concrete soon after the excavation was completed.

The construction sequence for the excavation to the Vincentown formation and the protection of the Vincentown are as follows:

1. Hydraulic dredging March 23, 1976 to October 29, 1976.

2. Dewatering of excavation completed January 11, 1977. (Refer to Section 2.4.13)

3. Dental work to remove dredge spoils completed April 7, 1977.

4. Immediately following the dental work, the competent Vincentown Formation was inspected and accepted by soil engineer. The surface is surveyed and covered with a minimum of two feet of backfill material to protect the Vincentown surface from vehicular traffic and from winter freezing conditions. Thickness of backfill was monitored for frost penetration during winter.

5. Mud mat placed between April 1977 and August 1977.

Excavation in the Vincentown formation extends to about Elevation 30 feet; at this depth highly cemented sands were encountered. These sands did not exhibit any loss of cementation upon partial drying during sampling in the field. Details of the cementation characteristics of the Vincentown sands are provided in Appendix D of Reference 2.5-57.

It should be noted that the Vincentown sand at HCGS, did not experience repetitive "drying and wetting". The groundwater level at the main power block area was lowered by pumping during construction and are expected to be raised to its natural level. The groundwater level is expected to remain at its natural equilibrium level throughout the plant life.

Settlement monitoring data for the power block structures are provided in FSAR Section 2.5.4.10. The data represents measurements before and after the groundwater level was raised slowly to its current level at Elevation 65 feet. The data shows that no significant changes have occurred in the settlement rates indicating no change in the compressibility characteristics of the supporting Vincentown sands.

2.5.4.5.2 Station Service Water Intake Structure Excavation

The location of the station service water intake structure is shown on, Figure 2.5-31. The structure, 100 x 120 feet in plan area, is adjacent to the Delaware River and will be the inlet for the power plant cooling water.

Excavation was performed by a clamshell dredge within a flooded sheet pile cofferdam. Excavation was originally to be to Elevation +30 feet which was estimated as the depth to unweathered Vincentown sand; the sheet pile structure was designed to this criterion. When the unweathered bearing stratum was actually not encountered until between Elevations +23 and +29 feet, additional strengthening of the cofferdam and further excavation were required. This additional work was carried out in December 1981.

2.5.4.5.3 Fills

Several sources of soils were considered for use as Category I backfill for the site. Following extensive laboratory testing and assessments, material from a borrow source at Oldman, New Jersey, was determined to be suitable (Reference 2.5-100). The material being used as Seismic Category I structural backfill is a relatively well graded coarse to fine sand with 3 to 12 percent fines and a gravel content ranging up to 20 percent. The values of the mean grain size (D_{50} corresponding to the upper and lower bounds of the grain size distribution envelope) are 0.5 and 1.1 millimeters, a narrow range indicating homogeneity of the deposit. Maximum dry densities obtained using ASTM Test Designation D 1557 procedures range from 117 to 133 lb/ft^3. Static strength parameters determined in consolidated drained triaxial compression tests included friction angles of 35 to 45° under confining pressures up to 80 $lb/in.^2$, and cohesion intercepts up to 1000 lb/ft^2 for samples compacted to 95 percent or greater of maximum dry density by ASTM D 1557. Dynamic stress controlled cyclic triaxial tests and subsequent liquefaction analyses indicate a factor of safety of 1.6 or greater against liquefaction of the compacted backfill (Reference 2.5-114).

In September 1981, in anticipation of exhaustion of the Oldman's borrow source, two alternative borrow areas at Hitchner and Mullica Hill, New Jersey, were investigated (Reference 2.5-101). Results indicated that soils from these sources have slightly more fines than the material from the Oldman's source, but were suitable for Seismic Category I structural backfill. Maximum dry densities ranged from 128 to 132 pounds per cubic foot for the Hitchner soil and 120.4 to 129 for the Mullica Hill soil. Dynamic stress controlled cyclic triaxial tests indicated cyclic shear strengths higher than that of the Oldman's borrow for both the Hitcher and Mullica Hill material. Two test fill embankments constructed of these soils compacted over the in-place Oldman's fill indicated that the Hitchner material meets the specified compaction requirements when placed and compacted in 8 inch lifts, while the Mullica Hill material requires smaller lifts to meet compaction specifications. Therefore the Hitchner is the preferred alternate source of Seismic Category I structural backfill material, although the Mullica Hill could be suitable if the placement criteria are altered appropriately (Reference 2.5-101).

The backfill material from these sources are placed in uniform lifts not exceeding 8 inches in loose thickness and compacted to an average compaction of 98 percent of the maximum dry density obtained by ASTM D 1557. Gradation and degree of compaction tests are performed daily during backfill operations to insure the quality of the backfill.

2.5.4.6 Groundwater Conditions

A detailed groundwater study of the plant site is presented in Section 2.4.13 and briefly summarized below.

The upper permeable sedimentary formations at the HCGS site comprise two aquifer systems, shallow and deep. The shallow aquifer system consists of river bottom sands and gravels of Quaternary age. The aquifer occurs across the entire site but is occasionally discontinuous. Its average and maximum thicknesses are 5 and 14 feet, respectively; it is encountered at Elevations between +70 and +65 feet. The piezometric head in this shallow aquifer in the vicinity of the main excavation is between Elevations +96 and +86 feet with a gradient of 0.7 percent. Available data indicate there is no connection between this aquifer and the adjacent Delaware River.

The deep aquifer system consists of several permeable sedimentary formations to a depth of 300 feet, confined by the overlying Kirkwood clay. Major flow toward the excavation is contributed by the basal sands at the Kirkwood formation and the upper strata of the Vincentown sands. A minor contribution is due to the influence of the lower Vincentown, Hornerstown, Navesink and Mount Laurel sands.

The mean static water level in the deep aquifer ranged between Elevations +77 and +83 feet. General pattern of flow is from north to south with a gradient of 0.3 percent in the vicinity of the site. Fluctuations and static groundwater level of up to 3 feet were due to tidal changes in the adjacent Delaware River.

2.5.4.6.1 Seepage

Large volumes of groundwater, primarily from the lower aquifer, could potentially seep into the main excavation. To allow construction in the dry, an extensive dewatering system is installed, as discussed in Section 2.4.13 and summarized in Section 2.4.6.2.

2.5.4.6.2 Dewatering During Construction

The dewatering system for the main excavation at the HCGS site consists of:

1. A ring of 32 deep dewatering wells spaced around the perimeter of the excavation. The wells are screened in both the shallow (riverbed sands and gravels) and Vincentown aquifers, and are intended to lower the piezometric level in the Vincentown aquifer sufficiently to allow removal of dredge spoils from the bottom of the excavation and construction of the well point system.

2. A system of 733 sand drains inside the ring of dewatering wells, at the top of the excavation slopes. These drains are 12 inch diameter holes filled with medium to coarse grained sand. The drains connect the shallow aquifer to the Vincentown aquifer and are intended to increase the effectiveness of dewatering the shallow aquifer.

3. Eight temporary deep wells between the sand drains and the foot of the excavation slopes. These wells are installed to increase the rate of lowering of the groundwater level sufficiently to allow construction of the well point system.

4. A well point system at the toe of the excavation slopes, just outside the building line. This system consists of 121 well points spaced around the perimeter of the

excavation bottom. The well point system serves as the primary dewatering system.

5. A surface water sump system in the bottom of the excavation, constructed to intercept runoff from rain or snow.

6. Eleven supplementary deep wells at the bottom of the excavation near the test pits to lower the piezometric levels in the vicinity of the test pits and trenches during the post excavation foundation studies.

2.5.4.6.3 Groundwater Monitoring During Construction of the Main Excavation

The monitoring system for the dewatering operation at HCGS consists of the following:

1. Observation wells around the perimeter of the excavation, which include ten 300 series wells constructed in boreholes drilled for the initial geologic and foundation investigations and two observation wells constructed during a later investigation. These wells provide baseline data on piezometric levels prior to the excavation and allow continued surveillance of the shallow aquifer and Vincentown aquifer piezometric levels during and after excavation.

2. Twenty four hydrostatic pressure cell piezometers located at the bottom of the excavation and midway down the excavation slopes. These piezometers are remotely monitored point piezometers which are installed to provide general surveillance of piezometric levels during excavation and on a long-term basis.

3. Five observation wells placed near the well point system and the temporary deep dewatering wells to monitor the

operation of these systems. In addition three point piezometers are installed in the bottom of the excavation near three of the pressure cell piezometer groupings to provide general data on piezometric levels and to verify the operation of the pressure cell piezometers.

4. Five observation wells for the test pits and trenches located at the bottom of the excavation to monitor the local groundwater levels near the trenches during the time that the trenches and test pits are open.

5. Two discharge weir boxes to gauge the overall output of the dewatering system and to recover any sand removed by the dewatering system.

6. Two pool level indicators installed at about elevations +65 and +35 feet to monitor and record the surface water level in the excavation during the dredging and pumpdown stages and to assure that the surface water level is not lowered below the Vincentown aquifer piezometric level.

7. Two deep observation wells at the SNGS.

8. A recording tide gauge and barograph, data from which are used in the computation of the tidal and barometric efficiencies of the shallow and Vincentown aquifers.

2.5.4.6.3.1 Groundwater Conditions Experienced During Construction

Prior to construction, exploratory borings were made throughout the site. Some of the borings were converted to observation wells. Based upon the exploratory borings, monitoring wells, and known conditions at the adjacent Salem Nuclear Generating Station site, two aquifers were encountered during the foundation excavation activities as expected. These two aquifers consist of a shallow water table aquifer and a deeper artesian aquifer. The shallow aquifer is composed of hydraulic dredge spoils and an underlying thin layer of granular river deposits. The deeper aquifer is the Vincentown aquifer composed of the Vincentown Formation sands and a

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relatively thin layer of Kirkwood Formation basal sands overlying the Vincentown Formation. Because of hydraulic continuity between the basal Kirkwood sands and the Vincentown Formation sands, the two units are considered together as one unit for hydraulic analysis. The Vincentown Formation is the foundation formation for HCGS, therefore, excavation did not proceed any deeper than the Vincentown Formation.

Prior to initiation of dredging operations for the HCGS excavation, ground water levels at the site were somewhat depressed due to dewatering activities at the adjacent Salem Nuclear Generating Station site. By the time dredging had started at the HCGS site, dewatering activities had ceased at the adjacent Salem Nuclear Generating Station site and ground water levels had returned to normal.

Normal ground water levels at the HCGS site consisted of a water table in the shallow aquifer which was generally less than five feet below the land surface, and a piezometric surface in the Vincentown aquifer which was similarly generally less than five feet below the land surface. There was a slight hydraulic gradient vertically downward from the shallow aquifer into the Vincentown aquifer.

The HCGS site was excavated by a large dredge. Spoils were pumped to a nearby holding area and the excess water was drained back to the excavation. Water levels were monitored in the monitoring wells and in the pool level in the excavation to verify that the surface water elevation in the excavation was always higher than the ground water level elevation in both the shallow and deep aquifers. Whenever the pool level lowered as a result of pumpage by the dredge, dredging was stopped to prevent a continued lowering pool level which could result in a hydraulic gradient upward from the ground into the pool. The water level in the pool was then maintained by pumping in river water to make up water losses caused by the dredging.

While dredging was going on, a dewatering system was being constructed around the periphery of the excavation. When the dewatering system was completed, a partial turn-on of the system was used to lower the ground water level to maintain the piezometric levels in the ground at a lower level than the pool water level in the excavation.

A floating pump was used to pump water from the excavation into the Delaware River. The pumping continued for one month in a controlled manner so as to prevent excessively rapid dewatering of the excavation slopes. In addition, pumping rates in the dewatering system were increased to maintain the ground water levels below the pool water level so that the hydraulic gradient would always be downward.

When the last bit of surface water was removed from the excavation, scraping of residual mud from the bottom of the excavation was accomplished by front-end loaders and trucks. Inspection of the slopes of the excavation at this time revealed that there were no ground water seeps from the sides of the excavation. The dewatering system composed of deep dewatering wells and the system of sand drains was successful in intercepting all ground water flowing radially toward the excavation. These two systems effectively prevented any ground water from seeping into the excavation. During subsequent activities the only water which had to be removed from within the excavation was runoff resulting from direct rainfall into the excavation.

The final phase of geotechnical investigations in the base of the excavation consisted of digging test pits and test trenches into the Vincentown formation. Because it was essential that these pits and trenches be dug in the dry, the water level in the Vincentown formation had to be lowered in the vicinity of the pits and trenches. For this purpose, additional dewatering wells and monitoring wells were installed in the vicinity of the pits and trenches. The monitoring wells were used to verify that the water levels were maintained below the levels of the bases of the pits and trenches before digging began.

After the test pits and trenches were filled in, the temporary dewatering wells were decommissioned allowing the water level to rise in the areas of the filled in pits and trenches. At all times, the ground water level was maintained at a minimum of three feet below the base of the excavation.

The lowest ground water levels attained in the Vincentown aquifer as shown by the pressure cell piezometers installed under the center of the excavation occurred during June 1977 at a level of about +12 feet, PSD.

By controlling the dewatering system output, the piezometric level in the Vincentown aquifer was allowed to rise to about +25 feet, PSD by January 1978. The water levels were maintained at about +25 to +30 feet PSD until October 1981. At this time a gradual rise in water level to about +40 feet PSD in March 1983 was allowed. A partial shutdown of the dewatering system on April 9, 1983 allowed a ground water recovery to about +63 feet PSD in August 1983.

Water level hydrographs of pressure cell piezometers P-1, P-2, and P-2A for the period November 1976 to August 1983 are shown in Figure 2.5-64.

Water level records for observation wells 39, 300A, 301, 302 and 303 for the period May 1976 to August 1983 are shown in Figure 2.5-65.

These water level hydrographs are representative of ground water conditions prior, during and after construction at the HCGS site. Additional ground water level hydrographs for the period May 1976 to June 1977 are given in Reference 2.3-154.

2.5.4.6.4 Continuing Groundwater Monitoring

During the plant construction/dewatering stage, groundwater levels were being continuously monitored in the observation wells at the site (References 2.4-49, 2.4-50 and 2.4-55). These data provide information on any change in the piezometric head or direction of groundwater flow that might occur.

During plant operation, the dewatering system is decommissioned and the groundwater levels are expected to return to the natural pre-existing conditions. The direction of groundwater flow in the shallow and deep aquifers will be primarily to the Southwest, toward the Delaware River. As long as HCGS production wells are in operation, the groundwater flow in upper Raritan aquifer will be toward the Hope Creek production wells (Section 2.4.13).

2.5.4.7 Response of Soil and Rock to Dynamic Loading

As described in Section 2.5.1, the HCGS site is underlain by 1500 to 2000 feet of coastal plain sediments overlying crystalline bedrock. The response of soil and rock to dynamic and seismic loading conditions is discussed in Section 2.5.2. Design dynamic properties of the subsurface materials at the site are presented in Tables 2.5-16 and 2.5-17. These values are based on a review of all field and laboratory tests and geophysical surveys performed at the site.

The field investigations and laboratory testing program which provided the data for evaluation of the dynamic soil and rock properties are described in Sections 2.5.4.3 and 2.5.4.2, respectively. Analyses based on the soil properties, including descriptions of design criteria and computer programs used, are discussed in Section 2.5.4.8. Effects of dynamic loading on buried pipelines are also discussed in Section 2.5.4.8. Soil structure interaction analyses are described in Section 3.7.2.

2.5.4.8 <u>Liquefaction Potential</u>

2.5.4.8.1 General

The soft to medium stiff soils in the stratigraphic zones above the Vincentown have been removed from the areas beneath all safety-related structures in the main power block by excavating to a depth of 70 feet. Lean concrete will be used as fill from the top of the Vincentown Formation to the founding grade for each foundation. All non-safety-related structures except the Turbine Building will be supported on piles extending to the Kirkwood or Vincentown Formations or on engineered backfill. The Turbine Building, although a non-safety-related structure, will be constructed on a mat foundation supported by the Vincentown sands. The service water piping is to be supported in engineered backfill bearing on Kirkwood clays, and the service water intake structure will be founded on lean concrete bearing on dense Vincentown sands.

Groundwater investigations, discussed in Section 2.5.4.6, indicate that under natural conditions the subsurface soils may be saturated as high as Elevation +96 feet. Therefore, the liquefaction potential of the Vincentown sands supporting safety-related structures is evaluated in detail, as well as each of the overlying strata, to assess their influence on the dynamic stability of safety-related structures.

Liquefaction potential for the site soils is analyzed by comparing the shear stresses generated at a point by the SSE and the cyclic shear strength of the soil under field conditions. The significant soil, site, and earthquake parameters influencing the liquefaction potential of the site are:

1. Soil properties

    a. Grain size characteristics

    b. Relative density

c. Dynamic properties

d. Cyclic shear strength

2. Subsurface profile and dynamic model

3. Earthquake parameters

a. SSE

b. Seismic response of soil

These parameters are discussed in detail in the following sections.

2.5.4.8.2 Soil, Site, and Earthquake Parameters

2.5.4.8.2.1 Soil Properties

2.5.4.8.2.1.1 Grain Size Characteristics

The significant grain size characteristics influencing the liquefaction potential of a site are the mean grain size (D_{50}, the uniformity coefficient ($U = D_{60}/_{10}$), and the percentage fines (materials passing through the No. 200 U.S. Standard Sieve). For the studies of the liquefied soil after the Niigata earthquake, Ohsaki (Reference 2.5-104) gave the following criteria for determining potentially liquefiable soils from the grain size distribution curve:

1. D_{10} (size opening permitting ten percent of the material by weight to pass) must be greater than .074 millimeters (No. 200 sieve)

2. D_{60} must be between 2 and 0.2 millimeters

3. Uniformity coefficient (D_{60}/D_{10}) must be less than 5

Kishida (Reference 2.5-103) in his studies identified liquefiable soils as those having:

1. D_{50} less than 2 millimeters
2. Uniformity coefficient less than 10
3. Relative density less than 75 percent

Grain size distributions for the site soils are discussed in Section 2.5.4.2 and graphically summarized in Figures 2.5-34 through 2.5-37. Using the Ohsaki or Kishida criteria, the basal sands are generally too fine and the river bottom sands generally too coarse to be susceptible to liquefaction. Portions of the hydraulic fill and Vincentown sands may be liquefiable by these criteria. However, these correlations are for uncemented soils and were developed from the results of cyclic load tests. It is apparent that the Vincentown is not a typical soil in which index properties can be directly correlated to liquefaction potential; therefore, lower bound dynamic characteristics were determined from cyclic testing, using index properties for correlation within the formation.

2.5.4.8.2.1.2 Relative Density

In general, the granular soils at the site have an amount of fines in excess of the maximum specified by ASTM for performance of its test for Relative Density of Cohesionless Soils (D 2049) (Reference 2.5-87), i.e. more than 12 percent. Therefore, evaluations of minimum and maximum densities by ASTM method were not performed, although numerous density measurements were made. In addition, as discussed in Section 2.5.4.1, Vincentown sands are cemented sands with varying degrees of cementation. For this reason there is not necessarily a direct correlation between density and cyclic strength in the Vincentown sands. In general the static and cyclic shear strengths of these soils are highly influenced by the degree of cementation.

Correlations were made between relative density and blow counts from standard penetration tests in the uncemented granular soils (Reference 2.5-104) for reference purposes, although these correlations may be affected by the presence of relatively high percentages of fines. Density measurements for the various soil strata are discussed in Section 2.5.4.2.

2.5.4.8.2.1.3 Dynamic Properties

The dynamic properties used for the seismic studies of the soil profiles at the site area are:

1. The elastic dynamic properties which include shear modulus of elasticity, damping ratio, and Poisson's ratio.

2. The cyclic shear strength of the soils.

Laboratory tests were performed to evaluate the shear modulus and damping ratio at various confining pressures and strain levels. These tests and results are discussed in Section 2.5.4.2. Poisson's ratios were estimated from the geophysical exploration results and cyclic static triaxial tests.

Cyclic shear strengths of the soils were evaluated by performing a series of cyclic stress controlled traxial tests. These tests were performed on the least cemented Vincentown sand samples to define the lowest bound of cyclic strength of the formation and on representative samples of the overlying formations. Tests were performed at different stress ratios, and failure was defined when ~5 percent axial strain was reached. From the scatter of values due to variable cementation within the samples, lower bound laboratory strength values were obtained. Correction factors suggested by Seed strengths under field conditions. Test procedures and results are and Peacock (Reference 2.5-105) were applied to represent the discussed in Section 2.5.4.2.

2.5.4.8.2.2 Soil Profile and Dynamic Soil Model

The dynamic analysis of the site requires the development of a one dimensional mathematical model of the site. Borings 229 and 201 were selected to represent the site for the purpose of defining the one dimensional mathematical model. Borings 201 and 229 are representative of the conditions below the reactor area and the cooling tower area, respectively (Tables 2.5-16 and 2.5-17). The model incorporates representative values of total unit weight, Poisson's ratio, shear modulus, and related parameters for each soil type.

2.5.4.8.2.3 Earthquake Parameters

2.5.4.8.2.3.1 Safe Shutdown Earthquake

The time histories shown in Figures 3.7-3 and 3.7-4 for the safe shutdown earthquake are synthesized accelerograms in the horizontal and vertical directions, respectively. The time histories used comply with the Regulatory Guide 1.60, as revised. The peak ground acceleration for this safe shutdown earthquake is 20 percent of gravity. It contains approximately 10 seconds of strong motion and has a total duration of 20 seconds. The response spectra for these time histories and their compliance to Regulatory Guide 1.60 are provided in Reference 3.7-1. Safe shutdown earthquake acceleration time history is discussed in Section 2.5.2. Although inappropriately severe for liquefaction analyses, the Regulatory Guide 1.60 time history was used in initial analyses to verify site suitability. As the safety factors against liquefaction with this artificially derived earthquake were more than adequate, additional analyses, using less severe but more realistic earthquake time histories, were not performed.

2.5.4.8.2.3.2 Seismic Response of the Subsurface

The response of the subsurface models subjected to the SSE as input was calculated assuming one dimensional vertical propagation of shear waves through a multilayered system. The computer program SHAKE, developed by Schnabel, Lysmer, and Seed (Reference 2.5-106) was used to compute the motion at the base of the layered system. A parametric study was performed to establish the adequacy of the dynamic subsurface models for the seismic response analysis, and a conservative interpretation resulting therefore was adopted for the analysis.

The values of the shear modulus, damping ratio, and strains determined by this analysis, along with the SSE time history, were used as input for a finite element model for the site. The finite element program used was QUAD4B, developed by Idriss and Lysmer (Reference 2.5-107). The program incorporates non-linear strain dependent modulus and damping properties for the soil in each element. The finite element technique was used to have a consistent mathematical model with the dynamic soil structure interaction analysis to be performed. From this analysis the maximum shear stresses and strains were obtained for each element.

2.5.4.8.3 Liquefaction Potential Analysis

The liquefaction potential is determined by comparing the shear stresses generated at a point by the SSE to the cyclic shear strength of the soil under field conditions. The maximum induced shear stresses at various points were obtained as described in the previous section. The shearing stress required to cause liquefaction, or cyclic shear strength, was determined for the Vincentown sands through laboratory tests performed with uniform amplitude stress cycles. A wave train of five uniform cycles, with amplitude of 0.65 times the maximum shear stress of the actual stress time history, was selected to represent to SSE loading (References 2.5-78, 2.5-108 and 2.5-109). From the site specific laboratory test data the shear stress ratio corresponding to five cycles to failure was selected and corrected to represent field conditions as discussed in Reference 2.5-105. The factor of safety against liquefaction was computed by comparing the field shear strength to the amplitude of uniform stress cycles equivalent to the

SSE induced shear stress at a particular point in the soil profiles. Based on conservative interpretations of test results and conservative assumptions throughout the analysis, the results yield adequate factor of safety against SSE induced liquefaction of the Vincentown sands that provide foundation support to safety-related structures.

The dynamic properties of the soil layers overlying the Vincentown sands (hydraulic fill, river bottom sands, Kirkwood clays, and basal sands) were not evaluated through extensive dynamic triaxial testing due to their secondary importance to plant safety. The dynamic strength of these layers was evaluated based on static strength tests, index properties, field tests, and data from literature, in addition to limited dynamic triaxial testing. Liquefaction potential was analyzed as for Vincentown sands by comparing dynamic strengths to the seismically induced shear stresses. Results indicate that the only soils which may experience SSE induced liquefaction are the sandy portions of the hydraulic fill, which occur generally in the upper 30 feet at the site.

Based on these analyses and analyses of sliding stability, (See Appendix 3G and Appendix 3H) it is concluded that the power block and intake structure are safe from sliding, even if the surrounding soil is completely liquefied. The structures will be embedded at least 60 feet in soil and only the upper 30 feet could liquefy, thus affording at least 30 feet of confinement to the power block structure. In addition, a non-liquefiable backfill will surround the structures up to final grade. Both factors will provide resistance to sliding. In the case of the service water pipes, which were evaluated by one and two dimensional analyses based on dynamic cyclic triaxial tests of the backfill material, the results of the liquefaction analysis indicate that they can be safely founded in the Kirkwood Formation (Reference 2.5-114). Clays such as these do not liquefy. The well compacted engineered fill which will support the pipes will resist lateral forces due to a surrounding liquefied natural soil in the unlikely occurrence of the SSE.

The dynamic settlements induced by the SSE to the Reactor Buildings and intake structure are estimated to be on the order of 0.1 inch. The settlements that the service water piping founded on the Kirkwood Formation could experience are estimated to be on the order of 0.2 inch. These settlements were analyzed by a procedure developed by Lee and Albaisa (Reference 2.5-110) based on the pore pressure increase within the formation.

Details of the analyses of liquefaction potential and dynamic stability of the site are presented in References 2.5-111, 112, 113, and 114.

Reference 2.5-79, Part I presents the results of liquefaction analyses of the foundation soils supporting the service water pipes. A non-dimensional shear wave propagation analysis and two dimensional finite element analysis were performed for this purpose. Although the case analyzed is slightly different in geometry from the as-built conditions, it is concluded that these changes will not affect the conclusions from the study. These analyses also indicate that the seismic induced stresses under SSE conditions are well below the respective cyclic strengths and that the materials surrounding and supporting the service water pipes will have an adequate factor of safety against a general sliding failure.

The above referenced analyses provided an overall assessment of the liquefaction potential and seismic stability of the hydraulic fill materials. The subsurface data along the service water pipes and an evaluation of the liquefaction potential of the materials based on index properties, soil classification and blowcount data shows the following:

1. Ninety percent of the hydraulic fill materials encountered in these borings are primarily cohesive and will not be susceptible to liquefaction.

2. Zones of liquefiable sandy materials generally occur in thin lenses or discontinuous layers. These materials amount to roughly 5 percent of the total materials encountered in the borings with another 5 percent potentially liquefiable. Duplicate borings were drilled at several locations to ensure a continuous profile of the subsurface materials.

3. The lenses/discontinuous layers of the sandy materials occur at random elevations and are typically 2 to 4 feet in thickness.

Based on these data, our analysis indicates that the service water pipes are stable under SSE conditions. The soils surrounding these pipes will not experience gross liquefaction that could cause mass instability of the entire hydraulic fill. Even if these localized sandy zones liquefy, they will be contained within the non-liquefiable cohesive materials and therefore will not affect the foregoing conclusions regarding the overall liquefaction potential and seismic stability.

2.5.4.9 Earthquake Design Basis

An SSE associated with a maximum horizontal ground acceleration of twenty percent of gravity (0.2g) has been determined for the site. Derivation of the SSE and the OBE is discussed in Section 2.5.2.

2.5.4.10 Static Stability

All safety-related structures as well as the turbine building are founded on lean concrete bearing on structural backfill placed on the dense to very dense sands of the Vincentown formation. Nonsafety-related structures, except the Turbine Building, are supported on piles extending into the Vincentown Formation or on structured backfill. Foundation levels, dimensions, and static loads for the major facilities of the station are presented in Table 2.5-18.

2.5.4.10.1 Structures on Foundation Mats

2.5.4.10.1.1 Main Power Block

The Reactor Building, the Auxiliary Buildings and the Turbine Building are supported on separate structural foundation basemats. The thickness of the structural mats for support of the buildings is 20 feet. To find the structures on competent Vincentown Sands, an excavation measuring 500 by 650 feet at the base, about 70 feet deep was required. Structural backfill required extends from the final depth of excavation to the bottom of the lean concrete mud mat, upon which the structural basemat for each building is placed.

The removal of approximately 70 feet of soil resulted in a stress relief of approximately 4000 lb/ft^2. As a result of this stress relief, elastic rebound of about 1/2 inch was estimated to occur. Most of this has occurred during the course of excavation to competent Vincentown Sand, Elevation 30 feet, as discussed in Section 2.5.4.13. Recompression of this heave commenced with the placement of structural fill and lean concrete, and it is estimated that nearly all recompression of heave had occurred by the time the basemats were constructed.

The Vincentown soils, ranging from poorly graded sands to silty sands, provide a uniform bearing stratum. Although the cemented zones of the Vincentown Formation within the main station area are sometimes discontinuous, the density, as indicated by the blow counts of standard penetration tests and by measurements of undisturbed samples, falls in the dense to very dense range. Bearing capacity analyses, utilizing the conventional bearing capacity methods, indicate a safety factory greater than 3 against a general shear failure of the soil.

Several methods were used to compute the settlement of the power block structures. These analyses were performed assuming that the mats were uniformly loaded.

Settlements, based on the Peck and Bazara method (Reference 2.5-117), were computed to be 0.4 inches and 1.3 inches for net contact pressures of 2.1 kips per square foot (ksf) and 8.0 ksf, respectively. A heave of 0.5 inches was computed using the same method for excavation to the Vincentown formation. Settlements, based on the Theirs, Salver and Gray method (Reference 2.5-116), were computed to be 0.6 inches and 1.5 inches for net contact pressures of 2.1 ksf and 6.5 ksf, respectively. A settlement of one inch was computed using the Janbu method (Reference 2.5-118) for a net contact pressure of 2.1 ksf. Table 2.5-18 provides the current net contact pressures for the individual structures.

Records of settlement monitoring during and after construction are provided in Figures 2.5-67 through 2.5-96.

The areas around the Reactor, Auxiliary, and Turbine Building structures are backfilled to final grade with compacted well graded granular soils. The walls of these structures are designed to resist the lateral pressures of the soils under static and dynamic loadings. The static earth pressures are based on "at-rest" conditions, whereas the dynamic earth pressures are determined based on soil structure interaction analysis discussed in Section 3.7.2.5. Figures 2.5-60 and 2.5-61 provide the earth pressures used as design bases. Although the static lateral earth pressures given in Figures 2.5-60 and 2.5-61 are low, the below grade walls have the capacity to resist substantially higher lateral earth pressures.

2.5.4.10.1.2 Service Water Intake Structure

The Service Water Intake Structure, approximately 100 x 120 feet in plan area, is a safety-related structure. It is located at the waterfront and consequently is partially submerged. The structure will be founded on a mat at Elevation +65.5. Tremie concrete will be placed between the base of the mat and the bearing level in the Vincentown sands. The unweathered greenish gray Vincentown sands considered suitable as a bearing stratum occur at approximately

Elevation +25 feet in the intake area of the site and, borings and initial excavation operations at the location of the Service Water Intake Structure encountered the unweathered Vincentown at approximately Elevation +23 to +29 feet (Reference 2.5-119). This lower occurrence of the bearing stratum in this area was taken into account in the configuration and calculated contact stresses of the intake structure.

The stress relief due to excavation of approximately 70 feet of submerged soil is expected to be 4000 lbs/ft^2. However, because the total excavation area is only 100 x 120 feet and because sheet piles extend below the excavation level, the elastic rebound is expected to be negligible. About 70 percent of the removed load will be restored by the time placement of lean concrete is completed at the proposed grade, Elevation +65.5 feet. The net load to be imposed by the proposed construction is calculated to be very small because of stress relief and buoyancy effects.

Under these conditions, the estimated settlement of the intake structure (assuming uniform loading) is less than 1/2 inch. The total settlement including recompression and rebound is estimated to be less than one inch.

Settlements of the service water intake structure are measured by optical means using reference markers located on the base mat. The measurements are taken on a periodic basis. The settlement plot is provided in Figure 2.5-66. The service water pipeline consists of 20 foot long segments which are designed to accommodate differential settlement of 1 inch. Since the calculated differential settlement is less than 1 inch, no subsurface instrumentation was provided for the service water pipe lines.

The allowable net static bearing capacity of the Vincentown Formation at the intake structure due to uniform loading conditions is estimated to be 12 kips per square foot. The allowable bearing pressure estimate is based on settlement considerations; the allowable bearing pressure based on general or local shear failure conditions will be considerably higher. The allowable net dynamic bearing capacity of the Vincentown Formation at the intake structure is estimated to be 15 kips per square foot. Because of the large size of the raft foundation for the intake structure, the factor of safety against a bearing capacity failure of the underlying sand will be great.

2.5.4.10.2 Structures on Piles

Most permanent site structures, except the main power block structures, the intake structure, and the condensate storage tank are supported on pile foundations. Permanent structures not on piles are either located on structural backfill near the power block or are lightly loaded so they may be supported on the hydraulic fill.

Prior to driving piling for major structures outside the power block, a pile load test program was conducted (Reference 2.5-120) to determine the proper piling for the site. The dewatering of the site is causing consolidation and thus settlement of the hydraulic fill. This condition causes a negative skin friction load (downdrag) on the piles. This downdrag load is added to the design load to obtain the ultimate capacity of the piles. All piles are designed with a safety factor of 2, including this downdrag load. The capacity of the piles was verified with a pile and load test performed during the production pile driving.

Major structures which are supported on piles are:

1. Hyperbolic cooling tower, approximately 450 feet in diameter at the base and about 500 feet high, including fill support columns and basin

2. Circulating water pump structure and canal

3. Switchyard structures

4. Other miscellaneous yard structures located on hydraulic fill as described above.

2.5.4.10.3 Lateral Earth Pressures

The lateral pressures exerted by the soil were analyzed in relation to the stability of the power block area, and the service water intake structure and piping in a study performed in 1975 (Reference 2.5-125). For the evaluation of the soil pressures and stability of the structures, the following assumptions and hypotheses were used.

1. The ground water table is located at the final grade elevation for simplicity and conservatism;

2. The water level in the river is at the mean low tide (MLT) elevation as given in Bechtel's drawing No. C-0182-0, Rev. B, dated November 22, 1974;

3. The models of the actual situations are represented by Figures 2.5-57, 2.5-58 and 2.5-59.

4. The friction angle between the foundation mat and the compacted granular fill is 3° (Reference 2.5-126 and preliminary estimate of fill material properties);

5. The friction angle of the compacted granular backfill is 37° (Reference 2.5-126);

6. The dead load and live load of the power block structure were considered in computing the inertia forces in the pseudo static analysis; and

7. Only the horizontal component of the SSE was considered for this pseudo static analysis; and

8. The factor of safety was defined as the ratio of shearing strength to shearing stress along the postulated sliding surface.

The analysis was performed using a pseudo static approach proposed by Matsuo and Ohara (References 2.5-127 and 2.5-128). While other methods were used for the comparison (References 2.5-129, 2.5-130, 2.5-131 and 2.5-132), this method was chosen, since it resulted in higher imposed pressures and, thus, a more conservative solution (Reference 2.5-131).

The lateral forces were computed for the intake structure for two conditions:

1. assuming the hydraulic fill completely liquefied, and

2. assuming the hydraulic fill does not liquefy. These two conditions represent two extremes of the possible field conditions expressed. However, the power block and the service water piping were analyzed assuming two basic liquefied conditions: the upper soils flowing into the river and non-flowing. The flowing condition is highly unlikely, and it is presented only for information purposes.

Field and laboratory test results used in this analysis are discussed in Sections 2.5.4.2 and 2.5.4.3.

In this study, the analyses were performed assuming the conservative assumptions.

Figures 2.5-57, 2.5-58, and 2.5-59 provide the details of the models analyzed. Table 2.5-19 provides a summary of the results obtained for the power block and the intake structure. Table 2.5-20 presents the results obtained for the service water piping.

An independent stability analysis has also been performed for the intake structure and power block as described in Appendixes 3G and 3H, respectively.

2.5.4.11 Design Criteria

The design criteria are based on established soil mechanics principles discussed in the references cited. Static bearing capacity was evaluated using shear strength test results and bearing capacity factors as discussed in Terzaghi and Peck (1967), W.C. Teng (1965), Peck, Hanson and Thornburn (1973), and Winterkorn and Fang (1975). Deep failure was evaluated using a computer program developed by Dames & Moore to analyze deep circular slip surfaces (Reference 2.5-118). Dynamic bearing capacity was evaluated according to Meyerhof, as discussed in Harr, 1966.

The temporary construction slopes were analyzed for stability using the Dames & Moore stability program (Reference 2.5-123), which employs the Bishop method of circular slip surface analysis (Reference 2.5-121). An additional check of stability was made using the sliding wedge method as discussed in Reference 2.5-122.

Settlements were computed by several methods (Reference 2.5-115, 116, 117, and 119). including Janbu's tangent modulus method. Dynamic settlements were estimated using a method developed by Lee and Albaisa (Reference 2.5-110). Liquefaction potential was evaluated by comparison of seismically induced stresses with dynamic shear strength of the soil, as discussed in Reference 2.5-124.

Allowable factors of safety for bearing capacity were selected according to accepted procedures and practice, using a minimum allowable factor of safety of 3.0 for static loads and 2.0 for dynamic loads.

2.5.4.12 Techniques to Improve Subsurface Conditions

The relatively soft and loose soils above approximately Elevation +30 feet were excavated from the main power block area and from the service water intake area in order to found these safety-related structures on the competent Vincentown sands. The relatively loose hydraulic fill and river bottom sands along the service water pipelines were excavated so that the pipes may be installed in compacted granular backfill bearing on the competent Kirkwood clays.

Subsequent to excavation, techniques to improve subsurface conditions consisted of normal foundation preparation for placement of fill or concrete on soil bearing surfaces. Foundation preparation consisted of removal of debris and disturbed or frozen soil, then proofrolling before placement of fill. Foundation preparation was inspected by a qualified engineer who determined that competent soil was attained at foundation bearing grades and that unsatisfactory material was removed.

2.5.4.13 Subsurface Instrumentation

An instrumentation program was initiated at the HCGS site to:

1. Provide information on the relative movements of the surface of the Vincentown Formation as a result of heave from excavation and settlements due to the placement of structural backfill and the imposition of the structural and service loads.

2. Provide information on the actual performance of the structures. Six linear extensometers were installed in March 1976, at the locations shown on Figure 2.5-55.

   The major features of the extensometers are illustrated on Figure 2.5-56.

Excavation at the site was performed using hydraulic dredging techniques from May to October, 1976. Following the demobilization of the dredge, a period of controlled dewatering of the excavation ensued in which the surface water in the dredged hole was lowered. Upon completion of the dewatering phase, excavation of the residual soft soils which had remained after the hydraulic dredging operations was carried out by conventional means. When the approximate recorded surface locations of the buried extensometers were approached, actual locations were established by conventional survey procedures. Excavation continued manually around the recorded surface location of each extensometer until either the grout column of the previously installed extensometer was encountered or the top of the instrument package was unearthed.

When the top of the instrument package was uncovered, the actual elevation of the top anchor, the actual location of each extensometer, and a reconnect reading for each extensometer was obtained. Permanent cables were then attached to the extensometer sensor housing and routed to the instrumentation terminal buildings and buried in trenches excavated into the bearing stratum in the Vincentown Formation. Periodic readings of each extensometer were taken to verify that each instrument was working properly and to measure the relative movements of the surface due to construction activities.

The data obtained upon excavation of the sensor package ranged from +0.37 inches at Extensometer No. 1 to +0.17 inches at Extensometer No. 6. These measured heaves agree favorably with the anticipated surface heave of approximately 1/2 inch due to excavation (Section 2.5.4.10). Because of the depth selected for the location of the fixed anchor point, an output of approximately 80 percent of the total heave/settlement was expected; when this factor was applied to the heave measurements made, the results were in relatively good agreement with the original estimates resulted (Reference 2.5-95).

During the active construction phases of the project, readings were obtained to monitor soil movements. Readings were taken immediately before and after a significant change in grade or construction, such as when a large pour of concrete occurred in the vicinity of an instrumented location. When groundwater was allowed to rise, readings were taken for every 10 feet of rise of the groundwater level to monitor the effects of buoyancy on the overall movements to date. Since completion of construction operations, readings are obtained on a yearly frequency or as required during plant operation.

2.5.4.14 Construction Notes

Construction techniques for the excavation of the power block and SSWS intake structure is discussed in Section 3.8.5.6. Engineered backfill is discussed in Section 3.8.6.1.

2.5.5 Stability of Slopes

There are no natural soil or rock slopes within the plant boundaries. Temporary excavated soil slopes were established in the excavation for the main power block as discussed in Section 2.5.4.5. The excavation extends to a depth of approximately 70 feet, (Elevation +30 feet). The slope of the cut is approximately 2 horizontal to 1 vertical to a depth of 37 feet (Elevation +63 feet) and 1 horizontal to 1 vertical to a depth of 70 feet (Elevation +30 feet), with a 25 foot berm at Elevation +63 feet.

Detailed analyses of the stability of these temporary slopes were performed, using the simplified Bishop and sliding wedge methods of analysis (References 2.5-121 and 2.5-122). The analyses considered both instantaneous drawdown ($\emptyset = 0$) and a steady state of lowered water table (effective stress analysis). Results for both cases indicate adequate safety factors against local and deep seated failures.

The main excavation will be backfilled to final grade prior to commencement of plant operation. Therefore any potential failure of temporary slopes will not adversely affect the safety of the plant.

Upon completion of construction, there will be no natural or constructed slopes within the plant boundaries which could affect plant safety.

2.5.6 Embankments and Dams

No earth, rock, or earth and rockfill embankments will be used for plant flood protection or for impounding cooling water required for the operation of the plant.

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TABLE 2.5-1

EARTHQUAKE LIST

Date	H	M (gmt)	S	Lat. (North)	Long. (west)	Intensity (MM)	Magnitude	Hypocentral Depth	Reference(1)	Distance (Miles)
1698				41.4	73.5	IV			ANY	168
1702				41.4	73.5	IV			ANY	168
1711				41.4	73.5	IV			ANY	168
6 AUG 1729				41.4	73.5	IV			ANY	168
19 DEC 1737	4	0	0.0	40.8	74.0	VII			EQH	119
29 NOV 1738	22	50		41.0	74.5	VI			D-M	118
25 APR 1758	2	30	0.0	38.9	76.5	V			EQH	67
21 FEB 1774	19	0	0.0	37.2	77.4	VII			EQH	189
30 NOV 1783	3	50	0.0	41.0	74.5	VI			PAG	116
27 AUG 1833	6	0	0.0	37.8	78.0	V-VI			D-M	180
9 AUG 1840	20	30	0.0	41.5	72.9	V			EQH	194
26 OCT 1845				41.0	73.8	V			WES	137
2 SEP 1847				40.2	72.0	V			ANY	192
9 SEP 1848	4	0	0.0	40.8	74.0	V			NYS	119
2 NOV 1852	18	35	0.0	37.8	78.0	VI			D-M	180
7 FEB 1855	4	30	0.0	42.0	74.0	V			WES	189
1 JUL 1858	3	45	0.0	41.3	73.0	V			WES	181
9 OCT 1871	14	40	0.0	39.7	75.5	VII	5.7(2)		EQH	13
11 JUL 1872	10	25	0.0	40.9	73.8	V			EQH	132
11 DEC 1874	3	25	0.0	40.9	73.8	V			WES	132
28 JUL 1875	9	10	0.0	41.8	73.2	V			EQH	199
22 DEC 1875	23	45	0.0	37.5	77.5	VI			D-M	175
10 SEP 1877	14	59	0.0	40.3	74.9	V			EQH	63
5 FEB 1878	16	20	0.0	40.7	73.7	V			WES	126
4 OCT 1878	7	30	0.0	41.5	74.0	V			EQH	159
25 MAR 1879	24	30	0.0	39.2	75.5	V			EQH	20
9 AUG 1880	20	30	0.0	41.5	72.9	V			ANY	194
11 MAR 1883	23	57	0.0	39.5	76.4	V			EQH	48
12 MAR 1883	6	0	0.0	39.5	76.4	V			EQH	48
31 MAY 1884				40.6	75.5	V			ANY	75
10 AUG 1884	19	7	0.0	40.6	74.0	VII			EQH	109
3 JAN 1885	2	16	0.0	39.2	77.5	V			EQH	109
8 MAR 1889	23	40	0.0	40.0	76.0	V			EQH	43
9 MAR 1893	5	30	0.0	40.6	74.0	V			EQH	109
1 SEP 1895	11	9	0.0	40.7	74.8	VI			EQH	90
18 DEC 1897	23	45	0.0	37.7	77.5	V			EQH	164
10 MAR 1902				39.6	77.2	IV			BOL	91
8 MAY 1906	17	41	0.0	38.7	75.7	V			EQH	56
10 Jan 1907	9	45	0.0	41.3	77.0	IV			ANY	147
11 FEB 1907	8	22	0.0	37.7	78.3	VI			D-M	190
5 FEB 1908	8	20	0.0	41.4	73.2	IV			ANY	178
31 MAY 1908	17	42	0.0	40.6	75.5	VI			EQH	75
23 AUG 1908	9	30	0.0	37.5	77.9	V			EQH	189

Date	H	M (gmt)	S	Lat. (North)	Long. (west)	Intensity (MM)	Magnitude	Hypocentral Depth	Reference[1]	Distance (Miles)
2 APR 1909	7	25	0.0	39.4	78.0	VI			EQH	133
23 APR 1910				41.0	73.0	IV			NJS	167
23 APR 1910	21	0	0.0	39.3	76.7	IV			BOL	65
8 MAY 1910	10	10	0.0	37.7	78.4	V			D-M	195
8 Jun 1916	21	15	0.0	41.0	73.8	V			EQH	137
10 APR 1918	2	9	0.0	38.7	78.4	VI			EQH	165
1 MAY 1918	4	48	5.0	41.0	77.0	VI			ISS	130
6 SEP 1919	2	46	0.0	38.8	78.2	VI			EQH	152
26 JAN 1921	23	40	0.0	40.0	75.0	V			EQH	43
7 AUG 1921	6	30	0.0	37.8	78.4	V			EQH	195
1 JAN 1924				39.2	78.0	V			BOL	135
26 JAN 1926	23	40	0.0	40.0	75.0	V			ANY	43
12 MAY 1926	3	30	0.0	40.9	73.9	V			EQH	128
1 JUN 1927	12	20	0.0	40.3	74.0	VII			EQH	96
10 JUN 1927	2	16		38.0	78.5	V			D-M	185
27 DEC 1929	2	56	0.0	38.1	78.5	VI			USE	188
1 JUL 1931	2	45	0.0	41.6	73.4	IV	3.6[2]		EPB	182
25 JAN 1933	2	0	0.0	40.2	74.7	V	4.3[2]		EPB	64
19 JUL 1937	3	51	0.0	40.7	73.7	IV			USE	127
15 JUL 1938	22	45	0.0	40.4	78.2	V			USE	157
2 AUG 1938	10	2	0.0	41.1	73.7	IV			USE	144
23 AUG 1938	3	36	34.0	40.2	74.2	V	4.6[2]		EPB	84
23 AUG 1938	5	4	55.0	40.2	74.2	VI	4.8[2]		EPB	84
23 AUG 1938	7	3	29.0	40.2	74.2	V	4.6[2]		EPB	84
23 AUG 1938	11	11	6.0	40.2	74.2	IV	3.7[2]		EPB	84
15 NOV 1939	2	53	48.0	39.6	75.2	V			CGS	17
25 MAR 1940	22	28	0.0	38.9	78.6	IV-V			D-M	141
24 OCT 1942	17	27	3.6	41.0	75.2	IV	3.4[2]		EPB	102
1943				41.1	74.2	V			NJS	130
5 FEB 1944	16	22	0.5	40.8	76.2	IV	3.7[2]		EPB	97
28 OCT 1946	20	36	6.0	41.5	76.6	IV	3.6[2]		EPB	149
4 JAN 1947	18	51	4.0	41.0	73.6	V	4.3[2]		EPB	146
4 JAN 1948	22	20	0.0	37.5	78.5	V			D-M	196
8 MAY 1949	6	1		37.5	78.0	IV-V			D-M	185
29 MAR 1950	14	43	2.0	41.0	73.6	IV	3.6[2]		EPB	146
3 SEP 1951	21	26	24.5	41.3	74.2	V	4.4[2]		EPB	137
23 NOV 1951	6	45	36.0	40.6	75.5	IV	3.6[2]		EPB	75
10 SEP 1952	22	15	0.0	38.1	78.5	IV			BOL	188
8 OCT 1952	21	40	0.0	41.7	74.0	V	4.3[2]		EPB	171
7 FEB 1953	3	0	0.0	37.7	78.2	IV			BOL	191
27 MAR 1953	8	50	0.0	41.1	73.5	V	4.3[2]		EPB	152
17 AUG 1953	4	22	50.0	41.0	74.0	IV	3.7[2]		EPB	130
7 JAN 1954	7	25	0.0	40.3	76.0	VI	5.0[2]		EPB	61
21 FEB 1954	20	0	0.0	41.2	75.9	VII	5.7[2]		EPB	120
24 FEB 1954	3	55	0.0	41.2	75.9	VI	5.0[2]		EPB	119

TABLE 2.5-1 (Cont)

Date	H	M (gmt)	S	Lat. (North)	Long. (west)	Intensity (MM)	Magnitude	Hypocentral Depth	Reference[1]	Distance (Miles)
31 MAR 1954	21	25	0.0	40.2	74.0	IV	3.6[2]		EPB	95
11 AUG 1954	3	40	0.0	40.3	76.0	IV	3.6[2]		EPB	60
20 JAN 1955	3	0	0.0	40.3	76.0	IV	3.6[2]		EPB	61
23 MAR 1957	19	2	0.0	40.6	74.8	VI	4.8		ANY	84
22 JAN 1960	20	53	22.0	41.5	75.5	IV	3.4		EPB	138
15 SEP 1961	2	16	56.0	40.6	75.4	V			ANY	76
27 DEC 1961	17	6	0.0	40.1	74.8	V			ANY	55
23 JAN 1962	2	33	0.0	39.8	75.9	I-II			D-M	30
3 MAR 1963	1	24	32.0	41.5	75.8	IV	3.4		ANY	138
10 OCT 1963	14	59	52.5	39.8	78.2	IV	3.6[2]		EPB	145
12 MAY 1964	6	45	14.1	40.2	76.5	VI	4.5 m	33	CGS	71
13 FEB 1964	19	46	38.8	40.5	77.9	VI	5.2 m	15	CGS	144
17 NOV 1964	17	8	0.0	41.2	73.7	V	4.3[2]		EPB	150
15 JUL 1965	14	16	7.0	37.3	74.4	VI	5.1		CGS	163
16 SEP 1965	19	51	9.7	37.3	74.4	VI	5.1		CGS	163
29 SEP 1965	20	57	39.5	41.4	74.4	IV			NYS	143
31 MAY 1966	6	19	2.1	37.6	78.0	V	3.1 m	33	CGS	188
22 NOV 1967	22	10	0.0	41.2	73.8	V			NYS	147
3 NOV 1968	8	33	52.5	41.3	72.6	V			EQH	197
10 DEC 1968	9	12	44.9	39.7	74.6	V	2.6 m PAL	23	USE	49
6 OCT 1969				41.0	74.6	IV			NJS	113
11 DEC 1969	23	44	39.2	37.8	77.4	V			USE	155
20 AUG 1970	16	34	15.0	38.9	72.4	V	4.2 m		CGS	172
12 SEP 1971	0	6	27.1	38.1	77.4	V			ERL	143
11 FEB 1972	0	16	0.0	39.7	75.7	II			D-M	17
8 DEC 1972	3	0	32.6	40.1	76.2	IV		4	ERL	58
28 FEB 1973	8	21	32.3	39.7	75.4	VI	3.8 m SLM	14	ERL	15
10 JUL 1973	4	38	0.0	near Wilmington, Delaware		IV			D-M	15
7 JUN 1974	19	45	36.8	41.6	73.9	V	3.3 m PAL		GS	164
11 MAR 1976	21	7	20.2	41.0	74.4	IV	2.5 m PAL		D-M	118
13 APR 1976	15	39	13.0	40.8	74.1	V	3.0 m PAL		PAL	120
21 JAN 1977	20	50	44.5	40.0	74.3	IV	2.7	6	D-M	70
30 JUN 1978	20	13	43.6	41.1	74.2	IV	2.9 m PAL	5	GS	128
16 JUL 1978	6	39	37.8	39.9	76.3	V	3.1 m	5	GS	50
6 OCT 1978	19	25	41.6	40.0	76.5	VI	3.0 m	5	GS	63
30 JAN 1979	16	30	52.1	40.3	74.3	V	3.3 m	5	GS	86
23 FEB 1979	10	23	57.2	40.8	74.8	IV	2.9 PAL	13	GS	97
10 MAR 1979	4	49	39.7	40.7	74.5	V	3.1 m PAL	3	GS	99
30 DEC 1979	14	15	11.6	41.1	73.7	V	2.5 C WES	5	GS	148
17 JAN 1980	10	13	16.1	41.3	73.9	IV	2.9 PAL	5	GS	149
5 MAR 1980	17	6	54.5	40.2	75.2	IV	3.5 m PAL	3	GS	50
6 MAR 1980	17	20	32.4	40.2	75.1	IV	3.5		LDO	50
11 MAR 1980	6	0	26.0	40.2	75.1	IV	3.7 PAL		GS	49
2 MAY 1980	19	2	24.4	40.2	75.0	IV	3.0		LDO	50

TABLE 2.5-1 (cont)

Date	H	M (gmt)	S	Lat. (north)	Long. (west)	Intensity (MM)	Magnitude	Hypocentral Depth	References[1]	Distance (miles)
24 OCT 1980	17	27	38.1	41.3	72.9	IV	3.1 m	6.68	GS	187
25 OCT 1980	0	41	28.3	41.3	72.9	IV	2.7		WES	187

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(1) The following abbreviations are used:

ANY	Earthquakes adjacent to New York State (N.Y. State Geological Survey)
BOL	Bollinger, G.A., 1973
CGS	Coast and Geodetic Survey
D-M	Dames & Moore
EPB	Earth Physics Branch Department of Energy, MInes & Resources, Ottawa, Canada
EQH	Earthquake History of United States, Coffman and VonHake
ERL	Environmental Research Laboratories (NOAA)
GS	United States Geological Survey
ISS	International Seismological Summary Kew, England, UK
LDO	Lamont-Doherty Geological Observatory
NJS	New Jersey State Geological Survey
NYS	New York State Geological Survey
PAG	Page et al, 1968
PAL	Palisades, New York
WES	Weston Geophysical, Inc
USE	United States Earthquakes
M	Richter Magnitude
m	body wave magnitude
M	Surface wave magnitude
m	Nuttli Magnitude
C	Coda Length
m	Nutti magnitude

(2) Earth Physics Branch uses the Richter conversion of magnitude M = 2/3 I + 1. There were no additional earthquakes of M = 3.0 within a 200 miles radius reported in the Southeastern United States Seismic Network Bulletins through June 30, 1982 and the Northeastern United States Seismic Network Bulletins through September 30, 1982.

TABLE 2.5-2

INDEX PROPERTIES
HYDRAULIC FILL[1]

Boring No.	Sample No.	Depth (ft)	Atterberg Limits Liquid Limit	Atterberg Limits Plasticity Index	Specific Gravity	Natural Moisture Content(%)	Dry Density (lb/ft^3)[3]
211	1	5	73	26		53.9	64.3
	4	20	61	23		57.8	65.5
216	1A	5	30	11		44.6	
	4	25	75	36		68.1	46.5
217	2	11				31.6	88.3
	5	25	91	48		65	59.9
222	2	10	94	48		80.4	53.1
229	2	10	29	10		20.1	100.2
	7	30	80	40		66.4	60.6
232	6A	25	69	38		52.3	
	6	28			2.58		
238	1	5	90	37		63.1	60.6
	5	20	73	30		64	60.8
239	1	6				20.4	109.9
	7	35	52	13		53.8	66.3
253	6	25	89	43		76	
	2	20			2.60		
AB-1	10	30	93	58	2.50	64	60
AB-1A	7	19	51	29		52	68
AB-2	8	19	74	45	2.58	64	
AB-3	6	15	43	17	2.65	40	
AB-4	2	5	70	41	2.69	54	
	5	13	72	40	2.54	68	58

TABLE 2.5-2 (Cont)

Boring No.	Sample No.	Depth (ft)	Atterberg Limits Liquid Limit	Plasticity Index	Specific Gravity	Natural Moisture Content(%)	Dry Density ($lb/ft^{(3)}$)
AB-5	3	6	66	37	2.54	64	61
	7	16	63	34		43	77
	14	33	79	47	2.58	69	

(1) Reference: 2.5-57

TABLE 2.5-3

INDEX PROPERTIES

RIVER BOTTOM SANDS

Boring No.	Sample No.	Depth (ft)	Specific Gravity	Natural Moisture Content (%)	Dry Density (lb/ft$^{(3)}$)
217	7	35.5		12.3	121.6
228	11A	50		19.8	
AB-1	12	35		18	111
	13	35	2.64	23	
AB-1A	11	34		19	108
	12	38		20	109
AB-2	14	34	2.67	18	
AB-3A	7	36		20	

TABLE 2.5-4

INDEX PROPERTIES
KIRKWOOD CLAYS[1]

			Atterberg Limits			Natural	Dry
Boring No.	Sample No.	Depth (ft)	Liquid Limit	Plasticity Index	Specific Gravity	Moisture Content(%)	Density ($lb/ft^{(3)}$)
AB-1	14	40	57	36	2.66	36	86
	15	42	60	35	2.73	46	
	21	57	66	34	2.63	51	
	24	65	58	29		42	77
AB-1A	18	68	71	35		60	
AB-2	15	37	53	32	2.61	35	88
	16	39	55	30	2.63	52	
	24	59	66	31		58	
	27	67	45	16			
AB-2A	8	35	37	20		22	104
	9	41	42	20		42	
	13	61	53	19		50	
AB-3	17	43	81	50	2.62	58	64
	22	55	58	26	2.67	52	
	25	63	63	28		51	69
AB-3A	13	65	55	18		42	74

(1) Reference: 2.5-96

TABLE 2.5-5

INDEX PROPERTIES
BASAL SANDS[1]

Boring No.	Sample No.	Depth (ft)	Specific Gravity	Natural Moisture Content (%)	Dry Density (lb/ft(3))
206	10A	60.4		26.0	108.33
211	8A	65.5		25.8	
214	8A	57.0		36.7	
217	10	50.5		24.2	
228	14	65.0		29.7	
231	10	50		26.2	100
238	11	50		18.3	112.6
242	12	60		27.2	99
249	10C	46		14.7	
AB-1	26	70		18.0	108
AB-2	28	69	2.63	20	
AB-3	26	65	2.57	38	
AB-4	23	57		26	
AB-5	20	48	2.70	27	

(1) References: 2.5-57, 2.5-96

TABLE 2.5-6

INDEX PROPERTIES
VINCENTOWN SANDS[1]

Boring No.	Sample No.	Depth (ft)	Atterberg Limits Liquid Limit	Atterberg Limits Plasticity Index	Specific Gravity	Natural Moisture Content(%)	Dry Density (lb/ft[3])
201	18	100.0				29.8	93.2
	19A	110.0				41.7	79.2
211	11A	80.5				29.5	92.0
214	10A	65.6				26.2	97.9
216	14A	75.5				34.8	86.4
	14B	76.1				29.9	92.9
225	10D	57.5				21.3	101.8
	11A-B	59.4				35.2	84.2
	11A-C	59.9				22.2	102.3
231	12	60.0				32.1	95.0
238	17	75.5			2.64		
239	14	70.5				35.4	84.9
	15	75.5			2.68		
	16	80.5				30.5	88.7
220	14	70.5			2.66		
	16	80.0			2.67		
	20	120.0			2.60		
257	13	65.5			2.69		
	17	85.0			2.68		
	20	100.0			2.70		
259	14	65.0			2.67		
275	33	91.0	44	15	2.68	25.1	
	44	113.4	47	20	2.72	26.8	

Boring No.	Sample No.	Depth (ft)	Atterberg Limits Liquid Limit	Atterberg Limits Plasticity Index	Specific Gravity	Natural Moisture Content(%)	Dry Density (lb/ft(3))
277	16C	98.0	27	6		31.4	
	16E	98.5	33	6	2.73	32.3	
	25C	116.0	37	10	2.70	26.7	
	26C	118.5	27	11	2.721	27.8	

(1) References: 2.5-57, 2.5-58

TABLE 2.5-7

CONSOLIDATION TEST DATA

Boring No.	Sample No.	Depth (ft)	Initial Water Content (%)	Void Ratio	Specific Gravity	Liquid Limit (%)	Plastic Limit (%)	Recompression Ratio(1)	Compression Ratio(2)	Compression Index	Maximum Past Pressure (tsf)	Soil Type	Formation
232(3)	2D	6.25	50.0	1.33	2.66	82	33	.02	.18	.42	1.55	CH	Kirkwood
216(3)	9A	50.25	58.4	1.53	2.62	86	40	.02	.20	.50	1.80	OH	Kirkwood
232(3)	6A	25.3	52.3	1.42	2.65	69	31	.02	.17	.42	1.30	CH	Fill
201(3)	10D	46.5	66.5	1.80	2.71	107	36	.04	.28	.79	2.10	CH	Kirkwood
201(3)	12D	57.75	20.4	0.59	2.78	26	-	.01	.11	.17	8.0	CL/ML	Kirkwood
AB1(4)	10	30.0	64.0	1.60	2.50	93	35	.02	.24	.62	0.75	CH	Fill
AB4(4)	5	13.0	86.0	2.18	2.54	72	32	.02	.22	.70	0.25	CH	Fill
AB5(4)	3	6.0	57.0	1.45	2.54	66	29	.02	.17	.42	0.43	CH	Fill
AB1(4)	14	40.0	32.0	0.85	2.66	57	21	.02	.18	.33	2.50	CH	Kirkwood

(1) Recompression ratio = $\Delta\epsilon/\log(p_2/p_1)$ with ϵ = strain at load p in recompression

(2) Compression ratio = $\Delta\epsilon\log(p_2/p_1)$ in compression

(3) Reference 2.5-57

(4) Reference 2.5-96

TABLE 2.5-8

COEFFICIENT OF CONSOLIDATION, C
(cm^2/sec)

Boring Sample, Depth (ft)	Load Increment (psf) From	Load Increment (psf) To	Based on Original Specimen Height Log 2t Method	Based on Original Specimen Height $\sqrt{t}$ Method	Based on Actual Specimen Height Log t Method	Based on Actual Specimen Height $\sqrt{t}$ Method
1/UD-10,	6,400	12,800	0.0041	0.0065	0.0035	0.0056
40 to 42	25,600	51,200	0.0015	0.0024	0.0010	0.0018
2UD-10,	6,400	12,800	-	0.0091	-	0.0079
40 to 42	25,600	51,200	-	0.0228	-	0.0202
2/UD-12,	6,400	12,800	-	0.0076	-	0.0068
45 to 47	25,600	51,200	0.0024	0.0030	0.0015	0.0018
3/UD-2,	1,600	3,200	0.0028	0.0033	0.0026	0.0030
5 to 7.5	6,400	12,800	0.0024	0.0046	0.0021	0.0039
	25,600	51,200	0.0026	0.0046	0.0020	0.0036
3/UD-13,	6,400	12,800	0.0084	0.0076	0.0072	0.0066
45 to 47	25,600	51,200	0.0031	0.0038	0.0019	0.0024
3/UD-17	3,200	6,400	-	0.0228	-	0.0216
60 to 62	6,400	12,800	0.0041	0.0051	0.0036	0.0045
	12,800	25,600	0.0027	0.0033	0.0020	0.0025
	25,600	51,200	0.0024	0.0033	0.0015	0.0021

Boring Sample, Depth (ft)	Load Increment (psf) From	To	Based on Original Specimen Height Log t Method	$\sqrt{t}$ Method	Based on Actual Specimen Height Log t Method	$\sqrt{t}$ Method
4/UD-4	400	800	-	0.0033	-	0.0030
19 to 21	800	1,600	0.0004	0.0011	0.0004	0.0010
	1,600	3,200	0.0003	0.0005	0.0003	0.0004
	3,200	6,400	0.0005	0.0006	0.0003	0.0004
4/UD-14	6,400	12,800	-	0.0114	-	0.0102
50 to 52	12,800	25,600	0.0041	0.0057	0.0032	0.0045
	25,600	51,200	0.0031	0.0038	0.0021	0.0025
	51,200	102,400	0.0021	0.0033	0.0011	0.0017
5/UD-7	1,600	3,200	0.0005	0.0011	0.0004	0.0008
25 to 27	3,200	6,400	0.0008	0.0011	0.0006	0.0008

TABLE 2.5-9

RESULTS OF UNCONFINED COMPRESSION AND UNCONSOLIDATED UNDRAINED TESTS

Type of Test	Boring No.	Sample No.	Depth (ft)	Material	Water Content (%)	Cell Pressure (psf)	Maximum Shear Strength (psf)
UU(1)	201	S-10C	46.2	Kirkwood clay	37.1	4860	1120
UU(1)	201	S-12C	57.5	Kirkwood clay	19.0	6200	1100
UU(1)	211	1A	30-31.5	Hydraulic fill	59.9	1512	881
UU(1)	211	5	26.0	Hydraulic fill	62.3	1008	682
UU(1)	211	6A	55-56.5	Kirkwood clay	51.8	2308	1707
UC(1)	216	3	20	Hydraulic fill	52.4	-	525
UC(1)	216	8	45	Kirkwood clay	59.8	-	1225
UU(1)	253	4	20	Hydraulic fill	64.1	2016	150
UC(2)	B 1	8	43-45	Kirkwood clay	35.5	-	2157
UC(2)	B 2	8	46-48	Kirkwood clay	50.5	-	1340
UU(3)	SS 2	2	4	Hydraulic fill	41.0	1000	530
UU(3)	SS 3	1	2	Hydraulic fill	36.3	1000	1120
UU(3)	SS 3	3	6	Hydraulic fill	75.2	1000	490
UU(3)	SS 3	9	18	Hydraulic fill	82.4	1000	480
UU(3)	SS 4	13	24	Hydraulic fill	69.7	1250	670
UU(3)	SS 1	13	25	Hydraulic fill	52.1	1250	770
UU(3)	SS 3	13	26	Hydraulic fill	64.4	1250	525
UU(3)	SS 2	13	26	Hydraulic fill	66.9	1250	700
UU(3)	SS 4	15	28	Hydraulic fill	58.4	1500	625
UU(3)	SS 3	15	30	Hydraulic fill	66.0	1500	600
UU(3)	SS 4	17	32	Hydraulic fill	72.2	1750	850
UU(3)	SS 2	16	32	Hydraulic fill	58.7	1750	900
UU(4)	AB 1	10	30	Hydraulic fill	65.0	2016	598
UU(4)	AB 1A	7	19	Hydraulic fill	52.0	1440	624
UU(4)	AB 2	9	22	Hydraulic fill	61.0	1584	643
UU(4)	AB 2	11	27	Hydraulic fill	60.0	2000	550
UU(4)	AB 3	3	8	Hydraulic fill	71.0	605	583
UU(4)	AB 5	3	6	Hydraulic fill	47.0	504	390
UU(4)	AB 5	7	16	Hydraulic fill	43.0	1195	542
UU(4)	AB 1	24	65	Kirkwood clay	42.0	5040	2670
UU(4)	AB 2	15	37	Kirkwood clay	35.0	3024	1208
UU(4)	AB 5	19	46	Kirkwood clay	40.0	4032	1019

(1) Reference 2.5-57
(2) Reference 2.5-98
(3) Reference 2.5-94
(4) Reference 2.5-96

TABLE 2.5-10

RESULTS OF CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TESTS[1]

Boring No.	Sample No.	Depth (ft)	Material	Water Content (%)	Cell Pressure (psf)	Deviator Stress at Failure (ksf)	Pore Pressure at Failure (ksf)
213	15	80	V[2]	30.0	6,408	19.50	2.00
213	25	140	H[3]	30.1	10,000	15.60	-15.00
223	5	25	R[4]	52.8	3,000	2.50	1.50
223	5	25	R	52.8	5,000	2.90	1.20
223	5	25	R	52.8	9,000	2.90	0.90
232	6	25	F[5]	69.2	1,500	2.10	1.40
244	12	60	K[6]	39.5	9,000	8.00	5.40
253	2	10	F	32.1	1,000	2.00	0.45
253	2	10	F	32.1	1,500	3.70	0.52
253	2	20	F	32.1	3,000	5.70	1.40
253	4	20	F	56.8	2,000	1.55	1.00
253	4	20	F	56.8	4,000	1.50	2.00
253	4	20	F	56.8	6,000	2.20	1.30
254	15A	80	V	51.2	5,040	1.45	0.02
254	27	160	N[7]	19.7	13,968	24.50	3.60
256	13	70	V	24.7	4,032	5.00	1.40
256	26	160	N	21.7	11.952	24.00	1.50
AB 2	9	22	F	61.0	1,010	1.25	0.75
AB 5	7	16	F	64.0	1,500	1.45	1.15
AB 2	11	27	F	65.0	2,000	1.70	1.45
AB 1	24	65	K	40.0	3,500	5.70	2.00
AB 1	24	65	K	49.0	7,060	6.30	5.00
AB 3	17	43	K	54.0	10,510	8.40	6.10

(1) For tests on samples from borings AB 1, AB 2, AB 3, and AB 5, see Reference 2.5-96; for others see Reference 2.5-57.
(2) Vincentown sands
(3) Hornerstown sands
(4) River bottom sands
(5) Hydraulic fill
(6) Kirkwood clays
(7) Navesink sands

TABLE 2.5-11

RESULTS FROM RESONANT COLUMN TESTS ON SAND

Boring No.	Sample No.	Depth (ft)	Elev. (ft)	Natural (Final) Moisture Content (%)	Natural Dry Unit Weight (pcf)	Soil Type & Formation (1)	Confining Stress (psf)	Single Amplitude Shear Strain (inch/inch)	Shear Modulus G, (ksf)	Shear Wave Velocity ft/s	Damping Ratio D(%)	Identi-fication No.
								2.0×10^{-6}	2475.0	878.01	2.3	1
								4.0×10^{-6}	2439.0	871.50	2.6	2
							3312.0	8.1×10^{-6}	2421.0	868.38	2.7	3
								1.7×10^{-5}	2368.0	858.82	2.8	4
								3.6×10^{-5}	2280.0	842.72	3.1	5
								1.6×10^{-6}	2677.0	913.14	3.9	6
								3.8×10^{-6}	2603.0	900.43	4.0	7
206	14	85	+14.6	22.3	84.5	SM	5760.0	7.8×10^{-6}	2543.0	890.87	4.1	8
				(21.7)								
				(21.7)		(Vincentown)		1.6×10^{-5}	2512.0	884.55	4.6	9
								3.3×10^{-5}	2475.0	878.01	4.7	10
								1.4×10^{-6}	3579.0	1055.83	4.2	11
								2.9×10^{-6}	3494.0	1043.22	4.5	12
							8150.4	6.2×10^{-6}	3265.0	1008.45	5.2	13
								1.3×10^{-5}	3083.0	979.94	5.4	14
								2.9×10^{-5}	2887.0	948.28	5.9	15
								3.0×10^{-6}	1595.0	651.08	3.0	16
								6.1×10^{-6}	1581.0	648.22	3.5	17
							2736.0	1.2×10^{-5}	1574.0	646.78	3.7	18
								2.5×10^{-5}	1568.0	645.54	4.1	19
								5.2×10^{-5}	1541.0	639.96	4.2	20
								2.3×10^{-6}	2167.0	758.90	3.3	21
								4.6×10^{-6}	2167.0	758.90	3.4	22
232	13	60	+47.4	23.7	97.9	SM	5040.0	9.2×10^{-6}	2152.0	756.27	3.4	23
				(16.9)								
				(16.9)		(River Bottom)		1.8×10^{-5}	2152.0	756.27	3.6	24
								3.9×10^{-5}	2136.0	753.45	3.7	25

Boring No.	Sample No.	Depth (ft)	Elev. (ft)	Natural (Final) Moisture Content (%)	Natural Dry Unit Weight (pcf)	Soil Type & Formation (1)	Confining Stress (psf)	Single Amplitude Shear Strain (inch/inch)	Shear Modulus G, (ksf)	Shear Wave Velocity ft/s	Damping Ratio D(%)	Identi-fication No.
								2.1×10^{-6}	2327.0	786.41	4.6	26
								4.3×10^{-6}	2294.0	780.82	5.8	27
							7488.0	8.9×10^{-6}	2230.0	769.85	5.5	28
								1.8×10^{-5}	2167.0	758.90	5.6	29
								3.1×10^{-5}	2136.0	753.45	5.3	30
								2.4×10^{-6}	2037.0	742.89	2.6	31
								4.8×10^{-6}	1978.0	732.05	3.0	32
							3312.0	9.8×10^{-6}	1949.0	726.66	3.0	33
								2.0×10^{-5}	1934.0	723.86	3.0	34
								4.2×10^{-5}	1905.0	718.42	3.1	35
								2.0×10^{-6}	2377.0	802.50	4.4	36
								4.2×10^{-6}	2329.0	794.35	5.3	37
232	15	75	+32.4	25.6	94.6	SM	5760.0	8.4×10^{-6}	2313.0	791.62	5.2	38
				(16.7)								
				(16.7)		(Vincentown)		1.7×10^{-5}	2282.0	786.30	5.2	39
								3.6×10^{-5}	2204.0	772.74	5.7	40
								1.4×10^{-6}	3612.0	989.24	3.9	41
								2.8×10^{-6}	3535.0	978.64	4.4	42
							8150.4	5.8×10^{-6}	3440.0	965.40	5.0	43
								1.1×10^{-5}	2982.0	898.84	6.4	44
								1.9×10^{-6}	2886.0	878.51	5.4	45
								3.9×10^{-6}	2804.0	865.94	4.5	46
							6710.4	8.2×10^{-6}	2643.0	840.71	4.6	47
								1.7×10^{-5}	2565.0	828.22	5.0	48
								2.8×10^{-5}	2526.0	821.90	4.2	49
								1.5×10^{-6}	3718.0	997.14	3.9	50
								3.0×10^{-6}	3695.0	994.05	4.5	51
232	25A	130	-22.6	27.6	94.4	SM	9216.0	6.1×10^{-6}	3649.0	987.84	4.6	52
				(18.0)								
				(18.0)		(Hornerstown)		1.4×10^{-5}	3032.0	900.46	5.1	53
								2.6×10^{-5}	2824.0	869.02	5.4	54

Boring No.	Sample No.	Depth (ft)	Elev. (ft)	Natural (Final) Moisture Content (%)	Natural Dry Unit Weight (pcf)	Soil Type & Formation (1)	Confining Stress (psf)	Single Amplitude Shear Strain (inch/inch)	Shear Modulus G, (ksf)	Shear Wave Velocity ft/s	Damping Ratio D(%)	Identi-fication No.
								1.2 X 10^{-6}	4622.0	1111.77	4.5	55
								2.4 X 10^{-6}	4596.0	1108.64	5.0	56
							11,606.4	4.9 X 10^{-6}	4520.0	1099.43	4.9	57
								1.0 X 10^{-5}	4419.0	1087.08	5.1	58
								1.8 X 10^{-5}	4220.0	1062.32	4.9	59
								1.5 X 10^{-6}	3458.0	946.05	4.2	60
								3.0 X 10^{-6}	3419.0	940.70	4.9	61
							6710.4	6.4 X 10^{-6}	3245.0	916.45	5.2	62
								1.2 X 10^{-5}	2892.0	865.17	7.0	63
								1.4 X 10^{-6}	3678.0	975.68	5.0	64
232	25B	130	-22.6	25.1	99.4	SM	7286.4	2.9 X 10^{-6}	3617.0	967.55	6.1	65
				(26.1)								
				(26.1)		(Hornerstown)		6.0 X 10^{-6}	3439.0	943.45	5.8	66
								1.1 X 10^{-5}	3132.0	900.35	5.7	67
								9.2 X 10^{-7}	5789.0	1224.06	3.2	68
								1.8 X 10^{-6}	5789.0	1224.06	3.8	69
							12,168.0	3.7 X 10^{-6}	5689.0	1213.44	3.8	70
								7.4 X 10^{-6}	5689.0	1213.44	4.0	71
								1.6 X 10^{-5}	5540.0	1197.45	4.5	72
254	25	140.0	-39.0	17.2	110.0	SM	7300.0	3.3 X 10^{-8}	5210.0		3.4	73
				(17.8)								
				(17.8)		(Hornerstown)		9.07 X 10^{-7}	5190.0		3.5	74
								1.20 X 10^{-6}	5170.0	1245	3.6	75
								2.71 X 10^{-6}	5080.0		3.7	76
								4.14 X 10^{-6}	5040.0		3.8	77
254	25	140.0	-39.0	17.6	115.8	SM	13,600.0	2.96 X 10^{-7}	6750.0		3.9	78
				(18.0)								
				(18.0)		(Hornerstown)		4.44 X 10^{-7}	6750.0		3.7	79
								8.54 X 10^{-7}	6720.0	1383	3.7	80
								1.30 X 10^{-6}	6700.0		3.8	81
								3.00 X 10^{-6}	6650.0		3.8	82

TABLE 2.5-11 (Cont)

Boring No.	Sample No.	Depth (ft)	Elev. (ft)	Natural (Final) Moisture Content (%)	Natural Dry Unit Weight (pcf)	Soil Type & Formation (1)	Confining Stress (psf)	Single Amplitude Shear Strain (inch/inch)	Shear Modulus G, (ksf)	Shear Wave Velocity ft/s	Damping Ratio D(%)	Identi-fication No.
254	25	140.0	-39.0	17.5	116.0	SM	14,400.0	3.41×10^{-7}	7320.0		3.1	83
				(18.6)								
				(18.6)		(Hornerstown)		6.16×10^{-7}	7290.0		3.2	84
								8.24×10^{-7}	7270.0	1436	3.3	85
								1.96×10^{-6}	7240.0		3.4	86
								2.82×10^{-6}	7170.0		3.5	87
255	18A	87	+14.7	29.8	91.7	SM	2016.0	1.7×10^{-6}	1600.0		1.1	88
				(30.1)								
				(30.1)		(Vincentown)		2.5×10^{-6}	1600.0		1.2	89
								4.0×10^{-6}	1600.0	750.0	1.2	90
								8.0×10^{-6}	1580.0		1.6	91
								1.1×10^{-5}	1540.0		1.9	92
256	21	115	-15.8	33.7	82.2	SM	6710.4	3.45×10^{-7}	3650.0		2.2	93
				(36.0)								
				(36.0)		(Hornerstown)		4.44×10^{-7}	3650.0		1.9	94
								1.56×10^{-6}	3650.0	1156.0	2.0	95
								2.71×10^{-6}	3650.0		2.0	96
								4.36×10^{-6}	3660.0		2.1	97
256	21	115	-15.8	31.5	91.1	SM	9216.0	3.69×10^{-7}	5030.0		1.4	98
				(31.7)								
				(31.7)		(Hornerstown)		4.81×10^{-7}	5030.0		1.3	99
								1.25×10^{-6}	5030.0	1336.0	1.6	100
								2.64×10^{-6}	5030.0		1.3	101
								4.51×10^{-6}	5010.0		1.2	102
256	21	115	-15.8	26.2	92.3	SM	11,606.4	3.95×10^{-7}	6070		0.9	103
				(30.0)								
				(30.0)		(Hornerstown)		5.03×10^{-7}	6070		1.3	104
								1.44×10^{-6}	5950		1.1	105
								3.13×10^{-6}	5920	1457	1.2	106
								1.18×10^{-5}	5810		1.4	107

(1) Soil types are in accordance with Unified Soil Classification System.

TABLE 2.5-12

RESULTS FROM RESONANT COLUMN TESTS ON CLAY

Boring No.	Sample No.	Depth (ft)	Elev (ft)	Natural (Final) Moisture Content (%)	Natural Dry Unit Weight (pcf)	Soil Type & Formation (1)	Confining Stress (psf)	Single Amplitude Shear Strain (inch/inch)	Shear Modulus G, (ksf)	Damping Ratio D(%)	Identi-fication No.
244	10A	50-52	+51-+49	56.6	66.5	MH	2016.0	6.3×10^{-6}	857	2.5	145
				(57.1)		(Kirkwood)		1.4×10^{-5}	857	2.6	146
								2.2×10^{-5}	840	2.6	147
								4.3×10^{-5}	824	2.9	148
								4.7×10^{-5}	808	2.6	149
244	10B	50-52	+51-+49	56.4	68.4	CH	7920.0	4.9×10^{-6}	1890		
	2.4		150								
				(53.0)		(Kirkwood)		1.5×10^{-5}	1860	2.3	151
								2.3×10^{-5}	1860	2.3	152
								2.9×10^{-5}	1860	2.3	153
								3.2×10^{-5}	1840	2.4	154
244	10C	50-52	+51-+49	67.9	58.6	MH	14,976.0	4.2×10^{-6}	1900	2.2	155
				(59.1)		(Kirkwood)		9.0×10^{-6}	1890	2.2	156
								1.3×10^{-5}	1870	2.5	157
								1.6×10^{-5}	1870	2.5	158
								2.7×10^{-5}	1890	2.2	159
254	8A	45-47	+56-+54	61.0	62.3	MH	2016.0	1.0×10^{-5}	562	2.6	160
				(60.9)		(Kirkwood)		1.8×10^{-5}	562	2.7	161
								2.7×10^{-5}	562	2.8	162
								3.4×10^{-5}	562	2.8	163
								6.1×10^{-5}	530	3.2	164
254	8B	45-47	+56-+54	52.9	68.8	CH	7920.0	8.9×10^{-7}	1480	3.1	165
				(47.7)		(Kirkwood)		1.2×10^{-6}	1480	3.2	166
								1.5×10^{-6}	1480	3.1	167
								4.0×10^{-6}	1480	3.3	168
								1.8×10^{-5}	1480	3.2	169
254	8C	45-47	+56-+54	54.4	67.8	CH	14,976.0	8.8×10^{-7}	2580	3.5	170
				(41.6)		(Kirkwood)		1.1×10^{-6}	2620	3.6	171
								1.8×10^{-6}	2580	3.6	172
								2.9×10^{-6}	2580	3.7	173
								4.8×10^{-6}	2580	3.6	174

Boring No.	Sample No.	Depth (ft)	Elev (ft)	Natural (Final) Moisture Content (%)	Natural Dry Unit Weight (pcf)	Soil Type & Formation (1)	Confining Stress (psf)	Single Amplitude Shear Strain (inch/inch)	Shear Modulus G, (ksf)	Damping Ratio D(%)	Identi-fication No.
255	9A	40	+61.7	46.7 (47.8)	73.6	MH (Kirkwood)	2016.0	9.90×10^{-7}	921.0	3.1	175
								2.64×10^{-6}	921.0	2.7	176
								5.53×10^{-6}	916.0	3.0	177
								8.62×10^{-6}	916.0	3.1	178
								1.15×10^{-5}	916.0	3.1	179
255	9B	40	+61.7	38.3 (32.7)	82.1	CH (Kirkwood)	14,976.0	2.96×10^{-7}	3320.0	3.4	180
								1.10×10^{-6}	3350.0	3.5	181
								1.75×10^{-6}	3350.0	3.5	182
								2.75×10^{-6}	3350.0	3.5	183
								3.47×10^{-6}	3370.0	3.5	184
232	3	10	+97.4	26.0 (25.7)	95.7	CL (Fill)	460.8	6.88×10^{-6}	581.0	3.4	185
								1.39×10^{-5}	571.0	3.7	186
								2.85×10^{-5}	550.0	3.7	187
								5.78×10^{-5}	540.0	3.9	188
								1.28×10^{-4}	488.0	4.0	189
232	7	30	+77.4	51.4 (50.0)	66.7	CL (Fill)	1368.0	7.92×10^{-6}	425.0	1.5	190
								1.60×10^{-5}	416.0	1.6	191
								3.23×10^{-5}	411.0	1.5	192
								6.50×10^{-5}	407.0	1.6	193
								1.41×10^{-4}	380.0	1.8	194

(1) Soil types are in accordance with Unified Soil Classification System.

TABLE 2.5-13

RESULTS FROM DYNAMIC STRAIN CONTROLLED TESTS ON SAND

Boring No.	Sample No.	Depth (ft)	Elev. (ft)	Natural (Final) Moisture Content (%)	Natural Dry Unit Weight (pcf)	Soil Type & Formation (1)	Confining Stress (psf)	Single Amplitude Shear Strain (inch/inch)	Shear Modulus G, (ksf)	Shear Wave Velocity ft/s	Damping Ratio D(%)	Identi-fication No.
201	22A	140.4	-40.5	26.2	99.6	SM	7344.0	4.45×10^{-4}	2190.0		6.5	108
				(24.3)								
				(24.3)		(Hornerstown)		9.5×10^{-4}	1480.0		10.3	109
								3.0×10^{-3}	510.0		18.4	110
								9.58×10^{-3}	210.0		13.8	111
201	22C	141.1	-41.2	25.2	101.7	SM	12,168.0	4.63×10^{-4}	2970.0		4.9	112
				(23.4)								
				(23.4)		(Hornerstown)		9.16×10^{-4}	2450.0		7.8	113
								2.92×10^{-3}	800.0		17.4	114
								9.37×10^{-3}	260.0		14.8	115
206	10B	61.0	+38.6	21.9	106.0	ML	2736.0	4.59×10^{-4}	870.0		10.5	120
				(20.2)								
				(20.2)		(Basal Sand)		9.54×10^{-4}	610.0		13.9	121
								2.94×10^{-3}	240.0		18.1	122
								9.45×10^{-3}	70.0		16.0	123
206	10C	61.6	+38.0	23.1	103.6	SM	6710.0	4.64×10^{-4}	1540.0		7.9	124
				(21.0)								
				(21.0)		(Basal Sand)		9.51×10^{-4}	1190.0		9.6	125
								2.97×10^{-3}	420.0		17.5	126
								2.97×10^{-3}	430.0		18.0	127
								9.54×10^{-3}	100.0		16.2	128
206	22A	130.3	-30.7	26.3	97.9	SM	11,606.0	4.44×10^{-4}	3230.0		4.2	129
				(25.1)								
				(25.1)		(Hornerstown)		9.24×10^{-4}	2620.0		7.4	130
								2.96×10^{-3}	1220.0		15.0	131
								9.40×10^{-3}	350.0		15.5	132
206	22C	131.6	-32.0	26.0	98.7	SM	6710.0	4.58×10^{-4}	2330.0		5.8	133
				(21.8)								
				(21.8)		(Hornerstown)		9.32×10^{-4}	1760.0		9.5	134
								2.97×10^{-3}	740.0		15.3	135
								9.43×10^{-3}	300.0		11.3	136

TABLE 2.5-13 (Cont)

Boring No.	Sample No.	Depth (ft)	Elev. (ft)	Natural (Final) Moisture Content (%)	Natural Dry Unit Weight (pcf)	Soil Type & Formation (1)	Confining Stress (psf)	Single Amplitude Shear Strain (inch/inch)	Shear Modulus G, (ksf)	Shear Wave Velocity ft/s	Damping Ratio D(%)	Identification No.
216	14A	75.5	+26.2	34.8	86.4	SM	8150.0	4.56 X 10^{-4}	2430.0		5.1	137
				(30.1)								
				(30.1)		(Vincentown)		9.32 X 10^{-4}	1710.0		8.2	138
								2.95 X 10^{-3}	480.0		14.9	139
								9.42 X 10^{-3}	100.0		12.0	140
216	14B	76.1	+25.6	29.9	92.9	SM	3312.0	4.41 X 10^{-4}	1350.0		7.7	141
				(28.4)								
				(28.4)		(Vincentown)		8.74 X 10^{-4}	890.0		12.4	142
								2.99 X 10^{-3}	248.0		16.8	143
								9.21 X 10^{-3}	80.0		12.0	144

(1) Soil types are in accordance with Unified Soil Classification System.

TABLE 2.5-14

RESULTS FROM DYNAMIC STRAIN CONTROLLED TESTS ON CLAY

Boring No.	Sample No.	Depth (ft)	Elev (ft)	Natural (Final) Moisture Content (%)	Natural Dry Unit Weight (pcf)	Soil Type & Formation (1)	Confining Stress (psf)	Single Amplitude Shear Strain (inch/inch)	Shear Modulus G, (ksf)	Damping Ratio D(%)	Identification No.
201	10B	45.6	+54.3	53.9 (51.8)	68.9	CH (Kirkwood)	2059.2	5.13×10^{-4}	418	7.0	195
								1.05×10^{-3}	389	10.1	196
								3.32×10^{-3}	187	14.6	197
								1.06×10^{-2}	84.9	16.8	198
201	12B	55.9	+40.0	17.7 (16.9)	114.6	ML (Kirkwood)	2577.6	5.06×10^{-4}	864	10.5	199
								1.08×10^{-3}	562	13.9	200
								3.29×10^{-3}	245	18.7	201
								1.05×10^{-2}	89.3	15.5	202
206	10A	60.4	+39.2	20.4 (18.8)	108.3	CL-ML (Kirkwood)	2736.0	5.05×10^{-4}	821	9.5	203
								1.04×10^{-3}	547	13.4	204
								3.30×10^{-3}	201	17.7	205
								1.04×10^{-2}	59	14.8	206

TABLE 2.5-15

SUMMARY OF COMPRESSION AND SHEAR WAVE VELOCITIES FROM GEOPHYSICAL SURVEYS[1]

(Velocities in ft/sec)

	Refraction	Uphole	LDIL[2] Downhole	Surface Wave Downhole	Design Velocities	Poisson's Ratio
V_{1p}	1800-1500	2500	2000	-	2000	.49
V_{1s}	-	-	210	-	210	
V_{2p}	4600	4200	4600	4400	4600	.40
V_{2s}	-	-	-	-	1850	
V_{3p}	6800	6250	6700	6400-6800	6800	.44
V_{3s}	-	1850	2250	-	2250	

(1) Reference 2.5-57

(2) Long-distance-in-line

(3) V_{np} = propagation velocity of compressional stress waves in nth soil layer

(4) V_{ns} = propagation velocity of shear stress waves in nth soil layer

TABLE 2.5-16

DYNAMIC SUBSURFACE MODEL
BORING 201

Depth (ft)	Thickness (ft)	Soil Type[1]	Total Unit Wt (pcf)	Poisson's Ratio	G (psf) (Low strain)	A (sand)	n	Curve Number for K Value (Fig. 10)
0-3	3	SM (fill)	119	0.48	0.1×10^6 (approx)	840	1	1 G.W.L.
3-22	19	CL (fill)	96	0.48	0.3×10^6	-	-	3
22-31	9	ML-CL (fill)	100	0.48	0.5×10^6	-	-	3
31-35	4	SP (River Bottom)	129	0.44	4.5×10^6 (approx)	4760	1	2
35-40	5	ML (River Bottom)	120	0.44	5.3×10^6 (approx)	4760	1	4
40-55	15	CH (Kirkwood)	105	0.44	7.6×10^6	-	-	4
55-60	5	ML (Kirkwood)	123	0.44	5.5×10^6 (approx)	3157	-	4
60-65	5	SM-GW (Basal Sand)	131	0.43	6.0×10^6 (approx)	3157	1	2
65-200	135	SM (Vincentown) (Hornerstown)	115.8	0.43	10.0×10^6 (approx)	3157	1	2
200 (halfspace assumed below this depth)		SM (Hornerstown) (sand)	115.8	0.43	$20. \times 10^6$ (approx at 200 ft)	3157	1	2

(1) Soil types are in accordance with Unified Soil Classification System.

TABLE 2.5-17

DYNAMIC SUBSURFACE MODEL
BORING 229

Depth (ft)	Thickness (ft)	Soil Type[1]	Total Unit Wt (pcf)	Poisson's Ratio	G (psf) (low strain)	A (sands)	n	Curve Number for K Value (Fig. 10)
0-13	13	ML (fill)	110	.48	0.2×10^6 (approx)	400	1	1 G.W.L.
13-23	10	SM (fill)	119	.48	0.44×10^6 (approx)	400	1	1
23-43	20	CL (fill)	96	.48	0.8×10^6	-	-	3
43-45	2	GM (River Bottom)	130	.44	8×10^6	(constant)	-	2
45-50	5	CH (Kirkwood)	105	.44	8.5×10^6	-	-	4
50-68	18	SW (Basal SM sands)	131	.44	5.0×10^6 (approx)	2100	1	2
68-200	132	SM (Vincentown) (Hornerstown)	115.8	.43	10.0×10^6 (approx)	2100	1	2
200 (half-space assumed below this depth)		SM (Mt Laurel or Hornerstown sands)	115.8	.43	15.0×10^6 (approx at 200 ft)	2100	1	2

(1) Soil types are in accordance with Unified Soil Classification System.

TABLE 2.5-18

FOUNDATION DESIGN DATA

Structure	Approximate Plan Dimension (ft)	Elevation of Bottom of Mat (ft)	Total Design Load[(1)] (ksf)	Approximate Elevation Top of Vincentown
Excavation	650 x 530	30	-	36
Reactor Containment Building	192.5 x 312	40	4.3	36
Auxiliary Building	165 x 312	40	3.1	36
Turbine Generator	364 x 195	40	2.7	36
Intake Structure	114 x 104	65.5	5.8	23 to 29
Cancelled Unit Area	192.5 x 312	40	2.0	36
Administration Facility	265.5 x 195	40	1.3	36

(1) Total design load is defined as the average contact pressure net of buoyancy.

TABLE 2.5-19

LATERAL FORCES DURING EARTHQUAKE EXCITATION AND FACTOR OF SAFETY AGAINST SLIDING FOR INTAKE AND POWER BLOCK STRUCTURES

Condition	Structure	Total Load[1] (kips/ft)	Internal Force (kips/ft)[2] I Inertia Force	P_1	P_2	P_3	P_4	P_5	P_6	P_7	P_8	Factor of Safety
No liquefaction	Intake	310.0	62.0	41.6	11.7	21.1	-17.1	4.0				1.3
Liquefied hydraulic fill	Intake	310.0	62.0	93.3	15.2					-17.1	4.0	1.1
	Power block[3]	3655.4	731.1	(5)	12.5	(5)	22.8	(5)	15.1	(5)	17.8	1.3
	Power block[4]	3655.4	731.1	76.2	12.5	26.2	22.8	134.0	15.1	-67.2	15.7	1.4

(1) Total loads are based on Bechtel's Drawing BK-C-622 Rev. A (12/16/74), and Table 1 contained in letter from Bechtel to PSE&G dated June 2, 1975 for Intake Structure and Power Block respectively.

(2) I and P's refer to Figures 2.5-58 and 2.5-59, and negative sign indicates an opposite direction.

(3) Assuming the liquefied hydraulic fill does not flow.

(4) Assuming the liquefied hydraulic fill flows and the water level on the river side is MLT.

(5) Forces that are present on both sides of the structure and therefore are in equilibrium have been omitted.

(6) Factor of safety is defined as the ratio of shearing strength to shearing stress along the postulated sliding surface.

TABLE 2.5-20

LATERAL FORCES DURING EARTHQUAKE EXCITATION AND FACTOR OF SAFETY AGAINST SLIDING FOR PIPELINE

				Internal Forces (kips/ft)[2]					
Condition	Height[1] (ft)	Base[1] Width (ft)	Total Load (kips/ft)	I Inertia Force	P_1	P_2	P_3	P_4	Factor of Safety[3]
No Flow	25	25	87.5	17.5	43.8	7.2	-43.8	10.2	1.1
Flow	25	25	87.5	17.5	43.8	7.2	- 9.4	2.2	0.6

(1) The selection of height and base width is for illustrative purpose only.

(2) I and P's refer to Figure 2.5-57 and negative sign indicates an opposite direction.

(3) Factor of Safety is defined as the ratio of shearing strength to shearing stress along the postulated sliding surface.

TABLE 2.5-21

EARTHQUAKES $\geq M_L = 4.0$ USED IN A 5° X 5° COMPARISON BETWEEN THE HOPE CREEK SITE AND MIRAMICHI, N.B.

Hope Creek

(See Table 2.5-1)

Miramichi, N.B.

Date	N. Lat.	W. Long.	Magnitude (M_L)
22 May 1817	46.0	69.0	5.0
09 Jul 1824	46.5	66.5	4.5
08 Feb 1855	46.0	64.5	4.5
22 Oct 1869	45.0	66.2	5.0
27 Feb 1874	44.8	68.7	4.0
31 Dec 1882	45.0	67.0	4.5
22 Mar 1896	45.2	67.2	4.0
21 Mar 1904	45.0	67.2	5.0
15 Jul 1905	44.3	69.8	4.5
14 May 1908	44.0	65.8	4.0
08 Aug 1908	46.3	67.6	4.5
11 Dec 1912	45.0	68.0	4.0
13 Jan 1914	45.1	67.2	4.5
27 Jul 1915	44.0	65.0	4.0
12 Jun 1917	49.0	68.0	4.0
02 Jul 1922	46.5	66.6	4.5
08 Feb 1928	45.3	69.0	4.5
04 Jan 1930	46.7	65.8	4.5
30 Sep 1937	45.5	65.8	4.5
17 May 1938	49.0	68.0	4.0
22 Aug 1938	44.7	68.8	4.0
23 Jun 1944	49.4	67.8	5.0
29 Jun 1950	49.5	67.4	4.5
28 Jun 1951	49.5	67.0	4.0
19 Sep 1951	49.3	66.3	4.5
24 Jan 1953	49.4	66.0	4.5
14 Sep 1953	49.4	65.3	4.5
21 Oct 1958	49.2	68.5	4.0
25 Mar 1962	47.5	66.0	4.0
14 Jan 1966	48.9	67.7	4.0
30 Sep 1967	49.3	65.9	4.5

TABLE 2.5-22

RECURRENCE PARAMETERS

Region	Area (mi^2)	$N/10^4 mi^2$				
		$M_L = \geq 4.0$	≥ 4.5	≥ 5.0	≥ 5.5	"b"
HCGS (5°x5°)	9.2×10^4	0.03	0.01	0.002	-	0.67
New Brunswick (5°x5°) (w/o Miramichi Events)	8.1×10^4	0.02	0.01	0.003	0.0007	0.85

TABLE 2.5-23

SEISMIC EVENTS WITHIN A 1° x 1° AREA
CENTERED ABOUT THE HCGS SITE AND THE MIRAMICHI
MAGNITUDE 5.7 EPICENTER

Magnitude	HCGS (1930-1980)	New Brunswick (1930-1981)
2.0 - 2.9	2	14
3.0 - 3.9	5	9
4.0 - 4.4	1	1
4.5 - 4.9	0	1
TOTAL	8	25

TABLE 2.5-24

EARTHQUAKES USED IN A 1° X 1° COMPARISON
BETWEEN THE HOPE CREEK SITE AND MIRAMICHI, NEW BRUNSWICK

Hope Creek

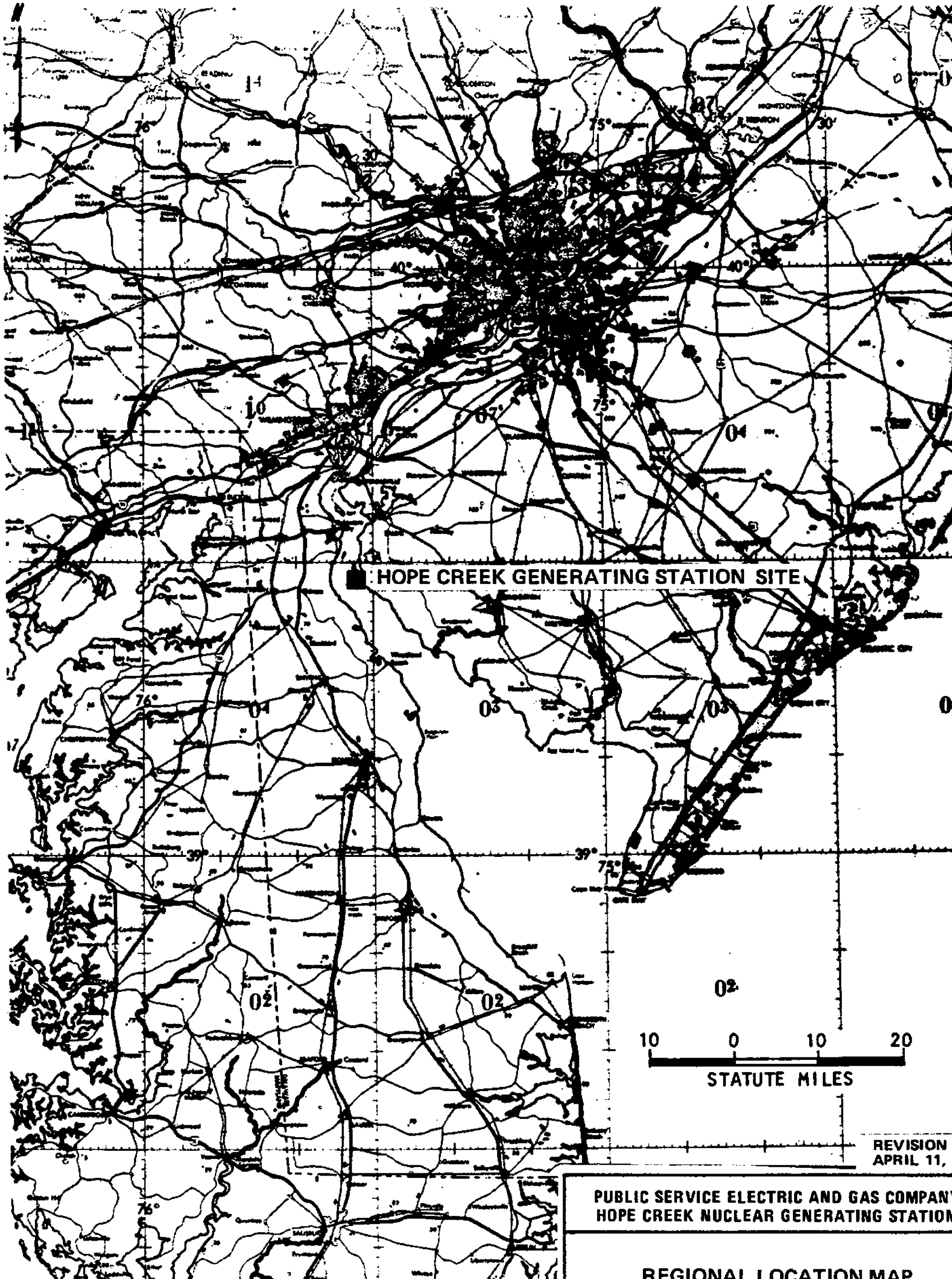
Date	North Lat.	West Long	Intensity	Magnitude (1)
15 Nov 1939	39.6	75.2	V	4.2
11 Aug 1954	40.3	76.0	IV	3.6
20 Jan 1955	40.3	76.0	IV	3.6
23 Jan 1962	39.8	75.9	I-II	2.5
11 Feb 1972	39.7	75.7	II	2.7
28 Feb 1973	37.7	75.4	VI	$3.8m_n$
10 Jul 1973	Near Wilmington		IV	3.7
02 May 1980	40.2	75.0	IV	3.7

Miramichi, New Brunswick

(14 events between M_L - 2.0-2.9 are taken from the Shearon-Harris SER, 1983), (Reference 2.5-151))

Date	Lat.	Long.	Magnitude (M_L)
04 Jan 1930	46.7	65.8	4.6
15 Jun 1938	46.5	66.8	3.3
04 Aug 1957	46.5	67.0	3.7
29 Jan 1961	46.3	66.9	3.8
31 Jan 1962	47.5	67.1	3.5
25 Mar 1962	47.5	66.0	4.0
01 Aug 1963	46.8	66.5	3.0
17 Oct 1964	47.6	67.2	3.9
27 May 1965	46.9	66.6	3.3
24 Oct 1977	47.0	67.0	3.0
28 Nov 1981	47.0	66.6	3.7

(1) Converted from MM Intensity of magnitude using $m_b = 1.75 + 0.5I_o$ (Nuttli and Herrmann, 1978, (Reference 2:5-152))



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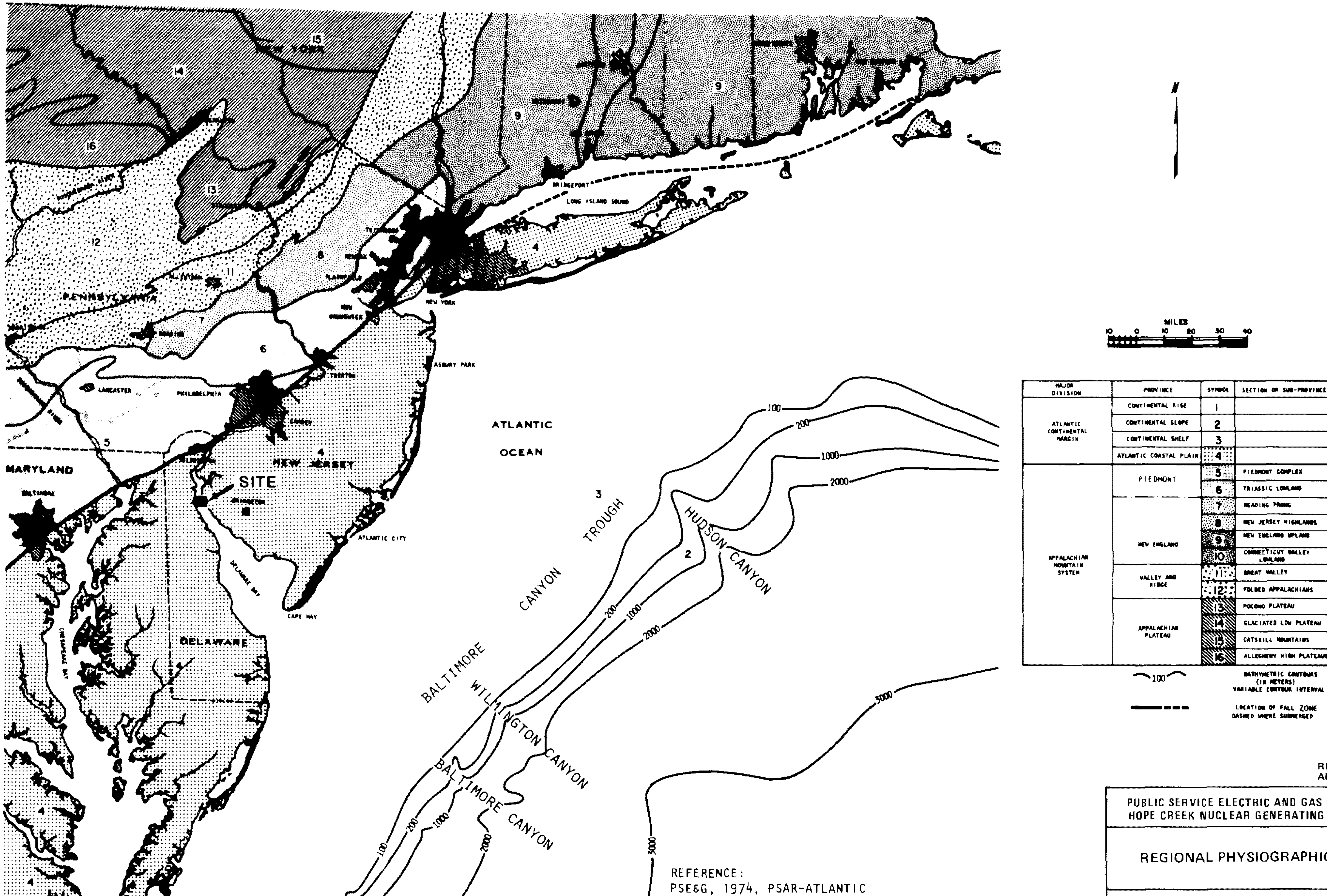
REFERENCE:
PSE&G, 1978, POST-EXCAVATION FOUNDATION STUDIES, HOPE CREEK GENERATING STATION, FIGURE 1-1.

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

REGIONAL LOCATION MAP

UPDATED FSAR

FIGURE 2.5-1



REFERENCE:
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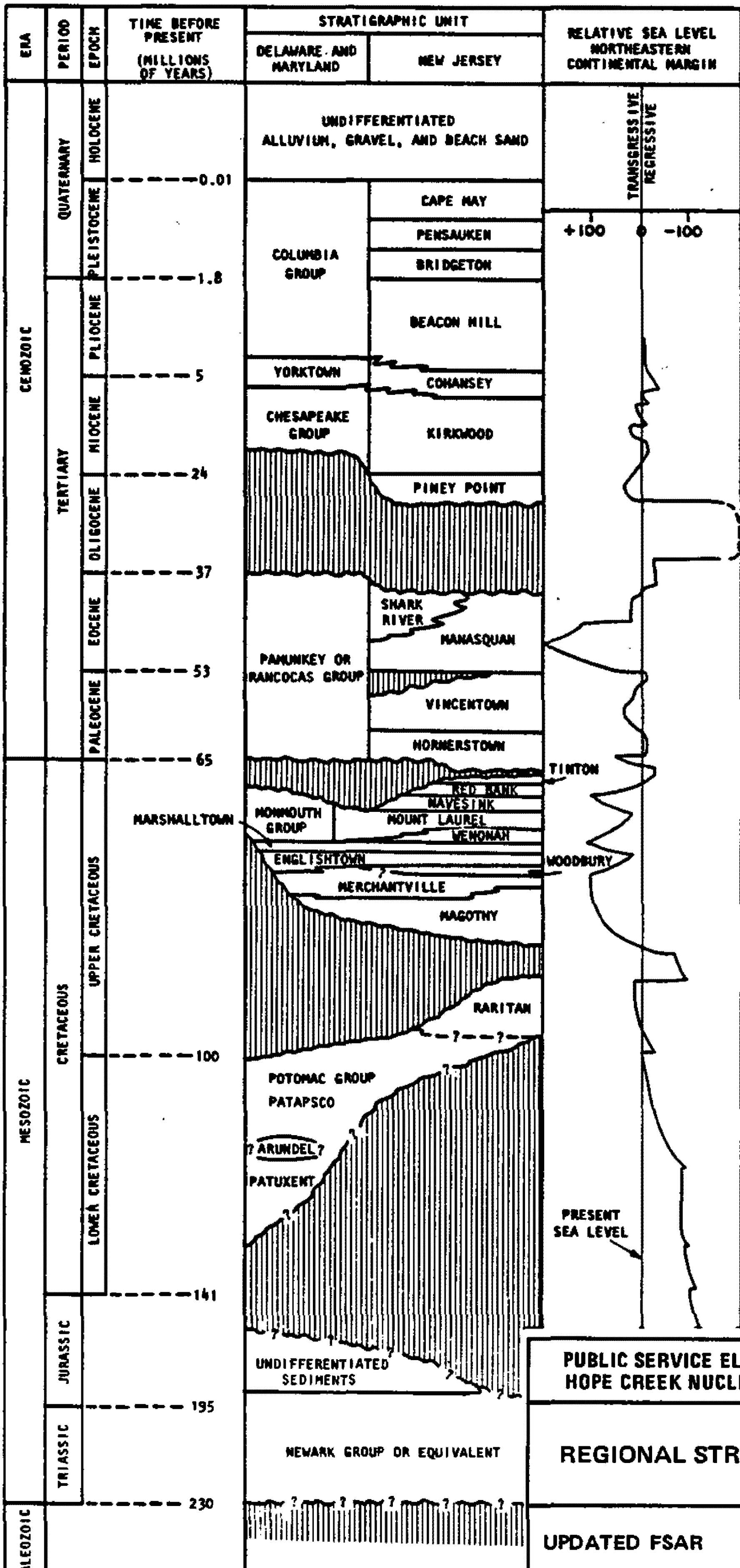
HOPE CREEK NUCLEAR GENERATING STATION

REGIONAL PHYSIOGRAPHIC MAP

UPDATED FSAR

FIGURE 2.5-2



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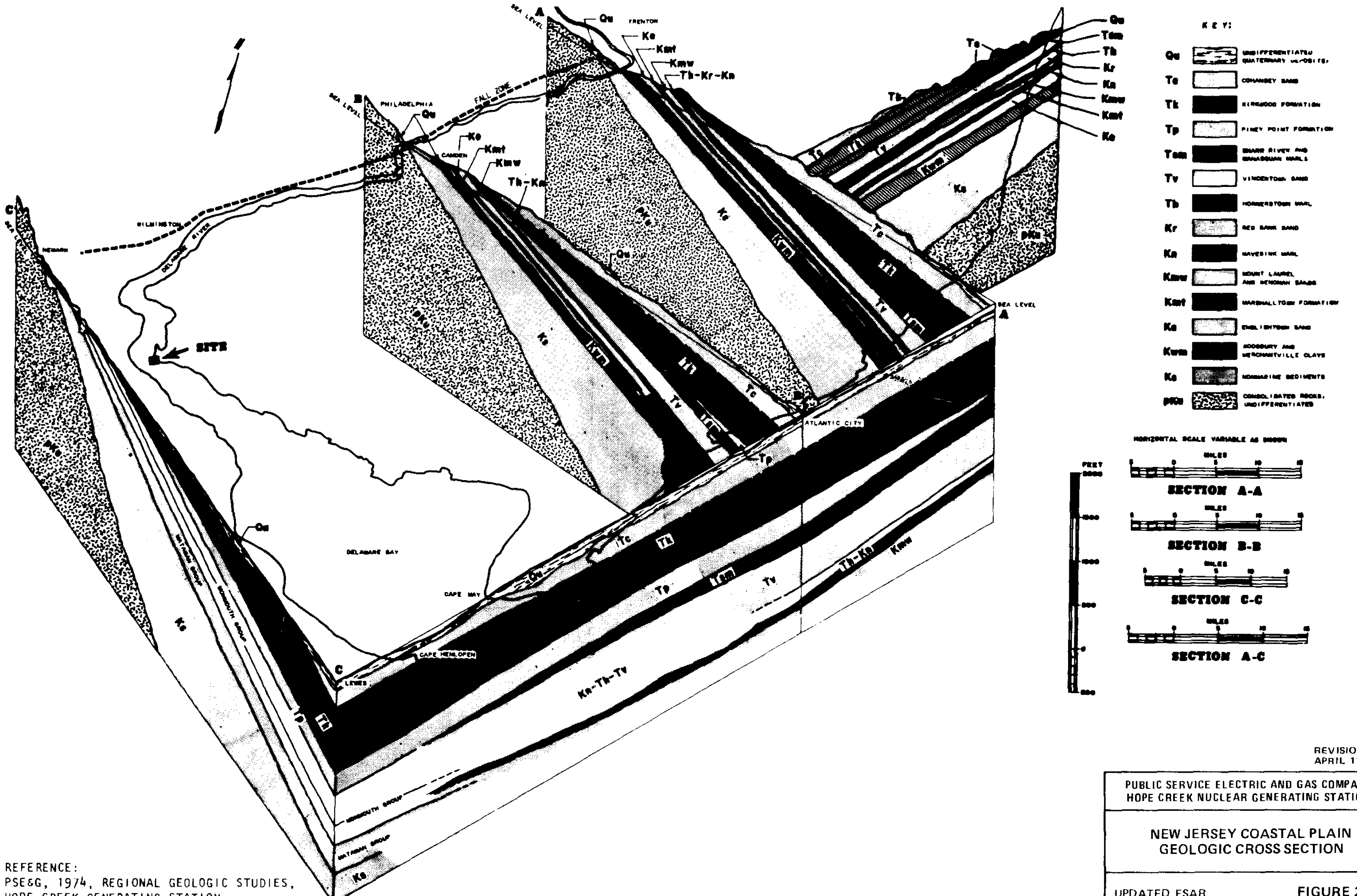
HOPE CREEK NUCLEAR GENERATING STATION

REGIONAL STRATIGRAPHIC COLUMN

UPDATED FSAR

FIGURE 2.5-3



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HOPE CREEK NUCLEAR GENERATING STATION

NEW JERSEY COASTAL PLAIN
GEOLOGIC CROSS SECTION

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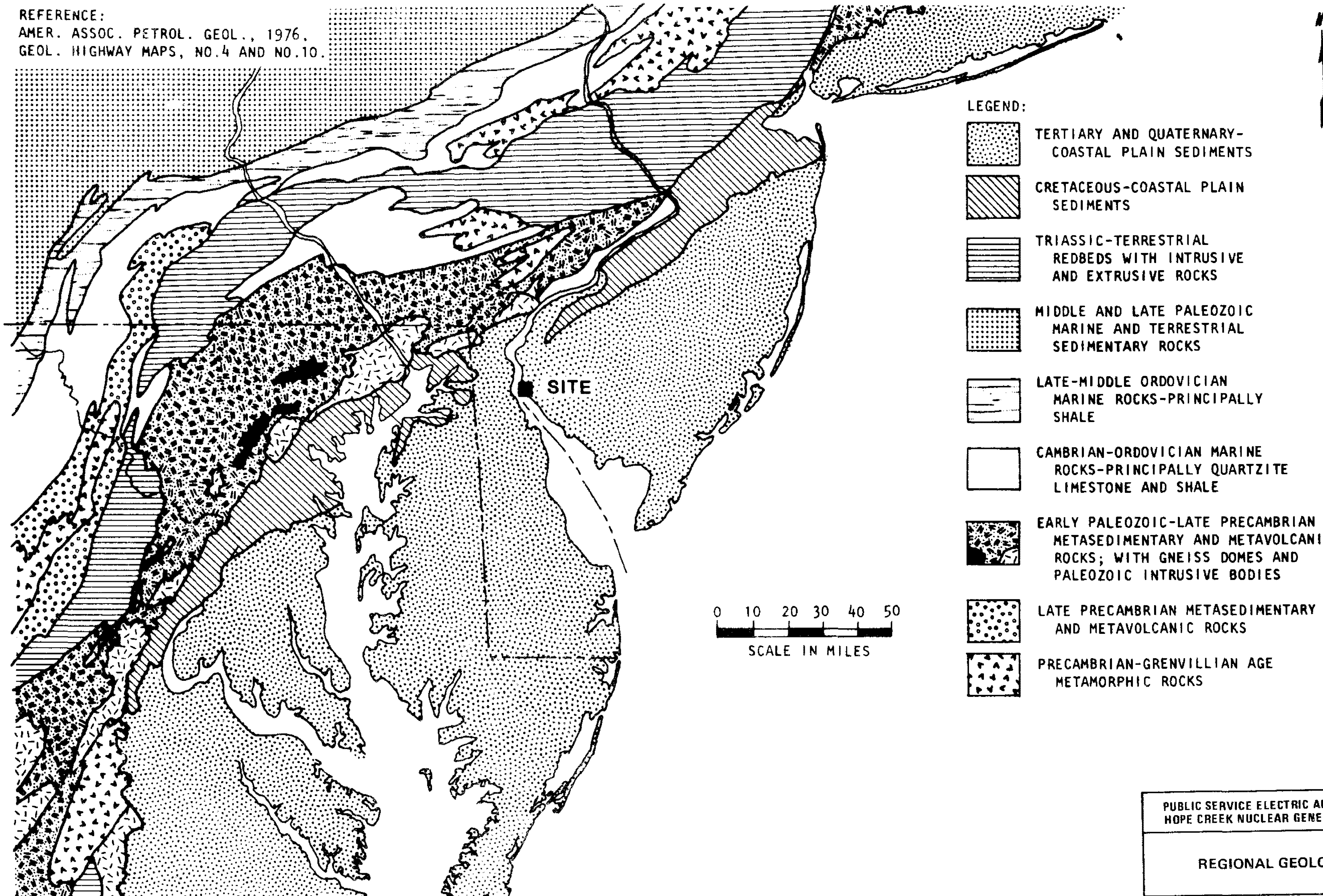
FIGURE 2.5-4



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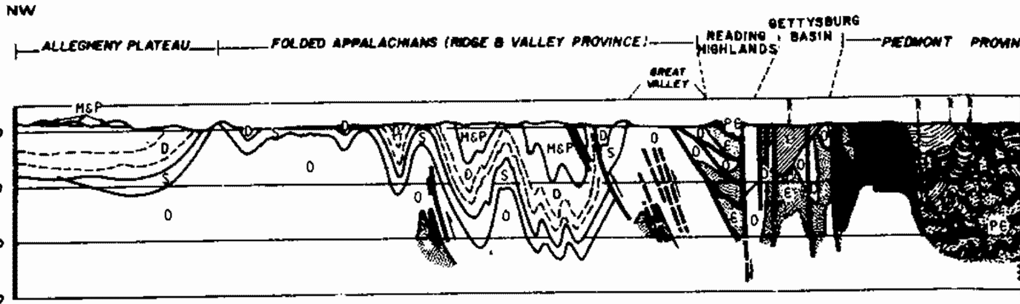
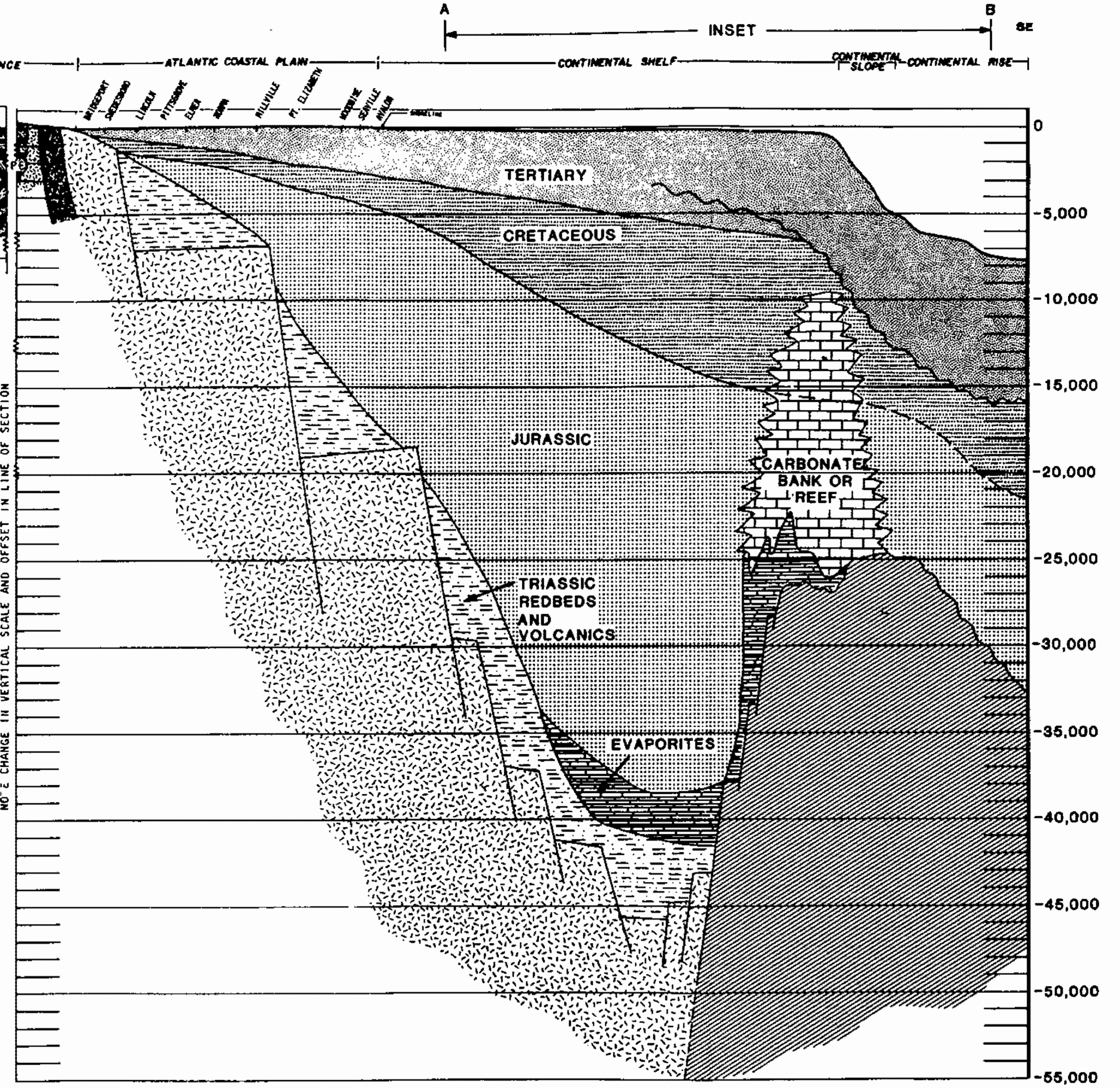
HOPE CREEK NUCLEAR GENERATING STATION

REGIONAL GEOLOGIC MAP

UPDATED FSAR

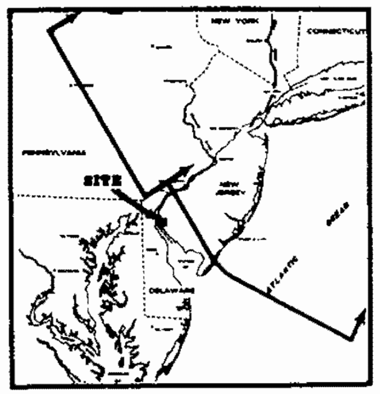
FIGURE 2.5-5





A. EXPLANATION:

- TRIASSIC
- M&P — MISSISSIPIAN & PENN.
- D — DEVONIAN
- S — SILURIAN
- O — ORDOVICIAN
- Є — CAMBRIAN
- PЄ — PRECAMBRIAN AND PRE-MESOZOIC GNEISS
- WISSAHICKON SCHIST AND EQUIVALENTS
- GABROIC INTRUSIVES
- SERPENTINITES AND ULTRAMAFICS
- TOP OF PRE-MESOZOIC BASEMENT OF UNDETERMINED LITHOLOGY



LOCATION OF SECTIONS

REFERENCES:

PSE&G, 1974, PSAR-ATLANTIC GENERATING STATION, FIG. 2.5.1-8.

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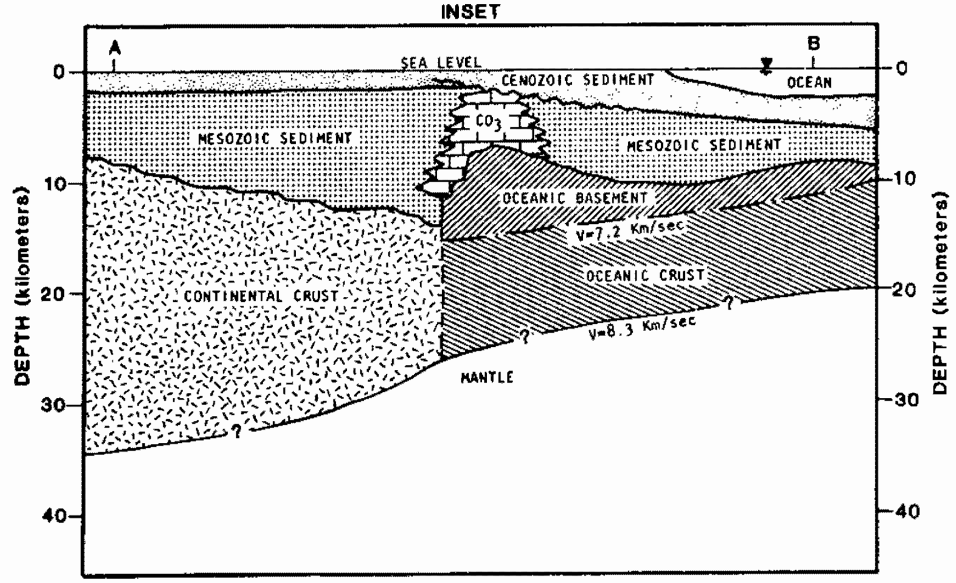
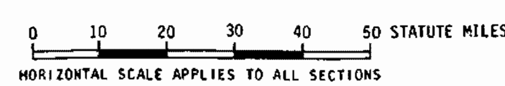
INSET





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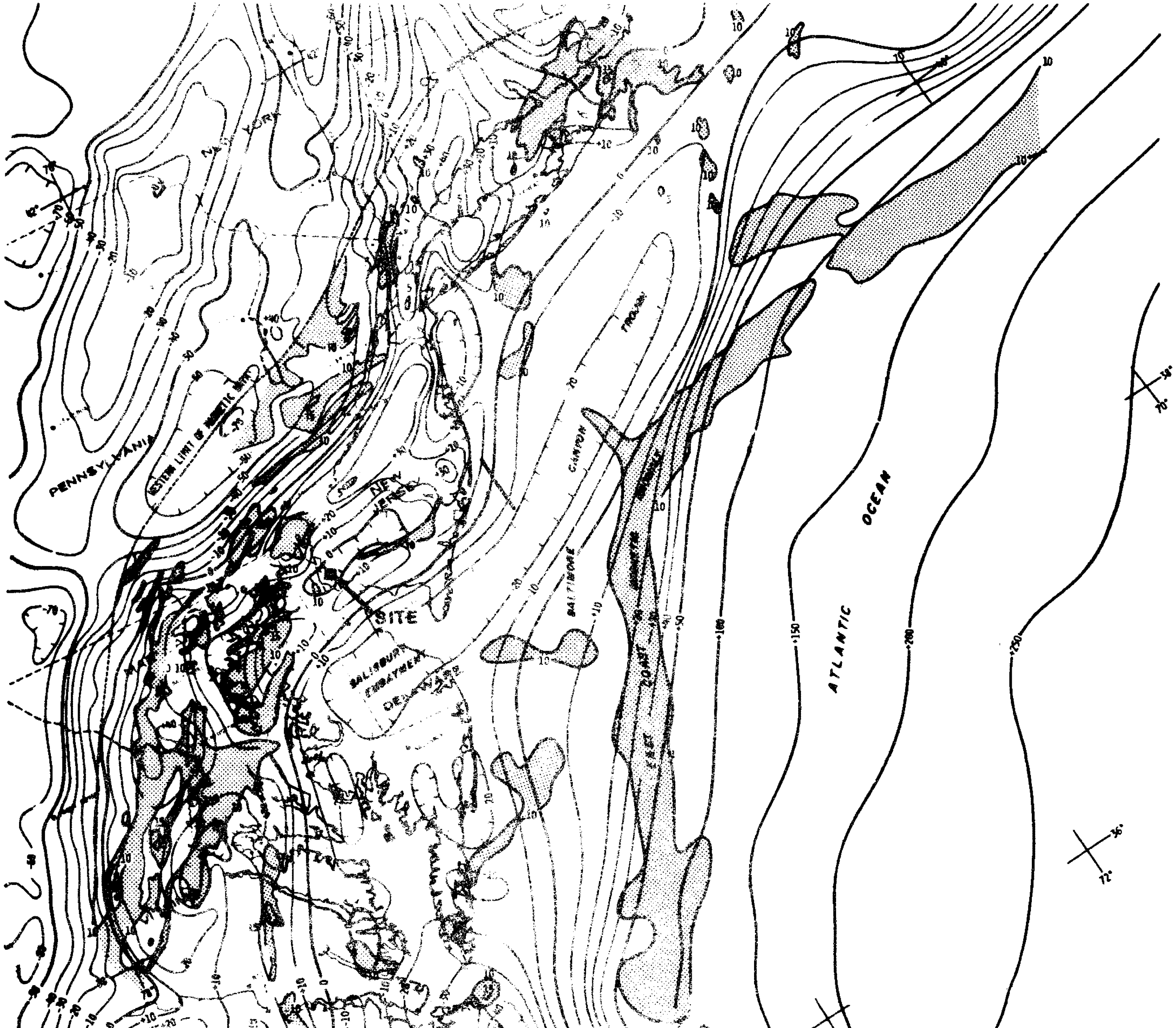
PUBLIC SERVICE ELECTRIC AND GAS COMPANY HOPE CREEK NUCLEAR GENERATING STATION	
GENERALIZED REGIONAL GEOLOGIC CROSS SECTION	
UPDATED FSAR	FIGURE 2.5-6



REFERENCE:
PSE&G, 1974, PSAR-ATLANTIC GENERATING STATION

MILES

KEY:

GRAVITY CONTOURS (MILLIGALS)

MAGNETIC CONTOURS (HUNDREDS OF GAMMAS)

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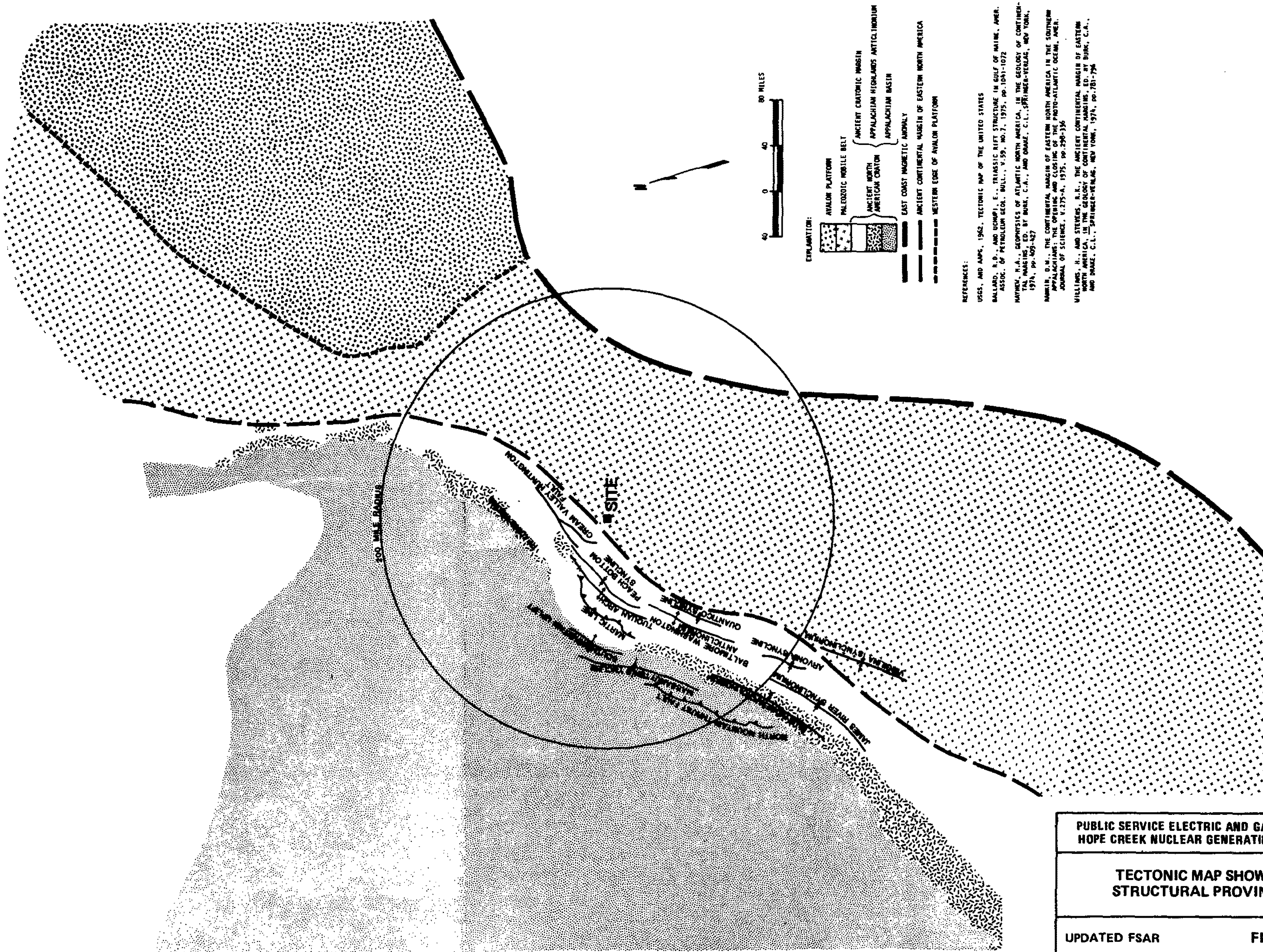
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

REGIONAL GRAVITY AND MAGNETIC MAP

UPDATED FSAR

FIGURE 2.5-7



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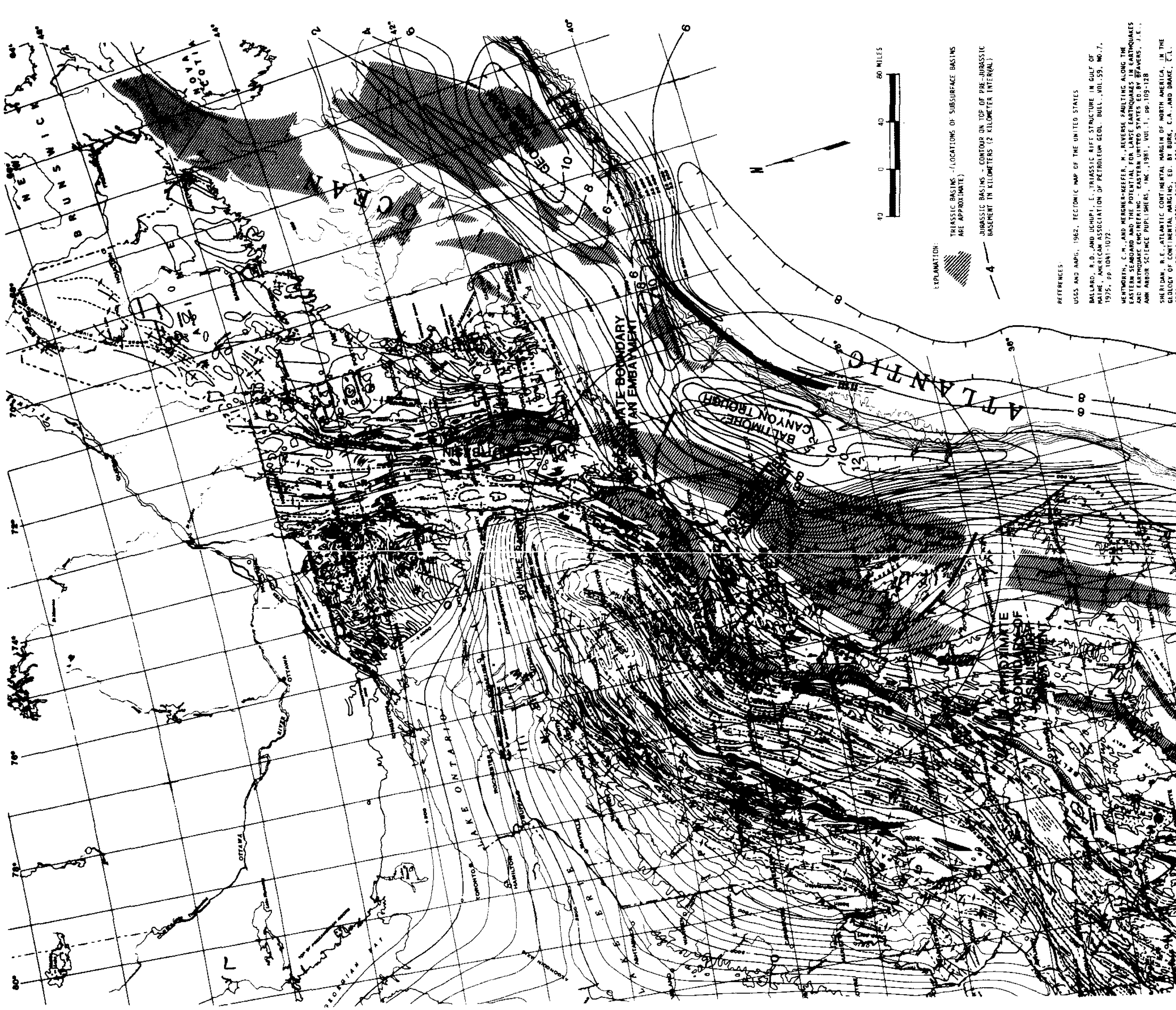
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

TECTONIC MAP SHOWING
STRUCTURAL PROVINCES

UPDATED FSAR

FIGURE 2.5-8



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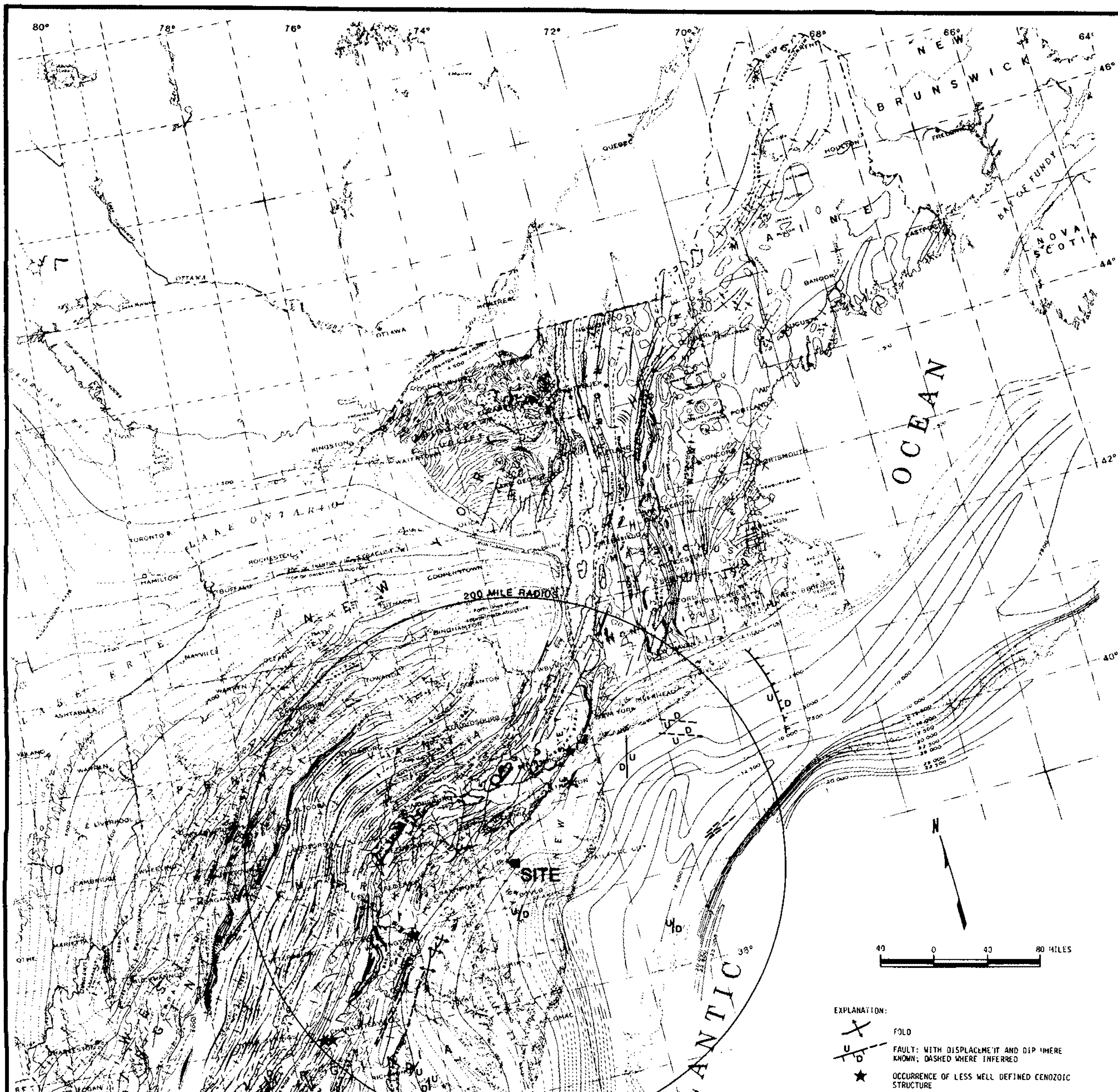
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

TECTONIC MAP SHOWING
TRIASSIC AND JURASSIC BASINS

UPDATED FSAR

FIGURE 2.5-9



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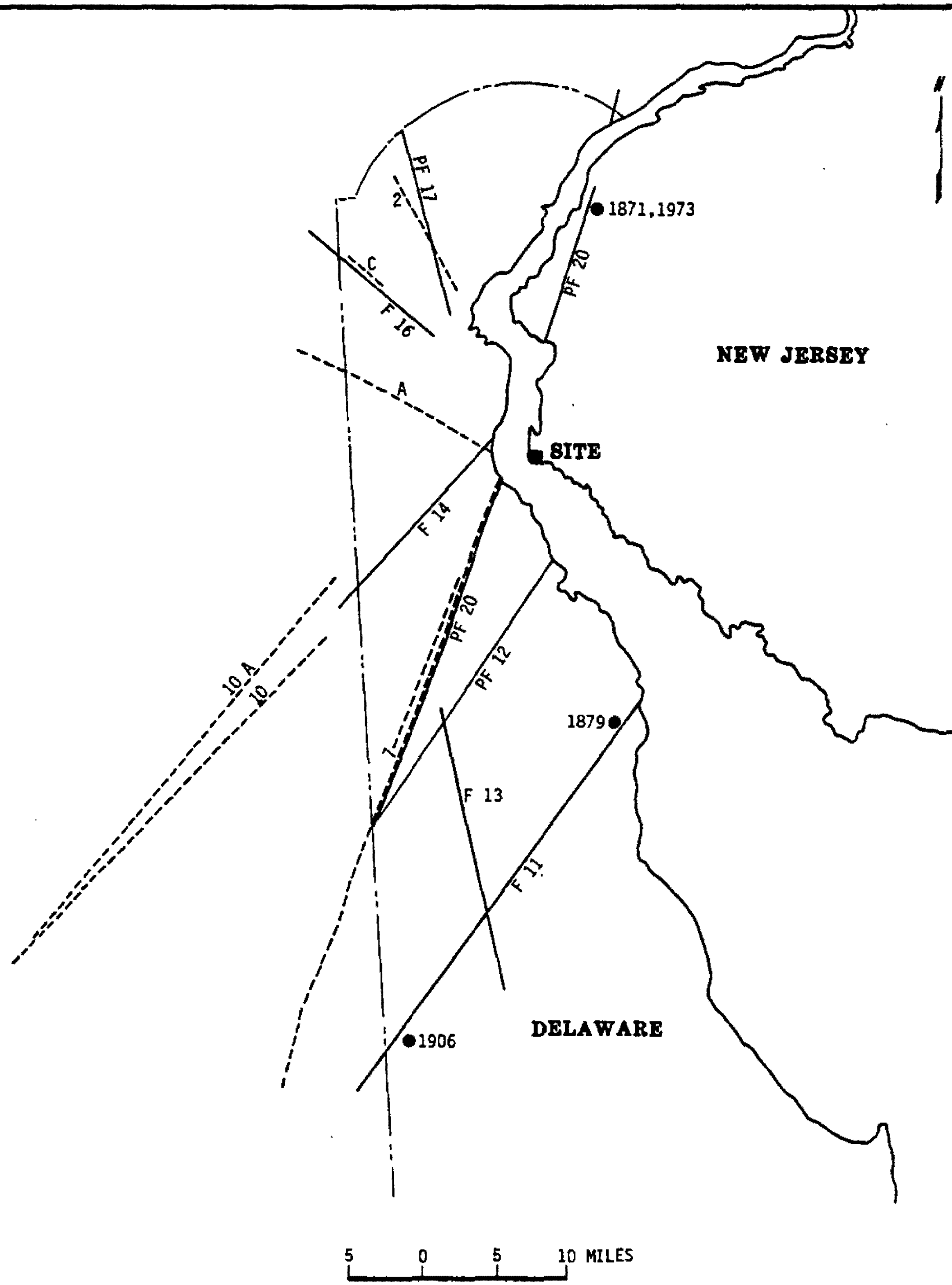
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

TECTONIC MAP SHOWING LATE CRETACEOUS AND CENOZOIC STRUCTURES

UPDATED FSAR

FIGURE 2.5-10



LEGEND:

● = EARTHQUAKE EPICENTERS (SPOLJARIC, 1979)

F = FAULT

PF = PROBABLE FAULT

----- LINEAMENT (DELMARVA SUMMIT SITE PSAR)

REFER TO SECTION 2.5.1.1.3.3 FOR DISCUSSION

REFERENCE: SPOLJARIC, 1979
DELMARVA P & L SUMMIT PSAR

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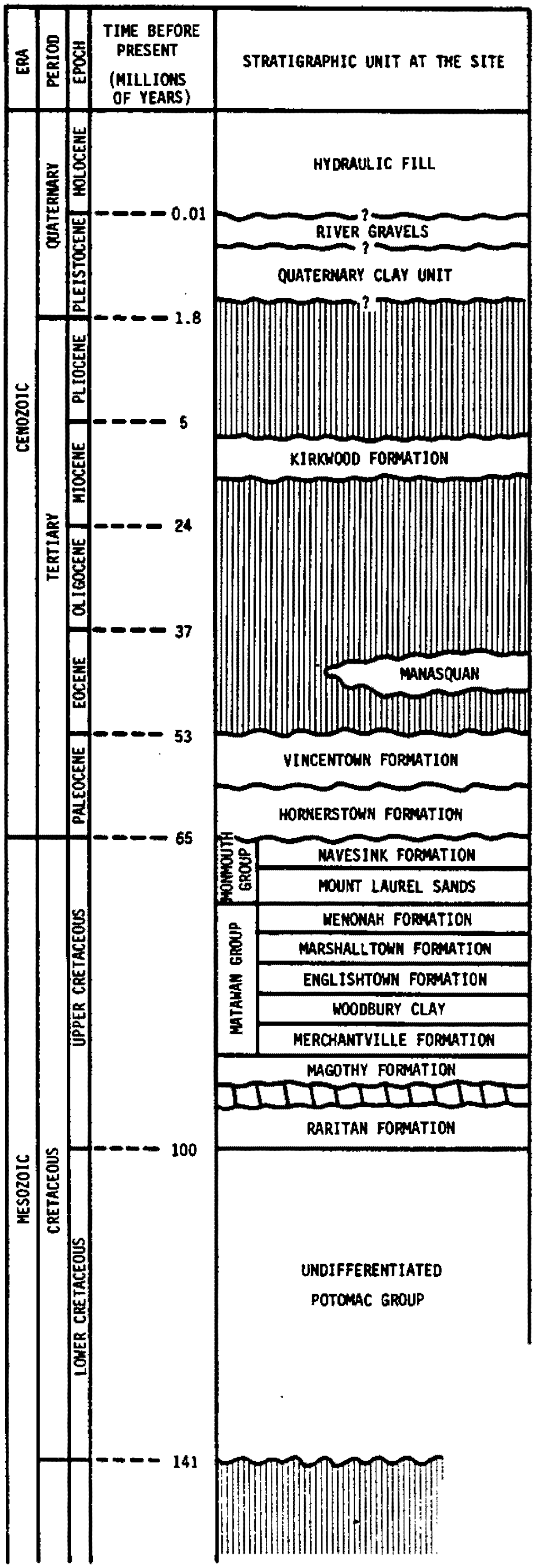
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

REVIEW OF RECENT LANDSAT ANALYSIS

UPDATED FSAR

FIGURE 2.5-10A



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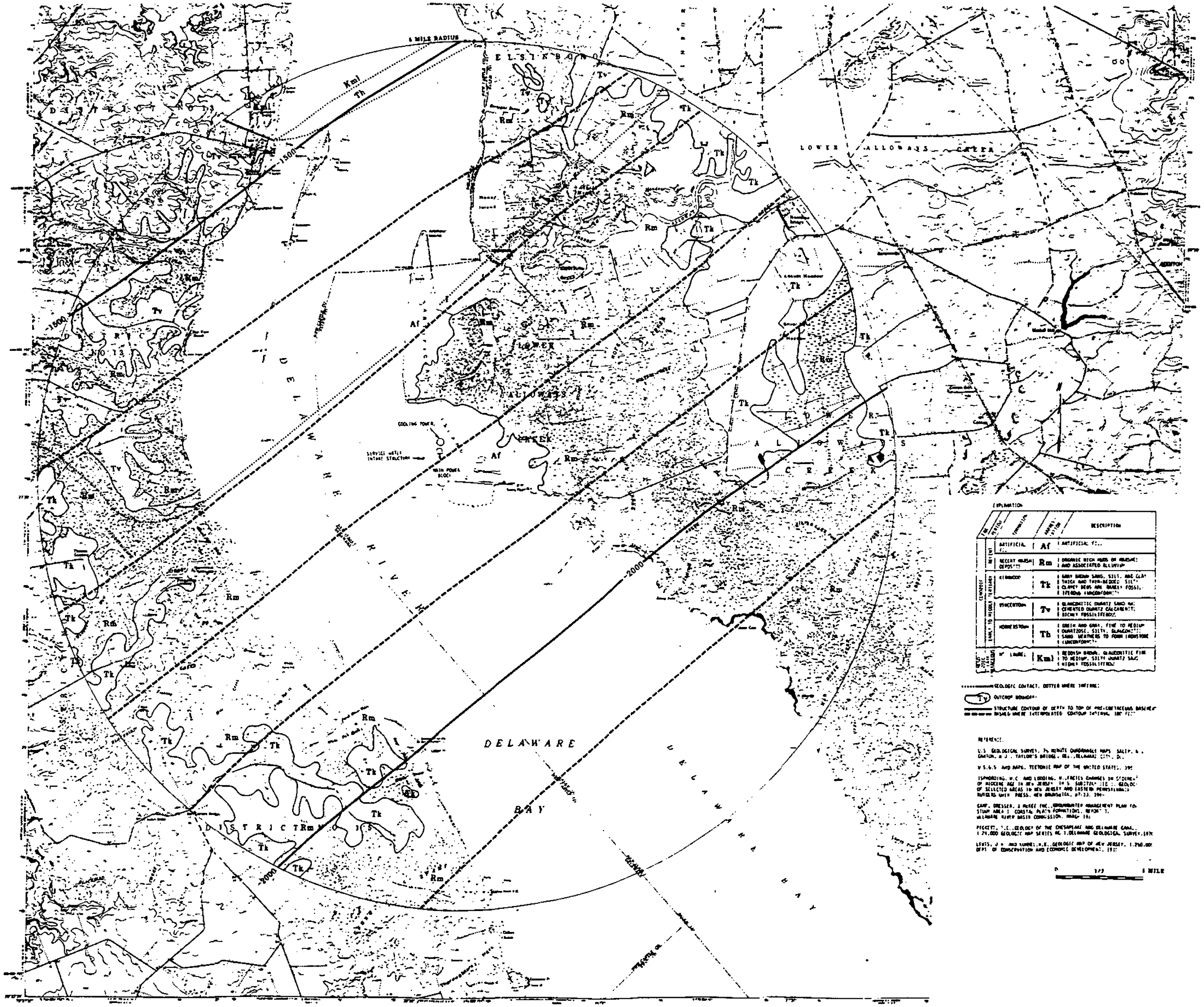
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

SITE STRATIGRAPHIC RELATIONSHIPS

UPDATED FSAR

FIGURE 2.5-11



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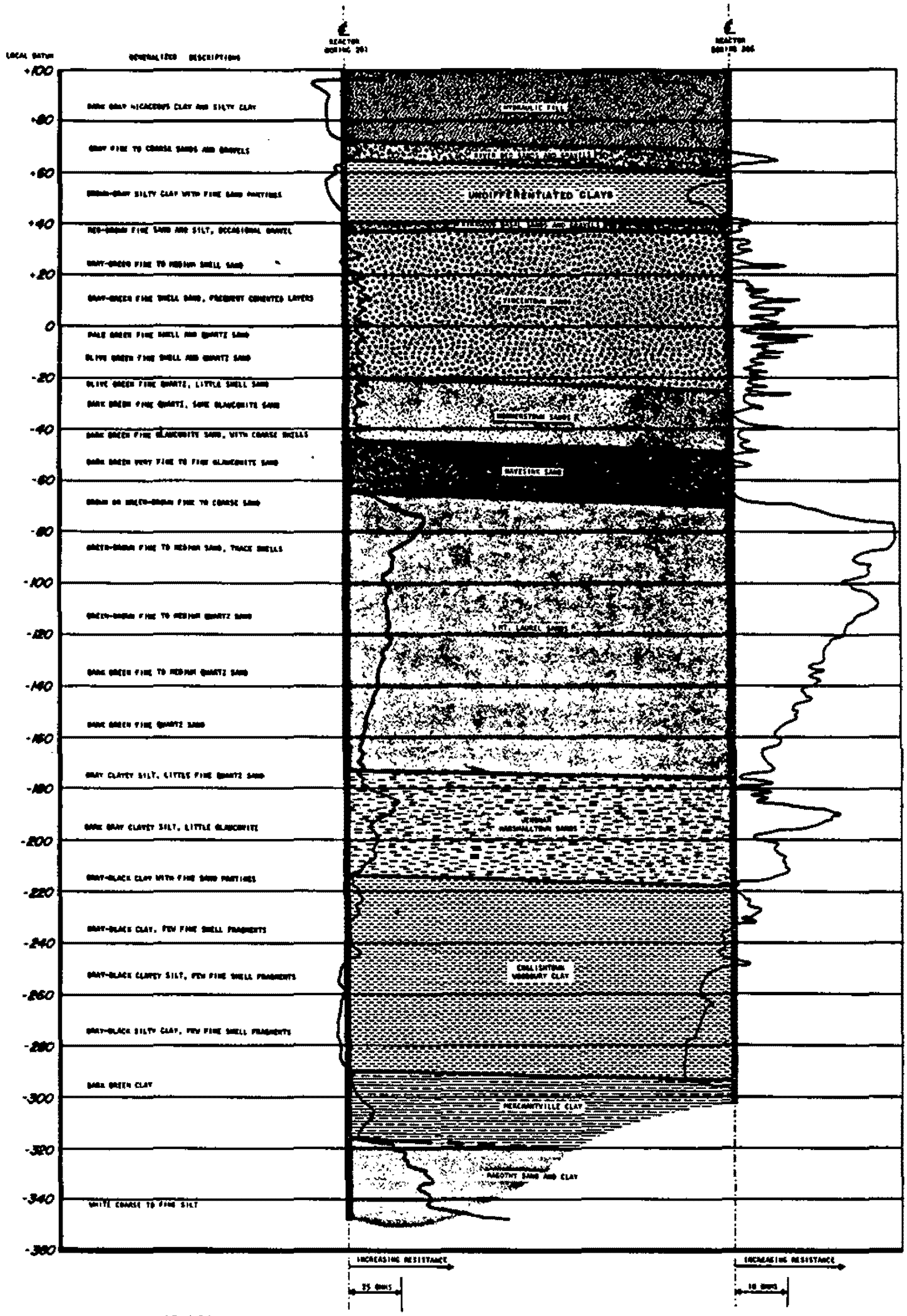
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

SURFICIAL GEOLOGIC MAP

UPDATED FSAR

FIGURE 2.5-11A



KEY:

SINGLE POINT RESISTANCE LOG

NOTES:

1. THE SUBSURFACE SECTION SHOWN REPRESENTS OUR EVALUATION OF THE MOST PROBABLE CONDITIONS BASED UPON INTERPRETATION OF PRESENTLY AVAILABLE DATA, SOME VARIATIONS FROM THESE CONDITIONS MUST BE EXPECTED.
2. ELEVATIONS REFER TO LOCAL DATUM 100'=11' M.S.L.

REFERENCE:

PSE&G, MAY 23, 1974, REPORT, FOUNDATION STUDIES, PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

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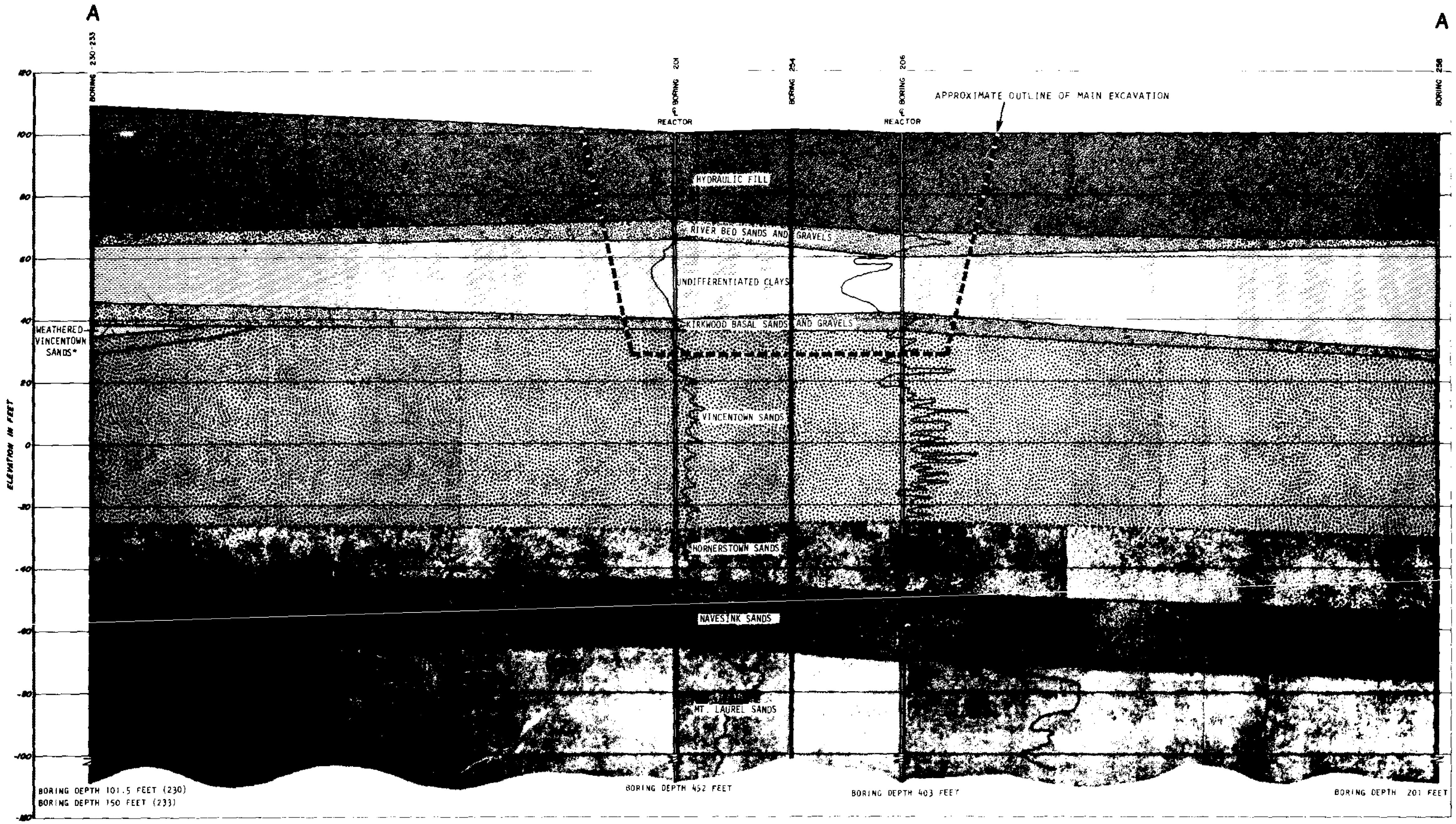
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

STRATIGRAPHY OF DEEP TEST BORINGS

UPDATED FSAR

FIGURE 2.5-12



REFERENCE:

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NOTES:

1. THE SUBSURFACE SECTIONS SHOWN REPRESENT OUR EVALUATION OF THE MOST PROBABLE CONDITIONS BASED UPON INTERPRETATION OF PRESENTLY AVAILABLE DATA. SOME VARIATIONS FROM THESE CONDITIONS MUST BE EXPECTED.
2. ELEVATIONS REFER TO LOCAL DATUM. 100 FEET LOCAL DATUM = 11 FEET MEAN SEA LEVEL.
3. LOCATION OF CROSS SECTION SHOWN ON FIGURE 2.5-19.
4. * OXIDIZED ZONE WITH BLOW COUNTS APPROXIMATELY LESS THAN 20 USING STANDARD 2 INCH O.D. SPLIT SPOON DRIVEN BY A 140 POUND HAMMER FALLING 30 INCHES.

KEY:

- BORING ON LINE
- BORING PROJECTED ON LINE
- SHARP CONFORMABLE CONTACT
- UNCONFORMABLE CONTACT
- GRADATIONAL CONFORMABLE CONTACT
- APPARENT DEPTH OF OXIDIZED ZONE
- SINGLE POINT RESISTANCE LOG

100 0 100 200

HORIZONTAL SCALE IN FEET

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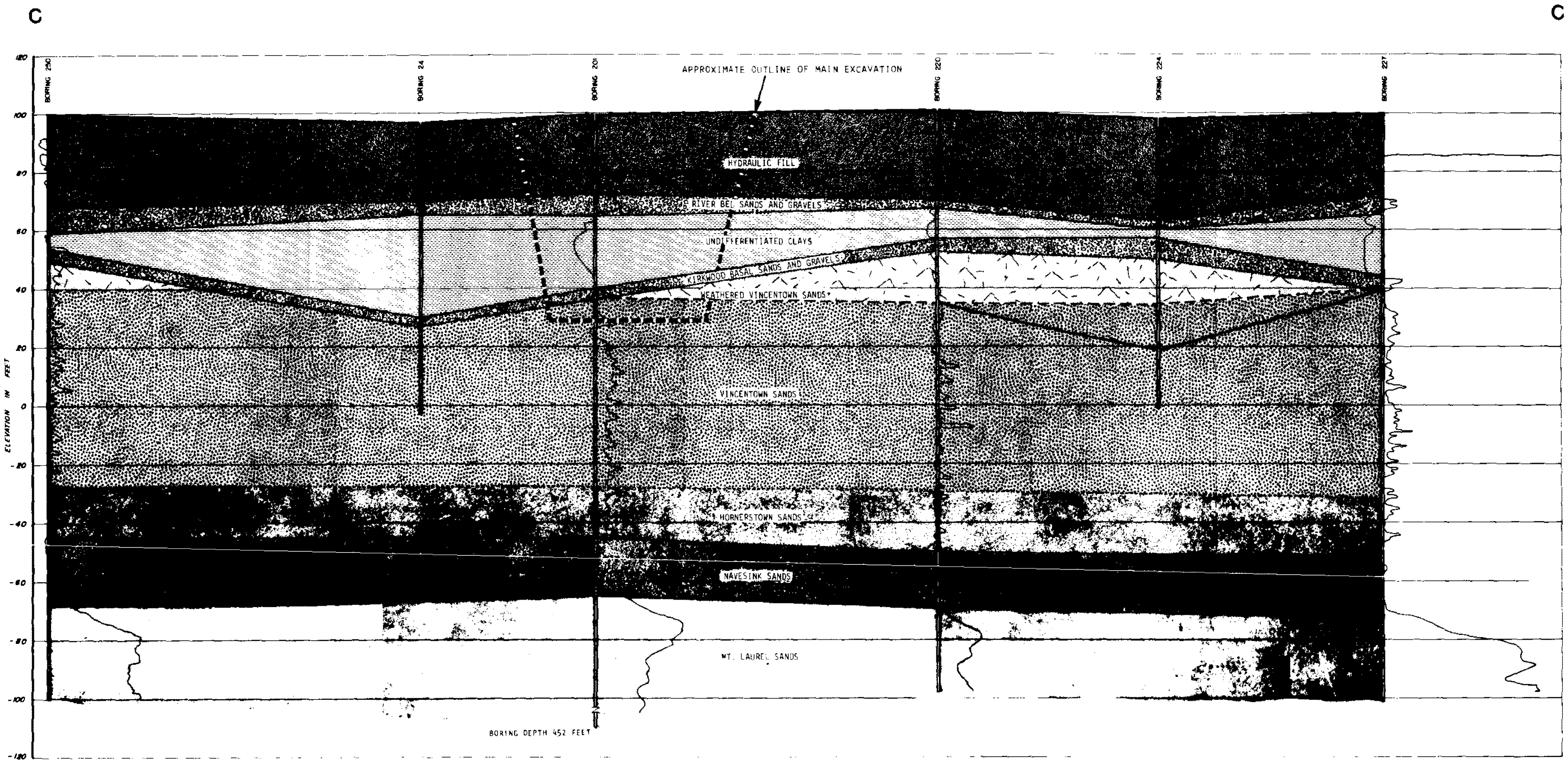
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

GEOLOGIC CROSS SECTION A-A

UPDATED FSAR

FIGURE 2.5-13



REFERENCE.
PSE&G, MAY 23, 1974, REPORT, FOUNDATION STUDIES, PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE

NOTES:

1. THE SUBSURFACE SECTIONS SHOWN REPRESENT OUR EVALUATION OF THE MOST PROBABLE CONDITIONS BASED UPON INTERPRETATION OF PRESENTLY AVAILABLE DATA. SOME VARIATIONS FROM THESE CONDITIONS MUST BE EXPECTED.

2. ELEVATIONS REFER TO LOCAL DATUM. 100 FEET LOCAL DATUM = 11 FEET MEAN SEA LEVEL.

3. LOCATION OF CROSS SECTION SHOWN ON FIGURE 2.5-19.

4. OXIDIZED ZONE WITH BLOW COUNTS APPROXIMATELY LESS THAN 20 USING STANDARD 2 INCH O.D. SPLIT SPOON DRIVEN BY A 140 POUND HAMMER FALLING 30 INCHES.

KEY:

SHARP CONFORMABLE CONTACT
UNCONFORMABLE CONTACT
GRADATIONAL CONFORMABLE CONTACT
INFERRED CONTACT
APPARENT DEPTH OF OXIDIZED ZONE
SINGLE POINT RESISTANCE LOG

100 0 100 200

HORIZONTAL SCALE IN FEET

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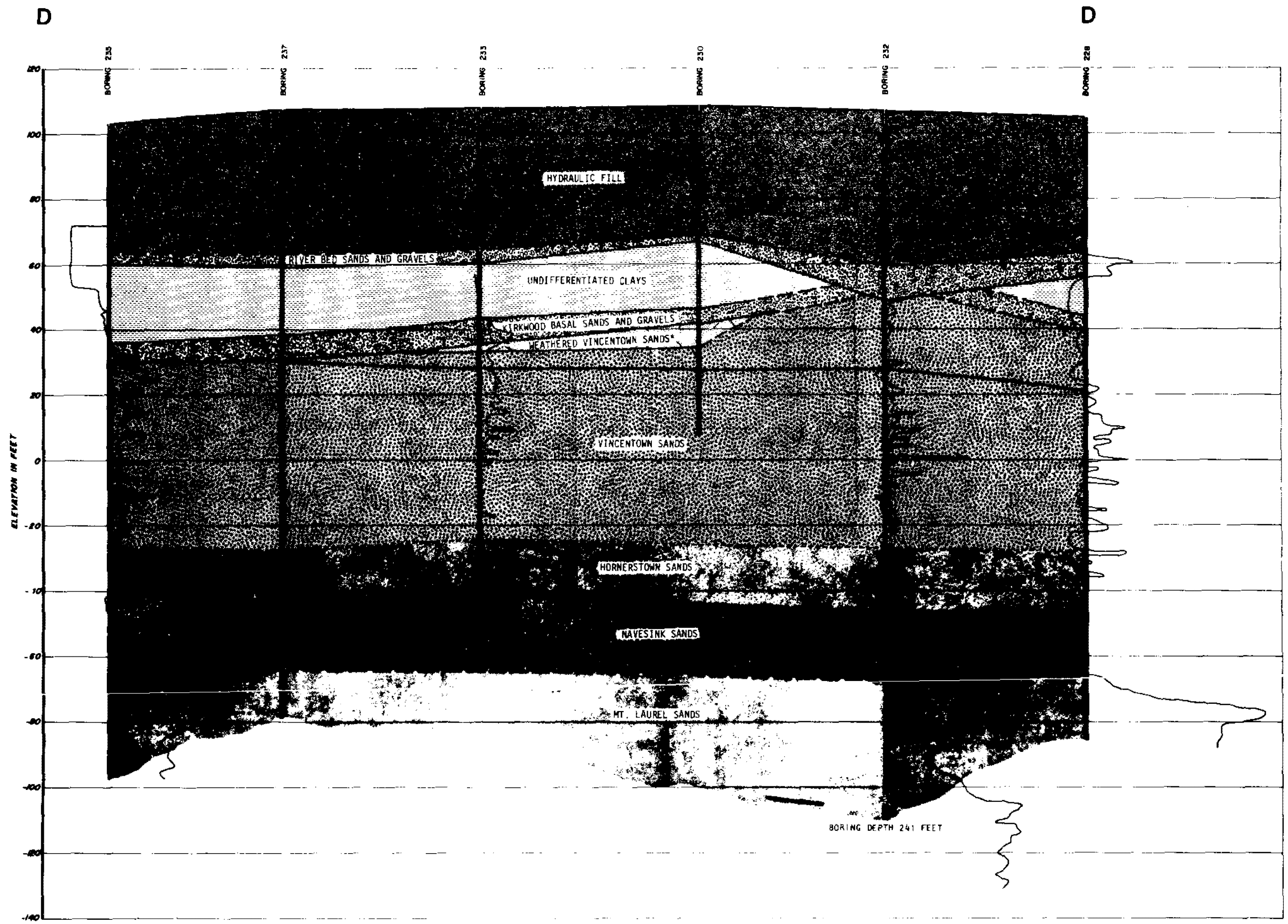
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

GEOLOGIC CROSS SECTION C-C

UPDATED FSAR

FIGURE 2.5-14



REFERENCE:
PSE&G, MAY 23, 1974, REPORT, FOUNDATION STUDIES, PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

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3. LOCATION OF CROSS SECTION SHOWN ON FIGURE 2.5-19.

4. * OXIDIZED ZONE WITH BLOW COUNTS APPROXIMATELY LESS THAN 20 USING STANDARD 2 INCH O.D. SPLIT SPOON DRIVEN BY A 140 POUND HAMMER FALLING 30 INCHES.

KEY:

SHARP CONFORMABLE CONTACT
UNCONFORMABLE CONTACT
GRADATIONAL CONFORMABLE CONTACT
INFERRED CONTACT
APPARENT DEPTH OF OXIDIZED ZONE
SINGLE POINT RESISTANCE LOG

100 0 100 200
HORIZONTAL SCALE IN FEET

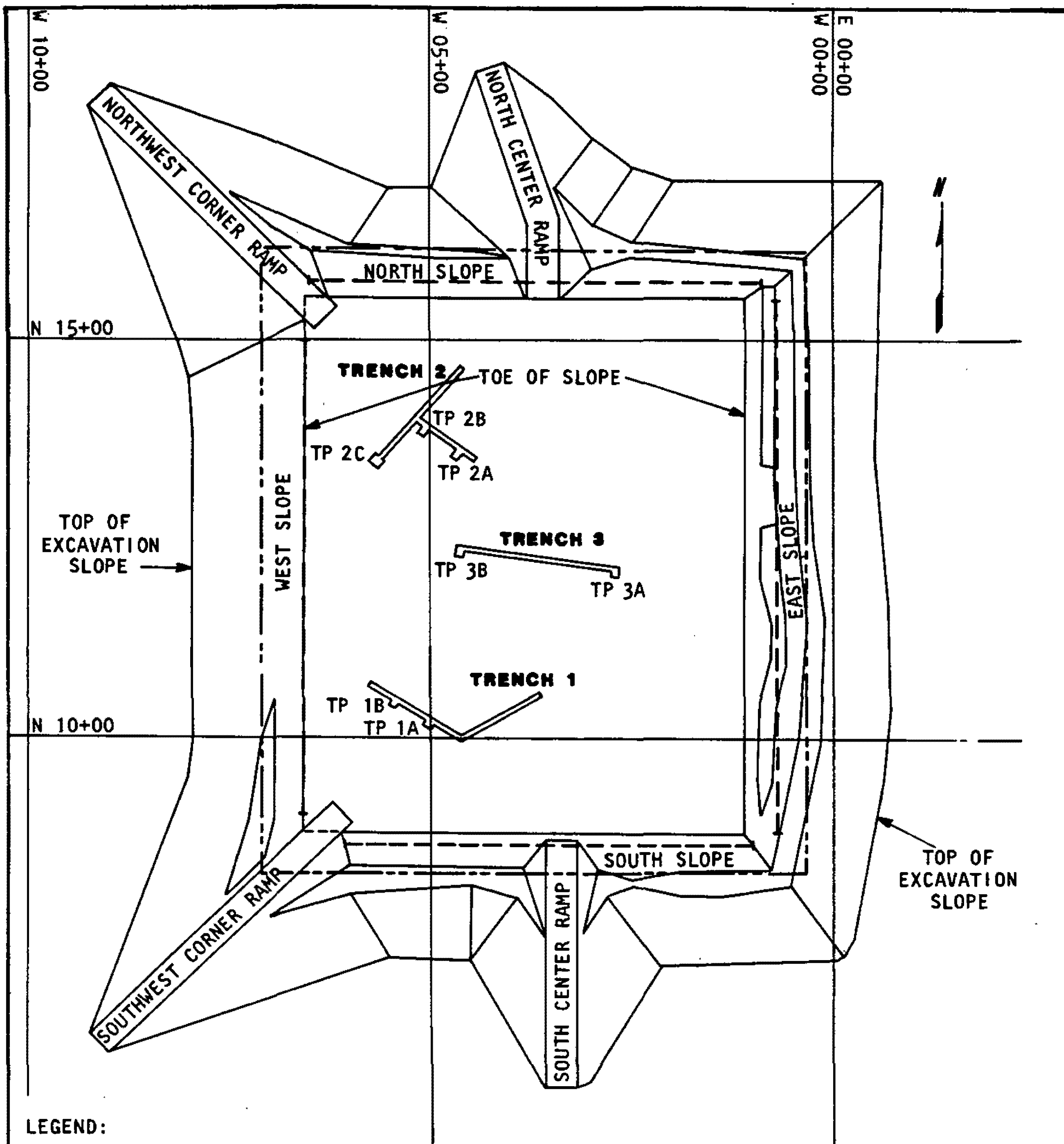
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

GEOLOGIC CROSS SECTION D-D

UPDATED FSAR

FIGURE 2.5-15



LEGEND:

TP 3B EXPLORATORY TRENCHES AND ADJACENT TEST PITS EXCAVATED FOR GEOTECHNICAL EXAMINATION

LOCATION OF EXCAVATION SLOPE PROFILES

LIMIT OF AREA INCLUDED IN STRUCTURAL CONTOUR MAPS

REFERENCE:
PSE&G, MAY 23, 1974, REPORT, FOUNDATION STUDIES, PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

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APRIL 11, 1988

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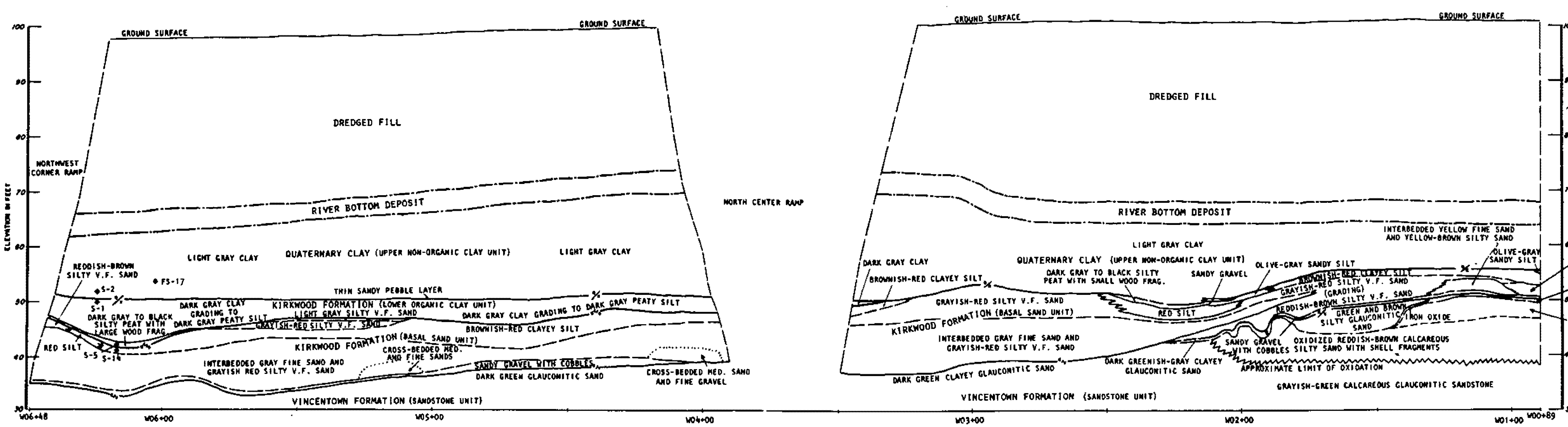
HOPE CREEK NUCLEAR GENERATING STATION

PLOT PLAN OF MAIN EXCAVATION

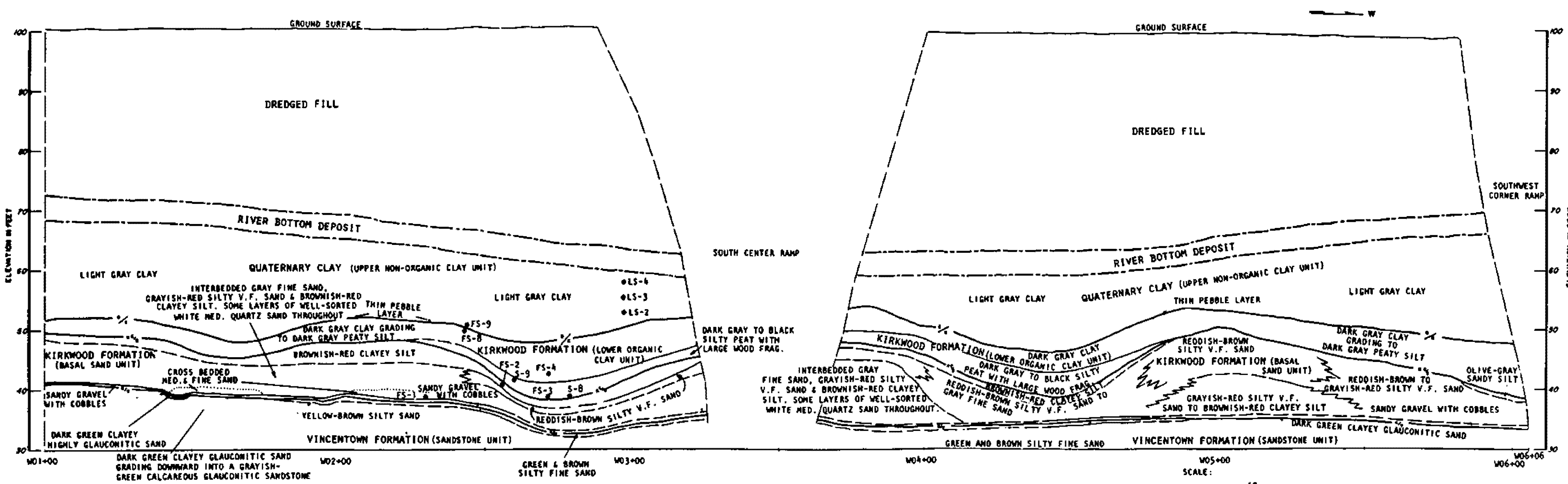
UPDATED FSAR

FIGURE 2.5-16



GEOLOGICAL PROFILE OF NORTH SLOPE

SHOWING PROMINENT STRATIGRAPHIC CONTACTS AND PROJECTED LITHOLOGIC CHANGES



SCALE:
HORIZONTAL 1" = 20'
VERTICAL 1" = 10'
TWO TIMES VERTICAL EXAGGERATION

GEOLOGICAL PROFILE OF SOUTH SLOPE

SHOWING PROMINENT STRATIGRAPHIC CONTACTS AND PROJECTED LITHOLOGIC CHANGES

LEGEND:

STRATIGRAPHIC NOMENCLATURE
STRATIGRAPHIC UNITS ARE IDENTIFIED BY THEIR FORMATIONAL NAME AND/OR DESCRIPTIVE LITHOSTRATIGRAPHIC NAME (IN PARENTHESIS). THIS NOMENCLATURE IS PRESENTED IN BOLDER TYPE. THE DETAILED LITHOLOGY OF EACH INTERNAL FACIES IS DESCRIBED IN SMALL TYPE.

CONTACTS
1. LITHOSTRATIGRAPHIC CONTACTS (OBSERVED —— INFERRED — —)
APPROXIMATE LIMITS OF UNNAMED RIVER BOTTOM DEPOSIT
Q/K — QUATERNARY CLAY/TERTIARY KIRKWOOD FORMATION (UPPER NON-ORGANIC CLAY / LOWER ORGANIC CLAY) OR (UPPER NON-ORGANIC CLAY / BASAL SAND)
K C/S — KIRKWOOD FORMATION (LOWER ORGANIC CLAY / BASAL SAND)
K/V — KIRKWOOD FORMATION/VINCENTOWN FORMATION (BASAL SAND / CALCAREOUS SANDSTONE)

2. LITHOFACIES CONTACTS WITHIN LITHOSTRATIGRAPHIC UNITS:
—— OBSERVED — — — INFERRED
IRREGULAR OR GRADATIONAL
.... APPROXIMATE LIMIT OF EXPOSURE

SAMPLE LOCATIONS
MICROPALEONTOLOGICAL SAMPLE
RADIOCARBON SAMPLE

NOTE: PROFILE LOCATIONS SHOWN ON FIGURE 2.5-16

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APRIL 11, 1988

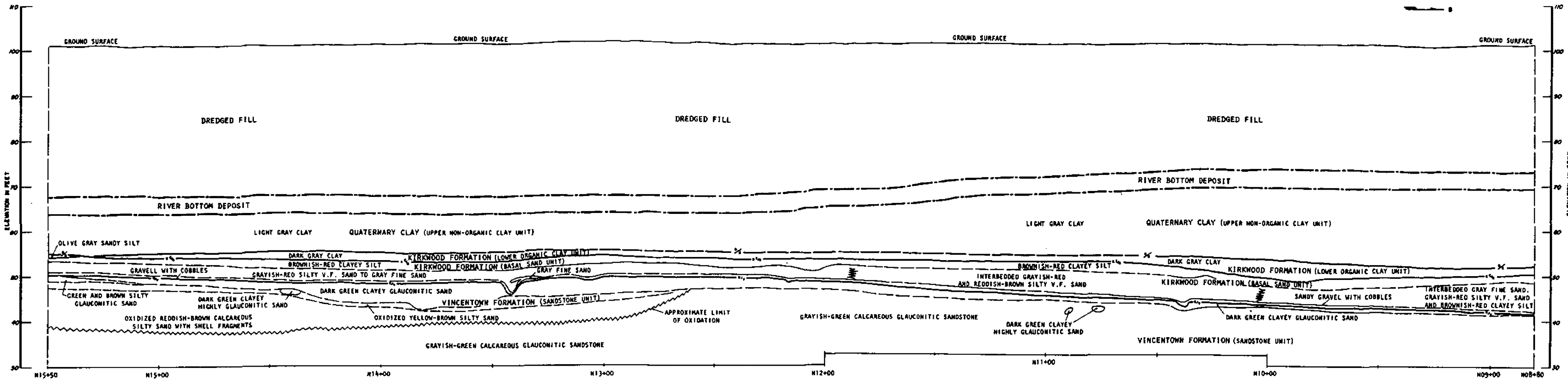
REFERENCE:
PSE&G, JANUARY 1978, REPORT, POST EXCAVATION FOUNDATION STUDIES, HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

GEOLOGICAL PROFILES OF MAIN EXCAVATION WALLS

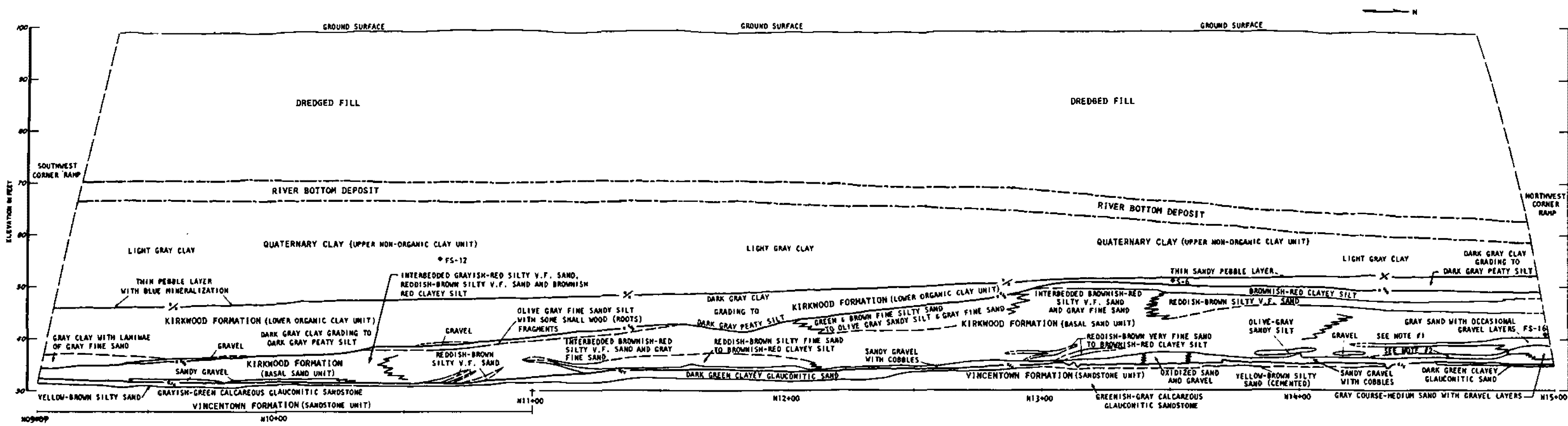
UPDATED FSAR

FIGURE 2.5-17



GEOLOGICAL PROFILE OF EAST SLOPE

SHOWING PROMINENT STRATIGRAPHIC CONTACTS AND PROJECTED LITHOLOGIC CHANGES



GEOLOGICAL PROFILE OF WEST SLOPE

SHOWING PROMINENT STRATIGRAPHIC CONTACTS AND PROJECTE LITHOLOGIC CHANGES

SCALE:

HORIZONTAL 1" = 20'
VERTICAL 1" = 10'
TWO TIMES VERTICAL EXAGGERATION

NOTES:

1. CYCLIC SEQUENCE OF WHITE WELL SORTED, MEDIUM QUARTZ SAND WITH RIPPLE MARKS (2" TO 3" THICK) ; GRAY FINE SAND TO GRAYISH-RED SILTY VERY FINE SAND (UP TO 4" THICK) AND REDDISH-BROWN CLAYEY SILT (1/4" THICK).
2. WHITE (WELL- SORTED) MEDIUM QUARTZ SAND WITH OCCASIONAL PEBBLE LAYERS AND RIPPLE MARKS.

REPORT:
PSE&G, JANUARY 1978, REPORT, POST EXCAVATION FOUNDATION STUDIES, HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE

LEGEND: STRATIGRAPHIC NOMENCLATURE

STRATIGRAPHIC UNITS ARE IDENTIFIED BY THEIR FORMATIONAL NAME AND/OR DESCRIPTIVE LITHOSTRATIGRAPHIC NAME (IN PARENTHESIS). THIS NOMENCLATURE IS PRESENTED IN BOLD-FACE TYPE. THE DETAILED LITHOLOGY OF EACH INTERNAL FACIES IS DESCRIBED IN SMALL TYPE.

CONTACTS

1. LITHOSTRATIGRAPHIC CONTACTS (OBSERVED —— INFERRED — —):

— - — APPROXIMATE LIMITS OF UNNAMED RIVER BOTTOM DEPOSIT

— Q/K — QUATERNARY CLAY/TERTIARY KIRKWOOD FORMATION ($\frac{\text{UPPER NON-ORGANIC CLAY}}{\text{LOWER ORGANIC CLAY}}$) OR ($\frac{\text{UPPER NON-ORGANIC CLAY}}{\text{BASAL SAND}}$)

— K C/S — KIRKWOOD FORMATION ($\frac{\text{LOWER ORGANIC CLAY}}{\text{BASAL SAND}}$)

— K/V — KIRKWOOD FORMATION/VINCENTOWN FORMATION ($\frac{\text{BASAL SAND}}{\text{CALCAREOUS SANDSTONE}}$)

2. LITHOFACIES CONTACTS WITH LITHOSTRATIGRAPHIC UNITS:

—— OBSERVED — — — INFERRED

IRREGULAR OR GRADATIONAL

·········· APPROXIMATE LIMIT OF EXPOSURE

SAMPLE LOCATIONS

◆ MICROPALEONTOLOGICAL SAMPLE

NOTE: PROFILE LOCATIONS SHOWN ON FIGURE 2.5-16

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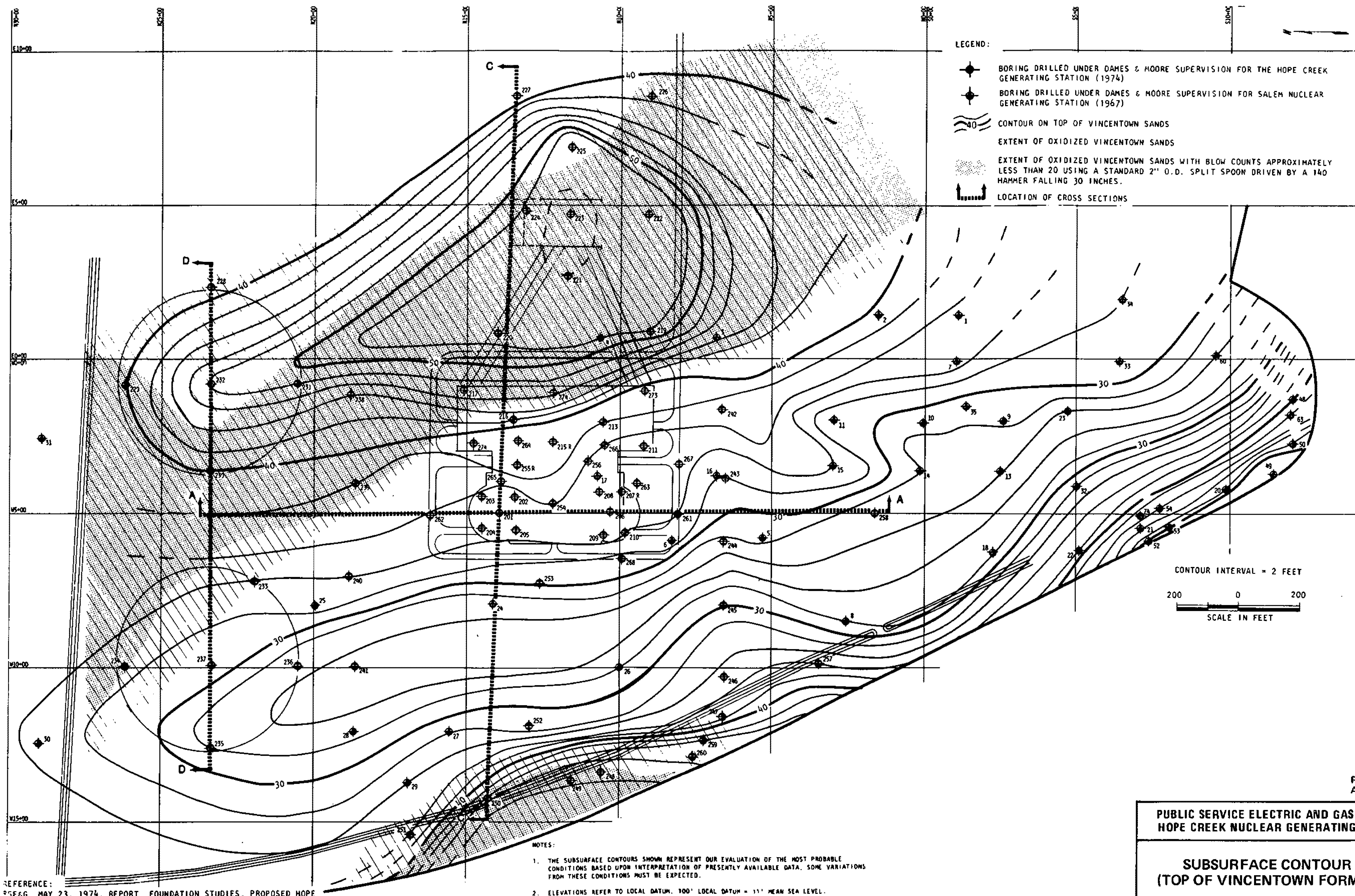
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

GEOLOGIC PROFILES OF MAIN EXCAVATION WALLS

UPDATED FSAR

FIGURE 2.5-18



NOTES:

1. THE SUBSURFACE CONTOURS SHOWN REPRESENT OUR EVALUATION OF THE MOST PROBABLE CONDITIONS BASED UPON INTERPRETATION OF PRESENTLY AVAILABLE DATA. SOME VARIATIONS FROM THESE CONDITIONS MUST BE EXPECTED.
2. ELEVATIONS REFER TO LOCAL DATUM. 100' LOCAL DATUM = 11' MEAN SEA LEVEL.
3. THE DISCUSSION IN THE TEXT OF THE REPORT IS NECESSARY FOR A PROPER UNDERSTANDING OF THE NATURE OF THE SUBSURFACE MATERIALS.

REFERENCE:
PSE&G, MAY 23, 1974, REPORT, FOUNDATION STUDIES, PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

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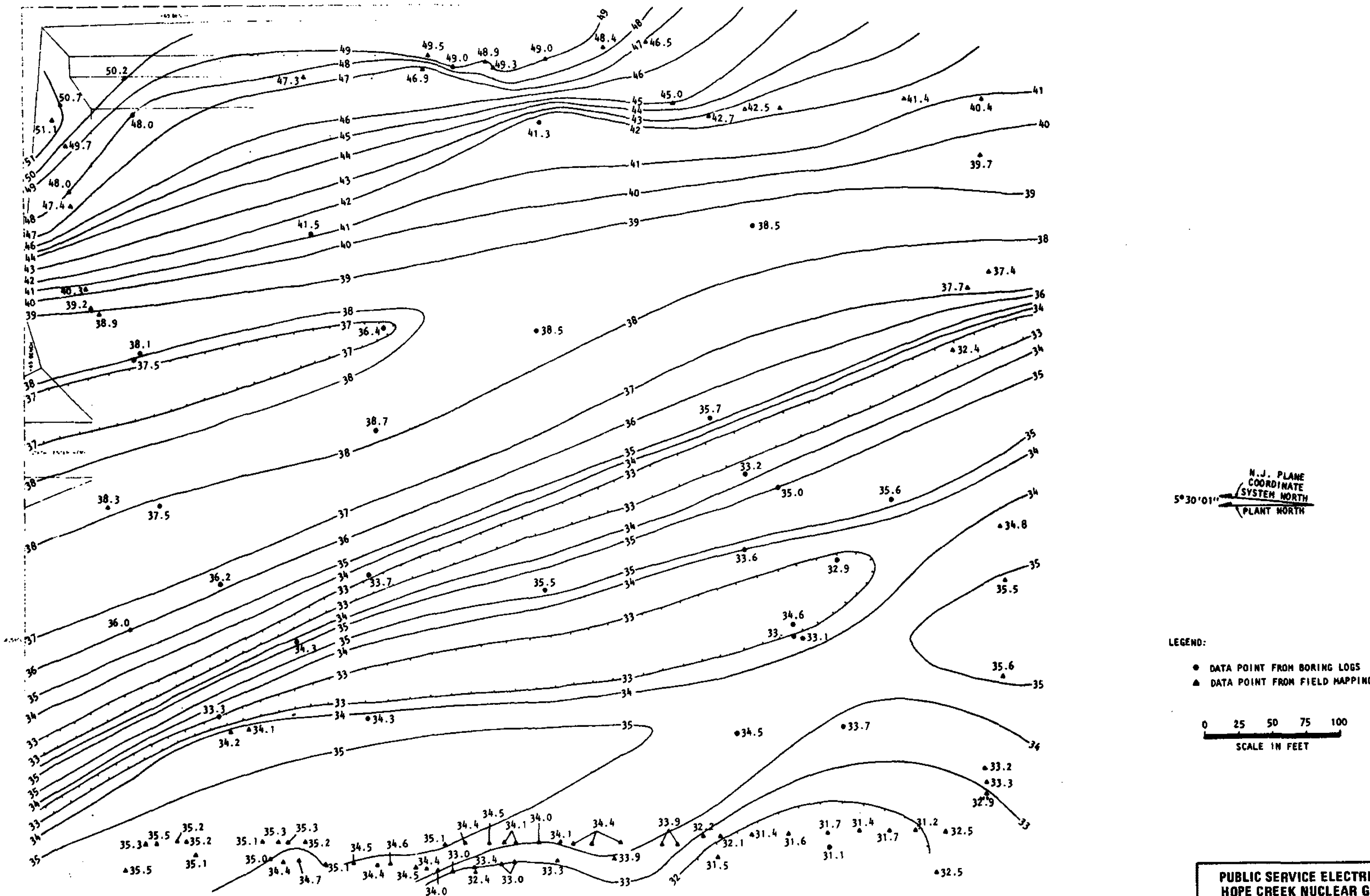
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

SUBSURFACE CONTOUR MAP
(TOP OF VINCENTOWN FORMATION)

UPDATED FSAR

FIGURE 2.5-19



NOTES:

1. CONTOUR INTERVAL IS ONE FOOT.
2. ELEVATIONS SHOWN REFER TO PUBLIC SERVICE DATUM.
3. AREA OF CONTOUR MAP SHOWN ON FIGURE 2.5-16

REFERENCE:
PSE&G, JANUARY 1978, REPORT, POST EXCAVATION FOUNDATION STUDIES, HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

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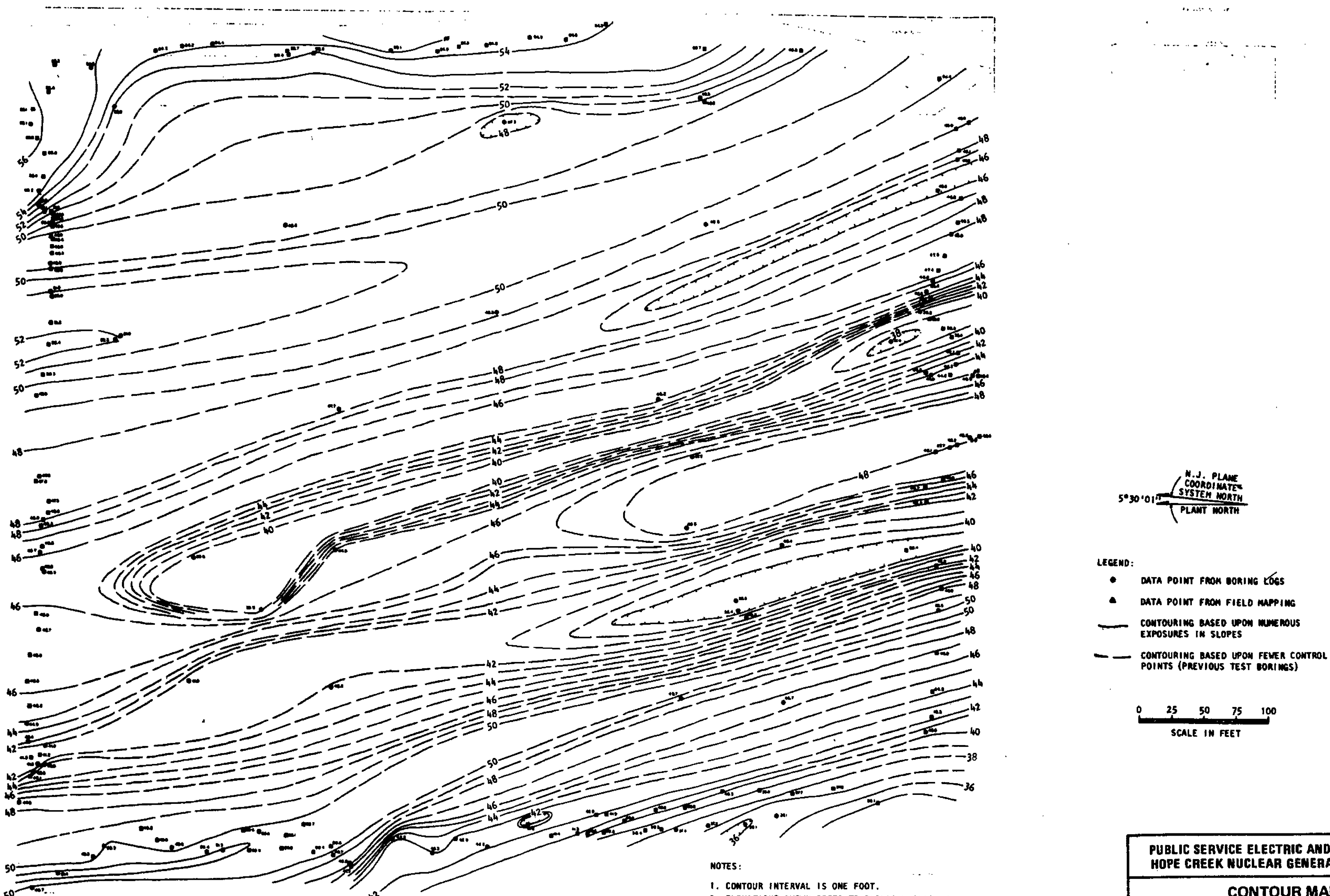
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

CONTOUR MAP
(TOP OF VINCENTOWN FORMATION)

UPDATED FSAR | **FIGURE 2.5-20**



NOTES:

1. CONTOUR INTERVAL IS ONE FOOT.
2. ELEVATIONS SHOWN REFER TO PUBLIC SERVICE DATUM
3. AREA OF CONTOUR MAP SHOWN ON FIGURE 2.5-16

REFERENCE:
PSE&G, JANUARY 1978, REPORT, POST EXCAVATION FOUNDATION STUDIES, HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

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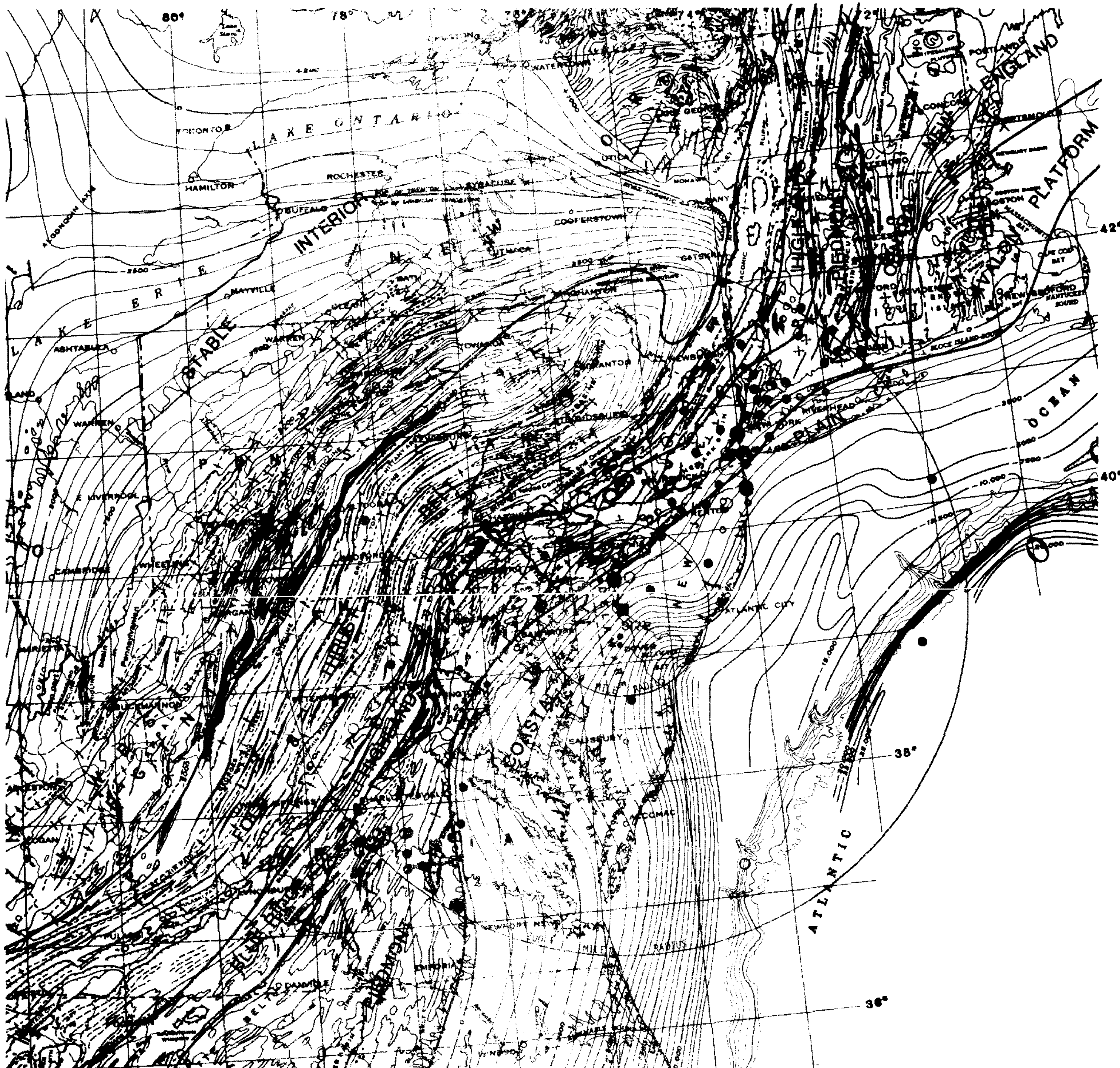
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

CONTOUR MAP
(KIRKWOOD FORMATION CLAY/
SAND CONTACT)

UPDATED FSAR

FIGURE 2.5-21



LEGEND:

MODIFIED MERCALLI SCALE INTENSITIES:

- II
- IV
- V
- VI
- VII

REFERENCE:

THE BASE MAP WAS DERIVED FROM "TECTONIC MAP OF THE UNITED STATES EXCLUSIVE OF ALASKA AND HAWAII" BY THE UNITED STATES GEOLOGICAL SURVEY AND THE AMERICAN ASSOCIATION OF PETROLEUM GEOLOGISTS, 1962, 1:2,500,000

40 0 40 80 MILES

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

REGIONAL EPICENTER MAP

UPDATED FSAR

FIGURE 2.5-22

10 0 10 20

MILES

LEGEND:

MODIFIED MERCALLI INTENSITIES

● I-II

○ IV

● V

△ VI

▲ VII

REFERENCE:
THE TIMES ATLAS OF THE WORLD COMPREHENSIVE EDITION 1980, N.Y. TIMES BOOKS, JOHN BARTHOLOMEW & SON LIMITED, NEW YORK; PLATE 103

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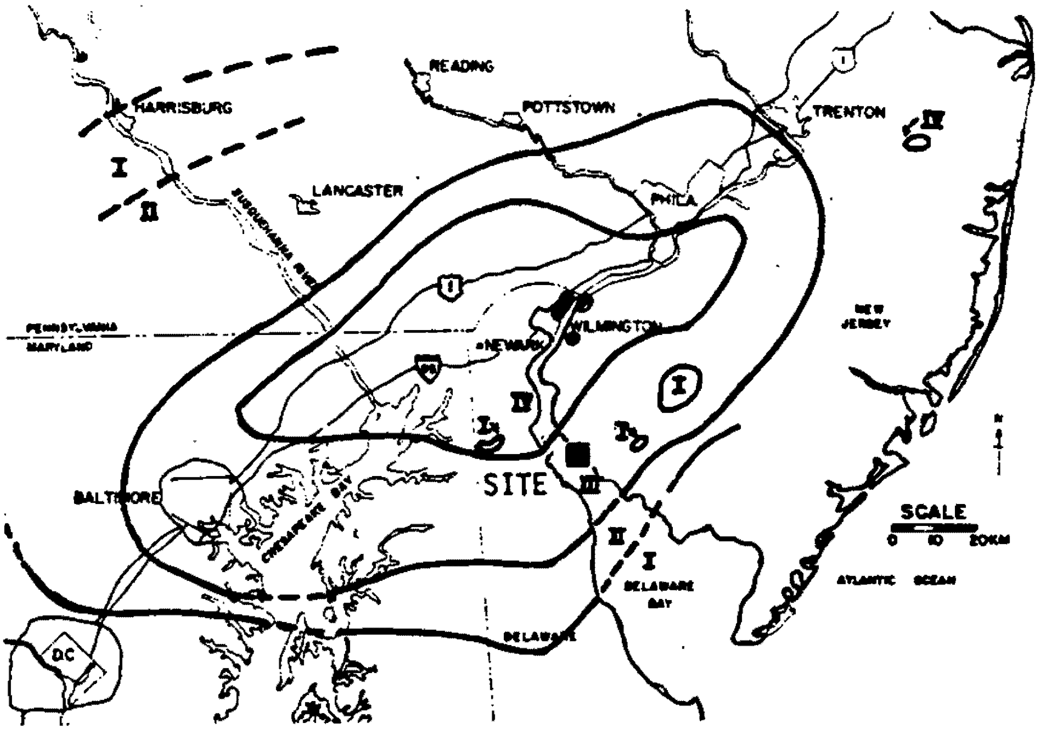
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOCATION OF EPICENTERS
(50 MILES)

UPDATED FSAR

FIGURE 2.5-23



LEGEND:

- MM INTENSITY
- NOAA EPICENTER
- MAX INTENSITY - V-VI
- AFTERSHOCKS

REFERENCE:
SHAR, M.L.; JORDON, R.R.; STEPHENS, C.D.; PICKETT, T.E.; WOODRUFF, K.D. AND SAMMIS, C.G., 1975, THE DELAWARE-NEW JERSEY EARTHQUAKE OF FEBRUARY 28, 1973, BSSA VOL.65, NO.1, p.86.

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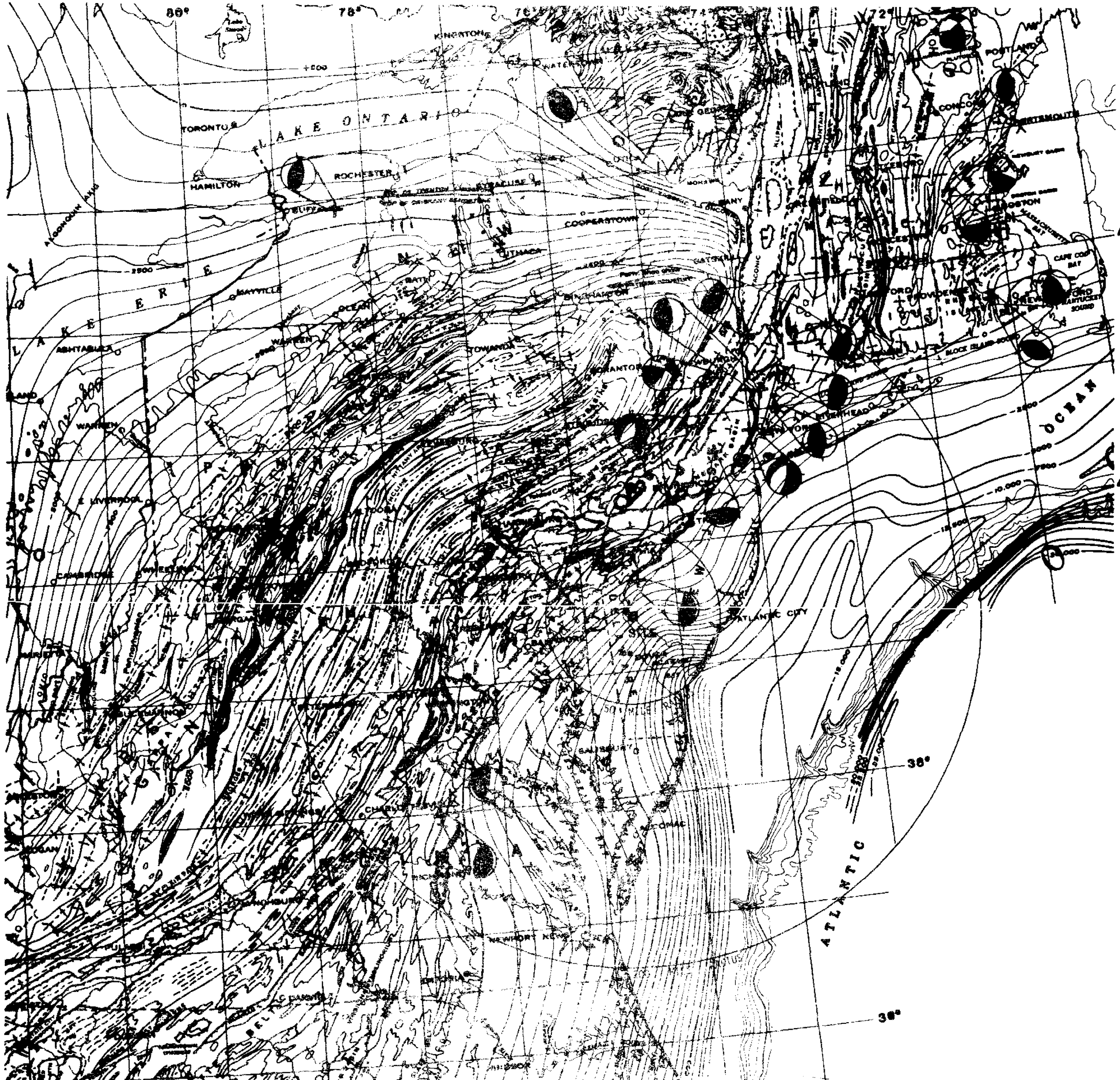
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

ISOSEISMAL MAP
OF FEBRUARY 28, 1973
WILMINGTON EVENT

UPDATED FSAR

FIGURE 2.5-24



N

LEGEND:

FOCAL MECHANISM SOLUTION
LOWER HEMISPHERE EQUAL AREA PROJECTION
P-WAVE COMPRESSION QUADRANTS SHOWN IN DARKENED AREA

REFERENCE:

THE BASE MAP WAS DERIVED FROM "TECTONIC MAP OF THE UNITED STATES EXCLUSIVE OF ALASKA AND HAWAII" BY THE UNITED STATES GEOLOGICAL SURVEY AND THE AMERICAN ASSOCIATION OF PETROLEUM GEOLOGISTS, 1962, 1:2,500,000

40 0 40 80 MILES

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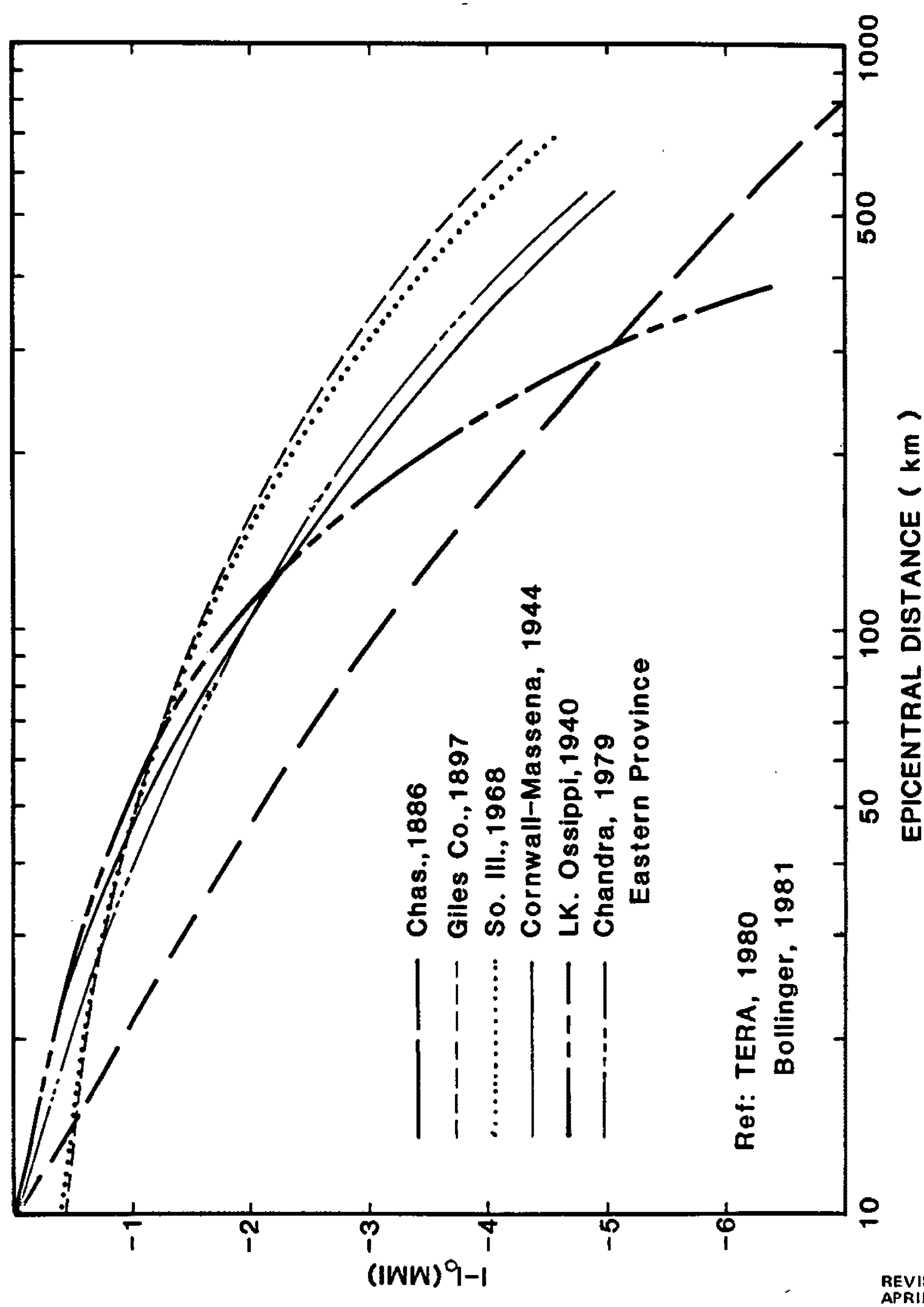
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

REGIONAL FOCAL MECHANISM SOLUTIONS

UPDATED FSAR

FIGURE 2.5-25



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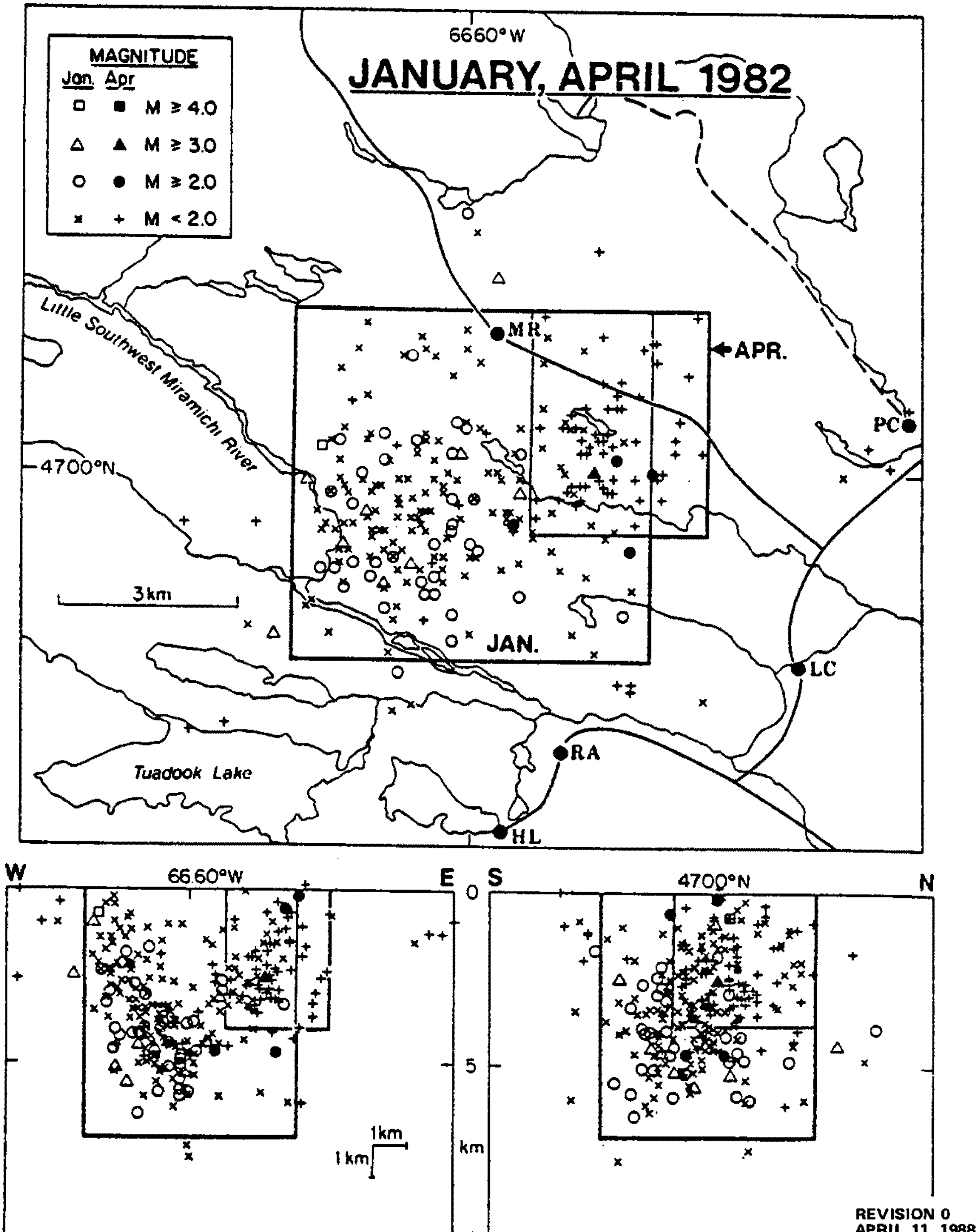
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

ATTENUATION RELATIONSHIPS

UPDATED FSAR

FIGURE 2.5-26



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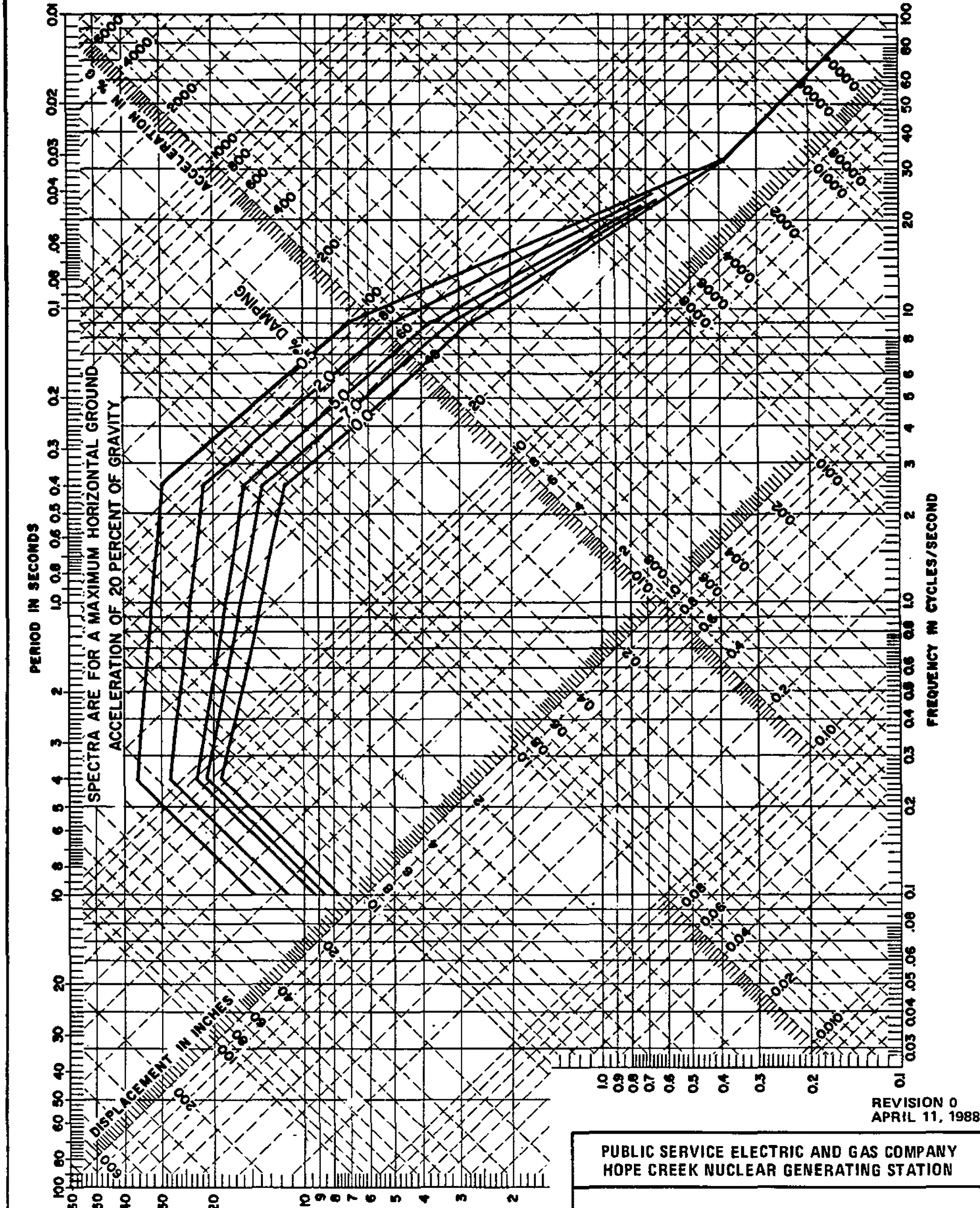
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

1982 MIRAMICHI
EARTHQUAKE SEQUENCE

UPDATED FSAR

FIGURE 2.5-26A

Reference: Aftershock Sequences of the 1982 Miramichi, New Brunswick Earthquakes; Wetmiller, et.al.,1983



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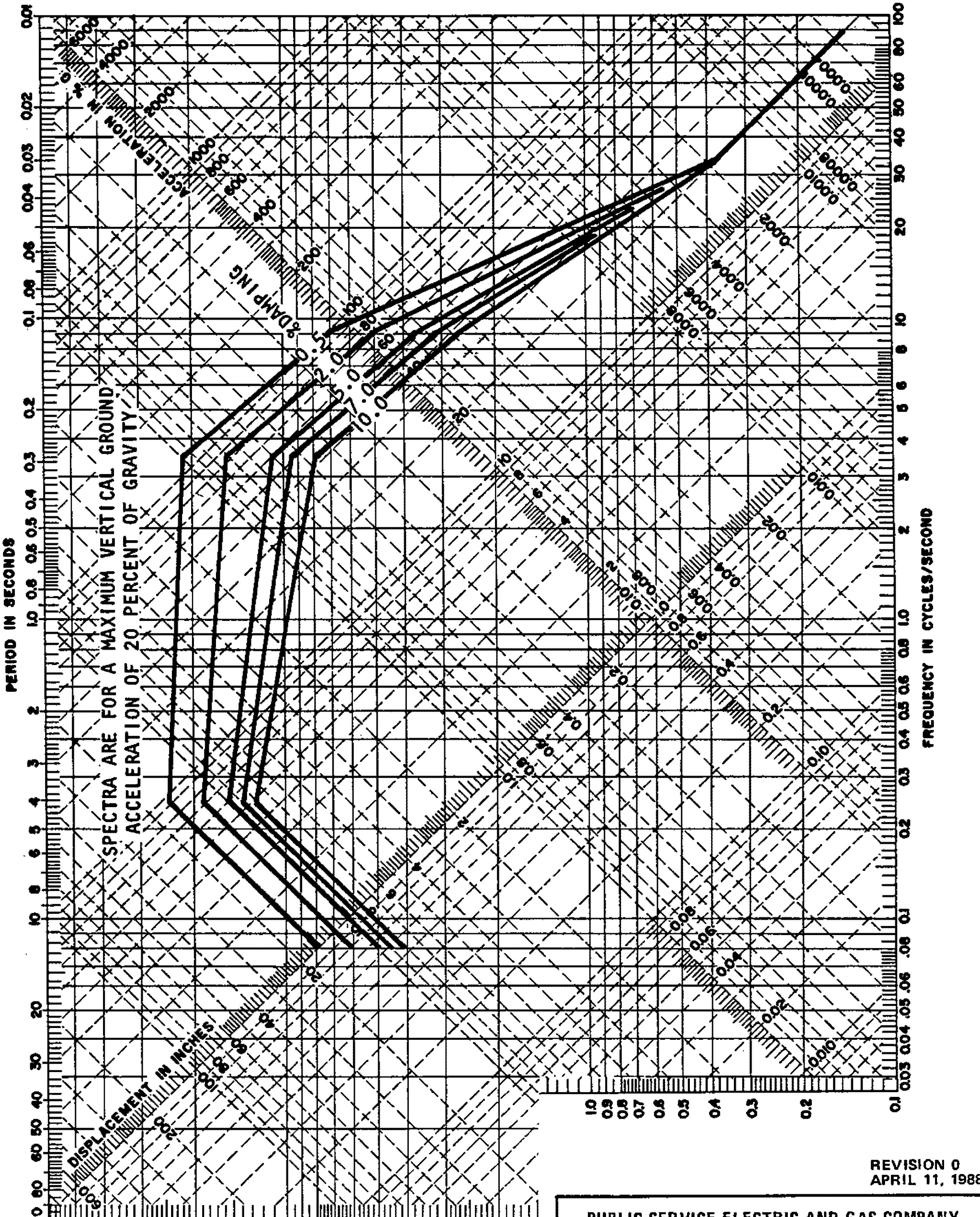
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

HORIZONTAL RESPONSE SPECTRA
SAFE SHUTDOWN EARTHQUAKE

UPDATED FSAR

FIGURE 2.5-27



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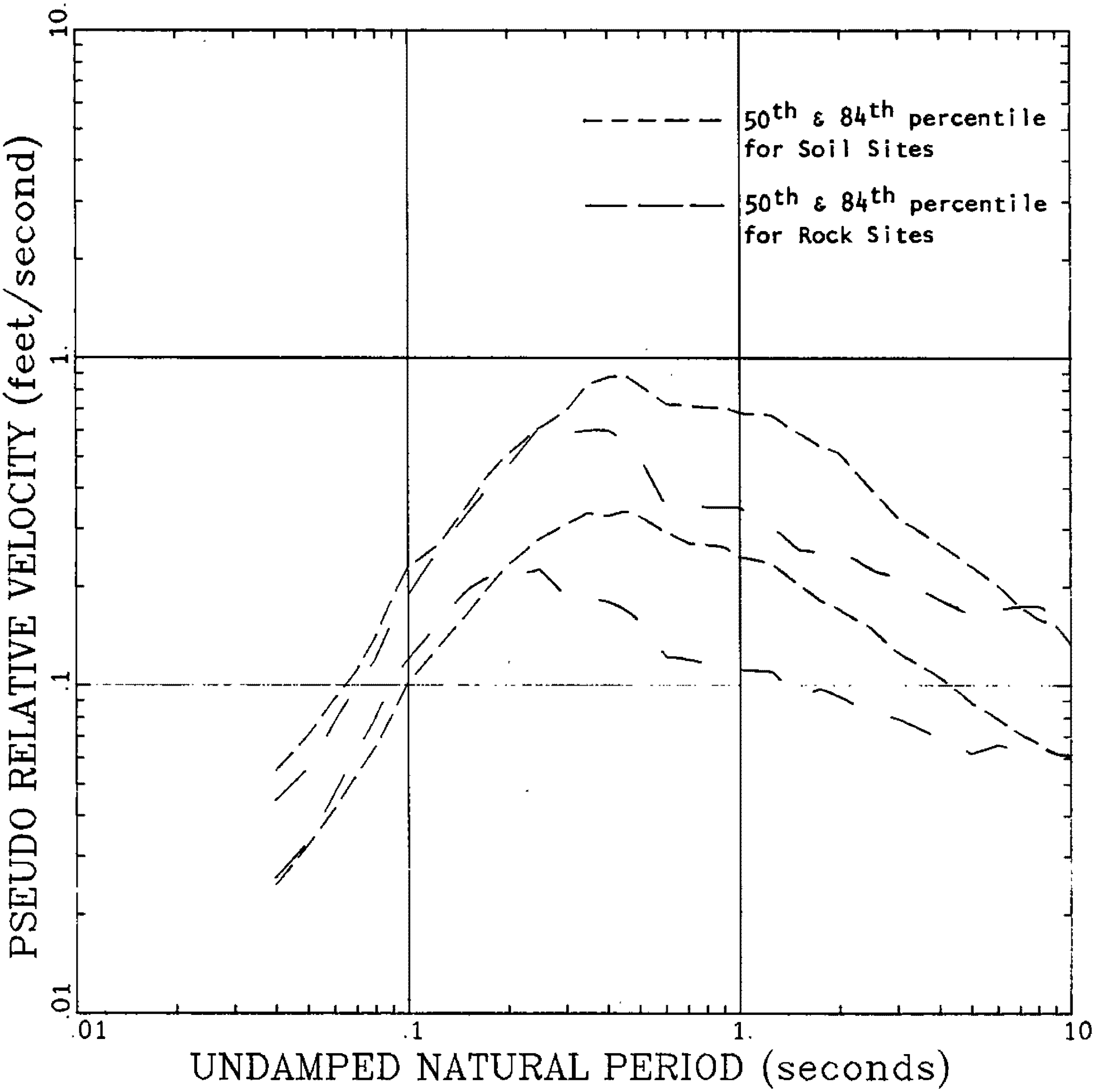
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

VERTICAL RESPONSE SPECTRA
SAFE SHUTDOWN EARTHQUAKE

UPDATED FSAR

FIGURE 2.5-28



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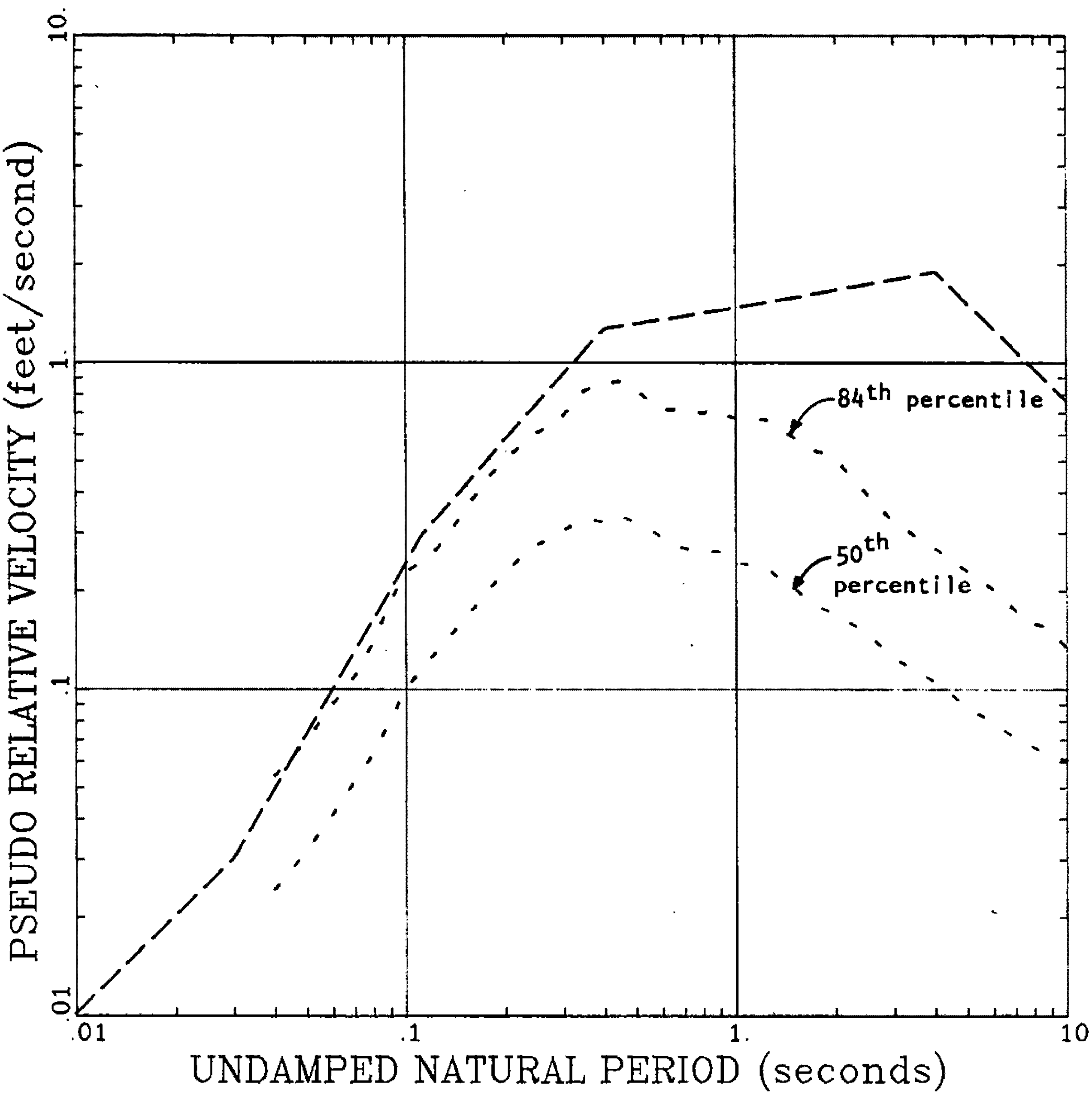
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

COMPARISON OF SITE SPECIFIC SPECTRA
FOR SOIL AND ROCK SITES
AT 50th AND 84th PERCENTILES
(DAMPING = 5.0)

UPDATED FSAR

FIGURE 2.5-28A



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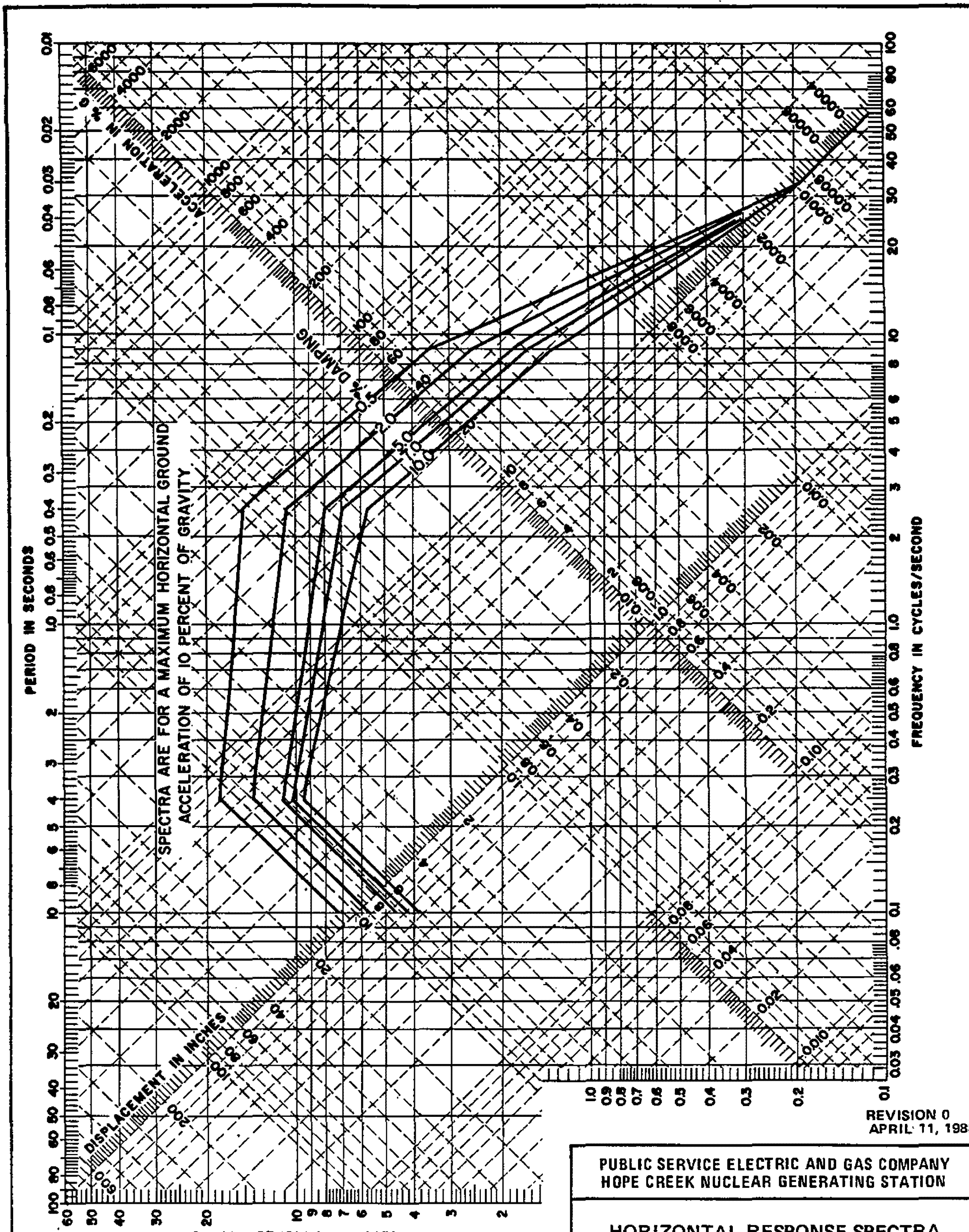
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

COMPARISON OF SITE SPECIFIC SPECTRA
FOR SOIL SITES AT 50th AND 84th PERCENTILE
TO HOPE CREEK HORIZONTAL
RESPONSE SPECTRUM AT 0.2g
(DAMPING = 5.0)

UPDATED FSAR

FIGURE 2.5-28B



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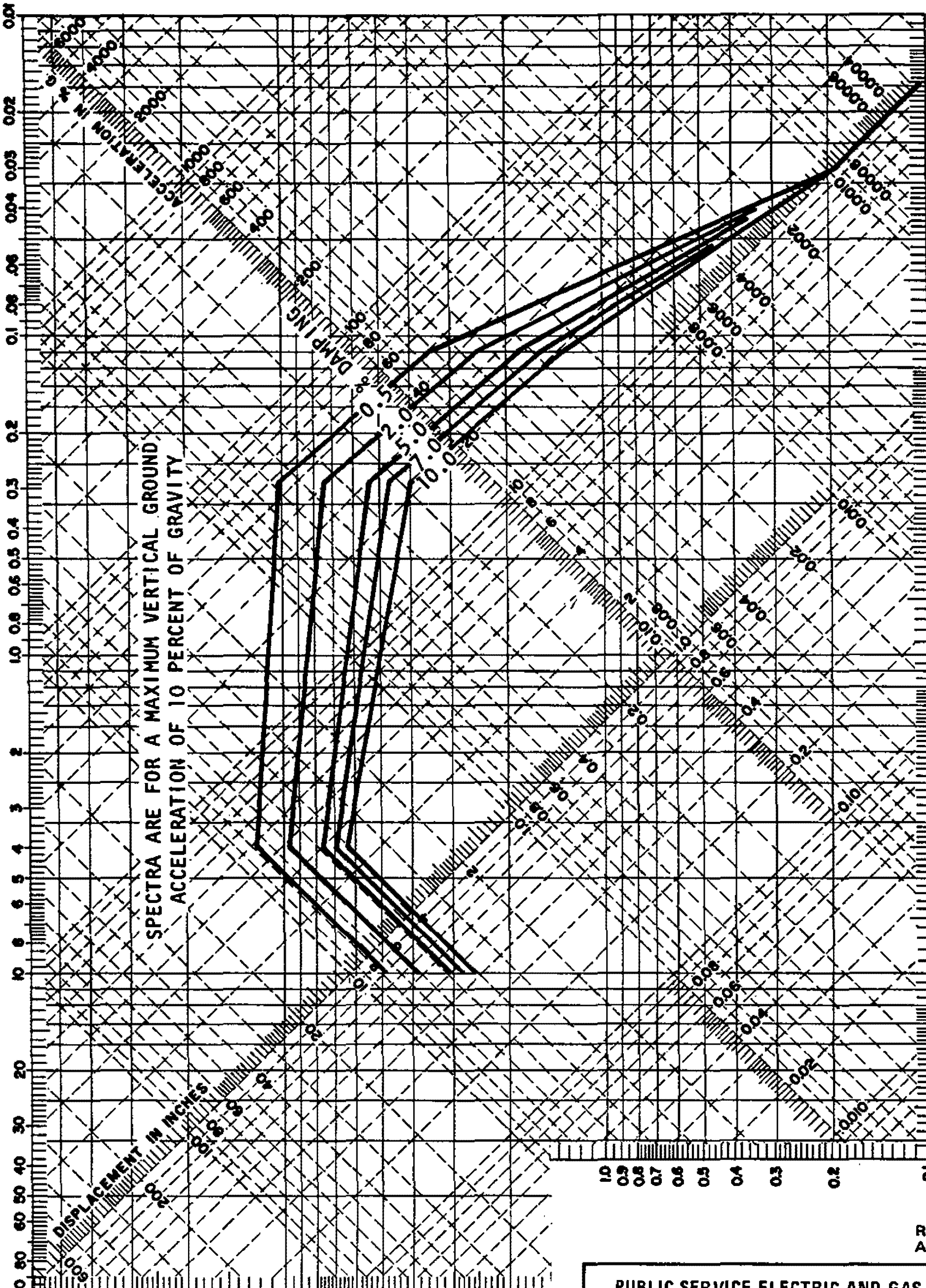
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

HORIZONTAL RESPONSE SPECTRA
OPERATING BASIS EARTHQUAKE

UPDATED FSAR

FIGURE 2.5-29



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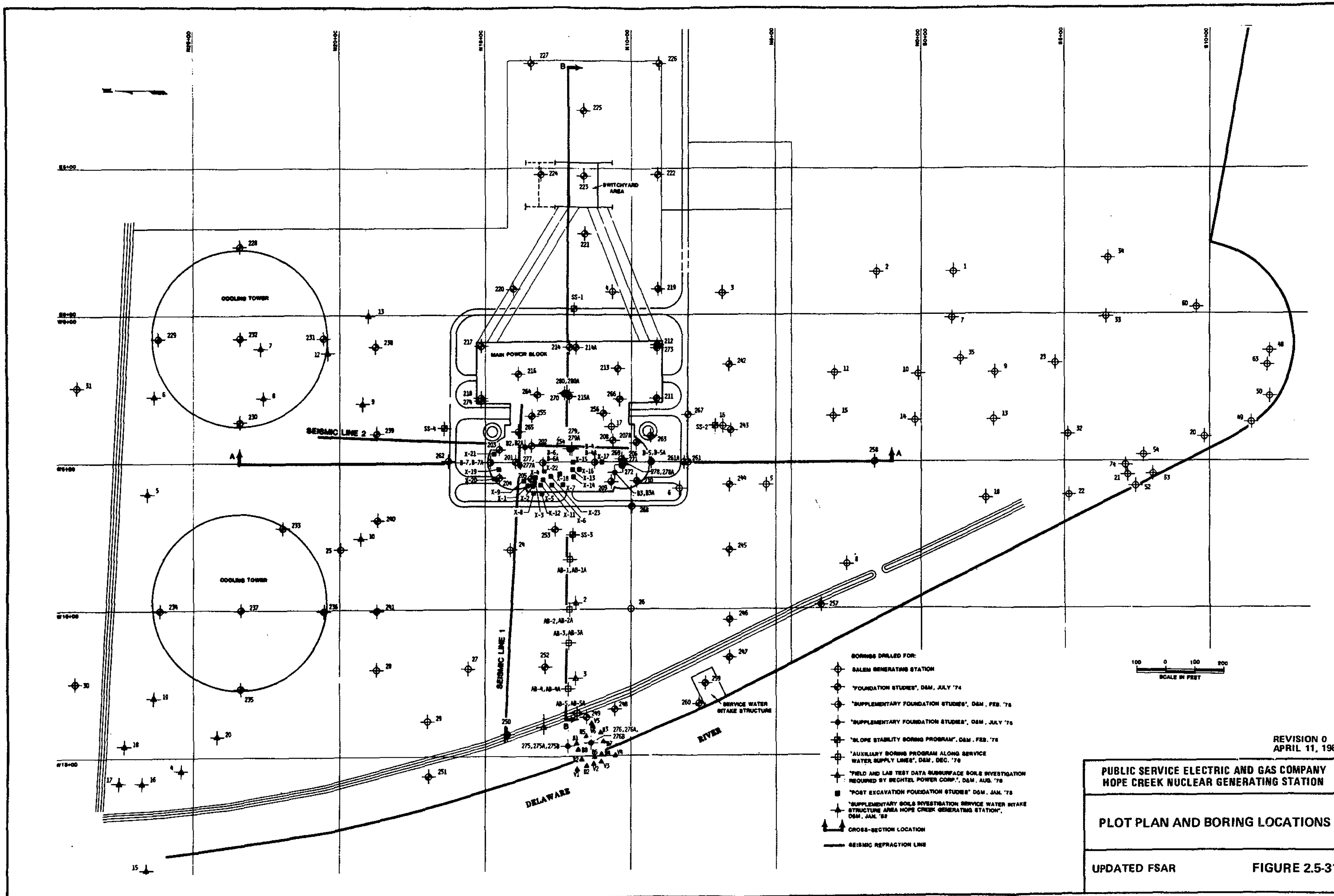
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

VERTICAL RESPONSE SPECTRA
OPERATING BASIS EARTHQUAKE

UPDATED FSAR

FIGURE 2.5-30



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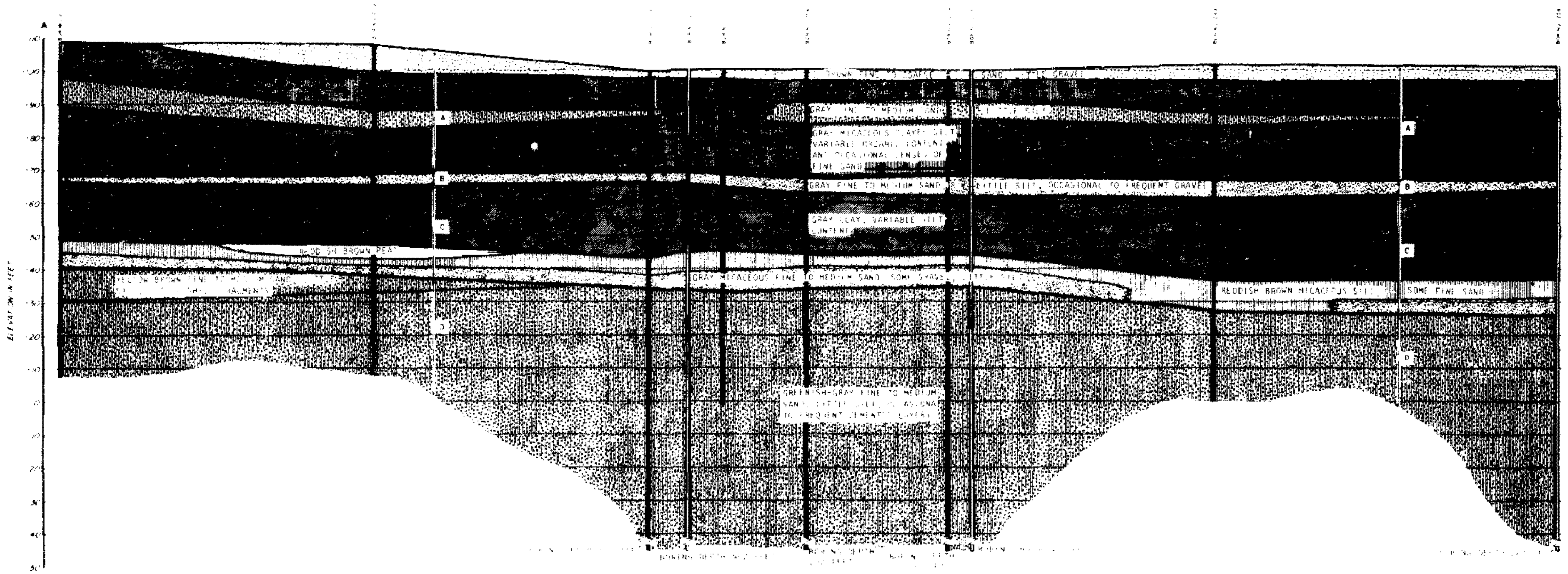
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

PLOT PLAN AND BORING LOCATIONS

UPDATED FSAR

FIGURE 2.5-31



SITE SUBSURFACE
SECTION A-A
HOPE CREEK GENERATING STATION

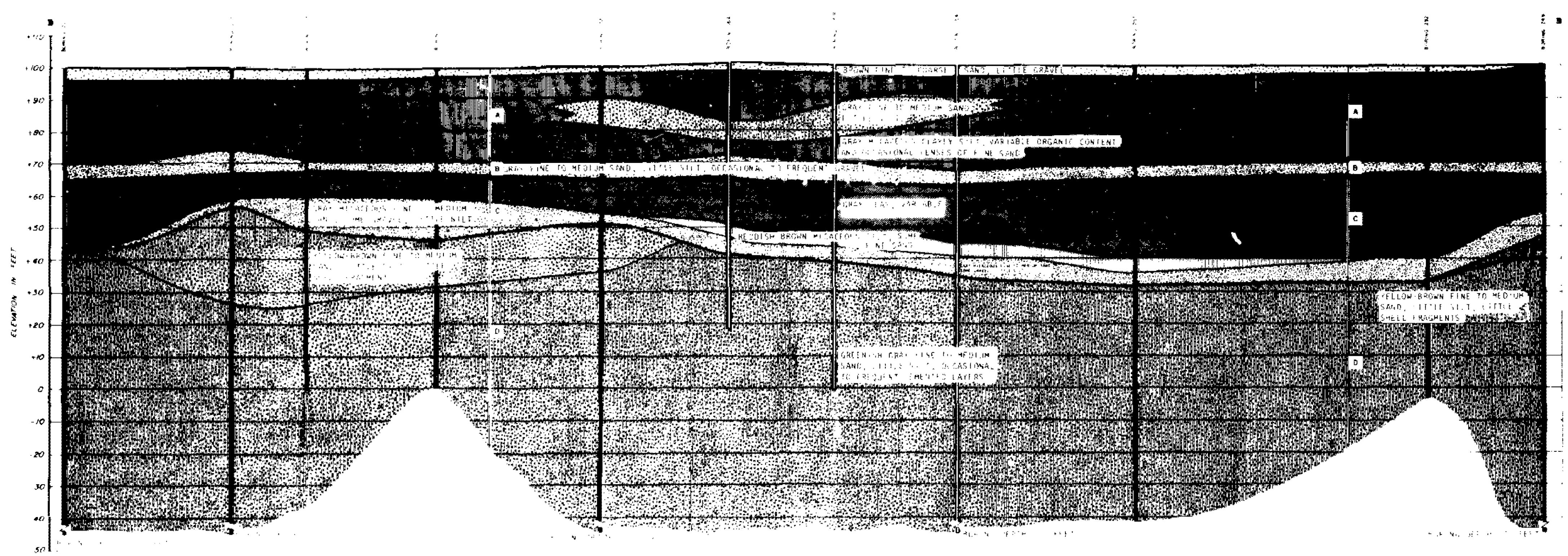
50 0 50 100
HORIZONTAL SCALE IN FEET

KEY

- BORINGS ON LINE
- BORINGS PROJECTED ON LINE
- A HYDRAULIC FILL
- B OLD RIVER BOTTOM
- C KIRKWOOD FORMATION
- D VINCENTOWN FORMATION

NOTES

1. VINCENTOWN FORMATION EXTENDS TO APPROXIMATE ELEVATION +30±
2. THE SUBSURFACE SECTIONS SHOWN REPRESENT OUR EVALUATION OF THE MOST PROBABLE CONDITIONS BASED UPON INTERPRETATION OF PRESENTLY AVAILABLE DATA. SOME VARIATIONS FROM THESE CONDITIONS MUST BE EXPECTED.
3. ELEVATIONS SHOWN REFER TO PUBLIC SERVICE PLANT DATUM
4. THE DISCUSSION IN THE TEXT OF THE REPORT IS NECESSARY FOR A PROPER UNDERSTANDING OF THE NATURE OF THE SUBSURFACE MATERIALS



SITE SUBSURFACE
SECTION B-B
HOPE CREEK GENERATING STATION

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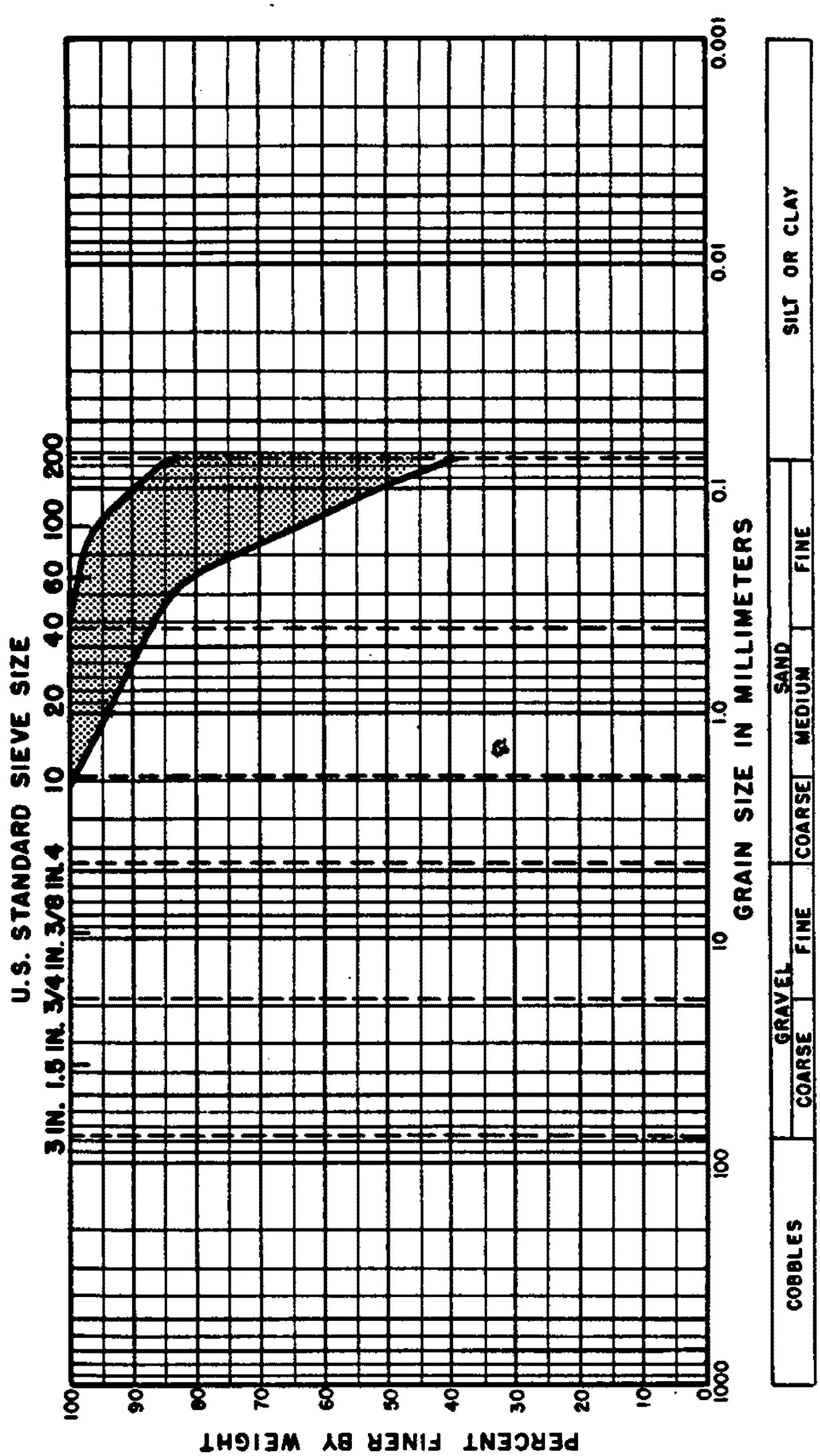
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

SITE SUBSURFACE SECTIONS

UPDATED FSAR

FIGURE 2.5-32



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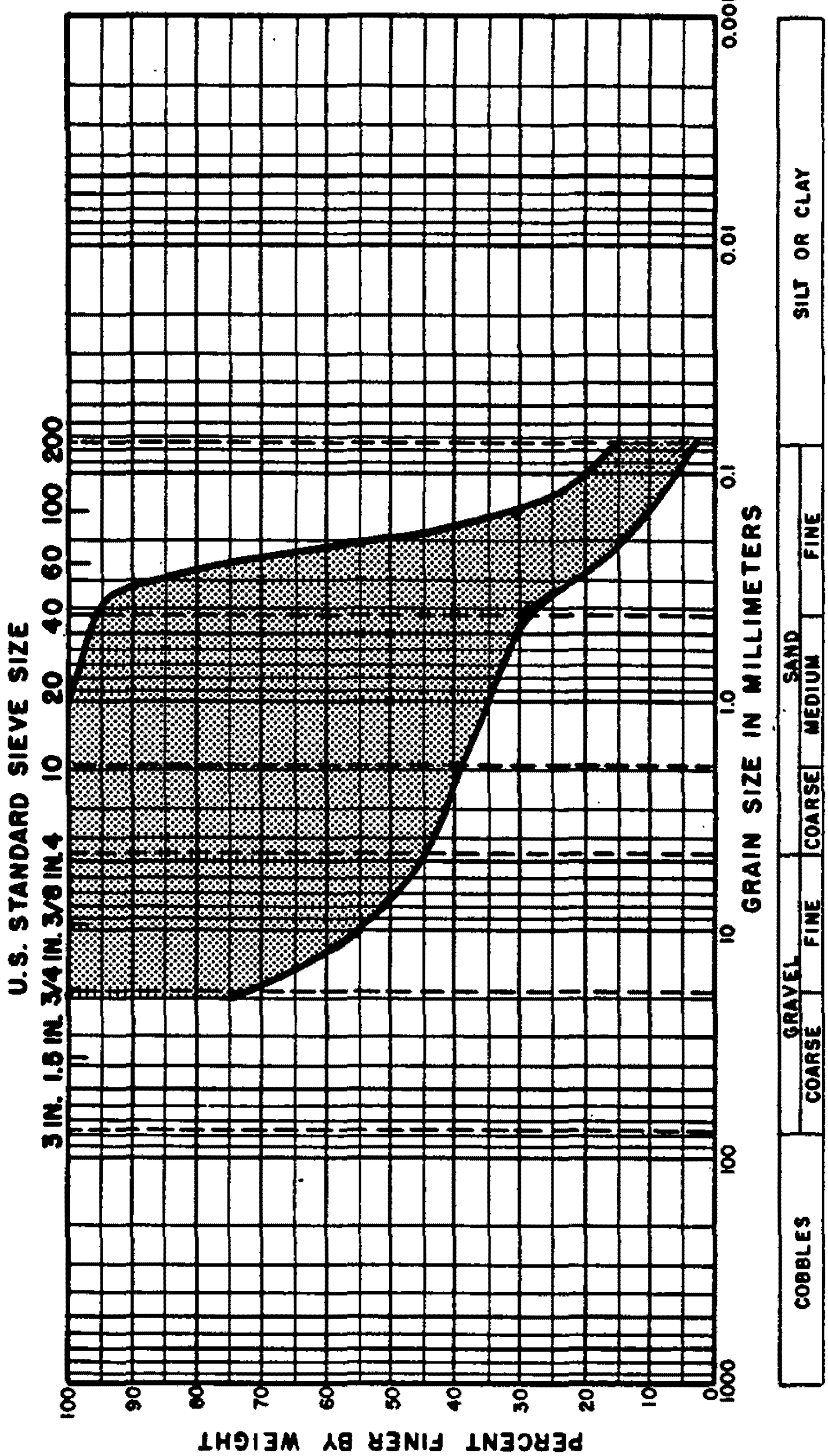
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

PARTICLE SIZE DISTRIBUTION
ENVELOPE FOR HYDRAULIC FILL

UPDATED FSAR

FIGURE 2.5-33



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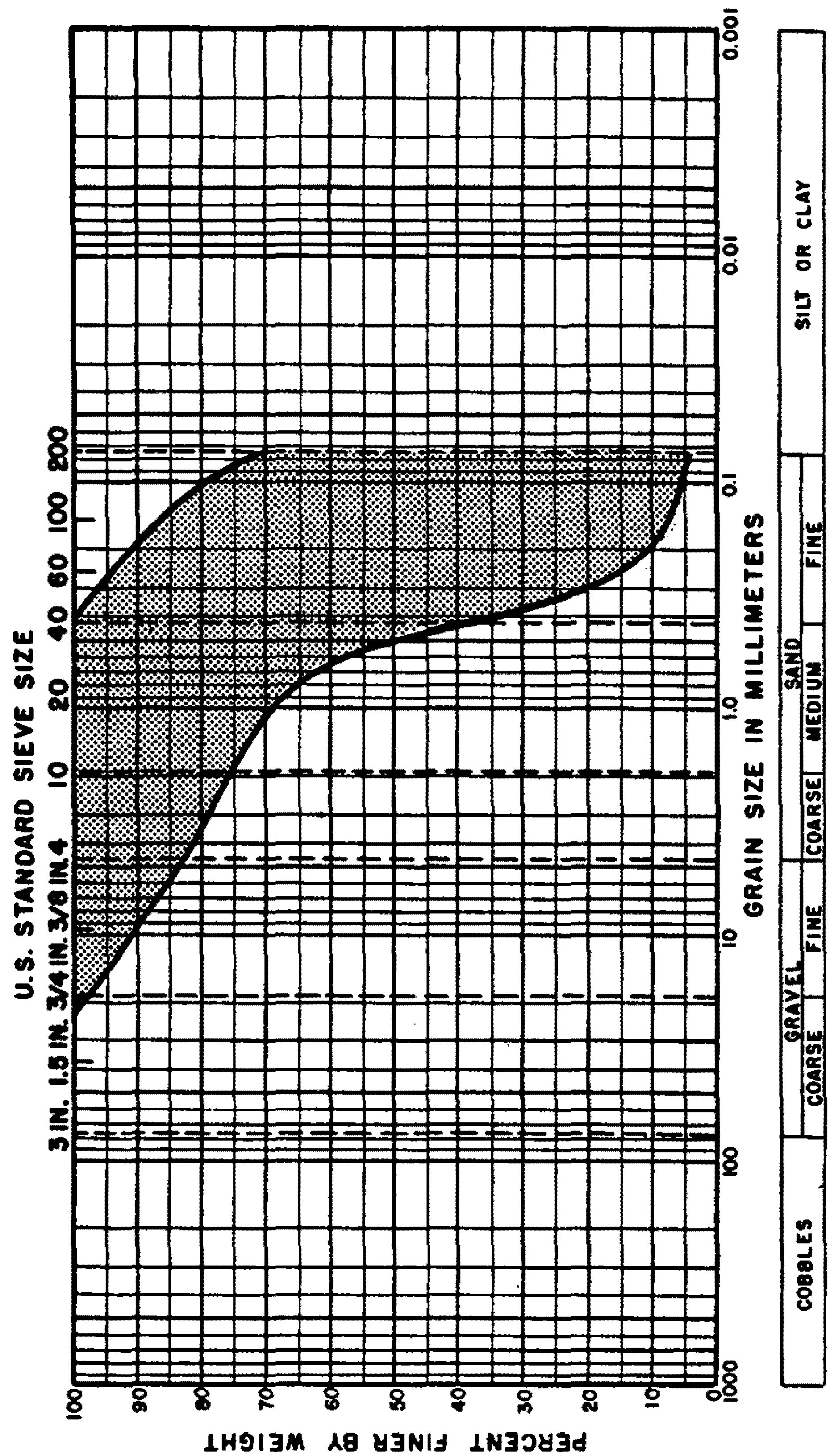
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

PARTICLE SIZE DISTRIBUTION
ENVELOPE FOR RIVER BOTTOM SANDS

UPDATED FSAR

FIGURE 2.5-34



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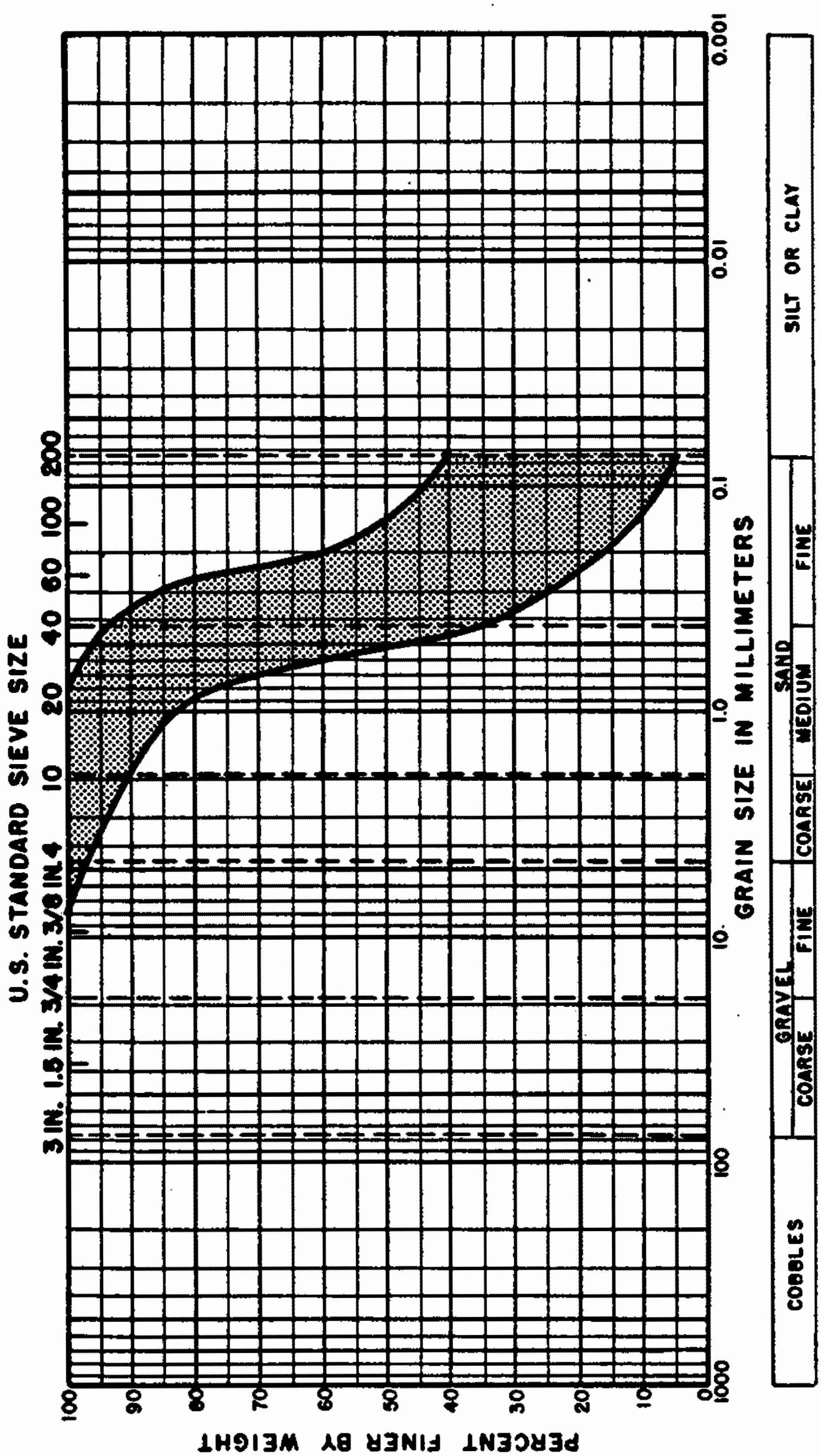
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

PARTICLE SIZE DISTRIBUTION
ENVELOPE FOR BASAL SANDS

UPDATED FSAR

FIGURE 2.5-35



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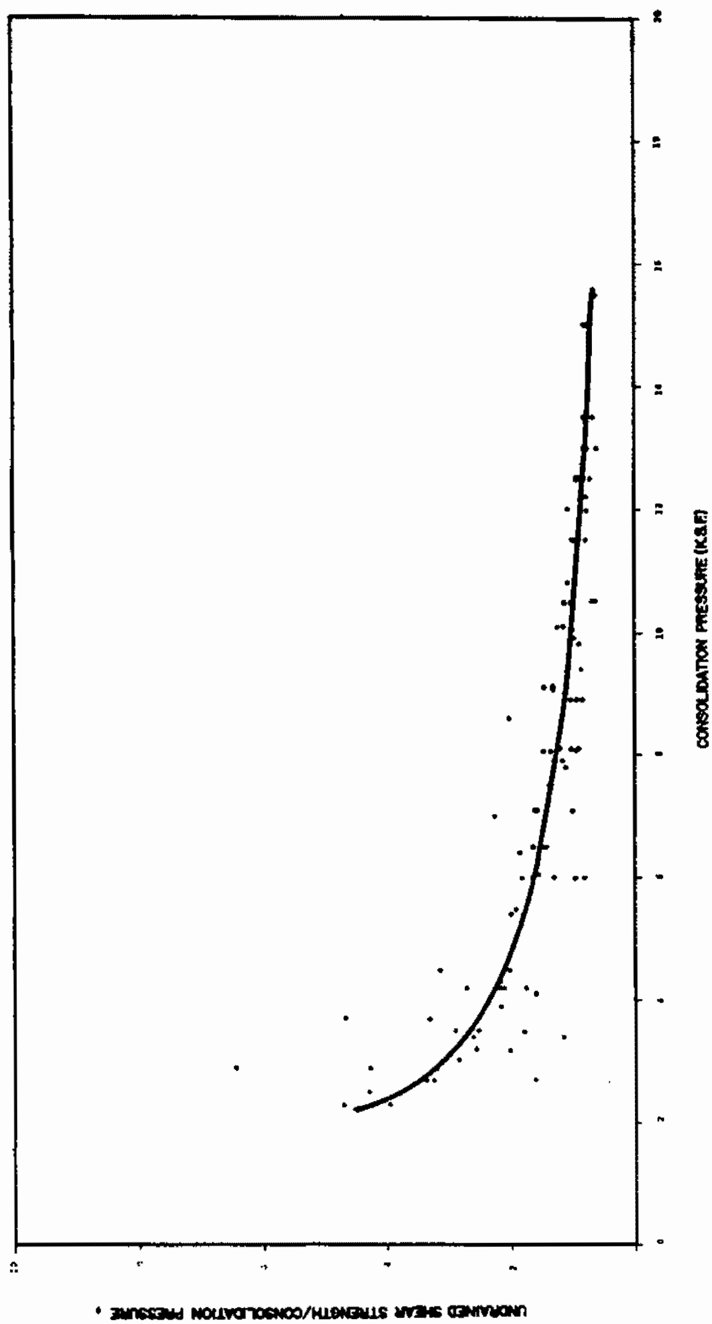
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

PARTICLE SIZE DISTRIBUTION
ENVELOPE FOR VINCENTOWN SANDS

UPDATED FSAR

FIGURE 2.5-36



REFERENCE:
PSE&G, JULY 75, REPORT, SUPPLEMENTARY FOUNDATION STUDIES, PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

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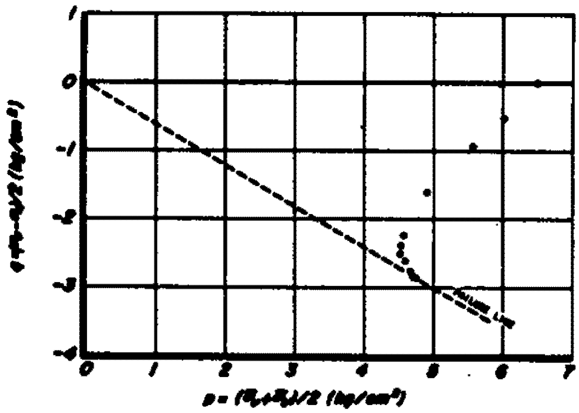
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

NORMALIZED UNDRAINED SHEAR STRENGTH vs CONSOLIDATION PRESSURE FOR VINCETOWN SANDS

UPDATED FSAR

FIGURE 2.5-37

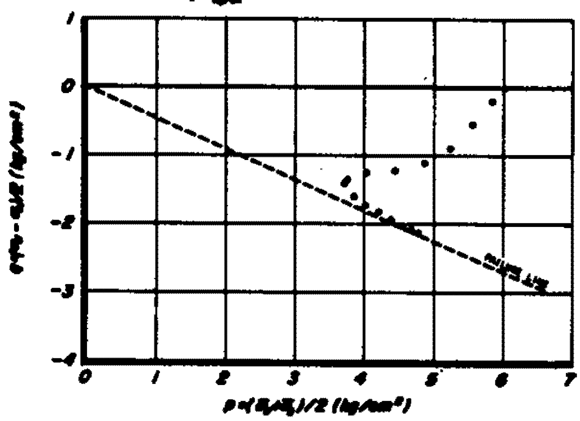


BORING - [illegible], SAMPLE - [illegible], DEPTHS [illegible]

TEST	CELL PRESS	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
CTXC	6.0*	19.6	10	31°	37°	0.30

kg/cm² = 2048.16 P.S.F.

* kg/cm²

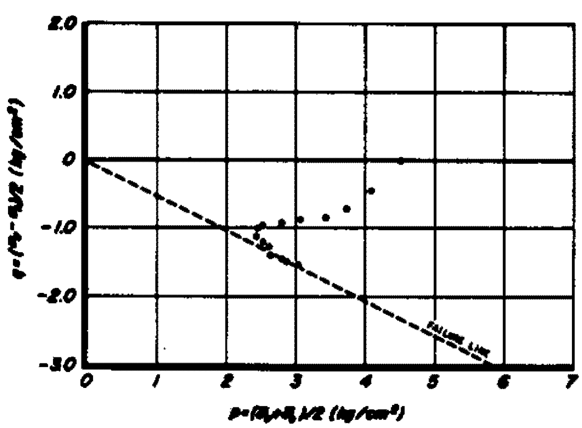


BORING - [illegible], SAMPLE - [illegible], DEPTHS [illegible]

TEST	CELL PRESS	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
CTXC	6.0*	21.6	10	34	[illegible]	0.311

kg/cm² = 2048.16 P.S.F.

* kg/cm²



BORING - [illegible], SAMPLE - [illegible], DEPTHS 65-67'

TEST	CELL PRESS	[illegible]	[illegible]	[illegible]	[illegible]
CTXC	4.5*	27.3	37°	30.6	0.342

kg/cm² = 2048.16 P.S.F.

* kg/cm²

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APRIL 11, 1988

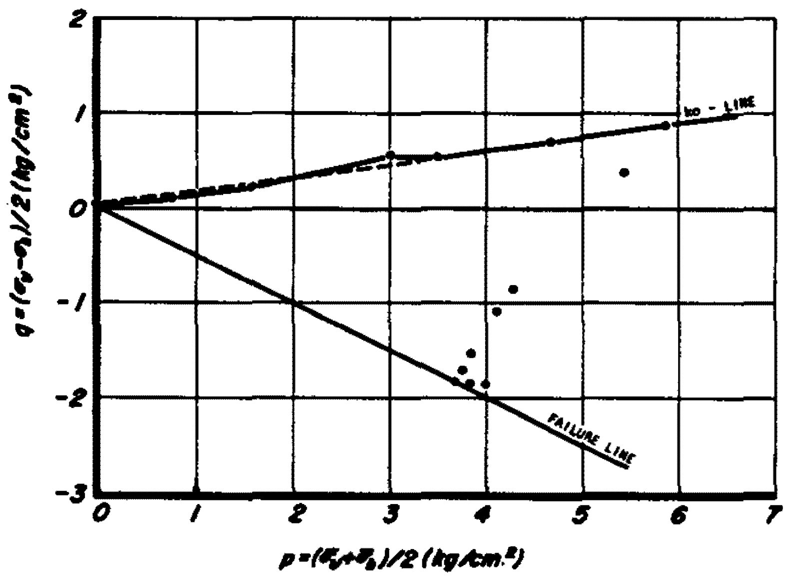
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

ISOTROPICALLY CONSOLIDATED UNDRAINED TRIAXIAL EXTENSION TESTS VINCENTOWN SANDS

UPDATED FSAR

FIGURE 2.5-38

REFERENCE:
PSE&G, MAY 23, 1974, REPORT, FOUNDATION STUDIES, PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.



BORING 216, SAMPLE 19, DEPTH 120-123'

TEST	CELL PRESS	$\%\epsilon_f$	k_o	ϕ_f	$\bar{\phi}_f$	σ_1/σ_{3c}
CTUE	5.0*	33.2	0.75	26°	29.5°	0.33

$1\,kg/cm^2 = 2048.16$ P.S.F.

* kg/cm^2

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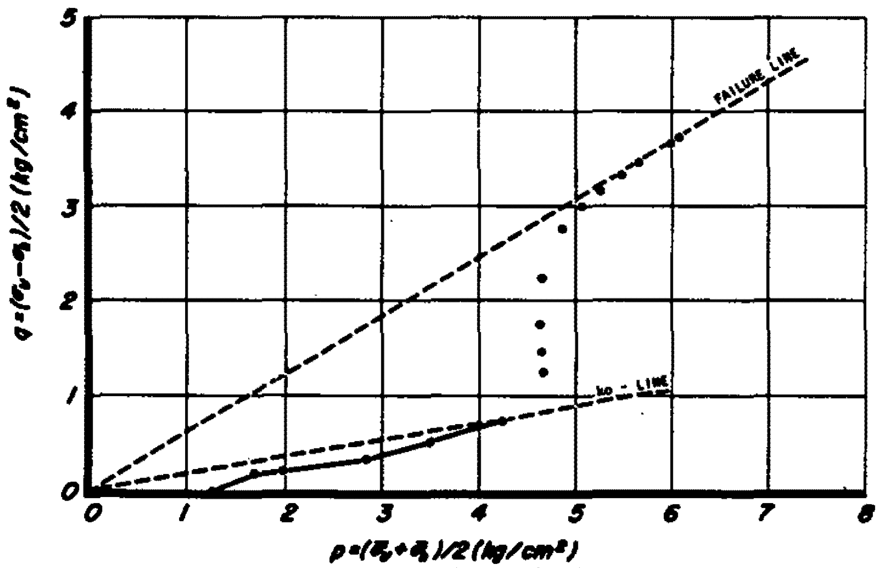
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

REFERENCE:
PSE&G, MAY 23, 1974, REPORT, FOUNDATION STUDIES, PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

ko – CONSOLIDATED UNDRAINED TRIAXIAL EXTENSION TEST VINCENTOWN SANDS

UPDATED FSAR

FIGURE 2.5-39

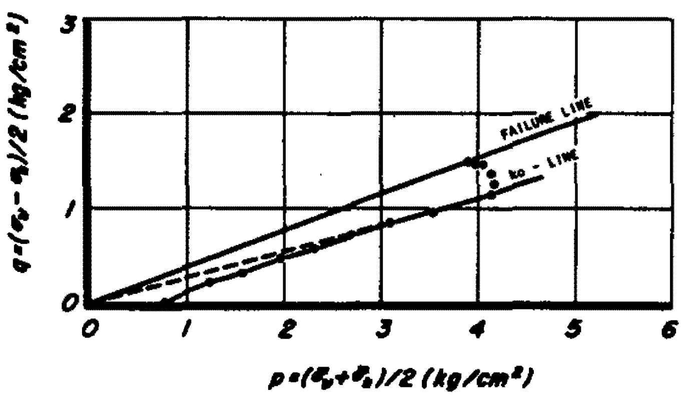


BORING - 259, SAMPLE - 14, DEPTHS 65 - 67'

TEST	CELL PRESS	w_f %	k_o	$\bar{\phi}_f$	$\bar{\phi}_f$	$q_f/\bar{\sigma}_{vc}$
Ck_oUC	5.0*	27	0.70	31.5	37.5	0.38

1kg/cm² = 2048.16 P.S.F.

* kg/cm²



BORING - 259, SAMPLE - 24, DEPTHS 35 - 37'

TEST	CELL PRESS	w_f %	k_o	$\bar{\phi}_f$	$\bar{\phi}_f$	$q_f/\bar{\sigma}_{vc}$
Ck_oUC	3.0*	43.9	0.57	20.2°	21.6°	0.275

1kg/cm² = 2048.16 P.S.F.

* kg/cm²

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APRIL 11, 1988

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HOPE CREEK NUCLEAR GENERATING STATION

ko – CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST KIRKWOOD CLAY

REFERENCE:
PSE&G, MAY 23, 1974, REPORT, FOUNDATION STUDIES, PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY, REPORT PREPARED BY DAMES & MOORE.

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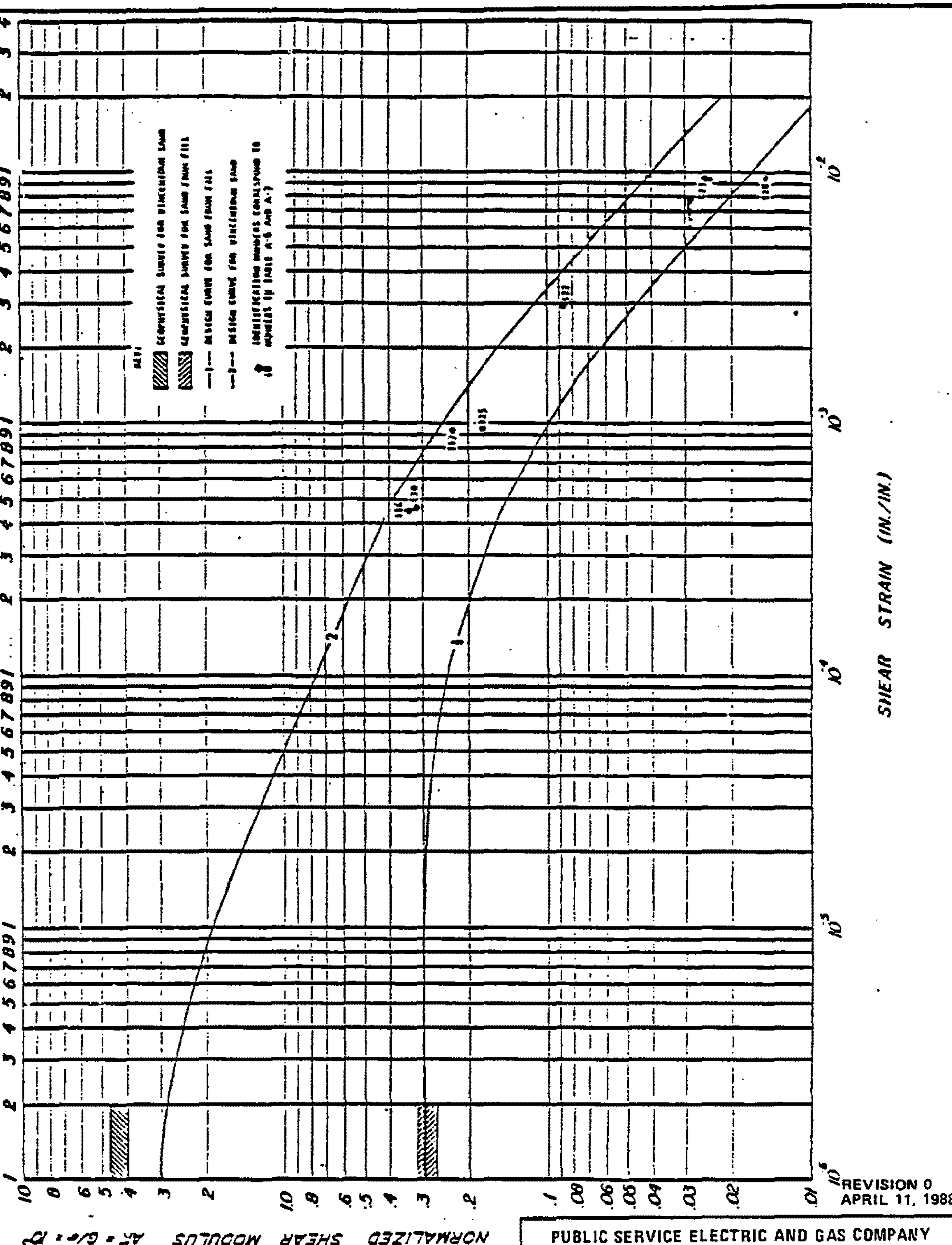
FIGURE 2.5-40



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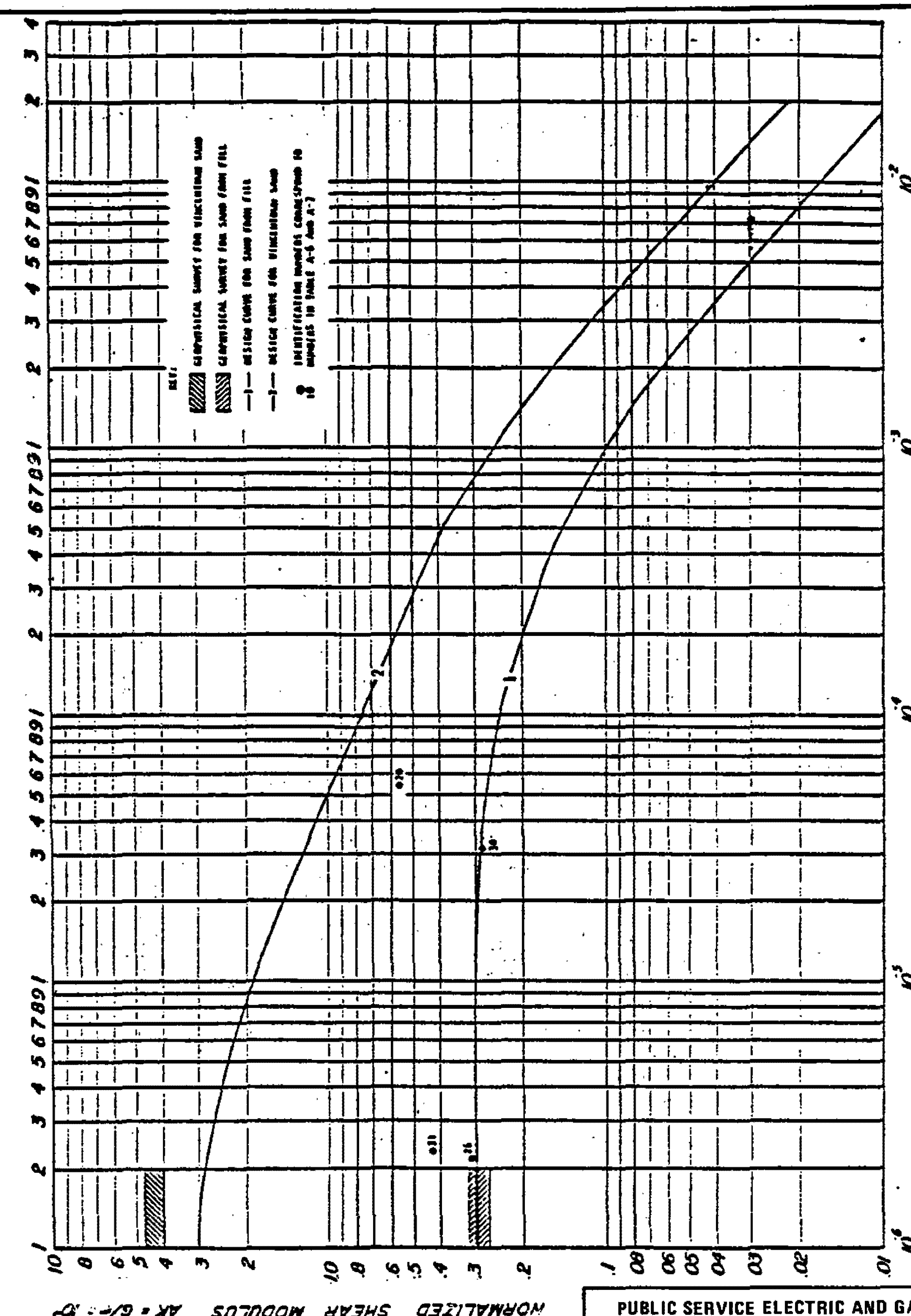
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

VARIATION OF NORMALIZED SHEAR MODULUS WITH SHEAR STRAIN FOR SAND (BASAL SANDS)

UPDATED FSAR

Sheet 1 of 4
FIGURE 2.5-41



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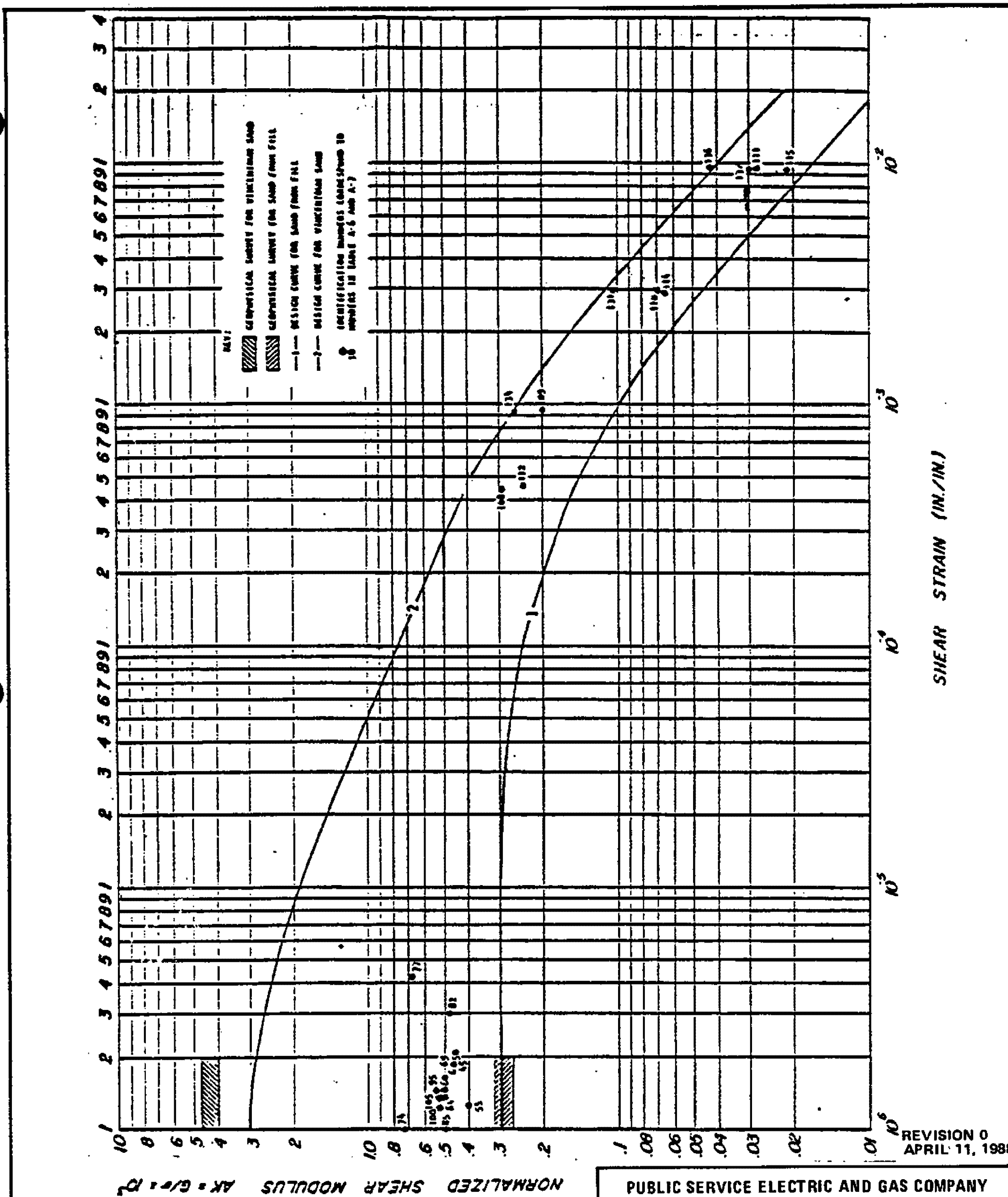
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

VARIATION OF NORMALIZED SHEAR MODULUS WITH SHEAR STRAIN FOR SAND (RIVER BOTTOM SANDS)

UPDATED FSAR

Sheet 2 of 4
FIGURE 2.5-41



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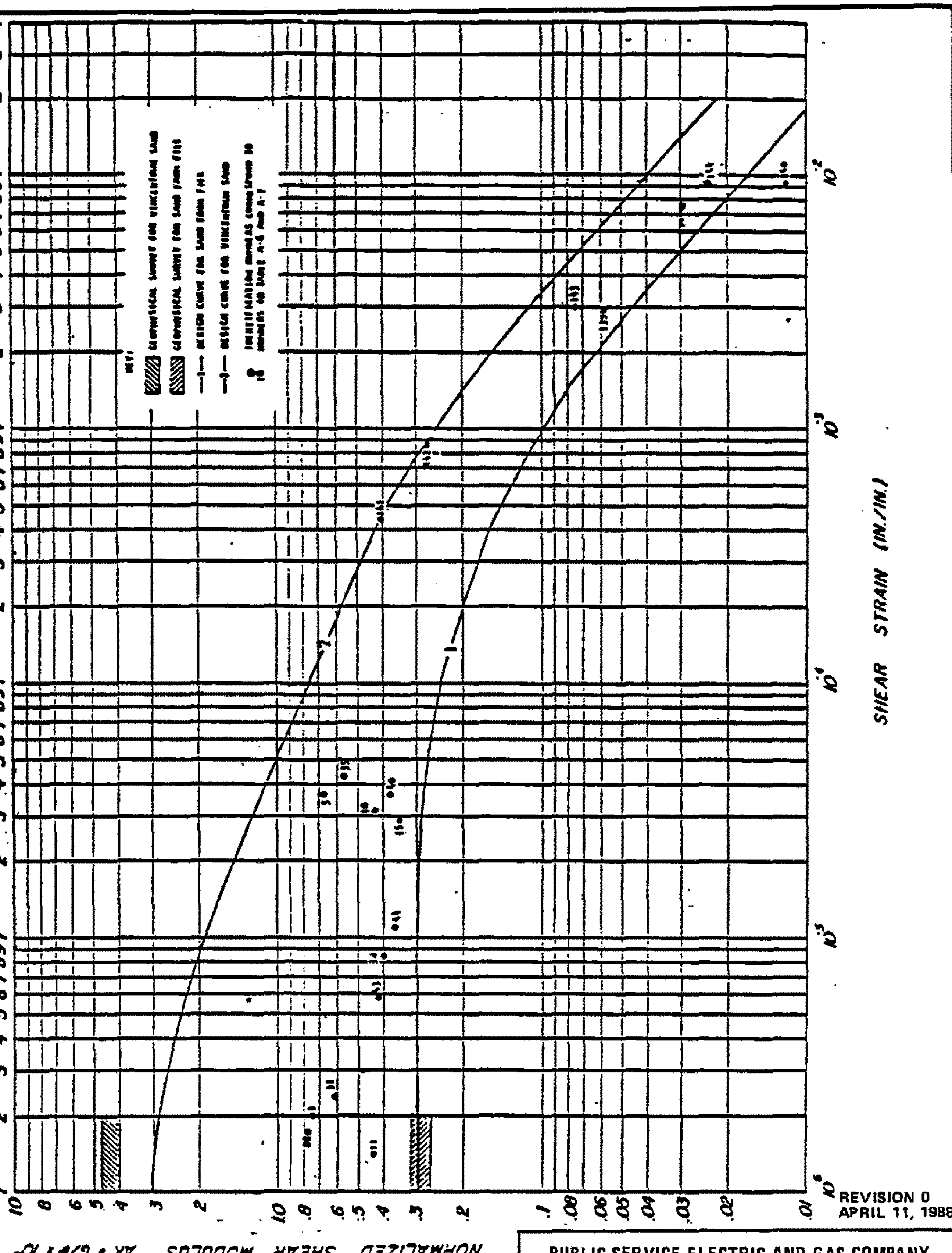
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

VARIATION OF NORMALIZED SHEAR MODULUS WITH SHEAR STRAIN FOR SAND (HORNERSTOWN SANDS)

UPDATED FSAR

Sheet 3 of 4
FIGURE 2.5-41



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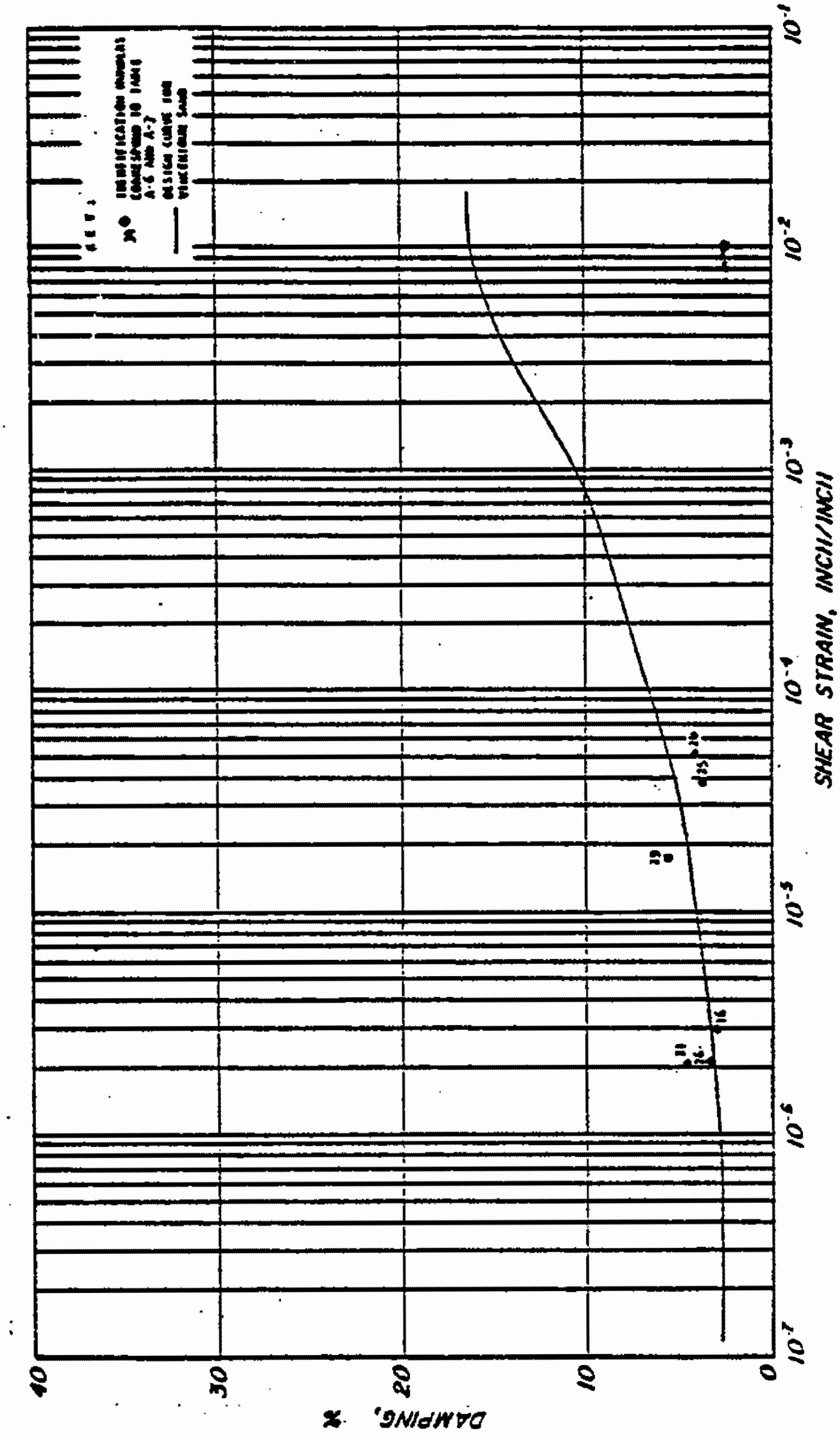
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

VARIATION OF NORMALIZED SHEAR MODULUS WITH SHEAR STRAIN FOR SAND (VINCENTOWN SANDS)

UPDATED FSAR

Sheet 4 of 4
FIGURE 2.5-41



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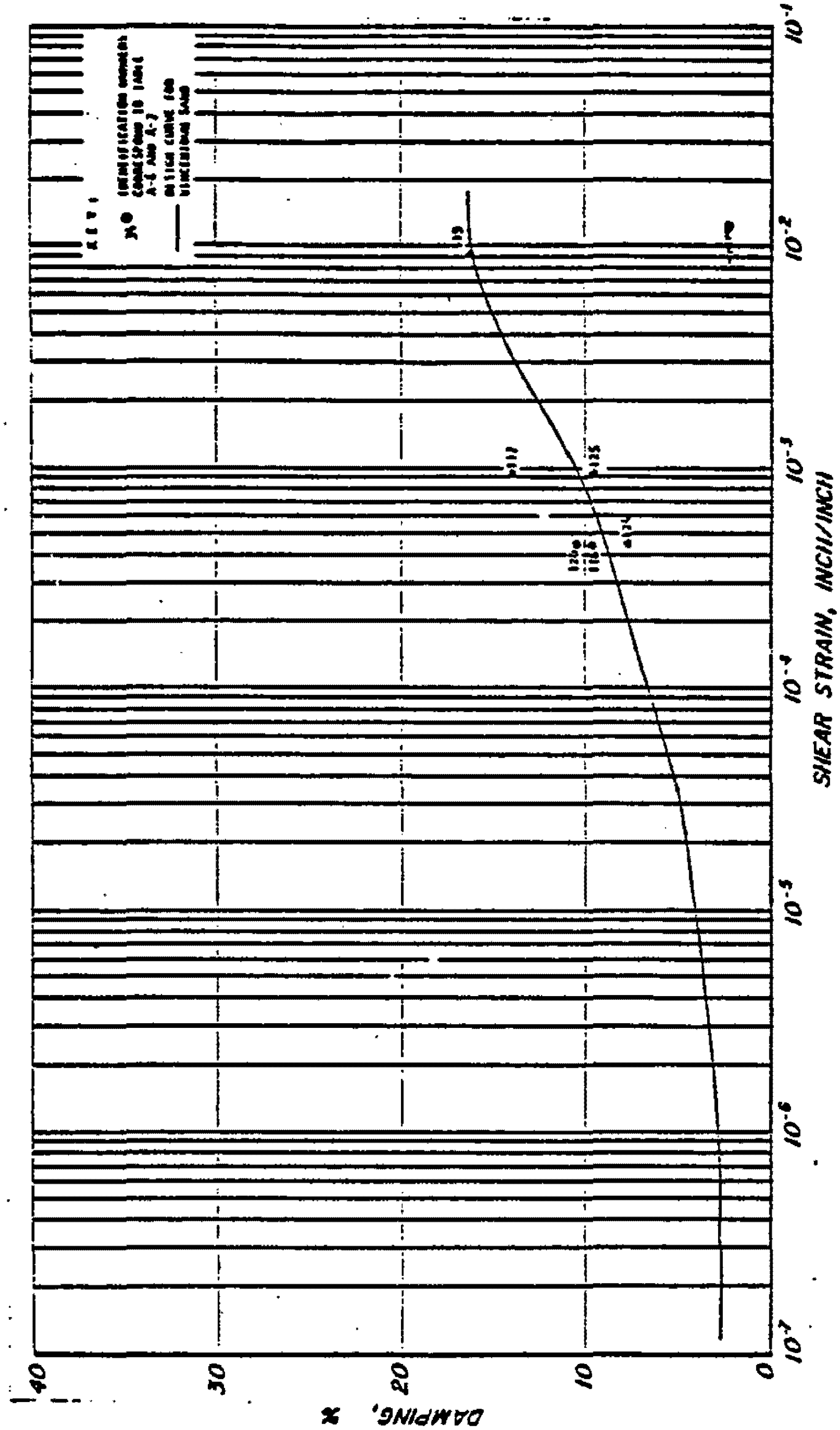
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

VARIATION OF DAMPING WITH SHEAR STRAIN FOR SAND (RIVER BOTTOM SANDS)

UPDATED FSAR

Sheet 1 of 4
FIGURE 2.5-42



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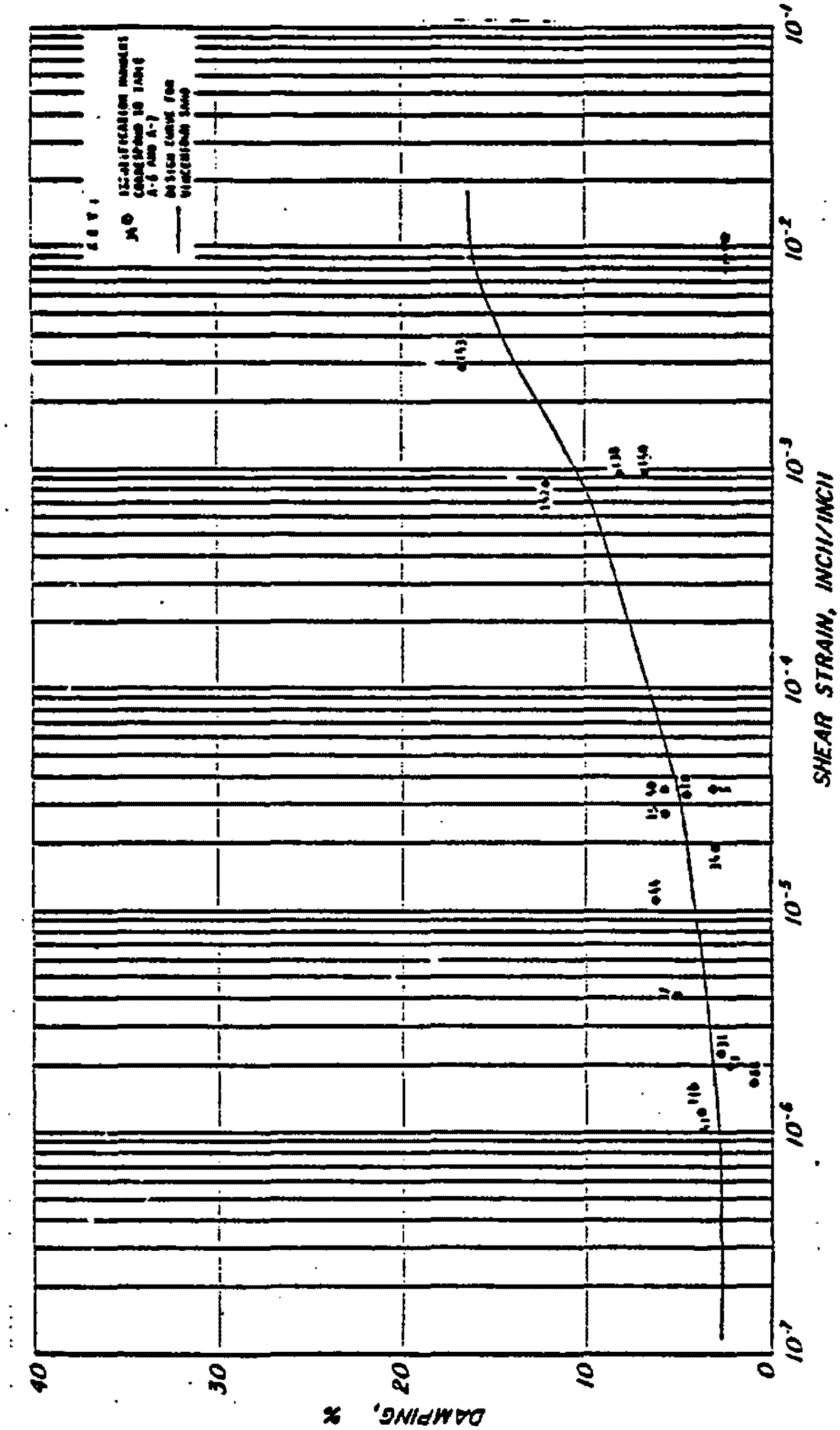
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

VARIATION OF DAMPING WITH SHEAR STRAIN FOR SAND (BASAL SANDS)

UPDATED FSAR

Sheet 2 of 4
FIGURE 2.5-42



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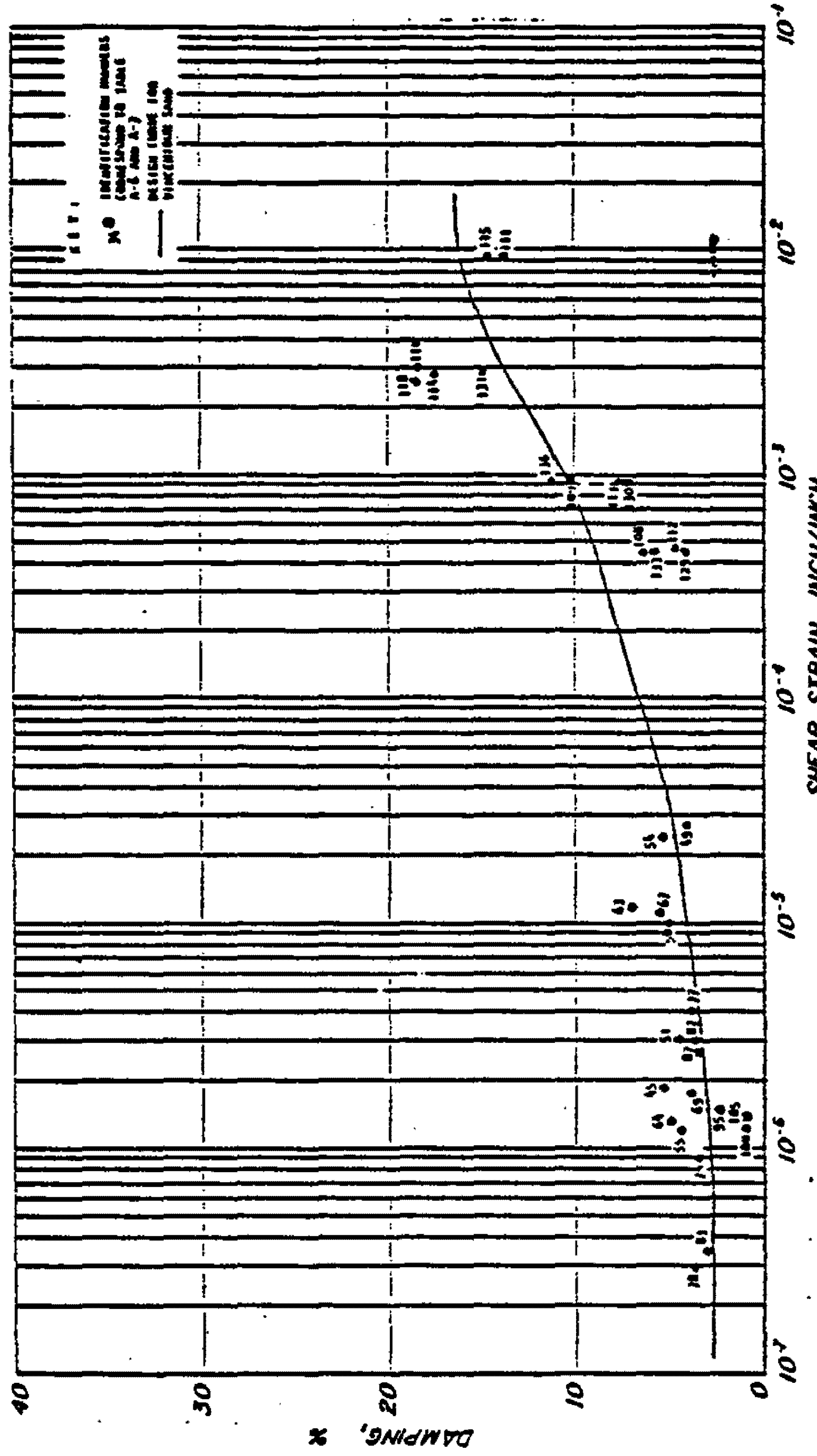
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

VARIATION OF DAMPING WITH SHEAR STRAIN FOR SAND (VINCENTOWN SANDS)

UPDATED FSAR

Sheet 3 of 4
FIGURE 2.5-42



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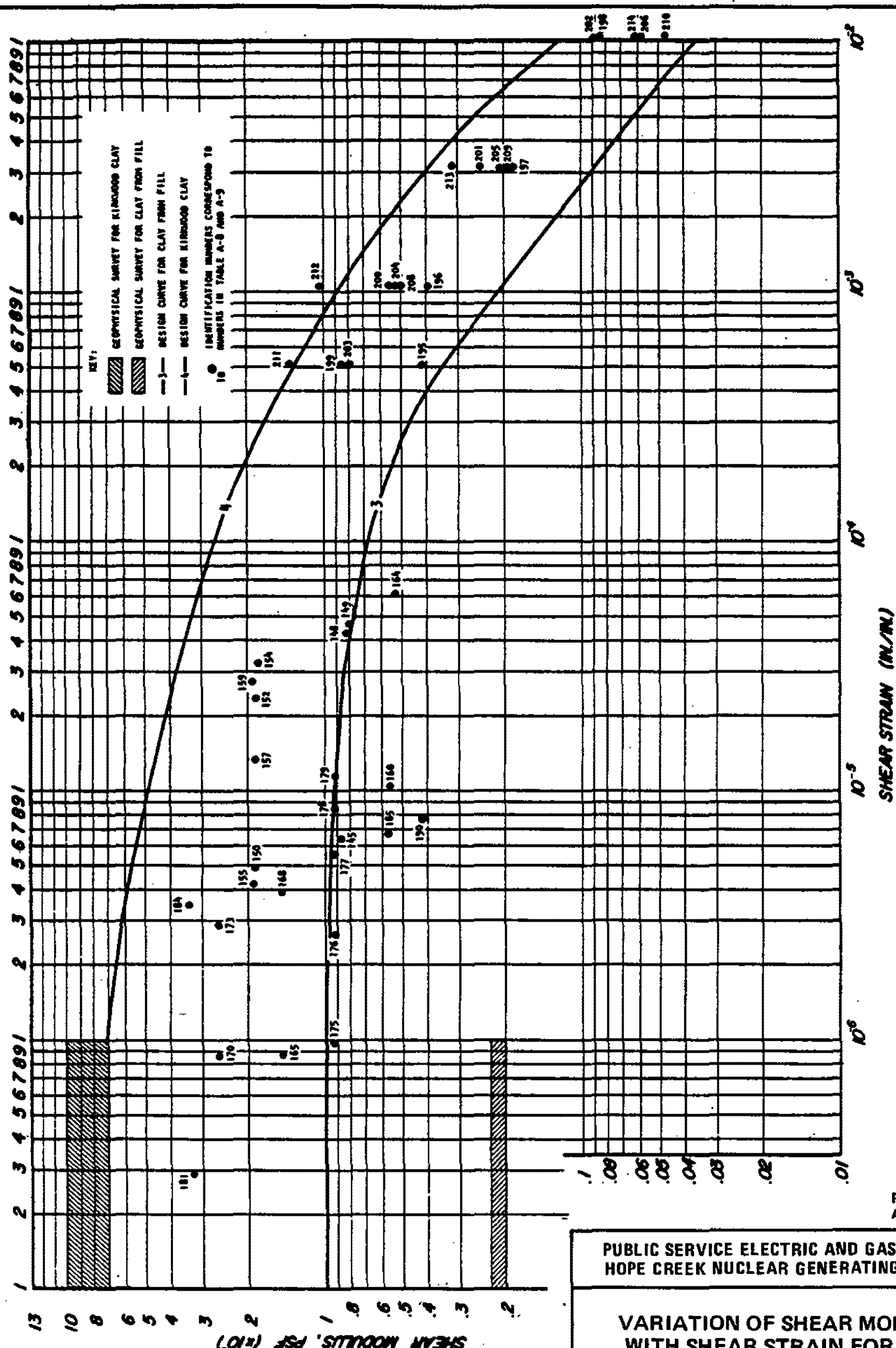
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

VARIATION OF DAMPING WITH
SHEAR STRAIN FOR SAND
(HORNERSTOWN SANDS)

UPDATED FSAR

Sheet 4 of 4
FIGURE 2.5-42



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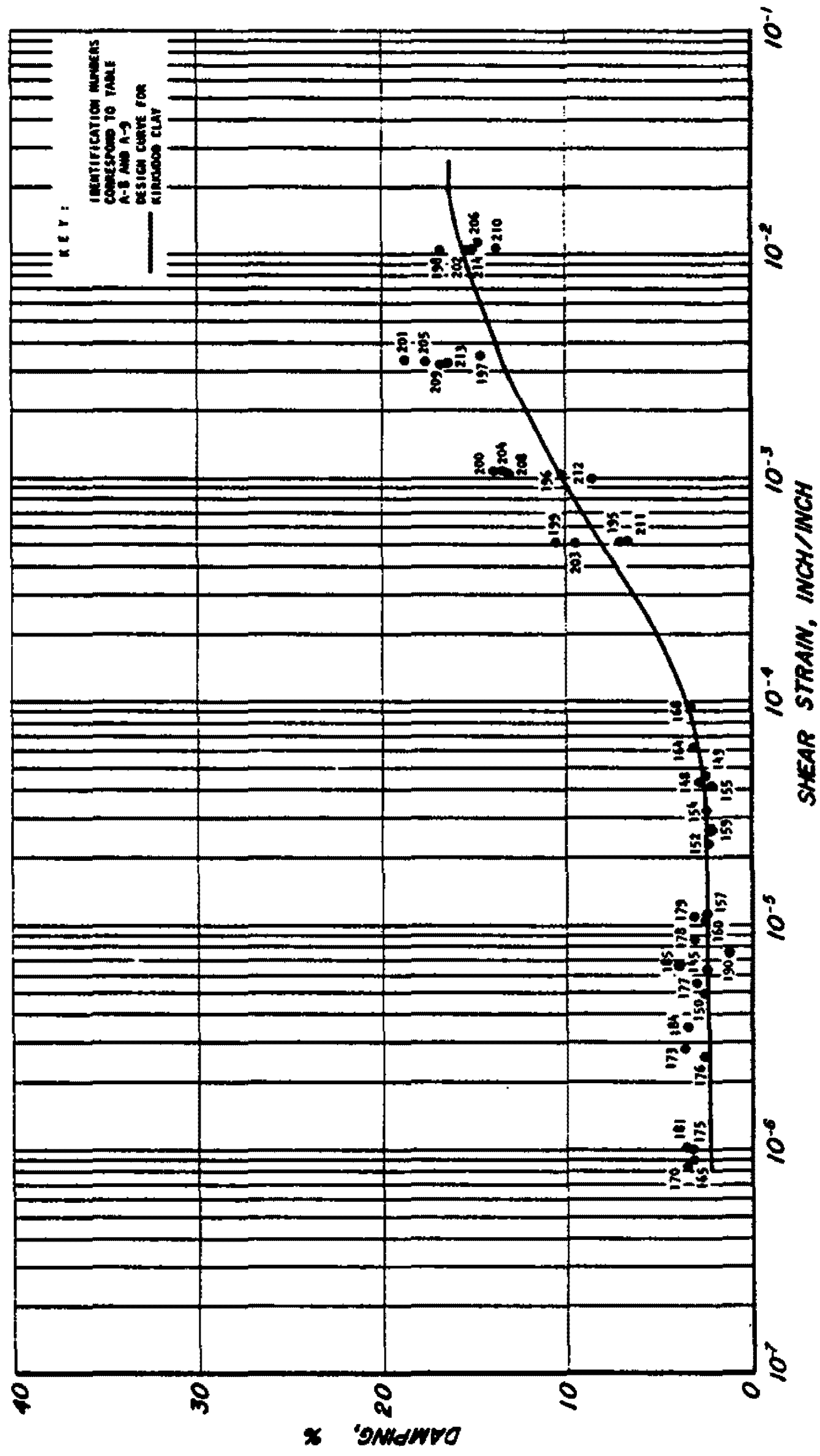
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

VARIATION OF SHEAR MODULUS WITH SHEAR STRAIN FOR CLAY

REFERENCE:
PSE&G, MAY 23, 1974, REPORT, FOUNDATION STUDIES, PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

UPDATED FSAR

FIGURE 2.5-43



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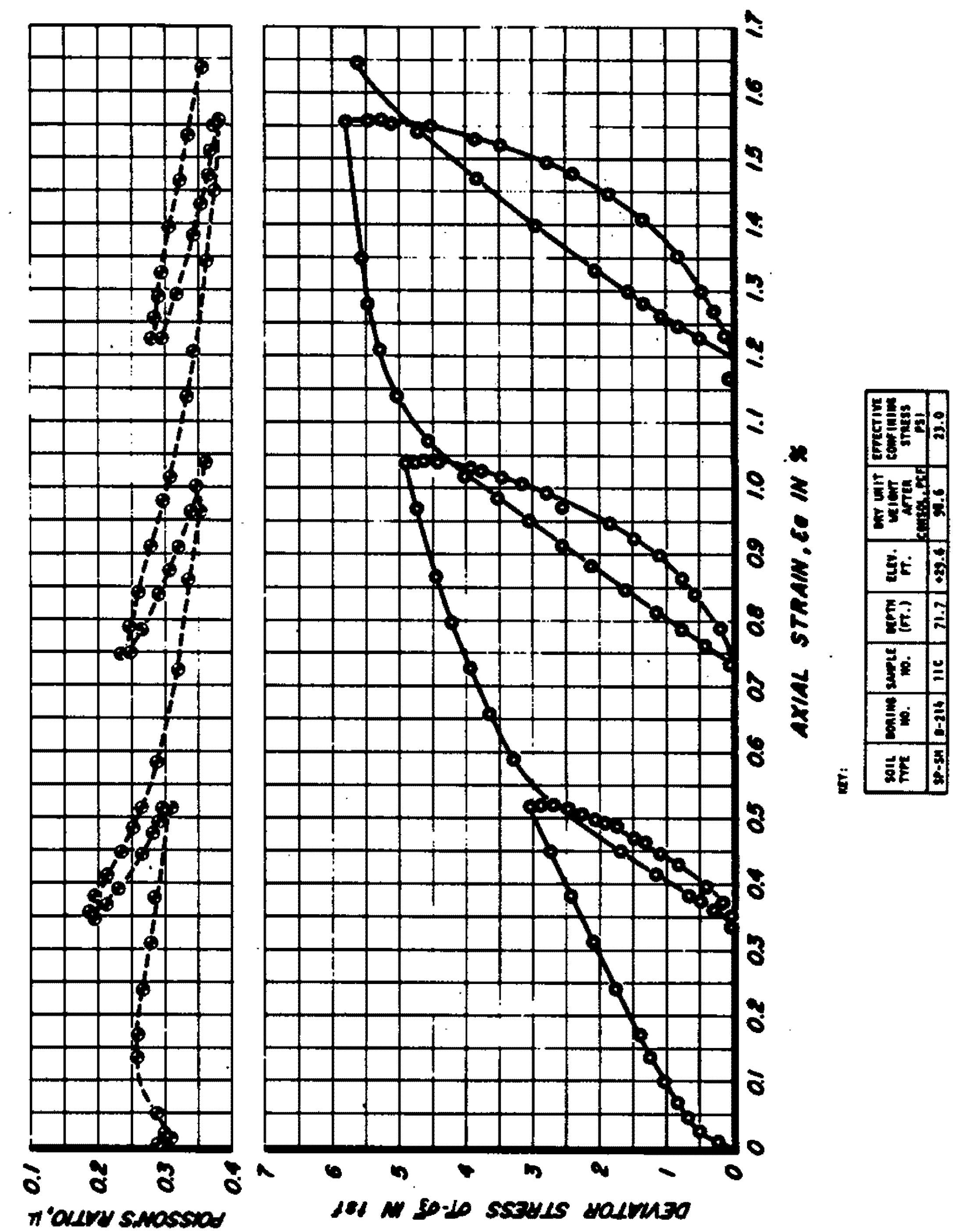
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

VARIATION OF DAMPING
WITH SHEAR STRAIN FOR CLAY

UPDATED FSAR

FIGURE 2.5-44

REFERENCE:
PSE&G, MAY 23, 1974, REPORT, FOUNDATION STUDIES, PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.



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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

TYPICAL CYCLIC STATIC
TRIAXIAL TEST DATA
FOR GRANULAR SOILS

REFERENCE:
PSE&G, MAY 23, 1974, REPORT, FOUNDATION STUDIES, PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

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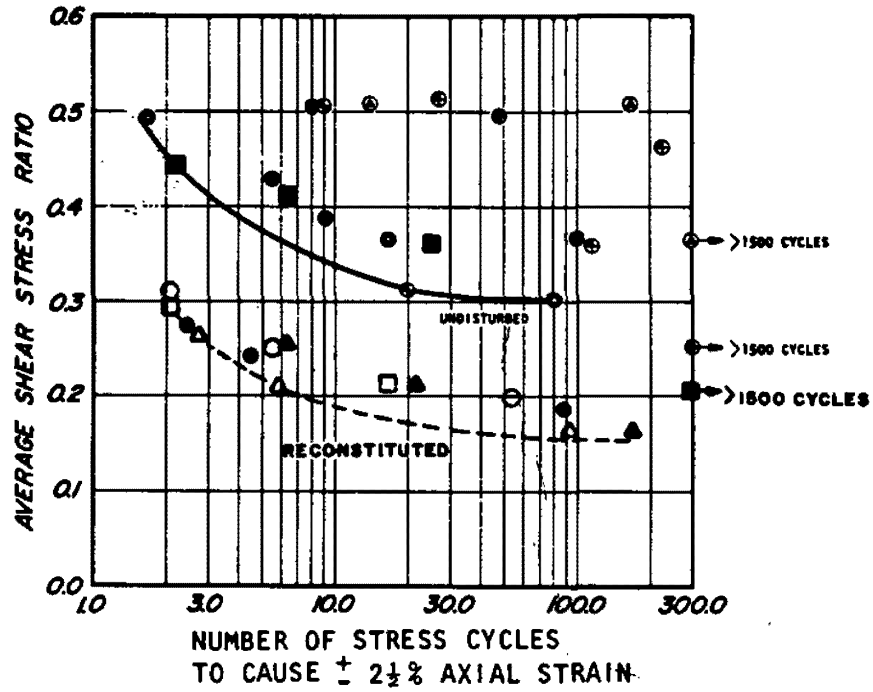
FIGURE 2.5-45



KEY:

SYMBOL RECONSTITUTED	SYMBOL UNDISTURBED	BORING	SAMPLE	APPROX. DEPTH (FEET)	APPROX. ELEV. (FEET)
○	⊕	201	19	110	-10
●	⊕	206	20	115	-18
△	⊙	206	13	80	+17
□	⊕	206	21	120	-23
▲	⊕	214*	10	65	+36
■	●	225	11A	58	+41
		225	100	56	+43

REFERENCE:
PSE&G, MAY 23, 1974, REPORT, FOUNDATION STUDIES, PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

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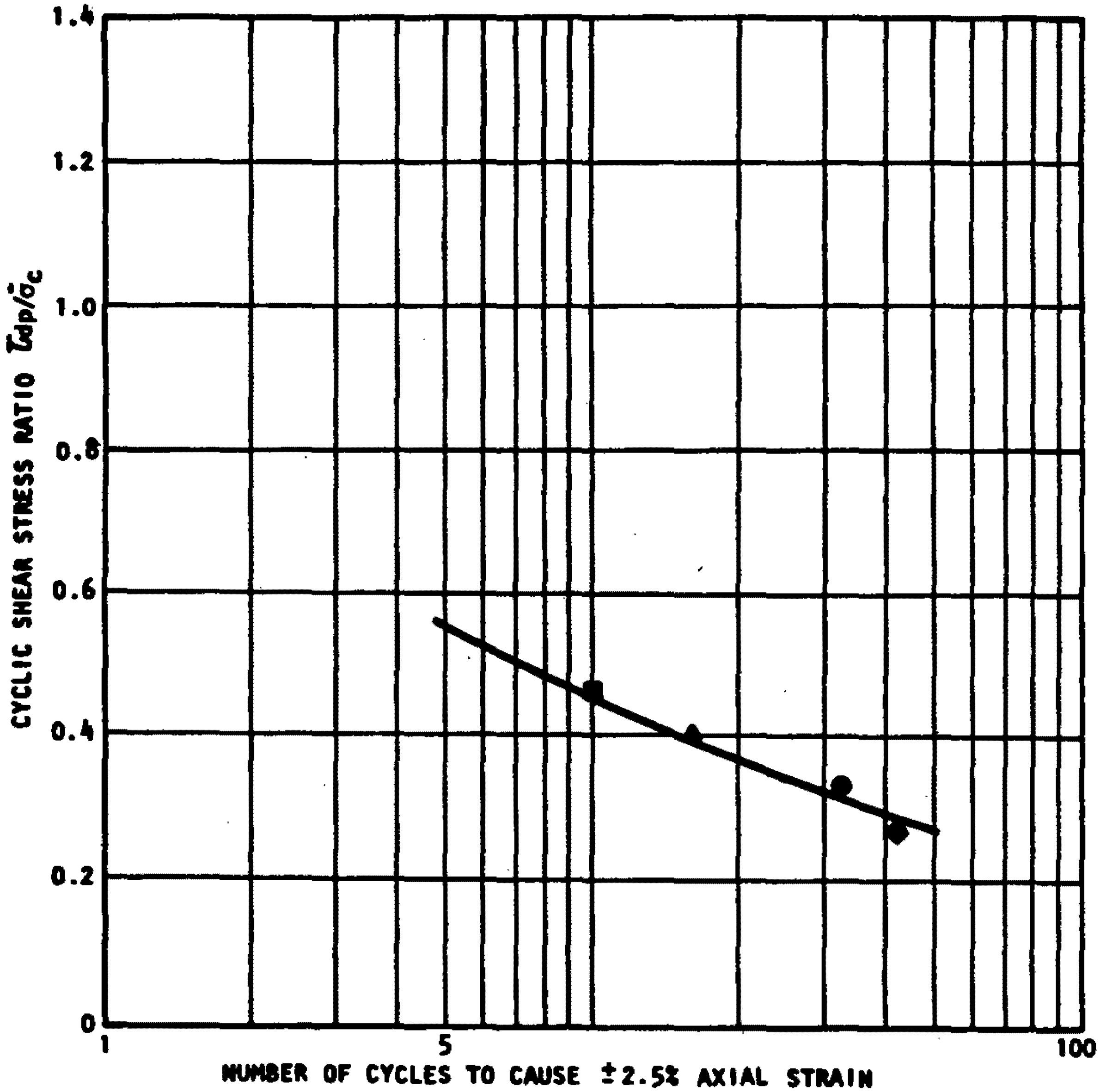
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

DYNAMIC STRENGTHS
VINCENTOWN SANDS

UPDATED FSAR

FIGURE 2.5-46



SYMBOL ±2.5% AXIAL STRAIN	BORING NO.	SAMPLE NO.	DEPTH (FT)	$\bar{\sigma}_c$** (P.S.I.)
●	AB-1	12	35	11
▲	AB-1A	11	34	11
■	AB-1A	12	38	11
◆*	AB-1	12	35	11

* RECONSTITUTED SAMPLE

** $\bar{\sigma}_c$ = EFFECTIVE CONFINING STRESS

REFERENCE:
PSE&G, JULY 1, 1974, REPORT, HOPE CREEK GENERATING STATION, SUPPLEMENTARY REPORT ON THE LIQUEFACTION STUDY. REPORT PREPARED BY DAMES & MOORE.

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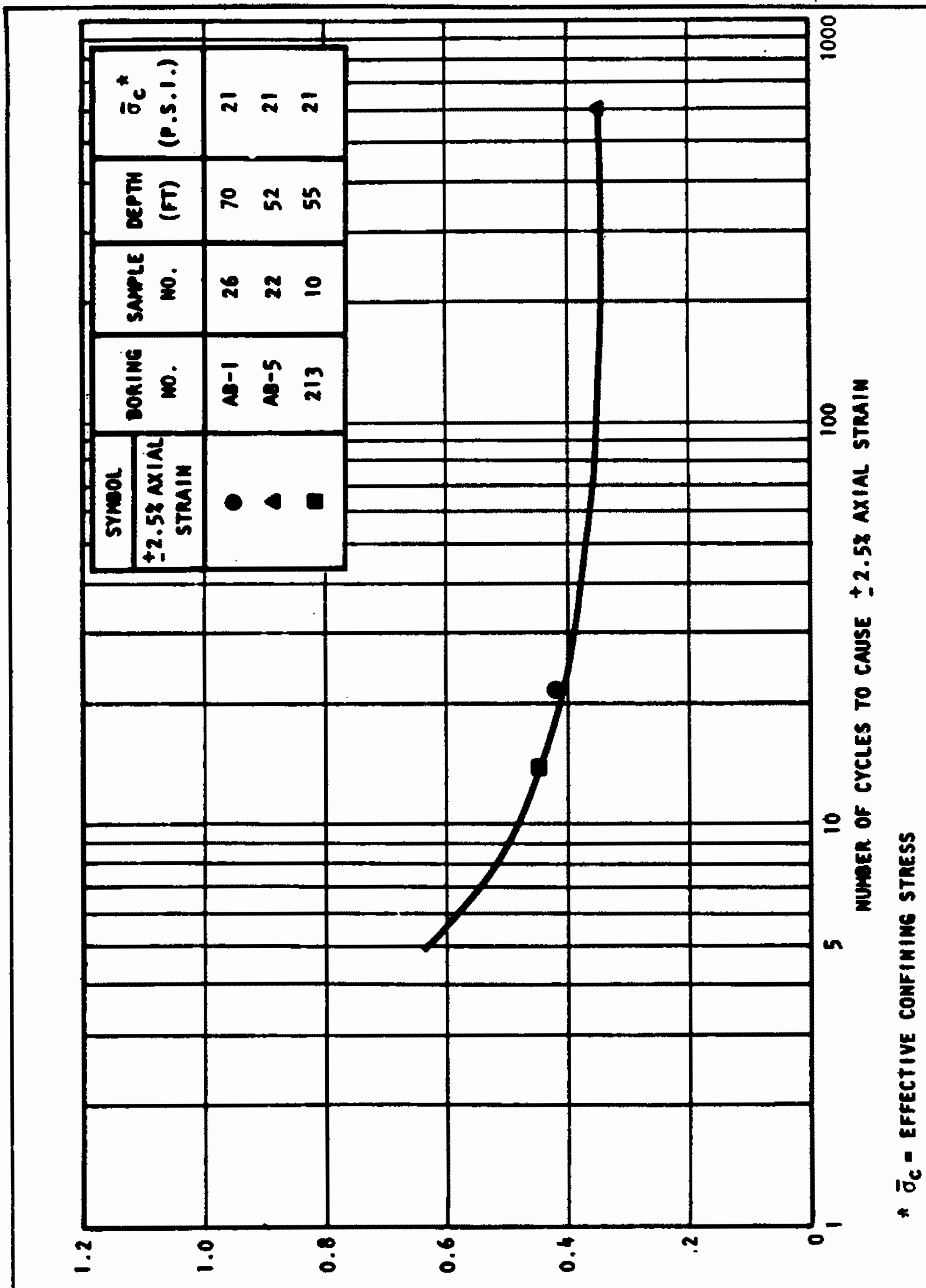
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

DYNAMIC STRENGTHS – RIVER BOTTOM SANDS

UPDATED FSAR

FIGURE 2.5-47



REFERENCE:
PSE&G, JULY 1976, AUXILIARY BORING PROGRAM ALONG SERVICE WATER SUPPLY LINES, HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

DYNAMIC STRENGTHS – BASAL SANDS

UPDATED FSAR

FIGURE 2.5-48

MAJOR DIVISIONS			GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE				SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

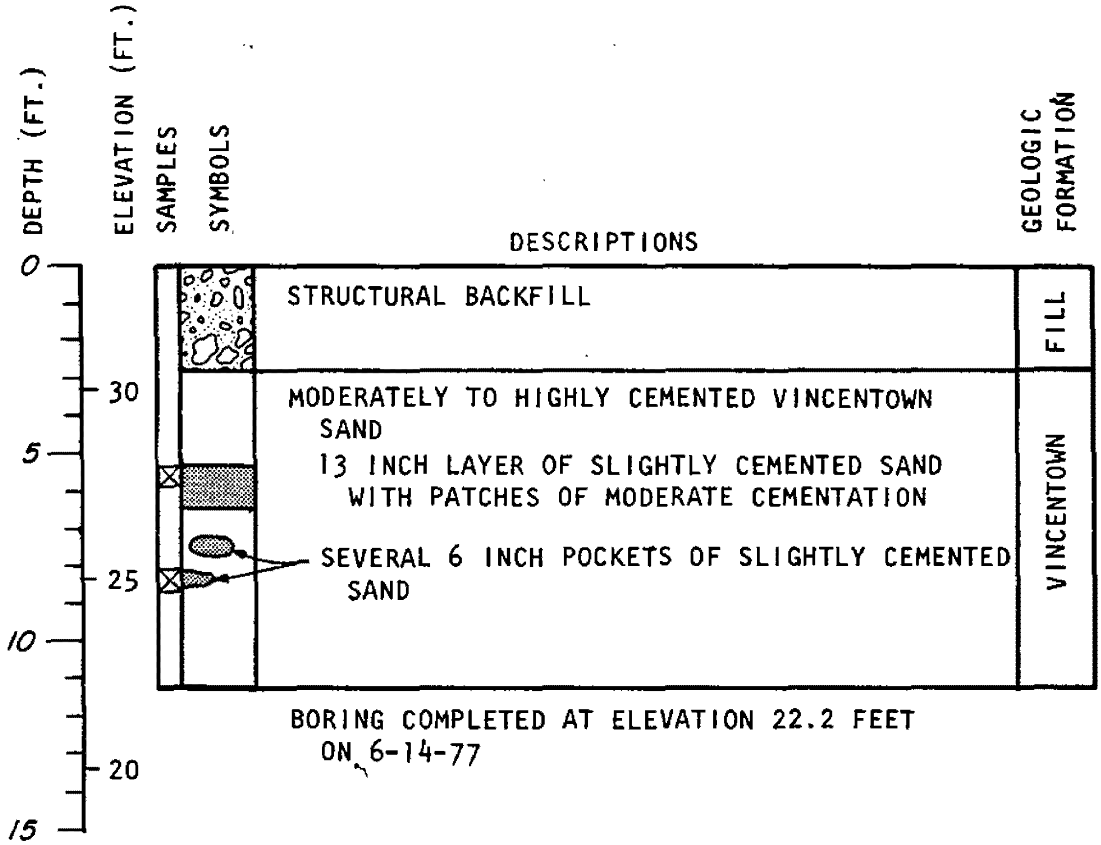
UNIFIED SOIL CLASSIFICATION SYSTEM

UPDATED FSAR

FIGURE 2.5-49

BORING X-1

SURFACE ELEVATION : 33.2
COORDINATES : N13+55
W05+76



Notes:

1. Elevations and Coordinates were provided by Bechtel Power Corporation and refer to Public Service Datum.
2. All borings were drilled with a 30-inch bucket auger.
3. Ground water was not encountered in the borings.
4. The discussion in the text of the report is necessary for a proper understanding of the nature and engineering properties of the subsurface materials.
5. Backfill of the borings performed by Bechtel Power Corporation after release of borings by Dames & Moore.

Key:

- Structural Backfill
- Moderately to Highly Cemented Vincentown Sands
- Slightly Cemented Vincentown Sands
- Apparently Uncemented Vincentown Sands
- ⊠ Bulk Sample

REFERENCE FOR BORINGS X-1 - X-23:
PSE&G, JANUARY 1978, POST EXCAVATION FOUNDATION STUDIES, PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

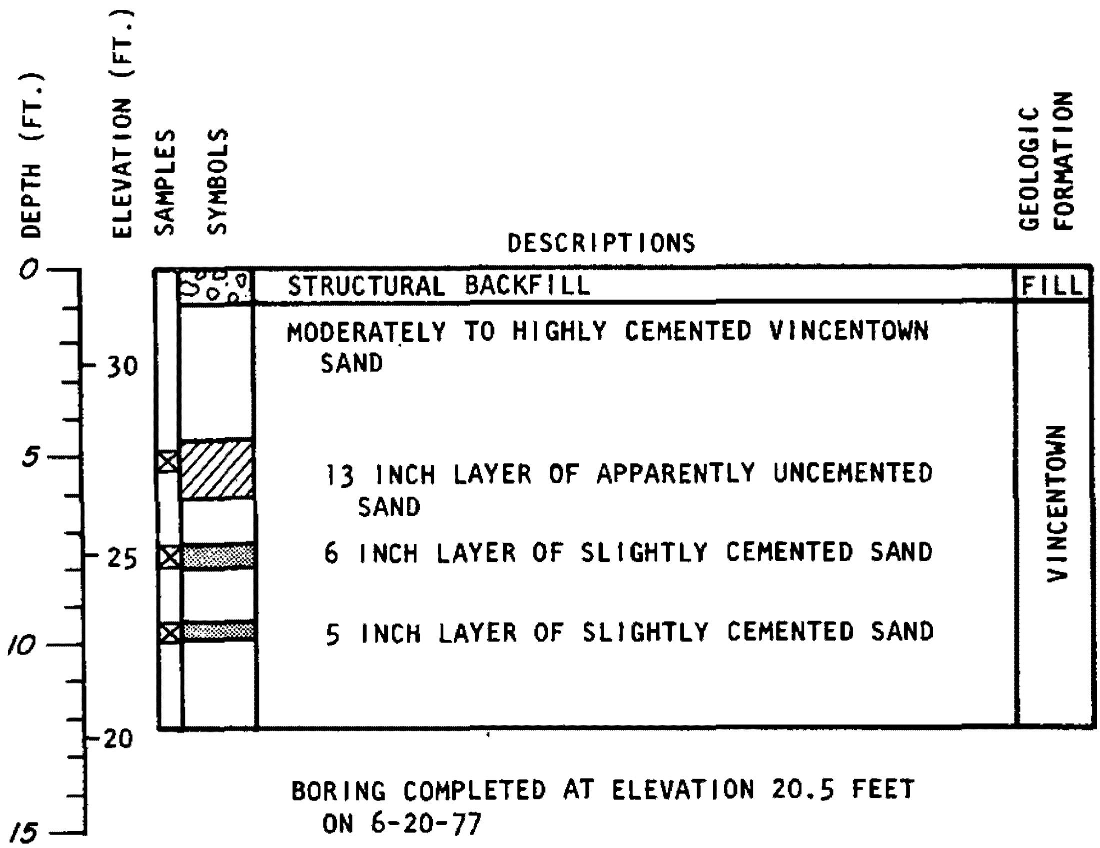
LOG OF BORINGS

UPDATED FSAR

Sheet 1 of 189
FIGURE 2.5-50

BORING X-2

SURFACE ELEVATION : 32.5
COORDINATES : N13+40
W05+74



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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

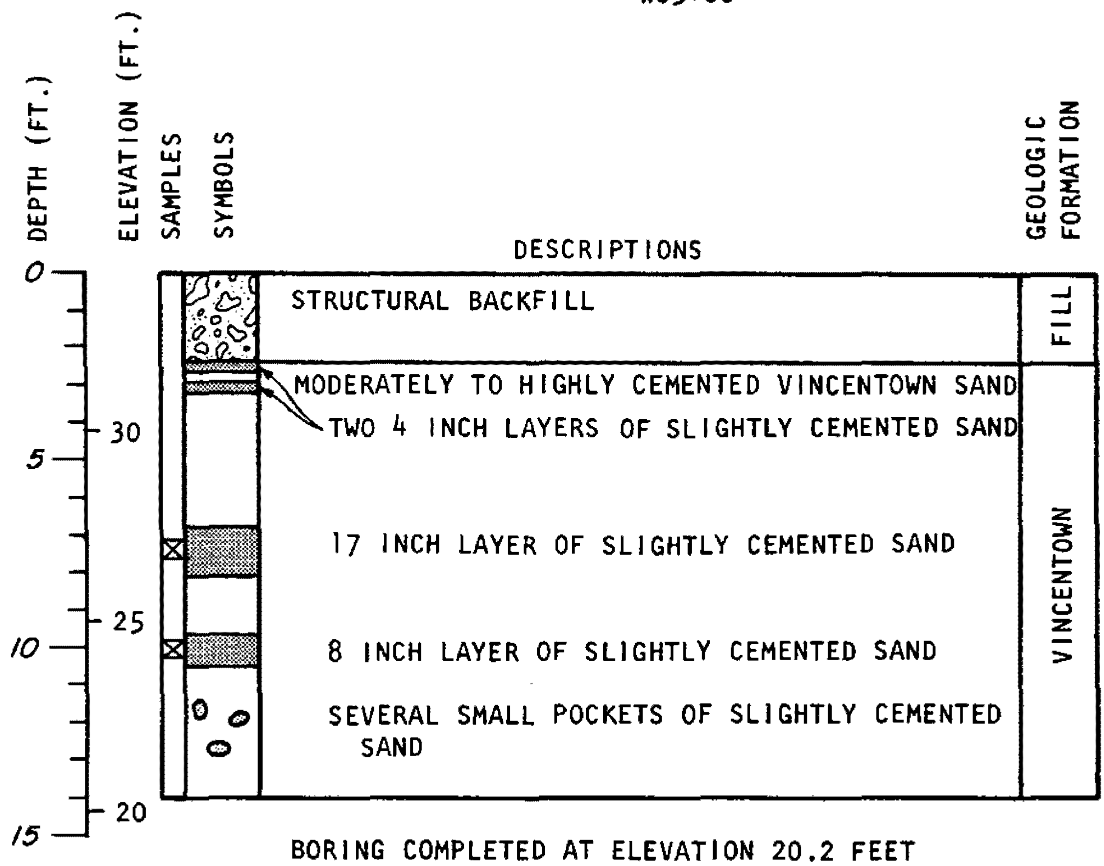
LOG OF BORINGS

UPDATED FSAR

Sheet 2 of 189
FIGURE 2.5-50

BORING X-3

SURFACE ELEVATION : 34.2
COORDINATES : N13+32
W05+66



BORING COMPLETED AT ELEVATION 20.2 FEET
ON 6-16-77

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

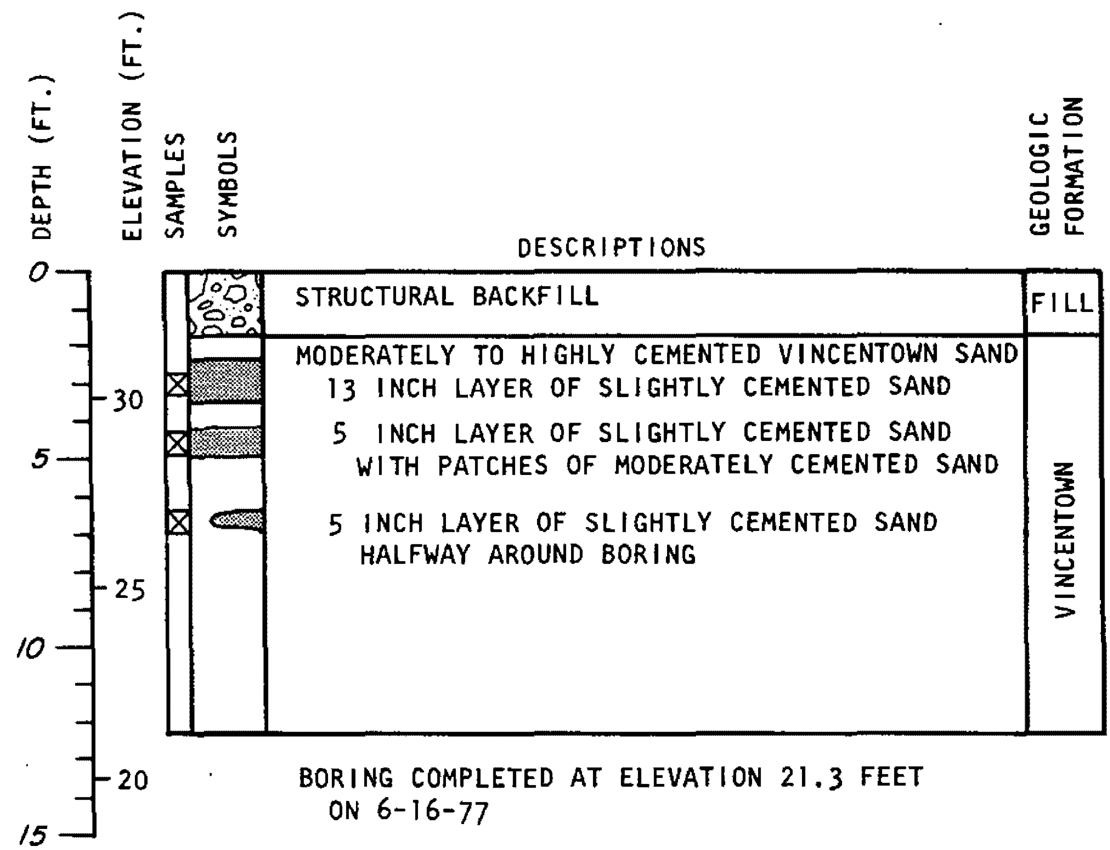
LOG OF BORINGS

UPDATED FSAR

Sheet 3 of 189
FIGURE 2.5-50

BORING X-4

SURFACE ELEVATION : 33.3
COORDINATES : N13+29
W05+48



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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

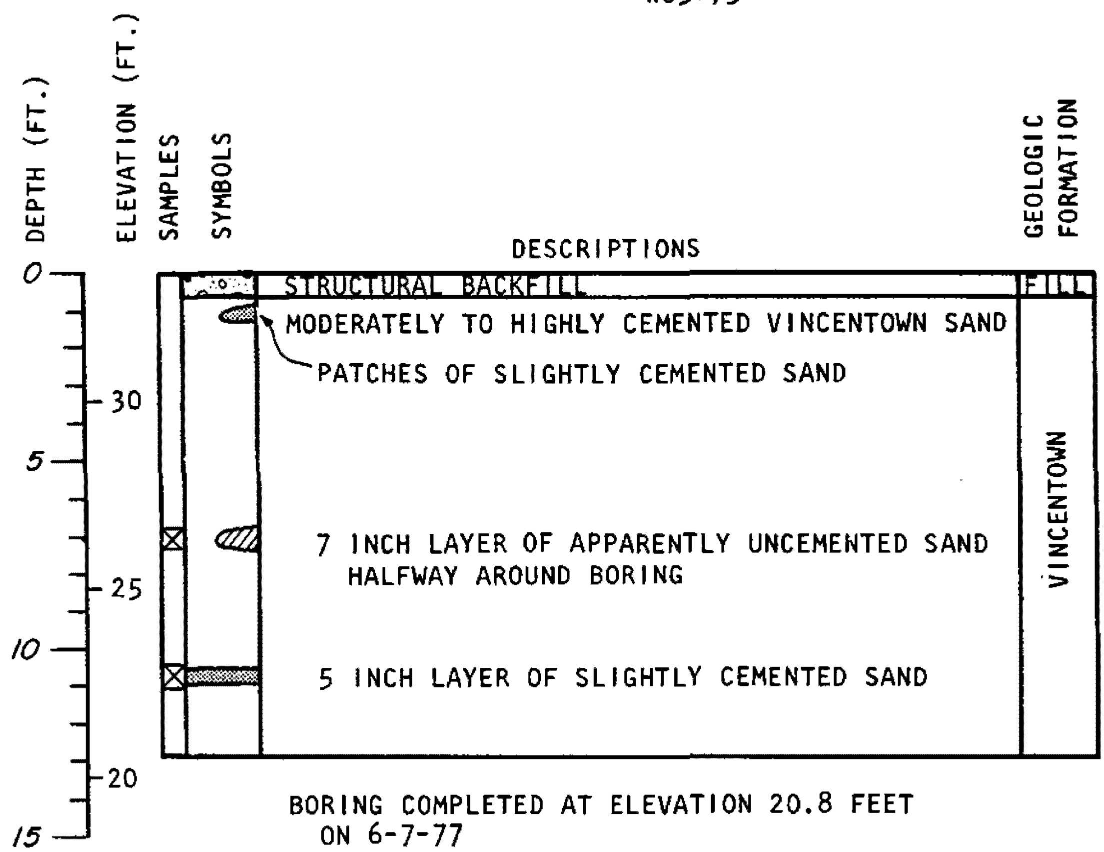
LOG OF BORINGS

UPDATED FSAR

Sheet 4 of 189
FIGURE 2.5-50

BORING X-5

SURFACE ELEVATION : 33.4
COORDINATES : N13+13
W05+73



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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

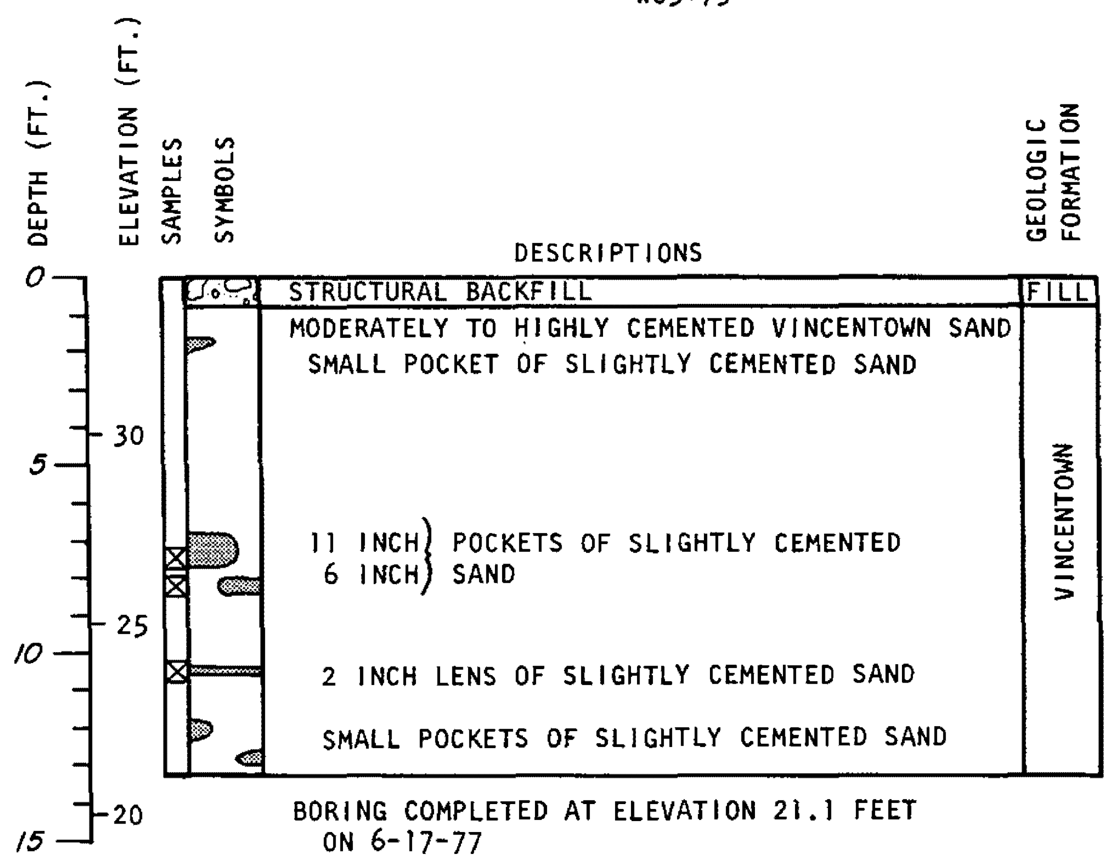
LOG OF BORINGS

UPDATED FSAR

Sheet 5 of 189
FIGURE 2.5-50

BORING X-6

SURFACE ELEVATION : 34.1
COORDINATES : N12+73
W05+73



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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

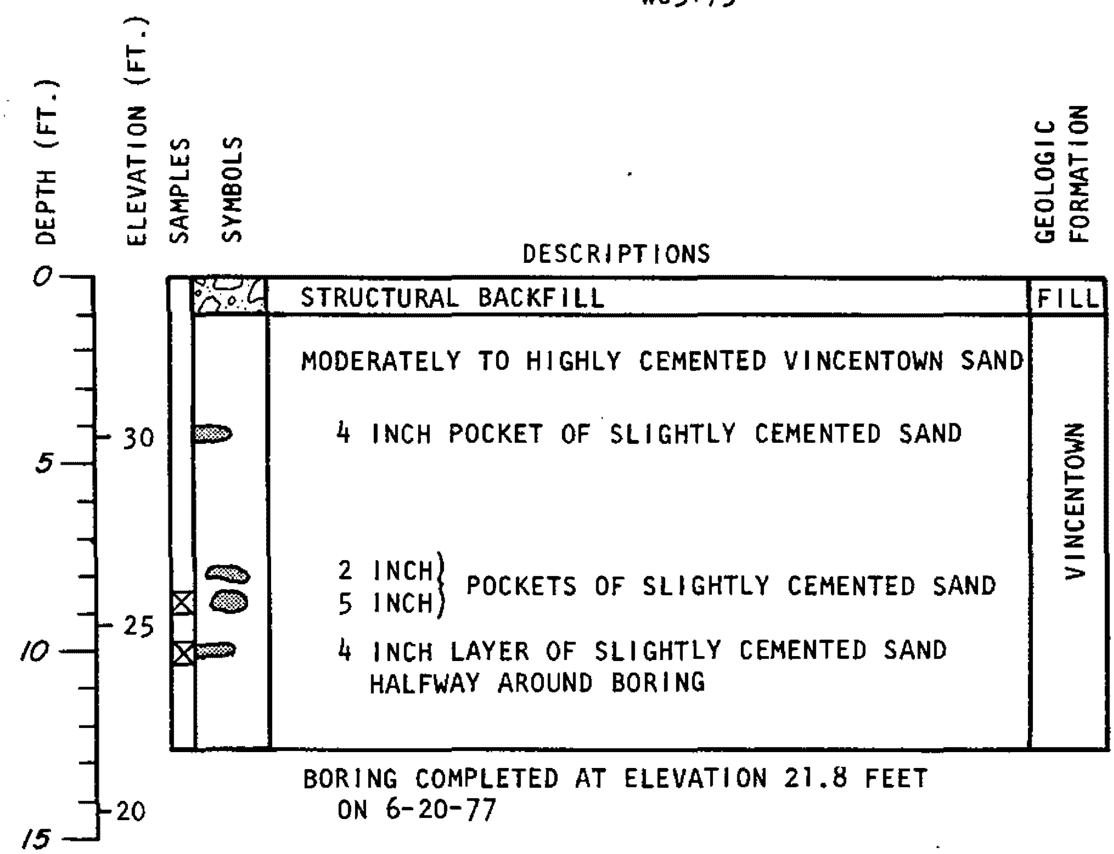
LOG OF BORINGS

UPDATED FSAR

Sheet 6 of 189
FIGURE 2.5-50

BORING X-7

SURFACE ELEVATION : 34.2
COORDINATES : N12+33
W05+73



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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

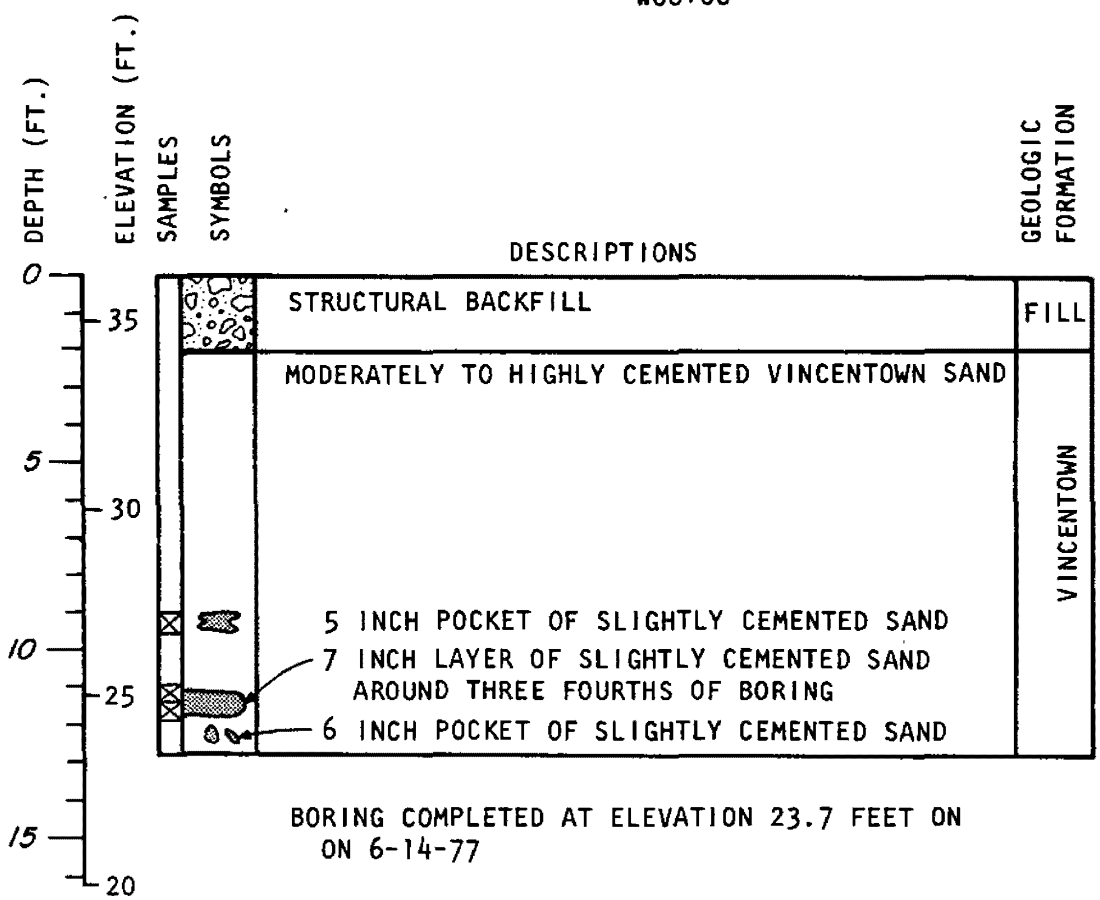
LOG OF BORINGS

UPDATED FSAR

Sheet 7 of 189
FIGURE 2.5-50

BORING X-8

SURFACE ELEVATION : 36.2
COORDINATES : N13+37
W06+00



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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

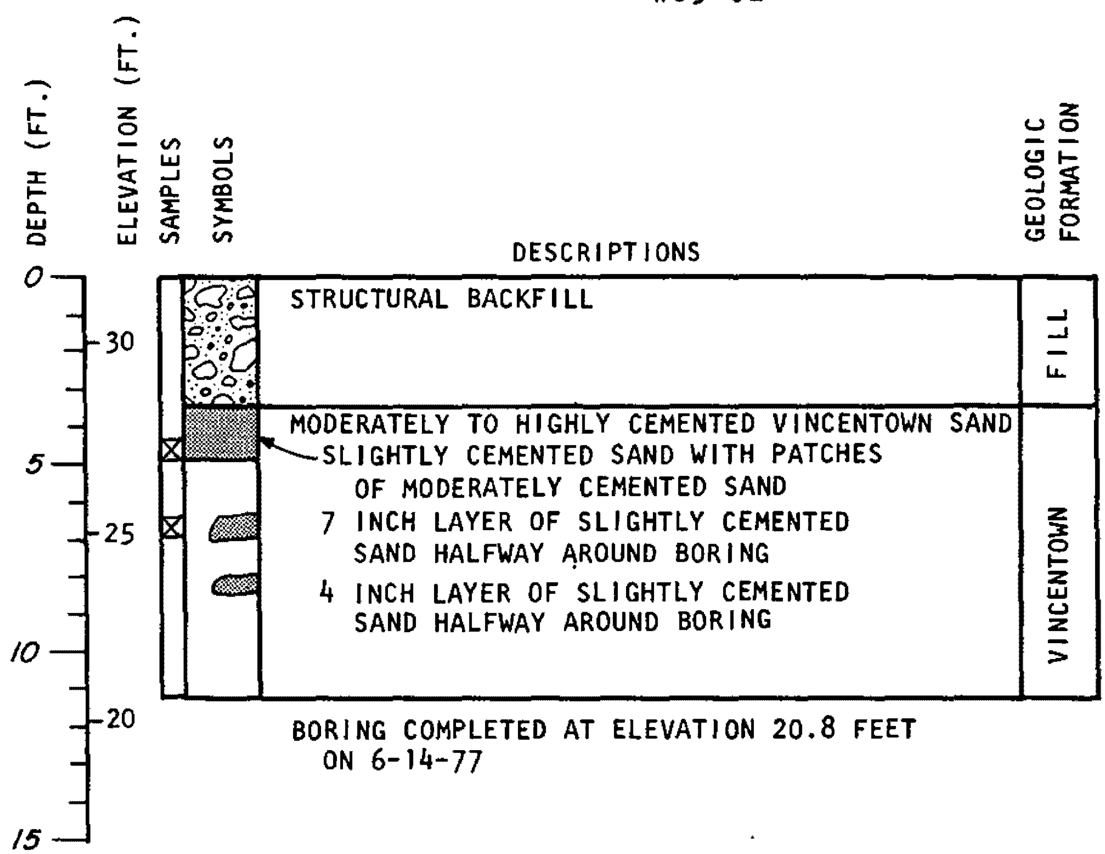
UPDATED FSAR

Sheet 8 of 189
FIGURE 2.5-50

BORING X-9

SURFACE ELEVATION : 31.8

COORDINATES : N13+70
W05+62



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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

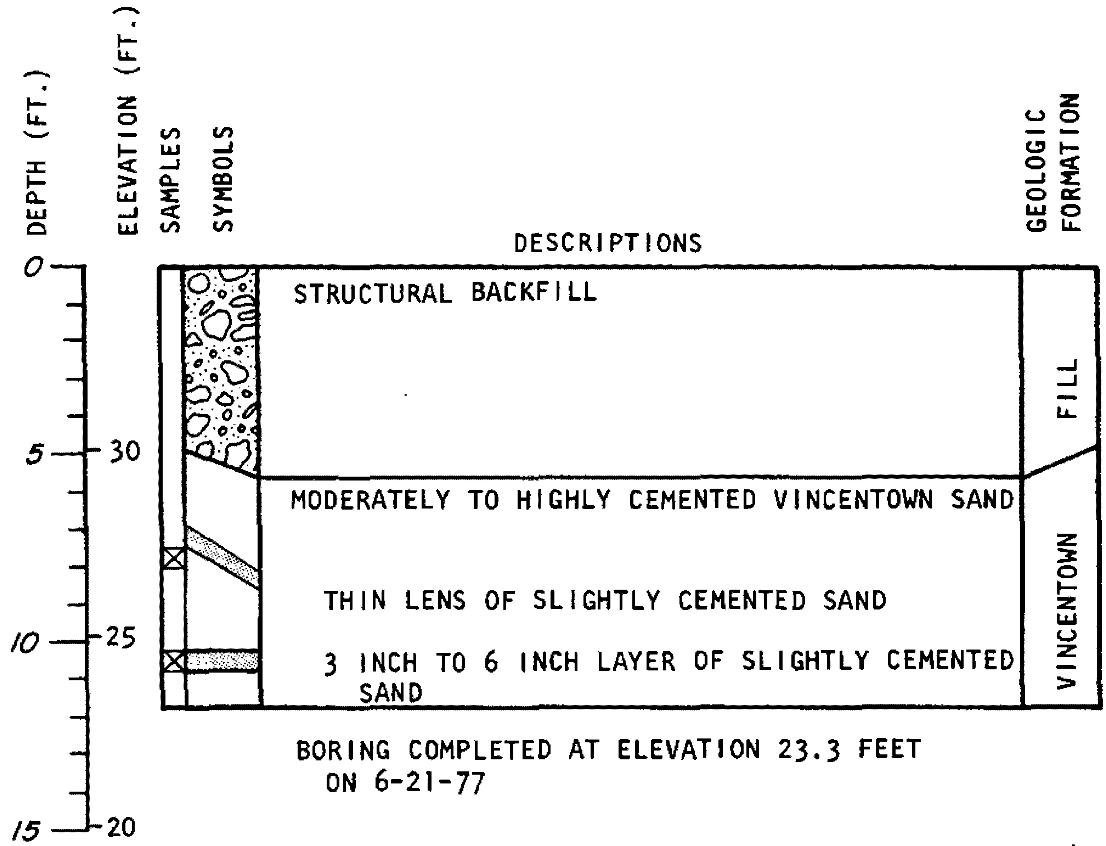
LOG OF BORINGS

UPDATED FSAR

Sheet 9 of 189
FIGURE 2.5-50

BORING X-10

SURFACE ELEVATION : 34.8
COORDINATES : N13+49
W05+39



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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

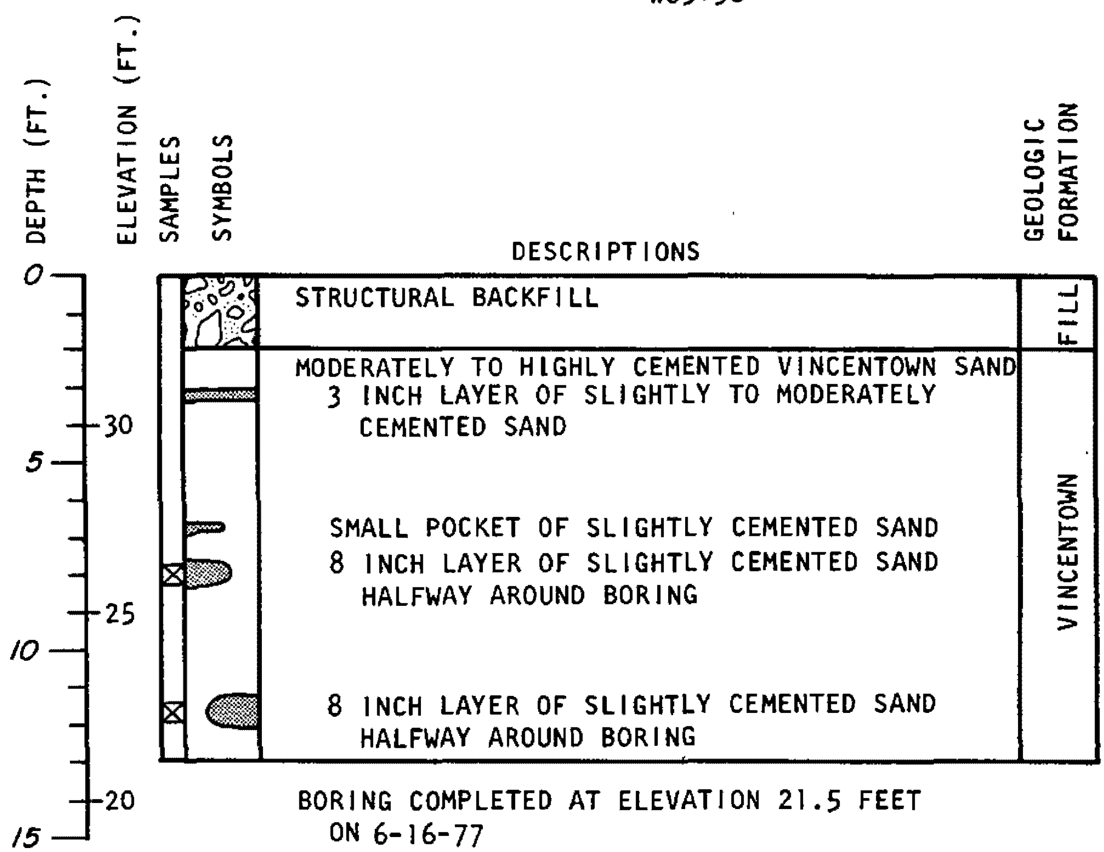
LOG OF BORINGS

UPDATED FSAR

Sheet 10 of 189
FIGURE 2.5-50

BORING X-11

SURFACE ELEVATION : 34.0
COORDINATES : N13+06
W05+56



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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

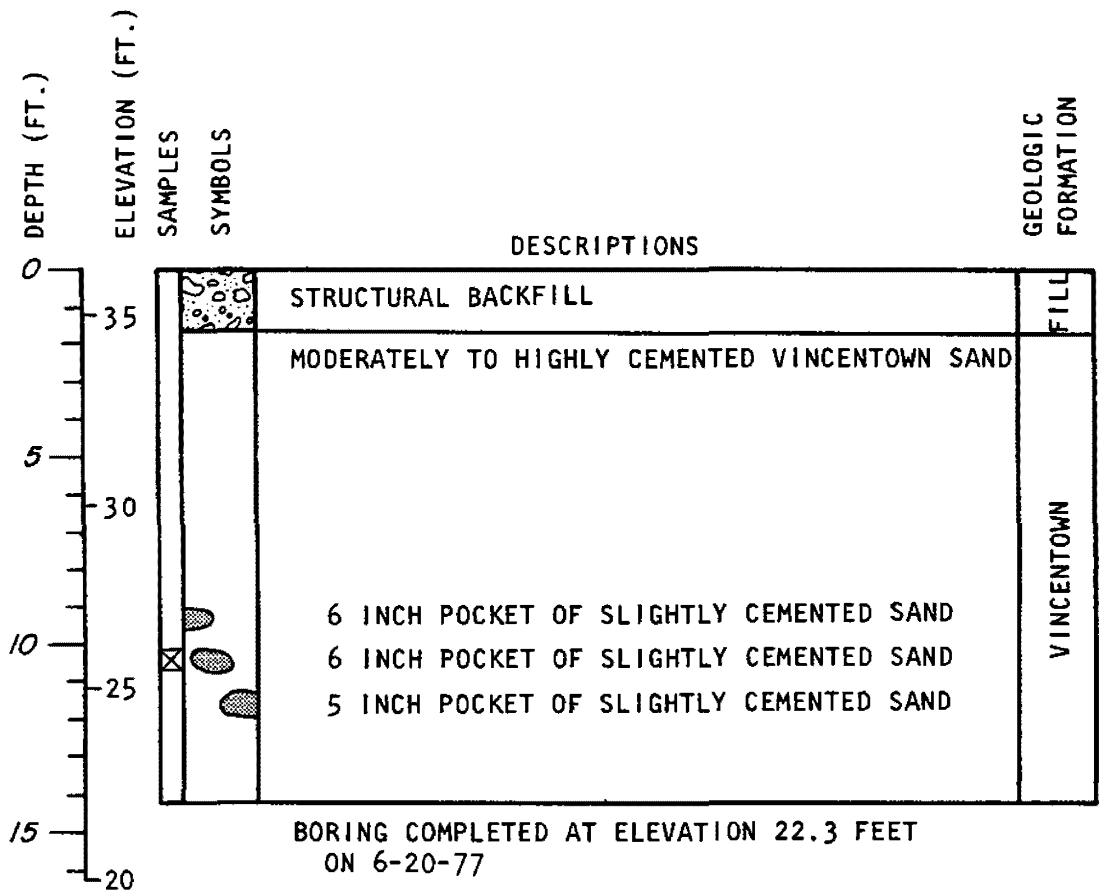
LOG OF BORINGS

UPDATED FSAR

Sheet 11 of 189
FIGURE 2.5-50

BORING X-12

SURFACE ELEVATION : 36.2
COORDINATES : N13+08
W06+04



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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

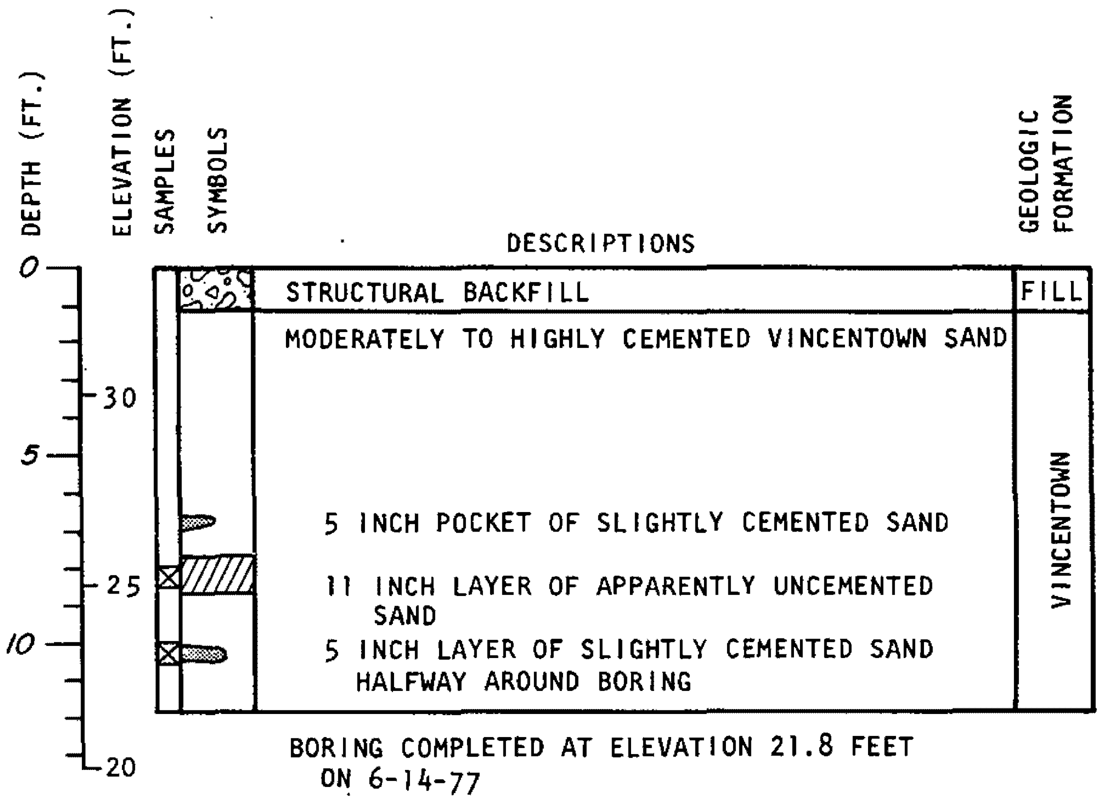
LOG OF BORINGS

UPDATED FSAR

Sheet 12 of 189
FIGURE 2.5-50

BORING X-13

SURFACE ELEVATION : 33.3
COORDINATES : N12+00
W05+25



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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

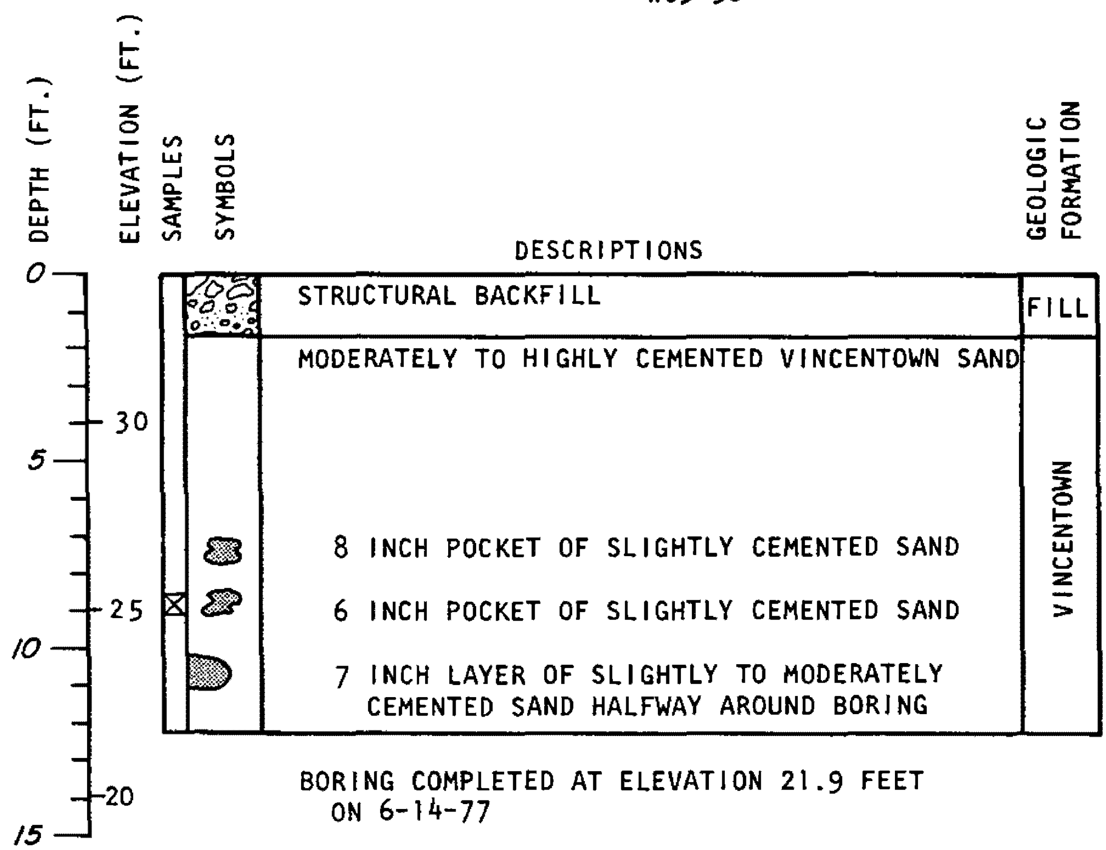
LOG OF BORINGS

UPDATED FSAR

Sheet 13 of 189
FIGURE 2.5-50

BORING X-14

SURFACE ELEVATION : 33.9
COORDINATES : N12+00
W05+50



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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

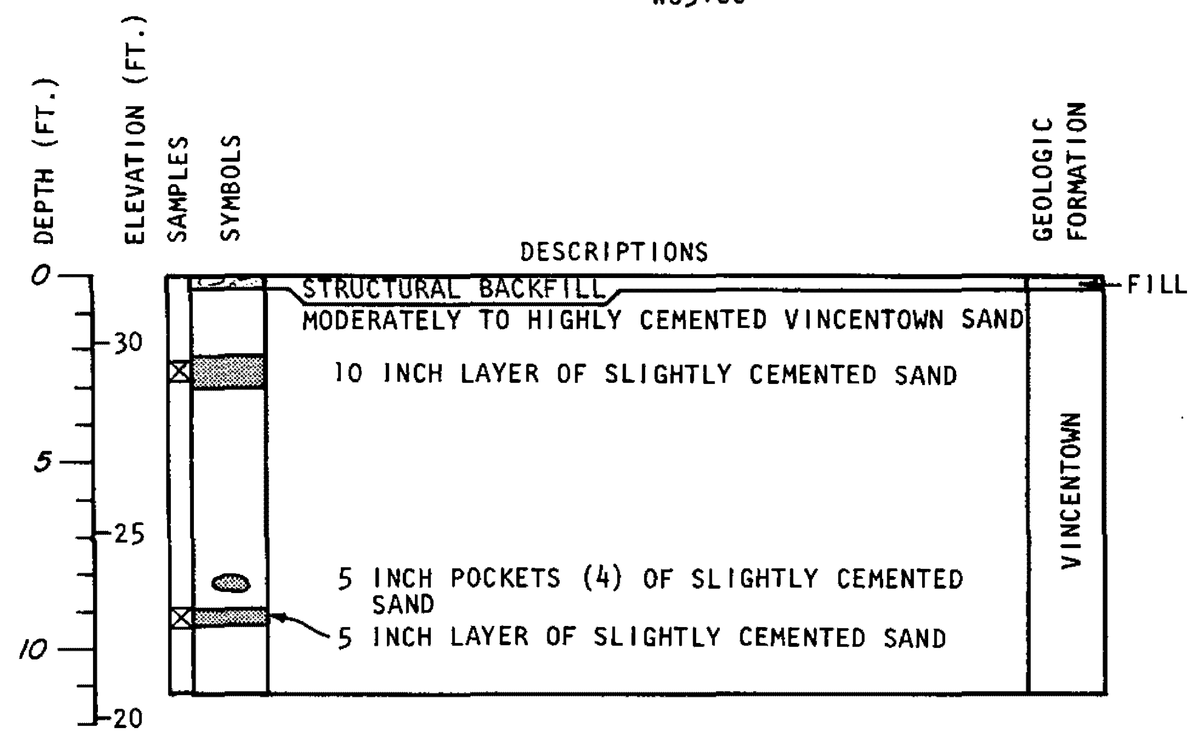
LOG OF BORINGS

UPDATED FSAR

Sheet 14 of 189
FIGURE 2.5-50

BORING X-15

SURFACE ELEVATION : 31.8
COORDINATES : N12+00
W05+00



BORING COMPLETED AT ELEVATION 20.9 FEET ON 6-14-77

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

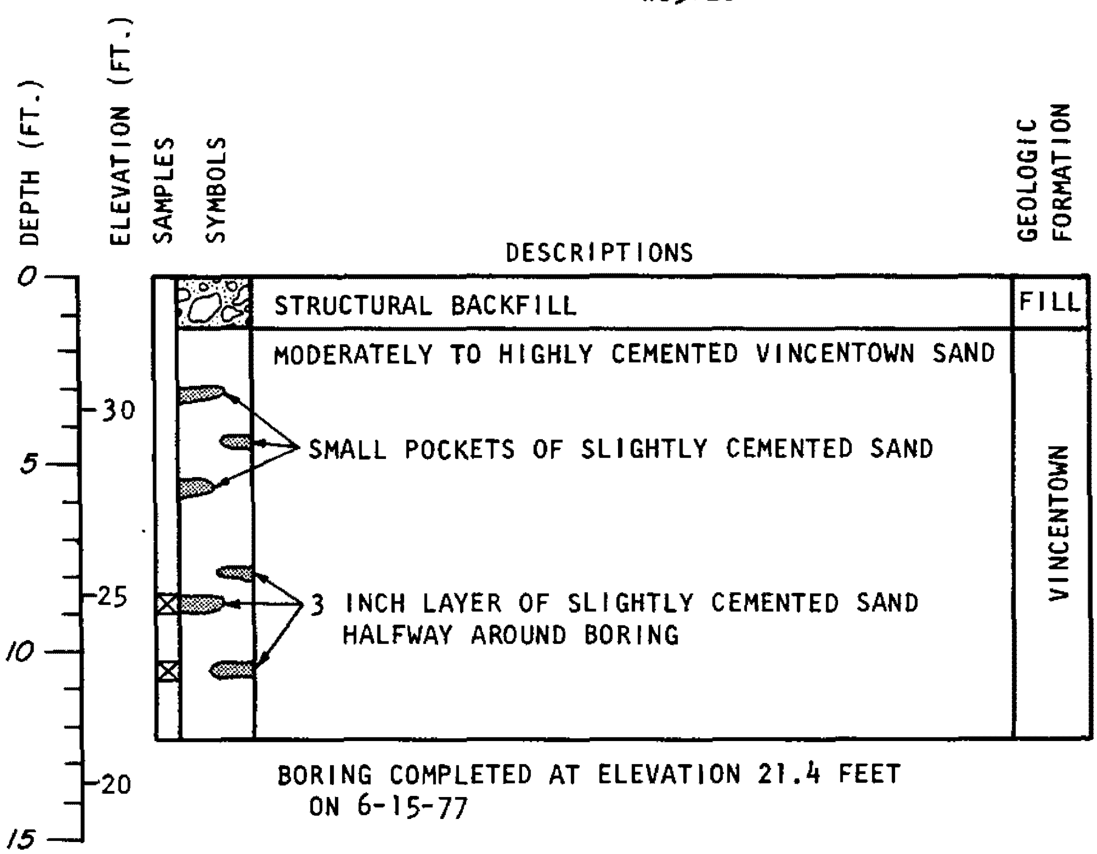
LOG OF BORINGS

UPDATED FSAR

Sheet 15 of 189
FIGURE 2.5-50

BORING X-16

SURFACE ELEVATION : 33.5
COORDINATES : N11+79
W05+26



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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

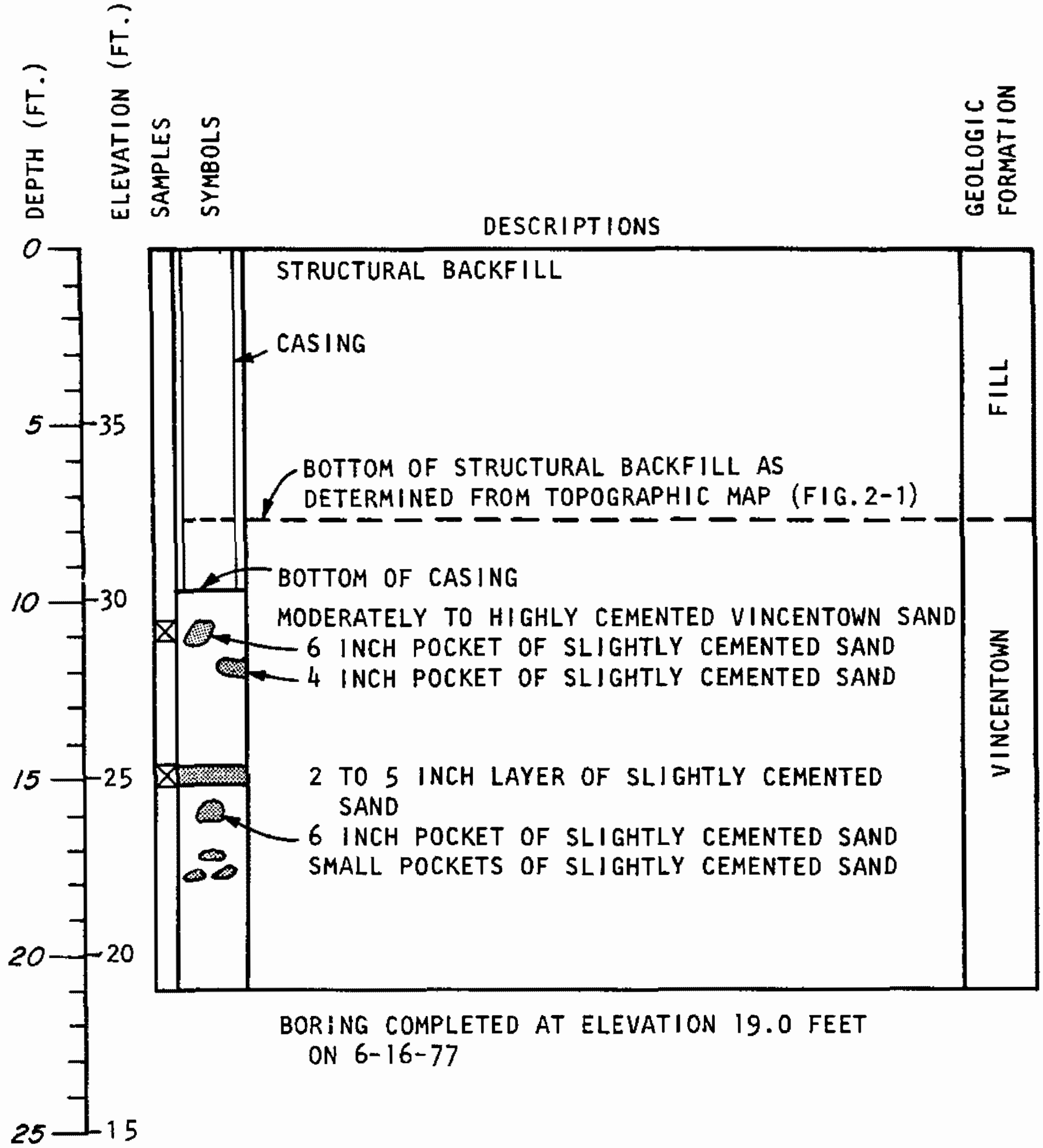
LOG OF BORINGS

UPDATED FSAR

Sheet 16 of 189
FIGURE 2.5-50

BORING X-17

SURFACE ELEVATION : 39.5
COORDINATES : N11+00
W05+00



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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 17 of 189
FIGURE 2.5-50

BORING X-18

SURFACE ELEVATION : 33.6
COORDINATES : N12+47
W05+39

DEPTH (FT.)	ELEVATION (FT.)	DESCRIPTIONS	GEOLOGIC FORMATION
0		STRUCTURAL BACKFILL	FILL
		MODERATELY TO HIGHLY CEMENTED VINCENTOWN SAND	VINCENTOWN
	30		
5			
		11 INCH LAYER OF SLIGHTLY CEMENTED SAND WITH PATCHES OF MODERATELY CEMENTED SAND	
	25		
10		2 TO 12 INCH LAYER OF SLIGHTLY CEMENTED SAND WITH PATCHES OF MODERATELY CEMENTED SAND	
	20		
15			

BORING COMPLETED AT ELEVATION 22.4 FEET
ON 6-15-77

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

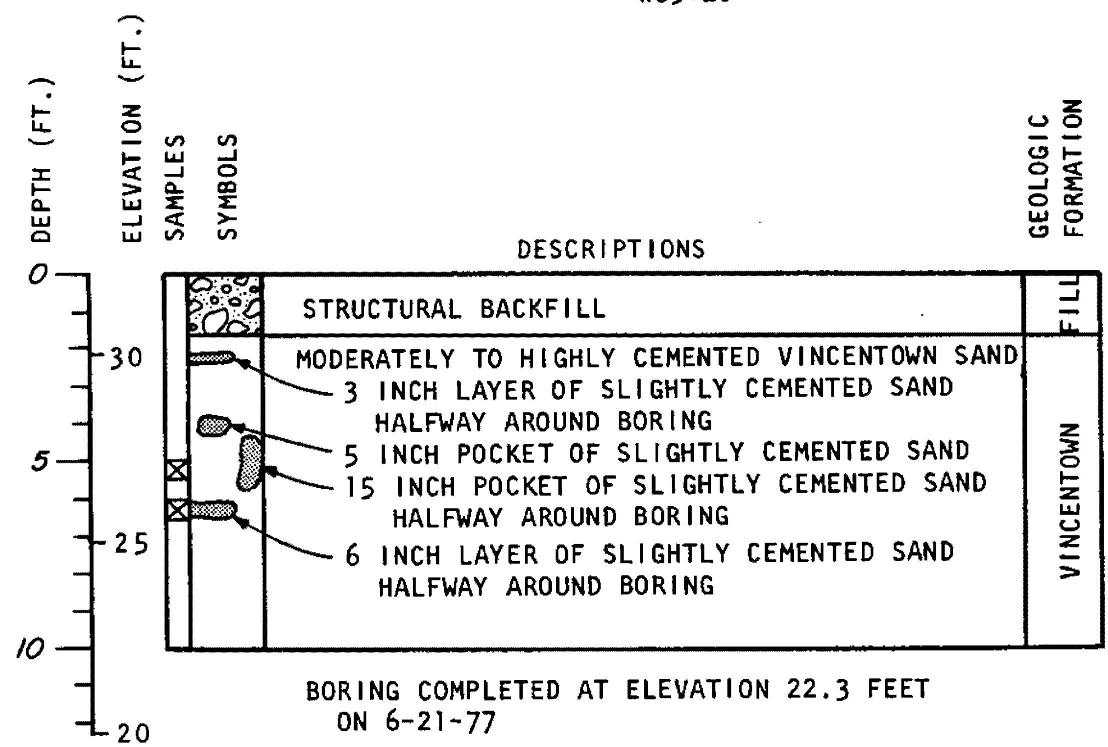
LOG OF BORINGS

UPDATED FSAR

FIGURE 2.5-50

BORING X-19

SURFACE ELEVATION : 32.1
COORDINATES : N14+55
W05+20



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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

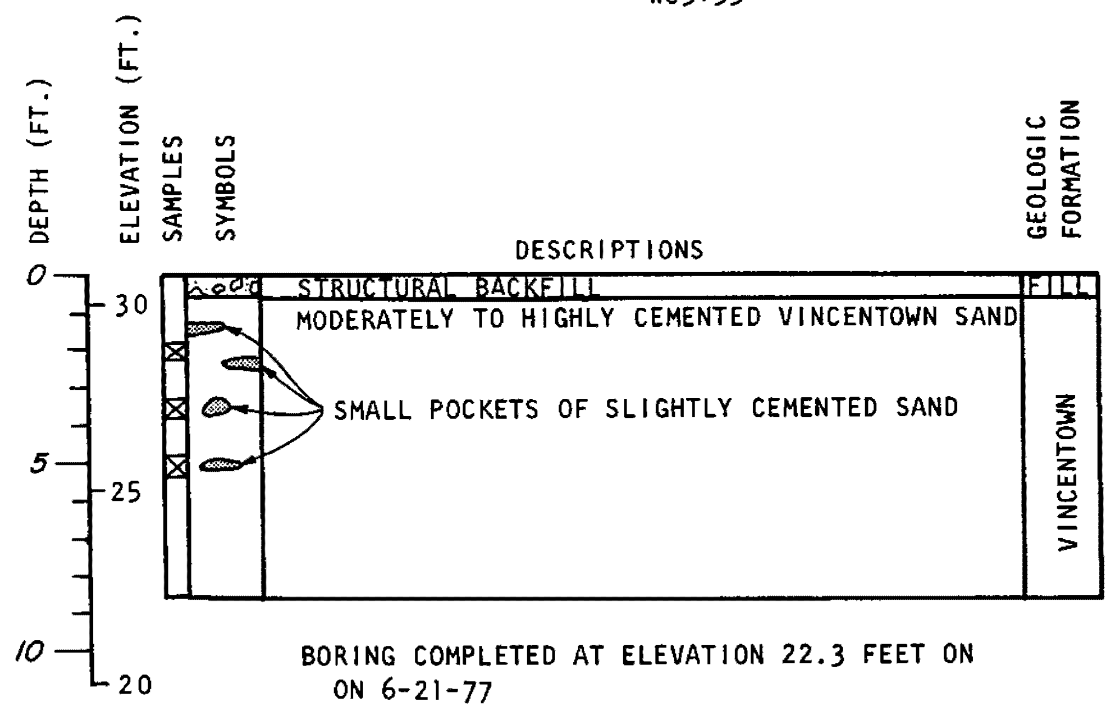
LOG OF BORINGS

UPDATED FSAR

Sheet 19 of 189
FIGURE 2.5-50

BORING X-20

SURFACE ELEVATION : 30.7
COORDINATES : N14+72
W05+55



BORING COMPLETED AT ELEVATION 22.3 FEET ON
ON 6-21-77

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

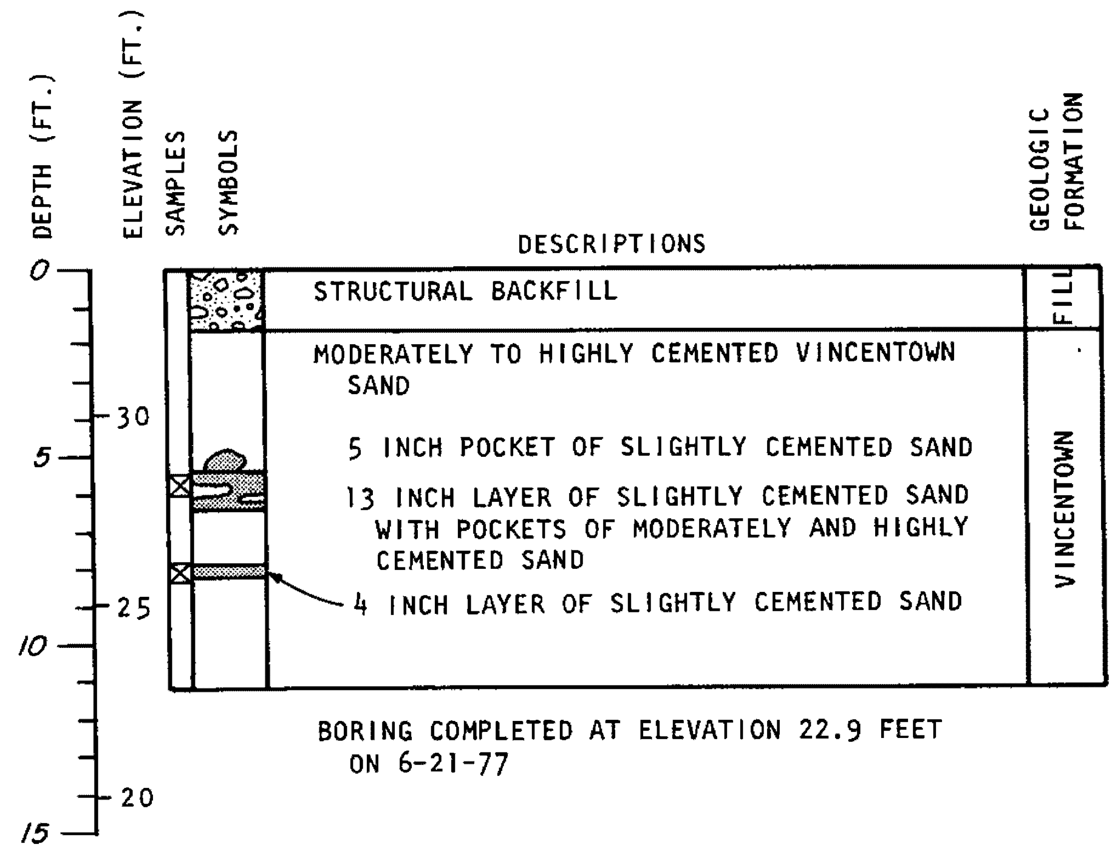
LOG OF BORINGS

UPDATED FSAR

Sheet 20 of 189
FIGURE 2.5-50

BORING X-21

SURFACE ELEVATION : 33.8
COORDINATES : N14+69
W04+71



BORING COMPLETED AT ELEVATION 22.9 FEET ON 6-21-77

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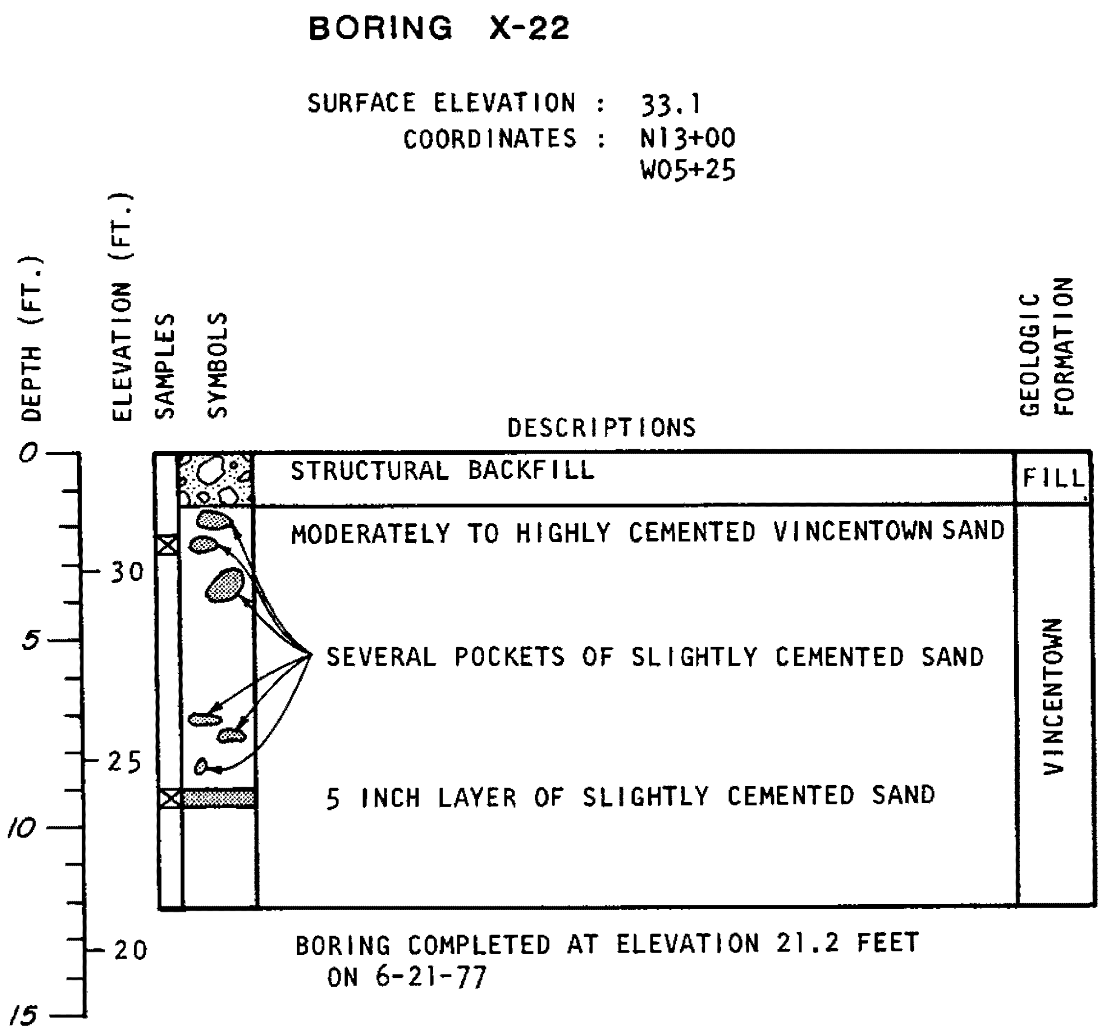
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 21 of 189
FIGURE 2.5-50



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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

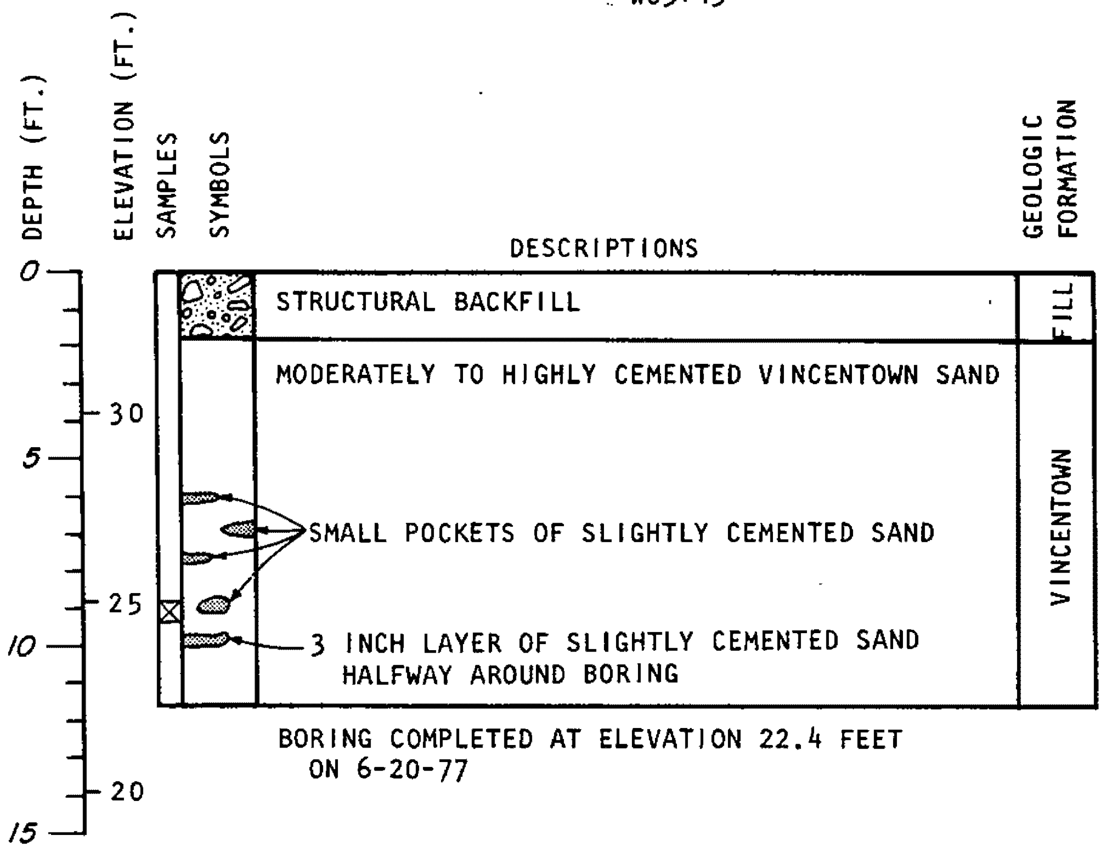
LOG OF BORINGS

UPDATED FSAR

Sheet 22 of 189
FIGURE 2.5-50

BORING X-23

SURFACE ELEVATION : 33.7
COORDINATES : N12+77
W05+45



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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 23 of 189
FIGURE 2.5-50

Location: N12+99, W13+99
Completion Date: 3/7/78

LOG OF BORING
BORING 1
EL. +97.8
DRILLING METHOD: TRUCK MOUNTED ROTARY MUD WASH DRILL RIG

SAMPLE Depth Feet	No.	SYM	Blows Ft.	UNIFIED SOIL CLASSIFICATION SYM.	Description	Lab Test
				GP	GRAVEL, CRUSHED SLAG, SAND (CONSTRUCTION FILL)	
				CL	DARK GRAY, SOFT, SILTY CLAY, TRACE FINE SAND, TRACE ORGANICS	
5	1		P			M&D, MA
	2		0	SM	GRAY, LOOSE, FINE SAND AND SILT, TRACE ORGANICS	
10	3		0			MA
				CH/OH	DARK GRAY, SOFT, CLAY, TRACE FINE SAND, TRACE ORGANICS	
15	4		P			AL, SA
	5		P			AL, MA, M&D, UU
20	6		0			
25	7		17	SM	GRAY, MEDIUM DENSE, FINE SAND AND SILT, THIN LENSES OF GRAY CLAY	SA
30	8		6			SA
35				GP	GRAY, VERY DENSE, FINE GRAVEL AND COARSE TO FINE SAND, TRACE SILT	
	9		72			SA
40				CL	GREENISH GRAY, STIFF, SILTY CLAY, TRACE ORGANICS, OCCASIONAL GRAVEL	

ELEVATIONS AND LOCATIONS REFER TO PSE&G PLANT DATUM
ELEVATIONS AND LOCATIONS WERE PROVIDED BY BECHTEL POWER CORPORATION

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APRIL 11, 1988

REFERENCE FOR BORINGS 1-10, 12, 13, 15-20, OW-140:
PSE&G, AUGUST 9, 1978, FIELD AND LABORATORY TEST DATA SUBSURFACE SOILS INVESTIGATION REQUIRED BY BECHTEL POWER CORP., PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 24 of 189
FIGURE 2.5-50

LOG OF BORING
BORING 1
EL. +97.8

Location: N 12+99, W 13+99
Completion Date: 3/7/78

SAMPLE				UNIFIED SOIL CLASSIFICATION		Lab Test
Depth Feet	No.	SYM	Blows/Ft.	SYM	Description	
	10		P			AL,M&D,Consol., G_s, UC
				CL		
	10A		38			
45	11		47			
50	12		37	SM	REDDISH BROWN AND GREENISH BROWN, DENSE, FOSSILIFEROUS, MEDIUM TO FINE SAND, SOME SILT	SA
55	13		48			
					GRADING TO GREENISH GRAY TO BROWNISH GREEN	
60	14		34	SM		
65	15		82/10"			
					BORING TERMINATED AT 66'4"	

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

FIGURE 2.5-50

LOG OF BORING
BORING 2
EL. +99.4
DRILLING METHOD: TRUCK MOUNTED ROTARY
MUD WASH DRILL RIG

Location: N 11+90, W 12+35
Completion Date: 3/7/78

SAMPLE			UNIFIED SOIL CLASSIFICATION		
Depth Feet	No.	Blows Ft.	SYM.	Description	Lab Test
	1	7	GP	MEDIUM DENSE TO LOOSE GRAVEL, CRUSHED SLAG, SAND-(CONSTRUCTION FILL)	SA
5	2	6			
	3	P			AL, M&D, UC
10			CH/OH	DARK GRAY, SOFT, CLAY, TRACE ORGANICS, FREQUENT THIN SEAMS OF FINE SAND	
15	4	P			M&D, MA
			SM	GRAY, MEDIUM DENSE, FINE SAND, SOME TO LITTLE SILT	
20	5	P			AL, M&D, UU
			CH/OH	DARK GRAY, MEDIUM STIFF TO STIFF, CLAY, FREQUENT THIN SEAMS OF FINE SAND	
25	6	P			AL, M&D
30			SM	GRAY, FINE SAND, SOME SILT	
	7	P	CH/OH	DARK GRAY, STIFF, CLAY	SA,AL,M&D
	8	4			
			SM	DARK GRAY, MEDIUM TO FINE SAND, SOME SILT	
35	9	P	CH/OH	DARK GRAY, MEDIUM STIFF, CLAY	AL,M&D,SA,UC
			SM	GRAY, DENSE, COARSE TO FINE SAND, LITTLE SILT, TRACE FINE GRAVEL	
40			CH/OH	GRAY, MEDIUM STIFF, CLAY, TRACE FINE SAND	

ELEVATIONS AND LOCATIONS REFER TO PSE&G PLANT DATUM
ELEVATIONS AND LOCATIONS WERE PROVIDED BY BECHTEL POWER CORPORATION

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 26 of 189
FIGURE 2.5-50

LOG OF BORING
BORING 2
EL. +99.4

PAGE 2 OF 2

Location: N 11+90, W 12+35
Completion Date: 3/7/78

Depth Feet	No.	Blows/Ft.	SYM.	Description	Lab Test
	10	P			AL,M&D,Consol., G_s,UC
	11	11			
45	12	P	CH/OH	BECOMING VERY STIFF	AL,M&D,Consol., G_s, UU
50	13	P			AL,M&D,MA
	14	14			
55	15	11			
60	16	P			M&D,SA,UC
	17	29	SM	REDDISH GRAY, DENSE, FINE SAND AND SILT	SA
65	18	25			
			SM	GREENISH GRAY, DENSE TO VERY DENSE, MEDIUM TO TO FINE SAND, SOME SILT	
70	19	102			
75	20	40			
				BORING TERMINATED AT 76'6"	
80					

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 27 of 189
FIGURE 2.5-50

LOG OF BORING PAGE 1 OF 2

BORING 3

EL. +100.3

DRILLING METHOD: TRUCK MOUNTED ROTARY MUD WASH DRILL RIG

Location: N 11+90, W 9+80

Completion Date: 3/8/78

SAMPLE				UNIFIED SOIL CLASSIFICATION		Lab Test
Depth Feet	No.	SYM	Blows Ft.	SYM.	Description	
				SP	ORANGE, DENSE, COARSE TO FINE SAND, LITTLE FINE GRAVEL, TRACE SILT - (CONSTRUCTION FILL)	
	1		9			
5				CL	DARK GRAY, VERY STIFF, SILTY CLAY AND FINE SAND	AL,M&D,MA,Consol Gs,UU
	2		P			
					GRADES TO	
10						
	3		P	ML	DARK GRAY, MEDIUM STIFF, SILT, TRACE FINE SAND	AL,M&D,UC
	4		4			
					GRADES TO	
15						
	5		P			AL,M&D,MA,UU
				CH	DARK GRAY, MEDIUM STIFF, CLAY, FREQUENT THIN SEAMS OF FINE SAND	
20						
	6		P			AL,M&D
	7		2			SA
25						
	8		P			AL,M&D,UC
30					BECOMING LESS PLASTIC	
	9		P			AL,M&D,UC
	10		4			
35	10A		55/3"			
				GP	DENSE, COARSE TO FINE GRAVEL AND SAND, OCCASIONAL COBBLES	
40				CH-OH	DARK GRAY, STIFF, CLAY	

ELEVATIONS AND LOCATIONS REFER TO PSE&G PLANT DATUM

ELEVATIONS AND LOCATIONS WERE PROVIDED BY BECHTEL POWER CORPORATION

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 28 of 189
FIGURE 2.5-50

LOG OF BORING
BORING 3
EL. +100.3

Location: N 11+90, W 9+80
Completion Date: 3/8/78

SAMPLE			UNIFIED SOIL CLASSIFICATION		
Depth Feet	No.	Blows Ft.	SYM.	Description	Lab Test
	11	11			
	12	P			AL
45	13	P	CH/OH		AL,M&D,MA,Consol G_s,UC
50	14	P			AL,M&D
	15	11			
55	16	P			AL,M&D,UC
60	17	P			AL,M&D,MA,Consol, G_s,UC
	18	16		BECOMING BROWNISH GRAY TO BROWN	AL(on Consol)
65	19	P	OH		AL,M&D
70	20	18	SM	GREENISH GRAY TO DARK GREEN, MEDIUM DENSE, FOSSILIFEROUS, MEDIUM TO FINE SAND, SOME SILT	
				BORING TERMINATED AT 71'6"	

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 29 of 189
FIGURE 2.5-50

LOG OF BORING
BORING 4
EL. +100.2
DRILLING METHOD: TRUCK MOUNTED ROTARY
MUD WASH DRILL RIG

Location: N 25+41, W 15+44
Completion Date: 3/21/78

SAMPLE				UNIFIED SOIL CLASSIFICATION		
Depth Feet	No.	SYM	Blows/Ft.	SYM.	Description	Lab Test
				GP	LOOSE, COARSE, CRUSHED STONE-(TEMPORARY ACCESS ROAD)	
5	1		12			
	2		12	SW/SM	LIGHT BROWN, MEDIUM DENSE, COARSE TO FINE SAND, LITTLE FINE GRAVEL, TRACE SILT	SA
10	2A		13			
15	3		7			
20	4		P			AL(on Consol), M&D, Consol, G_s, UU, AL(on UU)
	5/vs			CH/OH	DARK GRAY, SOFT, CLAY, LITTLE FIBROUS ORGANICS, FREQUENT THIN SEAMS OF FINE SAND	
25	6		P			AL, M&D, UC
	7/vs				BECOMING LESS PLASTIC	AL
					BECOMING MEDIUM STIFF	
	8		P			AL, M&D, UU
30	9		10			
35	10		P			
	11		15	SM	GRAY, MEDIUM DENSE, MEDIUM TO FINE SAND, SOME SILT	M, SA
					GRADING TO	
40				SP	GREENISH GRAY, VERY DENSE, COARSE TO FINE SAND, TRACE SILT	

ELEVATIONS AND LOCATIONS REFER TO PSE&G PLANT DATUM
ELEVATIONS AND LOCATIONS WERE PROVIDED BY BECHTEL POWER CORPORATION

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

FIGURE 2.5-50

LOG OF BORING
BORING 4
EL. +100.2

Location: N 25+41, W 15+44
Completion Date: 3/21/78

SAMPLE				UNIFIED SOIL CLASSIFICATION		Lab Test
Depth Feet	No.	SYM	Blows Ft.	SYM	Description	
	12		62	SP		
45	13		23			
50	14		P	CH	DARK GRAY, VERY STIFF, CLAY, TRACE ORGANICS	AL, M&D, Consol, G_s, AL (on Consol), UC
	15		15			
55	16		11			
60	16A		60/½"	GP	GRAVELLY LAYER DETECTED BY DRILLING OPERATIONS (NO RECOVERY)	
	17		35		GREENISH GRAY AND REDDISH BROWN, VERY DENSE TO DENSE, FINE SAND, SOME SILT	SA
65	18		53	SM		
					BECOMING GREENISH GRAY, OCCASIONAL THIN HIGHLY CEMENTED LAYERS	
70	19		64/9"			
					BORING TERMINATED AT 71'3"	
75						
80						

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 31 of 189
FIGURE 2.5-50

LOG OF BORING PAGE 1 OF 2

BORING 5

EL. +104.2

DRILLING METHOD: TRUCK MOUNTED ROTARY MUD WASH DRILL RIG

Location: N 26+43, W 06+01

Completion Date: 3/13/78

SAMPLE			UNIFIED SOIL CLASSIFICATION		
Depth Feet	No.	Blows/Ft.	SYM.	Description	Lab Test
			GP	VERY DENSE, COARSE TO FINE GRAVEL AND COARSE TO FINE SAND - (CONSTRUCTION ROADWAY)	
5	1	8		DARK GRAY, STIFF, SILTY CLAY, TRACE ORGANICS, FREQUENT THIN SEAMS OF FINE SAND	
10	2	P			AL,M&D,UU
	3	8			
15	4	0		BECOMING SOFT	
			CL/OL		
20	5	P			
	6	2			
25				BECOMING MORE PLASTIC AT 24 FEET	
	7	P			AL (on UC) M&D,Consol,G_s, UC,AL(on Consol)
	8	0			
30	9	P			
	10	5		BECOMING MEDIUM STIFF	
35	11	P			AL (on Consol), M&D, Consol,G_s, AL (on UC),UC
	12	2			
40			SP		

ELEVATIONS AND LOCATIONS REFER TO PSE&G PLANT DATUM

ELEVATIONS AND LOCATIONS WERE PROVIDED BY BECHTEL POWER CORPORATION

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

FIGURE 2.5-50

LOG OF BORING
BORING 5
EL. +104.2

Location: N 26+43, W 06+01
Completion Date: 3/13/78

SAMPLE				UNIFIED SOIL CLASSIFICATION		Lab Test
Depth Feet	No.	SYM	Blows Ft.	SYM.	Description	
	12A		28	SP	DENSE, COARSE TO FINE SAND, OCCASIONAL GRAVEL	
45	13		33	SM	BROWNISH GREEN, DENSE, COARSE TO FINE SAND, OCCASIONAL GRAVEL, LAYERS OF SILT	
50	14		12	CL	DARK GRAY, STIFF, SILTY CLAY, OCCASIONAL FINE GRAVEL	
55	15		16		GRADING WITH ORGANICS	
60	16		100			SA
65	17		73	SW/SM	VERY DENSE, COARSE TO FINE SAND, SOME FINE GRAVEL, LITTLE TO TRACE SILT	
70	18		36			
75	19		59	SM	GREENISH GRAY, VERY DENSE, FOSSILIFEROUS, MEDIUM TO FINE SAND, SOME SILT	SA
80	20		01/10'		BORING TERMINATED AT 80'10"	

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

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FIGURE 2.5-50

LOG OF BORING
BORING 6
EL. +103.8
DRILLING METHOD: TRUCK MOUNTED ROTARY MUD WASH DRILL RIG

Location: N 26+30, W 02+70
Completion Date: 3/14/78

Depth Feet	No.	SYM	Blows Ft.	SYM.	Description	Lab Test
				SP	BROWN, DENSE, COARSE TO FINE SAND AND GRAVEL (CONSTRUCTION FILL)	
5	1		41			
10	2		48	SM	DARK GRAY, DENSE, COARSE TO FINE SAND, SOME TO LITTLE SILT, LITTLE TO TRACE FINE GRAVEL, TRACE ORGANICS	
15	2A		17			
					GRADING TO	
20	3		P	CH	DARK GRAY, MEDIUM STIFF, CLAY, OCCASIONAL FINE GRAVEL, FREQUENT THIN SEAMS OF FINE SAND	AL, M&D, UU
	4		6	SM	DARK GRAY, FINE SAND AND SILT, OCCASIONAL FINE GRAVEL	SA
25	5		3			
30	6		P	CH	DARK GRAY, SOFT, CLAY, FREQUENT THIN SEAMS OF FINE SAND	
	7		1			
35	8		4			
				SP	SANDY MATERIAL DETECTED BY DRILLING OPERATION	
40				ML OL	GRAY, VERY STIFF TO STIFF, CLAYEY SILT, TRACE FINE SAND	

ELEVATIONS AND LOCATIONS REFER TO PSE&G PLANT DATUM
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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

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Sheet 34 of 189
FIGURE 2.5-50

LOG OF BORING
BORING 6
EL. +103.8

PAGE 2 OF 2

Location: N 26+30, W 02+70
Completion Date: 3/14/78

SAMPLE				UNIFIED SOIL CLASSIFICATION		
Depth Feet	No.	SYM	Blows/Ft.	SYM.	Description	Lab Test
	9		P	ML/OL		AL,M&D,Consol, G_s,UC,AL (on consol)
	10		14			
45	11		14			
50	12		49	SM	GREENISH BROWN AND PURPLISH BROWN, DENSE, MEDIUM TO FINE SAND, LITTLE TO TRACE SILT	SA
55	13		25	SM	GREENISH BROWN, MEDIUM DENSE, FOSSILIFEROUS, MEDIUM TO FINE SAND, SOME TO LITTLE SILT	
60	14		29		BECOMING REDDISH BROWN	
					BORING TERMINATED AT 61'6"	
65						
70						
75						
80						

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 35 of 189
FIGURE 2.5-50

LOG OF BORING
BORING 7
EL. +107.4
DRILLING METHOD: TRUCK MOUNTED ROTARY
MUD WASH DRILL RIG

Location: N 22+70, W 01+10
Completion Date: 3/1/78

SAMPLE Depth Feet	No.	Blows Ft.	UNIFIED SOIL CLASSIFICATION SYM.	Description	Lab Test
	0	9	SM	MEDIUM DENSE, COARSE TO FINE SAND, SOME GRAVEL, LITTLE SILT (CONSTRUCTION FILL)	
5	1	P			MA,AL,M&D,UC,UU
			CH	DARK GRAY, SOFT, CLAY, LITTLE FINE SAND, TRACE ORGANICS, TRACE GRAVEL	
10	2	117	SM	DARK GRAY, DENSE, COARSE TO FINE SAND, SOME SILT, SOME FINE GRAVEL	SA
				GRADES TO	
15	3	P	ML	DARK GRAY, SOFT, SILT AND MEDIUM TO FINE SAND	
20	4	P			SA,M&D
	5	8			
25	6	P			MA,AL,M&D,Consol, G_s,UC,AL(on consol)
			CH/OH	DARK GRAY, SOFT, CLAY, FREQUENT THIN SEAMS OF FINE SAND	
30	7	P			MA,AL,M&D
	8	2			
35	9	P			SA,M&D
				BECOMING MEDIUM STIFF TO STIFF	
40					

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

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LOG OF BORING
BORING 7
EL. +107.4

Location: N 22+70, W 01+10
Completion Date: 3/1/78

Depth Feet	No.	SYM	Blows Ft.	SYM.	Description	Lab Test
	10		P	CH/OH		SA,AL,M&D,Consol G_s
	11		26	SM	GRAY, DENSE TO MEDIUM DENSE, MICACIOUS FINE SAND AND SILT	SA
45	12		P	CH	DARK GRAY, STIFF,CLAY	
	13		102	SM	YELLOWISH BROWN, VERY DENSE, COARSE TO FINE SAND, LITTLE FINE GRAVEL, LITTLE SILT	
50	14		P			SA,M
				SM	GRAY, DENSE, FINE SAND, SOME TO LITTLE SILT	
55	15		47			
					GRADING WITH SOME GRAVEL AT 59 FEET	
60	16		45			
					GREENISH GRAY AND ORANGE, DENSE,FOSSILIFEROUS, MEDIUM TO FINE SAND, SOME SILT	
65	17		42		BECOMING REDDISH BROWN	
				SM		
70	18		33			
75	19		38		BECOMING REDDISH BROWN TO GREENISH BROWN	
					BORING TERMINATED AT 76'6"	
80						

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

FIGURE 2.5-50

LOG OF BORING
BORING 8
EL. 108.4
DRILLING METHOD: TRUCK MOUNTED ROTARY
MUD WASH DRILL RIG

Location: N 22+60, W 02+80
Completion Date: 3/2/78

SAMPLE Depth Feet	No.	SYM	Blows Ft.	UNIFIED SOIL CLASSIFICATION SYM.	Description	Lab Test
	1		12	GP	MEDIUM DENSE, COARSE TO FINE GRAVEL, SOME COARSE TO FINE SAND, TRACE SILT (CONSTRUCTION FILL)	SA
5	2		16	SM	DARK GRAY, MEDIUM DENSE, COARSE TO FINE SAND, SOME SILT, TRACE FINE GRAVEL	SA
10	3		5		GRADING WITH MORE SILT	MA
15	4		11	CH/OH	DARK GRAY, MEDIUM STIFF, CLAY, TRACE FIBROUS ORGANICS, TRACE FINE SAND	
20	5		P			MA, AL, M&D, Consol, UC, UU, G_s,
	6		0			AL (on consol)
25	7		18	SM	GRAY, MEDIUM DENSE, FINE SAND, LITTLE SILT	SA
					DARK GRAY, MEDIUM STIFF, CLAY, FREQUENT THIN SEAMS OF FINE SAND	
30	8		P			MA, AL, M&D, Consol, G_s, UC,
	9		3			SA
35	10		8	CH/OH		
40						

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HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

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FIGURE 2.5-50

LOG OF BORING
BORING 8
EL. 108.4

Location: N 22+60, W 02+80
Completion Date: 3/2/78

SAMPLE				UNIFIED SOIL CLASSIFICATION		
Depth Feet	No.	SYM	Blows/Ft.	SYM.	Description	Lab Test
	11		P	CH/OH	BECOMING STIFF	SA,AL,M&D,Consol Gs,UC,UU
	12		25			
45	13		65	SP/SM	BROWN AND GRAY, VERY DENSE, COARSE TO FINE SAND, LITTLE COARSE TO FINE GRAVEL, TRACE SILT	SA
50	14		16	CL	BROWN,STIFF,SILTY CLAY, TRACE FINE SAND	SA,AL
55	15		P		GRAY AND REDDISH BROWN, DENSE, FINE SAND, SOME SILT IN LENSES	SA,M&D
	16		47	SM		SA
60	17		93		BECOMING VERY DENSE	
65	18		9		ORANGE BROWN, LOOSE, MEDIUM TO FINE SAND, SOME TO LITTLE SILT	SA
70	19		19	SM	BECOMING MEDIUM DENSE	
75	20		29		BECOMING DENSE	
					BECOMING GREENISH GRAY	
80						

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

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FIGURE 2.5-50

LOG OF BORING
BORING 8
EL. 108.4

Location: N 22+60, W 02+80
Completion Date: 3/2/78

SAMPLE				UNIFIED SOIL CLASSIFICATION		
Depth Feet	No.	SYM	Blows Ft.	SYM.	Description	Lab Test
	21		40			
				SM		
85	22		38			
					BORING TERMINATED AT 86'6"	
90						

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 40 of 189
FIGURE 2.5-50

LOG OF BORING
BORING 9
EL. 108.2
DRILLING METHOD: TRUCK MOUNTED ROTARY
MUD WASH DRILL RIG

Location: N 19+20, W 03+00
Completion Date: 3/15/78

SAMPLE				UNIFIED SOIL CLASSIFICATION		Lab Test
Depth Feet	No.	SYM	Blows / Ft.	SYM.	Description	
				SP	MEDIUM DENSE, COARSE TO FINE SAND AND CRUSHED STONE (CONSTRUCTION FILL)	
5	0		12			
				CH / OH	DARK GRAY, STIFF TO MEDIUM STIFF, CLAY, FREQUENT THIN SEAMS OF FINE SAND, TRACE ORGANICS	
10	1		11			
15	1A		10			
20	2		P			AL,M&D,UC
	3		15			
25	4		P			AL,M&D,UC
	5		4			
30	6		P			AL,M&D,Consol, G_s,AL(on consol) UC
	7		4			
35	8		0			
				GP	GRAVELLY MATERIAL-DETECTED BY DRILLING OPERATIONS	
				CL	GRAY, STIFF TO MEDIUM STIFF, SILTY CLAY	
40						

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HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

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FIGURE 2.5-50

LOG OF BORING
BORING 9
EL. 108.2

Location: N 19+20, W 03+00
Completion Date: 3/15/78

Depth Feet	No.	SYM	Blows Ft.	SYM.	Description	Lab. Test
	9		P			
	10		9	CL		
45	11		P	SC	GRADING TO GRAY BROWN, MEDIUM TO FINE SAND, SOME CLAY	
	12		11			
50	13		13	CH/OH	GRAY, VERY STIFF TO STIFF, CLAY, TRACE ORGANICS	
55	14		P			AL, M&D, Consol, G_s, AL (on consol), UC
	15		12			
60	16		23	ML	PURPLISH, STIFF, SILT, SOME FINE SAND, TRACE ORGANICS	SA, M
				SM	REDDISH-BROWN, VERY DENSE, MEDIUM TO FINE FOSSILIFEROUS SAND, SOME SILT	
65	17		65			
70	18		30		BECOMING GREENISH GRAY, DENSE	
75	19		41		BECOMING REDDISH BROWN	SA, M
80						

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

FIGURE 2.5-50

LOG OF BORING
BORING 9
EL. 108.2

PAGE 3 OF 3

Location: N 19+20, W 03+00
Completion Date: 3/15/78

SAMPLE				UNIFIED SOIL CLASSIFICATION		Lab Test
Depth Feet	No.	SYM	Blows Ft.	SYM.	Description	
	20		31	SM		
					BORING TERMINATED AT 81'6"	
85						

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

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Sheet 43 of 189
FIGURE 2.5-50

LOG OF BORING
BORING 10
EL. 101.7
DRILLING METHOD: TRUCK MOUNTED ROTARY MUD WASH DRILL RIG

PAGE 1 OF 2

Location: N 19+25, W 07+60
Completion Date: 3/17/78

Depth Feet	No.	SYM	Blows Ft.	SYM.	Description	Lab Test
				GP	CRUSHED SLAG (ROADWAY)	
				SP	MEDIUM DENSE, COARSE TO FINE SAND AND CRUSHED STONE (CONSTRUCTION FILL)	
5	1		10			
					DARK GRAY, MEDIUM STIFF, CLAY, TRACE FINE SAND, LITTLE FIBROUS ORGANICS	
10	2		P			AL,M&D,UU
					GRADING WITH MORE SAND	
	3		14			SA,M
15					BECOMING MEDIUM STIFF TO SOFT	
	4		3	CH/OH		
20					GRADING WITH FREQUENT THIN SEAMS OF FINE SAND	
	5		P			AL,M&D,UU
	6		2			
25	7		2			
30	8		P			AL,M&D,UC
	9		3			
35				GP	GRAVELLY MATERIAL - DETECTED FROM DRILLING OPERATIONS	
	10		7			
				CL/OL	DARK GRAY, STIFF, SILTY CLAY, TRACE ORGANICS	
40						

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HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 44 of 189
FIGURE 2.5-50

LOG OF BORING
BORING 10
EL. 101.7

Location: N 19+25, W 07+60
Completion Date: 3/17/75

Depth Feet	No.	SYM	Blows/Ft.	SYM.	Description	Lab Test
	11		P			
	12		7			
45	13		8	CL/OL		
50	14		P			
	15		47		GREENISH AND PURPLISH, VERY DENSE, FINE SAND, SOME SILT	
55	16		52			SA,M
				SM		
60	17		85			
65	18		111	SW/SM	GRAY BROWN, VERY DENSE, COARSE TO FINE SAND AND FINE GRAVEL, TRACE SILT	SA,M
					GREENISH GRAY, VERY DENSE TO DENSE, MEDIUM TO FINE SAND, SOME SILT	
70	19		34	SM		
75	20		59			
					BORING TERMINATED AT 76'6"	
80						

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

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FIGURE 2.5-50

LOG OF BORING
BORING 12
EL. 105.6
DRILLING METHOD: TRUCK MOUNTED ROTARY MUD WASH DRILL RIG

PAGE 1 OF 2

Location: N 20+40, W 01+25
Completion Date: 3/2/78

SAMPLE				UNIFIED SOIL CLASSIFICATION		Lab Test
Depth Feet	No.	SYM	Blows Ft	SYM.	Description	
	1		P		ORANGE, VERY DENSE, COARSE TO FINE SAND, SOME TO LITTLE COARSE TO FINE GRAVEL, LITTLE SILT (CONSTRUCTION FILL)	SA
5	2		104	SM		
10	3		P	CH	DARK GRAY, STIFF TO MEDIUM STIFF, CLAY AND COARSE TO FINE SAND, TRACE FINE GRAVEL	SA
	4		P	SP	DARK GRAY, MEDIUM DENSE, COARSE TO FINE SAND, TRACE SILT	SA
15					DARK GRAY, STIFF TO MEDIUM STIFF, CLAY, FREQUENT THIN SEAMS OF FINE SAND, TRACE ORGANICS	
20	5		P			TOP: MA, AL, M&D, UC; BOTTOM: SA
	6		10	CH/OH		SA
25	7		1			AL
30	8		P			MA, AL, M&D, UC, UU
35	9		P			MA, AL, M&D, UC
	10		3			
40				SW/SM	TAN AND DARK GRAY, VERY DENSE, COARSE TO FINE SAND, SOME FINE GRAVEL, LITTLE SILT	

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

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FIGURE 2.5-50

LOG OF BORING
BORING 12
EL. 105.6

Location: N 20+40, W 01+25
Completion Date: 3/2/78

Depth Feet	No.	SYM	Blows/Ft.	SYM.	Description	Lab Test
	11		62	SW/SM		SA
					GRAY, STIFF, CLAY, TRACE ORGANICS	
45	12		10	CH/OH		AL
50	13		P	CL/ML	GRADING TO: BROWNISH GRAY, VERY STIFF, CLAYEY SILT, SOME FINE SAND	TOP: SA,AL,M&D, Consol,UC,G_s; BOTTOM: SA,M&D
					GRADING TO	
55	14		42	SM	GRAY, DENSE, COARSE TO FINE SAND, SOME SILT, LITTLE FINE GRAVEL	
60	15		27			SA
					BORING TERMINATED AT 61'6"	
65						

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 47 of 189
FIGURE 2.5-50

LOG OF BORING
BORING 13
EL. 101.6
DRILLING METHOD: TRUCK MOUNTED ROTARY MUD WASH DRILL RIG

PAGE 1 OF 3

Location: N 19+00, E 00+00
Completion Date: 3/6/78

Depth Feet	No.	SYM	Blows/Ft.	SYM.	Description	Lab Test
				GP	COARSE TO FINE GRAVEL, SOME COARSE TO FINE SAND (CONSTRUCTION FILL)	
5	1		P		DARK GRAY, MEDIUM STIFF, CLAY, LITTLE ORGANICS, FREQUENT THIN SEAMS OF FINE SAND	AL
10	2		P			SA,M&D,UU
15	3		12			
				CH/OH		
20	4		0			
	5		P			AL,M&D,UC
25						
30	6		2			
					GRADING TO GRAY, MEDIUM DENSE, COARSE TO FINE SAND, LENSES OF CLAY, LITTLE FINE GRAVEL	
35	7		P	SC		MA,M&D
					GRADING TO DARK GRAY, MEDIUM STIFF, CLAY, TRACE ORGANICS	
40				CH/OH		

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 48 of 189
FIGURE 2.5-50

Location: N 19+00, E00+00
Completion Date: 3/6/78

LOG OF BORING
BORING 13
EL. 101.6

SAMPLE Depth Feet	No.	SYM	Blows/Ft.	UNIFIED SOIL CLASSIFICATION SYM.	Description	Lab Test
	8		15	CH/OH		
				GP	GRAVELLY MATERIAL DETECTED BY DRILLING OPERATIONS	
45	9		19	SP	GREENISH GRAY, MEDIUM DENSE, MEDIUM TO FINE SAND, TRACE SILT	
50	10		3		REDDISH BROWN, MEDIUM DENSE TO LOOSE, MEDIUM TO FINE, FOSSILIFEROUS SAND, SOME TO LITTLE SILT	SA,M
55	11		14			
60	12		57	SM	BECOMING VERY DENSE TO DENSE	
65	13		35			
70	14		35		BECOMING GREENISH GRAY	
75	15		54			
80						

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

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LOG OF BORING
BORING 13
EL. 101.6

Location: N 19+00, E 00+00
Completion Date: 3/6/78

SAMPLE				UNIFIED SOIL CLASSIFICATION		
Depth Feet	No.	SYM	Blows/Ft.	SYM.	Description	Lab Test
	16		101			
				SM		
85	17		65			
					BORING TERMINATED AT 86'6"	
90						

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

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FIGURE 2.5-50

LOG OF BORING
BORING 15
EL. 75.3
DRILLING METHOD: BARGE MOUNTED ROTARY
WATER WASH DRILL RIG

PAGE 1 OF 2

Location: N 26+60, W 18+80
Completion Date: 3/31/78

Depth				
to Mud Line	13'	12½'	11'10"
Time	09:15	10:00	10:20
Date	3/30/78	3/30/78	3/30/78

SAMPLE Depth Feet	No.	SYM	Blows Ft.	UNIFIED SOIL CLASSIFICATION SYM.	Description	Lab Test
	1		0	SM	DARK GRAY, VERY LOOSE, MEDIUM TO FINE SAND, SOME SILT	
5	2		0			SA
				SP	GRADING TO GRAY, MEDIUM DENSE, MEDIUM TO FINE SAND	
10	3		10			
					GRADING WITH OCCASIONAL THIN LAMINATIONS OF SILTY SAND	
15	4		11			
				CH	GRAY, STIFF, CLAY	
20	5		15			AL,MA
				SM	GRAY, MEDIUM DENSE, MEDIUM TO FINE SAND, LITTLE SILT, LITTLE COARSE TO FINE GRAVEL	
25	6		26			
					BECOMING DARK GREEN	
30	7		27			SA
35	8		27	SM	GREENISH GRAY, DENSE TO MEDIUM DENSE, MEDIUM TO FINE SAND, SOME SILT	
40						

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 51 of 189
FIGURE 2.5-50

LOG OF BORING
BORING 15
EL. 75.3

Location: N 26+60, W 18+80
Completion Date: 3/31/78

Depth			
to Mud Line	13½'	16'	17½'
Time	11:45	13:20	15:00
Date	3/30/78	3/30/78	3/30/78

SAMPLE Depth Feet	No.	SYM	Blows Ft.	SYM.	UNIFIED SOIL CLASSIFICATION Description	Lab Test
	9		41	SM	BORING TERMINATED AT 41'6"	
45						

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 52 of 189
FIGURE 2.5-50

LOG OF BORING PAGE 1 OF 2
BORING 16
EL. 100.7
DRILLING METHOD: TRUCK MOUNTED ROTARY MUD WASH DRILL RIG

Location: N 26+72, W 15+84
Completion Date: 3/17/78

Depth Feet	Sample No.	Blows Ft.	SYM.	Description	Lab Test
			GW	COARSE GRAVEL (TEMPORARY ROADWAY)	
				DARK GRAY, SOFT TO VERY SOFT, CLAY, TRACE FIBROUS ORGANICS	
5	1	P			AL,M&D,UU
10	2	P			AL,M&D,UC
	3/vs				AL
15	4	P			M&D,UU
	5/vs				
	6	P	CH/OH		AL(on UC),M&D,UC, Consol, G_s, AL (on consol)
20	7/vs				AL
	8	P			AL,M&D,UC
25	9/vs				AL
	10	P		BECOMING MEDIUM STIFF TO SOFT, GRADING WITH FREQUENT THIN SEAMS OF FINE SAND	AL,M&D,UC
30	11	P		BECOMING GRAYISH BROWN	AL,M&D,UC
35	12	P			
	13/vs	7	ML	GRAY, SOFT, SILT AND FINE SAND	
40					

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

FIGURE 2.5-50

LOG OF BORING
BORING 16
EL. 100.7

Location: N 26+72, W 15+84
Completion Date: 3/17/78

Depth Feet	No.	SYM.	Blows Ft.	SYM.	Description	Lab Test
	14		7	ML		SA
45	15		44	SW/SM	GRAY, DENSE, COARSE TO FINE SAND, SOME FINE GRAVEL, LITTLE TO TRACE SILT	SA
50	16		6	CL	GRAY, MEDIUM STIFF, SILTY CLAY, TRACE FINE SAND	
55	17		P			AL,M&D,UC
			GW		COARSE GRAVEL AND SAND	
					BORING TERMINATED AT 55'	

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HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

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Sheet 54 of 189
FIGURE 2.5-50

LOG OF BORING
BORING 17
EL. 102.1
DRILLING METHOD: TRUCK MOUNTED ROTARY
MUD WASH DRILL RIG

Location: N 27+50, W 15+84
Completion Date: 3/14/78

SAMPLE				UNIFIED SOIL CLASSIFICATION		Lab Test
Depth Feet	No.	SYM	Blows Ft.	SYM.	Description	
				GW	CRUSHED SLAG (CONSTRUCTION FILL)	
5	1		P	CH/OH	DARK GRAY, SOFT, CLAY, TRACE FIBROUS ORGANICS	AL,M&D,UC
	2/vs					AL
10	3		P			AL,M&D,UU
	4/vs					AL
15	5		P			AL,M&D,UC
	6/vs					AL
	7		P			AL,M&D,UU
20	8/vs					AL
	9		P			AL,M&D,UC
	10/vs					AL
25	11		P		GRADING WITH OCCASIONAL SEAMS OF FINE SAND	AL,M&D,UC
	12/vs					AL
30	13		P			AL,M&D,UC
	14		9	ML	GRAY, SOFT, SILT AND FINE SAND	SA
35	15		P			
	16		4			
40				CH/OH		

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HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

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FIGURE 2.5-50

LOG OF BORING
BORING 17
EL. 102.1

PAGE 2 OF 2

Location: N 27+50, W 15+04
Completion Date: 3/14/78

SAMPLE				UNIFIED SOIL CLASSIFICATION		Lab Test
Depth Feet	No.	SYM	Blows Ft.	SYM.	Description	
				CH/OH	GRAY, MEDIUM STIFF TO SOFT, CLAY, FREQUENT THIN SEAMS OF MEDIUM TO FINE SAND	
	17		P			AL(on UC), M&D, Consol, G_s,
45	18		5			AL(on consol), UC
				SP/SM	GRAY, DENSE, MEDIUM TO FINE SAND, TRACE SILT	
50	19		P			SA, M&D
	20		39		GRADING WITH LITTLE GRAVEL	
55	21		7	CL	GRAY AND BROWN, STIFF, SILTY CLAY, TRACE FINE SAND	
60	22		P			
					BORING TERMINATED AT 62'	
65						
70						
75						
80						

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 56 of 189
FIGURE 2.5-50

LOG OF BORING
BORING 18
EL. 102.1
DRILLING METHOD: TRUCK MOUNTED ROTARY
MUD WASH DRILL RIG

Location: N 27-36, W 14+62
Completion Date: 3/9/78

SAMPLE Depth Feet	No.	SYM	Blows Ft.	UNIFIED SOIL CLASSIFICATION SYM.	Description	Lab Test
				GP	COARSE GRAVEL AND CRUSHED SLAG (CONSTRUCTION FILL)	
				CH/OH	DARK GRAY, SOFT TO VERY SOFT, CLAY, TRACE FIBROUS ORGANICS	
5	1		P			AL(on UU),MA, M&D, Consol, G_s,AL(on consol), UU
10	2		P			AL,M&D,UC
	3		2			
15	4		P		GRADING WITH FREQUENT THIN SEAMS OF FINE SAND	AL,M&D,UC
20	5		P			AL,M&D,Consol, G_s
	6		P			AL
25	7		P			AL,MA,M&D,UU,UC
30	8		P			AL,M&D
	9		4			
35	10		P			AL(on UC),MA,M&D, Consol,G_s,AL (on consol),UC
40				SP		

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 57 of 189
FIGURE 2.5-50

LOG OF BORING
BORING 15
EL. 102.1

Location: N 27+36, W 14+62
Completion Date: 3/9/78

SAMPLE				UNIFIED SOIL CLASSIFICATION		
Depth Feet	No.	SYM	Blows Ft.	SYM.	Description	Lab Test
	11		P	SP	GREENISH GRAY, LOOSE, MEDIUM TO FINE SAND, TRACE SILT	SA,M&D
	12		6	CH/OH	GRADING TO DARK GRAY, MEDIUM STIFF, CLAY, FREQUENT THIN SEAMS OF FINE SAND, TRACE FIBROUS ORGANICS	
45	13		P		DARK GRAY, MEDIUM DENSE, MEDIUM TO FINE SAND, LITTLE SILT	SA,M&D
50	14		56	SM	BECOMING VERY DENSE	
55					BECOMING GREEN, DENSE	
	15		39			
					GRAY AND BROWN, STIFF, CLAY, LITTLE FIBROUS ORGANICS	
60	16		P			AL,M&D,UC
	17		4	CH/OH		
65						
				SM	GREENISH GRAY,DENSE,MEDIUM TO FINE, FOSSILIFEROUS SAND,SOME SILT	
70	18		38			SA,M
					BORING TERMINATED AT 71'6"	
75						
80						

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

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FIGURE 2.5-50

Location: N 26+37, W 12+95
Completion Date: 3/21/78

LOG OF BORING
BORING 19
EL. 100.9
DRILLING METHOD: TRUCK MOUNTED ROTARY
MUD WASH DRILL RIG

SAMPLE: Depth Feet	No.	SYM	Blows/Ft.	UNIFIED SOIL CLASSIFICATION: SYM.	Description	Lab Test
				GP	COARSE GRAVEL (CONSTRUCTION FILL)	
					DARK GRAY, SOFT TO VERY SOFT, CLAY, LITTLE FIBROUS ORGANICS	
5	1		P	CH/OH		AL,M&D,UU
	2/vs					AL
10	3		P			AL,M&D,UC
	4/vs			SM	GRAY, MEDIUM DENSE, FINE SAND AND SILT	
	5		P		DARK GRAY, SOFT TO VERY SOFT, CLAY, LITTLE FIBROUS ORGANICS	AL(on UC),M&D,
15						Consol,G_s, AL(on consol),UC
	6/vs					AL
	7		P			AL,M&D
20	8/vs					AL
	9		P	CH/OH		AL,M&D,UU
	11/vs				GRADING WITH FREQUENT THIN SEAMS OF FINE SAND	AL
25	12		P			AL,M&D,UC
	13/vs					AL
30	14		P			M&D
				SM	GRAY,MEDIUM DENSE,FINE SAND,SOME TO LITTLE SILT	
35	15		P	CH/OH		AL,M&D,UC
	16/vs					AL
40						

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HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

FIGURE 2.5-50

LOG OF BORING
BORING 19
EL. 100.9

Location: N 26+37, W 12+95
Completion Date: 3/21/78

SAMPLE				UNIFIED SOIL CLASSIFICATION		Lab Test
Depth Feet	No.	SYM	Blows Ft.	SYM.	Description	
	17		P	CH/OH		AL,M&D,UC
				SP	GRAY, DENSE, FINE SAND, TRACE SILT	
45	18		40			
				CL	BROWNISH GRAY, STIFF TO MEDIUM STIFF, SILTY CLAY	
50	19		8	CL/ML	REDDISH BROWN, STIFF TO MEDIUM STIFF, SILT, SEAMS OF CLAY, LITTLE TO TRACE MEDIUM TO FINE SAND	
55				CH/OH	GRAYISH BROWN, VERY STIFF TO STIFF, CLAY	
	20		P			AL,M&D,UC
				PT	BROWN, VERY STIFF, CLAYEY PEAT	
60	21		27			
					BORING TERMINATED AT 61'6"	
65						

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 60 of 189
FIGURE 2.5-50

LOG OF BORING　　　PAGE 1 OF 3
BORING 20
EL. 101.1
DRILLING METHOD: TRUCK MOUNTED ROTARY MUD WASH DRILL RIG

Location: N 24+20, W 14+30
Completion Date: 3/10/78

SAMPLE				UNIFIED SOIL CLASSIFICATION		
Depth Feet	No.	SYM	Blows Ft.	SYM.	Description	Lab Test
				SP	LIGHT BROWN, DENSE, COARSE TO FINE SAND, LITTLE COARSE TO FINE GRAVEL, TRACE SILT (CONSTRUCTION FILL)	
5	1		35			
				CH/OH	DARK GRAY, MEDIUM STIFF TO SOFT, CLAY, LITTLE FIBROUS ORGANICS	
10	2		P			AL(on UU), M&D, Consol, G_s,
	3		5			AL(on consol), UU
15	4		P			
20	5		P			AL, M&D, UC
	6		0			
25	7		0			
30					GRADING WITH FREQUENT THIN SEAMS OF FINE SAND	
	8		P			
	9		7			
35	10		85	SW/SM	BROWNISH GRAY, VERY DENSE, COARSE TO FINE SAND, SOME FINE GRAVEL, LITTLE SILT	SA, M
40						

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HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

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FIGURE 2.5-50

LOG OF BORING
BORING 20
EL. 101.1

Location: N 24+20, W 14+30
Completion Date: 3/10/78

SAMPLE				UNIFIED SOIL CLASSIFICATION		Lab Test
Depth Feet	No.	SYM	Blows Ft.	SYM.	Description	
	11		P	ML	BROWN, STIFF, SILT	
					GREEN, DENSE, MEDIUM TO FINE SAND, TRACE SILT	
	12		32	SP/SM		
45	13		17	ML	GRAY BROWN, VERY STIFF, SILT, WITH SANDY SILT LAYERS, OCCASIONAL WOOD FRAGMENTS	
					GRADING TO GRAY, VERY STIFF TO STIFF, CLAY, TRACE ORGANICS	
50	14		P			AL(on UC), M&D, Consol, G_s,
	15		5			AL(on consol), UC
55	16		19	CH/OH		
60					BECOMING BROWN	
	17		P			AL, M&D, UC
	18		7			
65	19		82/9"	SM	GREENISH BROWN, VERY DENSE, COARSE TO FINE SAND, SOME COARSE TO FINE GRAVEL, SOME TO LITTLE SILT	SA, M
70	20		60		GREENISH GRAY, VERY DENSE, MEDIUM TO FINE FOSSILIFEROUS SAND, SOME SILT	
75	21		103	SM		SA, M
80						

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

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FIGURE 2.5-50

LOG OF BORING
BORING 20
EL. 101.1

Location: N 24+20, W 14+30
Completion Date: 3/10/78

SAMPLE				UNIFIED SOIL CLASSIFICATION		
Depth Feet	No.	SYM	Blows/Ft.	SYM.	Description	Lab Test
	22		58	SM		
					BORING TERMINATED AT 81'6"	
85						

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

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FIGURE 2.5-50

LOG OF BORING PAGE 1 OF 1

BORING OW-14D
EL. +101
DRILLING METHOD: TRUCK MOUNTED ROTARY REVERT WASH DRILL RIG

Location: N 16+99, E 5+74
Completion Date: 3/22/78

SAMPLE				UNIFIED SOIL CLASSIFICATION		Lab Test
Depth Feet	No.	SYM	Blows Ft.	SYM.	Description	
					STRAIGHT DRILL TO 55 FEET. SAMPLED WITH D&M TYPE U SAMPLER IN WELL OW14D 300 LB. HAMMER, 24 INCH DROP	
50						
55	1		8	SM	DARK GREEN, LOOSE, FINE SAND AND SILT	
	2		7	ML	DARK GRAY, SOFT, SILT AND FINE SAND, TRACE FINE GRAVEL	
60	3		6	SM	BROWN AND GREEN, MEDIUM DENSE TO LOOSE, MEDIUM TO FINE SAND, SOME TO LITTLE SILT	
	4		11			
65	5		12	SM	REDDISH BROWN, DENSE, MEDIUM TO FINE SAND, SOME SILT	
					SAMPLING TERMINATED AT 65 FEET	
70						

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 64 of 189
FIGURE 2.5-50

LOG OF BORING:
BORING NUMBER: B-1
ELEVATION: 101
DRILLING METHOD: TRUCK MOUNTED ROTARY WASH (MUD) DRILL RIG

Location: SEE FIGURE 1
Completion Date: 5/7/79

Depth Feet	No.	Blows/Ft.	SYM.	Description	Lab Test	BLOWS Two(2) Inches 2 4 6 8 10 12
5	1	5 / 11	ML	GRAY, MEDIUM CLAYEY SILT, LITTLE F-M SAND		
10	2	12 / 13	SP / SM	GRAY, MEDIUM DENSE SILTY SAND, TRACE FINE GRAVEL		
15	3	3 / 7		GRAY, LOOSE SANDY SILT, LITTLE FINE SAND		
20	4	2 / 1	ML	GRAY, VERY SOFT CLAYEY SILT, LITTLE F-M SAND		
25	5	2 / 4		GRADING MORE CLAY		
30	6	4 / 7		GRADING MEDIUM		
35	7	7 / 75	SP / GP	GRAY AND BROWN, VERY DENSE SAND AND GRAVEL		
40						

REFERENCE FOR BORINGS B-1 AND B-2:
PSE&G, MAY 1979, ADDITIONAL SUBSURFACE SOILS INVESTIGATION FOR DESIGN OF PIPE FOUNDATION, HOPE CREEK GENERATING STATION LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 65 of 189
FIGURE 2.5-50

LOG OF BORING:
BORING NUMBER: B-1
ELEVATION: 101
DRILLING METHOD: TRUCK MOUNTED ROTARY WASH (MUD) DRILL RIG

Location:
Completion Date: 5/7/79

SAMPLE Depth Feet	No.	Blows Ft.	UNIFIED SOIL CLASSIFICATION SYM.	Description	Lab Test	BLOWS Two(2) Inches 2	4	6	8	10	12
		47	SP/GP	GRADING MEDIUM DENSE							
		21		(NO RECOVERY)							
	8	P	CL	GRAY SILTY CLAY, TRACE TO LITTLE FINE SAND	AL,MD						
45					UC						
	9	7			AL						
		18									
	10	P	SP/SM		AL,SA						
50				RED BROWN, MEDIUM DENSE, SILTY SAND	MD						
		25			SA						
	11	52									
		28			SA	3	5	5	5	6	4
	12	28				4	4	5	5	5	5
		38			SA	3	5	4	4	9	13
55	13	59				16	15	10	5	6	7
		18	SM	GRADING GREENISH GRAY	SA	1	3	3	4	3	4
	14	29				4	6	6	5	5	3
		17			†	2	3	2	3	4	3
60	15	23		GRADING RED BROWN		5	5	3	3	4	3
		10			SA	1	2	2	2	2	1
	16	20				3	3	3	4	4	3
		5			SA	1	0	0	0	2	2
	17	29				2	7	6	4	5	5
65		23			SA	3	4	3	4	4	5
	18	34				4	5	7	6	5	7
		25			SA	2	4	4	4	6	5
	19	29				5	3	6	6	5	4
		34			SA	2	4	5	7	6	10
	20	107				17	16	13	13	20	28
70		37	SM		SA	7	12	6	4	4	4
	21	33				4	4	5	5	6	9
		41			SA	2	6	5	11	10	7
	22	38				7	6	7	6	6	6
75		28			SA	3	4	4	5	6	6
	23	51				6	6	8	11	11	9
		48			SA	5	5	4	6	8	20
	24	305/5"				47	153	*	-	-	-
		26		GRADING GRAY GREEN	SA	3	4	3	4	5	7
80	25	41				12	6	6	5	6	6

*100/1"
†INSUFFICIENT SAMPLE RECOVERY FOR SIEVE ANALYSIS

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PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

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Sheet 66 of 189
FIGURE 2.5-50

LOG OF BORING:
BORING NUMBER: B-1
ELEVATION: 101
DRILLING METHOD: TRUCK MOUNTED ROTARY
WASH (MUD) DRILL RIG

Location:
Completion Date: 5/7/79

SAMPLE Depth Feet	No.	Blows Ft.	UNIFIED SOIL CLASSIFICATION SYM.	Description	Lab Test	BLOWS Two(2) Inches 2	4	6	8	10	12
	26	47		GRAY GREEN, DENSE TO VERY DENSE, MEDIUM TO	SA	3	22	9	6	3	4
		48		FINE SAND, SOME SILT		4	4	5	5	15	15
	27	46			SA	3	6	5	10	14	8
		136				6	5	35	62	14	14
85	28	25			SA	2	4	4	5	6	4
		43				6	6	6	14	7	4
	29	28			SA	1	0	0	0	11	16
		84				10	7	5	6	8	48
		-				-	-	-	-	-	-
90	30	73			SA	36	9	3	14	7	4
		19				3	3	2	3	2	6
	31	9			SA	2	0	0	0	1	6
		22				4	3	3	4	4	4
	32	121/7"			SA	7	5	9	*	-	-
95	33	20			SA	2	3	3	4	4	4
		46				5	16	7	7	6	5
	34	11			SA	1	1	1	1	4	3
		73				5	7	21	17	14	9
	35	25			SA	2	5	4	4	5	5
100		52				5	7	7	8	16	9
	36	21			SA	2	4	4	4	4	3
		81				4	6	5	32	20	14
	37	25			SA	2	4	4	5	5	5
		235/5"				7	128	*	-	-	-
105	38	31			SA	3	4	5	6	6	7
		138				8	44	50	14	13	9
	39	43			SA	2	4	4	4	12	17
		48				11	7	9	7	7	7
	40	23			SA	3	3	3	4	5	5
110		45				5	5	7	9	9	10
	41	22			SA	1	1	2	3	3	12
		44				11	8	6	6	7	6
	42	47			SA	3	6	5	16	9	8
		46				8	6	7	8	8	9
115	43	73			SA	3	6	37	13	8	6
		171				5	7	10	96	35	18
	44	23			SA	3	4	3	4	4	5
		74				5	4	5	9	37	14
	45	24			SA	3	3	4	4	5	5
120		73				10	17	12	11	11	12

*100/1"

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

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FIGURE 2.5-50

LOG OF BORING:
BORING NUMBER: B-1
ELEVATION: 101.0
DRILLING METHOD: TRUCK MOUNTED ROTARY
WASH (MUD) DRILL RIG

Location:
Completion Date: 5/7/79

SAMPLE				UNIFIED SOIL CLASSIFICATION		Lab Test	BLOWS Two(2) Inches					
Depth Feet	No.	SYM	Blows Ft.	SYM	Description		2	4	6	8	10	12
	46		207/	SM	GRAY GREEN, DENSE TO VERY DENSE MEDIUM TO FINE SAND, SOME SILT	SA	2	5	20	80	*	-
			-				-	-	-	-	-	-
			29			SA	2	4	4	6	7	6
	47		48				6	7	7	8	9	11
125			30			SA	2	3	5	8	6	6
	48		49				7	7	8	8	10	9
			61			SA	4	9	11	13	14	10
	49		113				11	10	10	11	13	58
			120			SA	13	36	25	18	15	13
130	50		93				25	20	13	11	10	14
			24			SA	3	4	3	5	4	5
	51		48				6	7	8	9	8	10
			45			SA	3	5	8	10	9	10
	52		113				13	12	13	18	25	32
135			37			SA	4	5	6	6	8	8
	53		60				9	9	8	11	11	12
			34			SA	4	6	5	6	7	6
	54		60				7	9	12	9	10	13
			57			SA	4	6	5	7	9	26
	55		183				55	53	27	20	15	13
140			31			SA	4	4	6	5	6	6
	56		95				8	9	9	10	20	39
					BORING COMPLETED TO A DEPTH OF 142 FEET ON 5/7/79							
145												
150												
155												
160												

*100/1"

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 68 of 189
FIGURE 2.5-50

LOG OF BORING:
BORING NUMBER: B-2
ELEVATION: 100
DRILLING METHOD: TRUCK MOUNTED ROTARY WASH (MUD) DRILL RIG

Location: SEE FIGURE 1
Completion Date: 4/27/79

Depth Feet	No.	Blows Ft.	SYM.	Description	Lab Test	BLOWS Two(2) Inches 2 4 6 8 10 12
5	1	9 / 15	CL / ML	GREENISH GRAY STIFF SILTY CLAY, LITTLE SAND		
10	-	7 / 6		(NO RECOVERY)		
15	2	3 / 14				
20	3	0 / 2		GRADING SOFT		
25	4	1 / 5	ML / CL	GRADING MORE SILT		
30	5	2 / 5				
35	6	28 / 56	SP	GREENISH GRAY M-F SAND, LITTLE SILT	SA	
40						

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 69 of 189
FIGURE 2.5-50

LOG OF BORING:
BORING NUMBER: B-2
ELEVATION: 100
DRILLING METHOD: TRUCK MOUNTED ROTARY
WASH (MUD) DRILL RIG

Location:
Completion Date: 4/27/79

Depth Feet	No.	Blows Ft.	SYM.	Description (Unified Soil Classification)	Lab Test	Blows Two(2) Inches: 2	4	6	8	10	12
	7	13	SP								
		10	CL	DARK GREENISH GRAY STIFF SILTY CLAY	AL						
45		P		(UNSUCCESSFUL TUBE ATTEMPT)							
					UC,MD						
	8	P			AL						
	9	16									
50		36		BROWN, VERY STIFF SANDY SILT	AL,M						
	10	P	SM	GRAYISH BROWN AND YELLOW, FINE SILTY SAND, LITTLE GRAVEL	MD,SA						
55	11	53		BROWN, DENSE, SILTY SAND, SOME CLAY, LITTLE GRAVEL	SA						
		34									
	12	4		GRADING RED-BROWN, VERY LOOSE	SA						
		0									
60	13	1			SA	1	0	0	0	0	0
		1				1	0	0	0	0	0
	14	4			SA	0	0	0	0	1	3
		31				4	5	5	4	6	7
		24			SA	3	2	3	3	8	5
65	15	30		GRADING MEDIUM DENSE		5	4	6	6	4	5
	16	24			SA	3	4	4	4	5	4
		27				4	4	5	4	5	5
	17	41			SA	3	4	4	5	11	14
70		96				17	9	8	13	21	28
	18	37			SA	7	10	6	5	5	4
		34				4	5	5	5	8	7
	19	33			SA	5	5	5	7	6	5
		61				8	9	7	8	15	14
75	20	33			SA	5	5	5	6	6	6
		31				4	5	6	5	5	6
	21	98/11	SM	GRADING, GRAY GREEN	SA	4	3	4	7	80	*
		-				-	-	-	-	-	-
	22	44			SA	3	2	3	4	16	16
80		72				22	12	10	10	10	9

*100/1"

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

FIGURE 2.5-50

LOG OF BORING:
BORING NUMBER: B-2
ELEVATION: 100
DRILLING METHOD: TRUCK MOUNTED ROTARY
WASH (MUD) DRILL RIG

Location:
Completion Date: 4/27/79

SAMPLE			UNIFIED SOIL CLASSIFICATION		Lab Test	BLOWS Two(2) Inches					
Depth Feet	No.	Blows Ft.	SYM.	Description		2	4	6	8	10	12
	23	19		GRAY GREEN, VERY DENSE, C-F SAND, SOME SILT	SA	4	5	3	2	2	3
		87	SM			5	5	4	5	10	58
		27			SA	6	4	4	4	5	4
	24	149/11"				9	19	7	7	7	*
85		-		HARD ZONES INTERBEDDED WITH SOFTER ZONES, BASED ON DRILLING		-	-	-	-	-	-
		-				-	-	-	-	-	-
		-				-	-	-	-	-	-
		-				-	-	-	-	-	-
		69			SA	4	5	28	23	5	4
90	25	114				2	54	14	15	15	14
		177			SA	3	3	3	80	71	17
	26	40				10	7	6	7	5	5
	27	18			SA	1	1	2	7	4	3
		68				7	25	11	9	8	8
95		18			SA	3	3	3	2	4	3
	28	141/9.7"				4	5	6	6	**	-
		-				-	-	-	-	-	-
		32			SA	3	4	4	5	5	11
	29	120				14	8	8	6	7	77
100		31			SA	2	5	4	6	8	6
	30	57				9	8	11	12	8	9
		36			SA	8	6	6	7	5	4
	31	60				5	5	15	14	11	10
		54			SA	10	19	9	7	4	5
105	32	38				5	6	6	5	8	8
		30			SA	4	3	4	6	7	6
	33	59				7	8	9	11	11	13
		126			SA	2	4	20	68	21	11
	34	56				6	8	7	8	9	18
110		23			SA	5	4	2	2	5	5
	35	54				12	9	8	7	7	11
		22			SA	2	3	4	3	6	4
	36	146/11"				5	5	5	5	26	*
		-				-	-	-	-	-	-
115	37	20			SA	3	4	3	4	3	3
		121/5"				11	10	*	-	-	-
		23			SA	3	3	4	4	4	5
	38	45				5	7	7	8	9	9
		90			SA	2	3	3	9	57	16
120	39	101				11	8	18	25	17	22

*100/1"
**120/1.7"

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

FIGURE 2.5-50

LOG OF BORING:
BORING NUMBER: B-2
ELEVATION: 100
DRILLING METHOD: TRUCK MOUNTED ROTARY
WASH (MUD) DRILL RIG

Location:
Completion Date: 4/47/79

SAMPLE			UNIFIED SOIL CLASSIFICATION		Lab Test	BLOWS Two(2) Inches					
Depth Feet	No.	Blows Ft.	SYM.	Description		2	4	6	8	10	12
		100			SA	4	4	3	40	33	16
	40	49	SM	GRAY GREEN, DENSE TO VERY DENSE, M-F SAND,		11	7	7	7	8	9
	41	24		SOME SILT	SA	2	4	4	5	4	5
		147/11"				7	7	7	14	12	*
125		-				-	-	-	-	-	-
		28			SA	3	6	4	5	6	4
	42	48				7	7	8	8	8	10
		20			SA	1	2	3	4	5	5
	43	143				12	17	13	18	40	43
130		71			SA	6	18	11	10	14	12
	44	62				12	9	10	8	11	12
		36			SA	2	5	5	7	9	8
	45	64				11	9	12	9	11	12
		290			SA	3	11	19	52	25	80
	46	122				19	21	18	19	22	23
135		36			SA	3	5	6	7	8	7
	47	62				9	9	9	10	11	14
		48			SA	4	6	7	10	9	12
	48	155/7"				11	13	31	*	-	-
		-				-	-	-	-	-	-
140	49	42			SA	4	5	7	7	9	10
		224/11.6"				16	12	13	12	21	**
				HOLE COMPLETED TO A DEPTH OF 142 FEET ON 4/27/79. HOLE GROUTED ON 4/30/79							
145											
150											
155											
160											

*100/1"
**150/1.6"

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 72 of 189
FIGURE 2.5-50

ELEVATION: +100
COORDINATES: N 1189
W 1455
DATUM: SITE DATUM

BORING B1

Location: HOPE CREEK
Completion Date: 10/30/81

Water Level	NA		
Time			
Date			
Casing Depth			

SAMPLE				UNIFIED SOIL CLASSIFICATION		
Depth Feet	No.	SYM	Blows/Ft.	SYM.	Description	GEOLOGIC FORMATION
					WATER	
65					DREDGED DEPTH (33)	
	1		38	SM	REDDISH BROWN DENSE FINE TO MEDIUM SILTY SAND	WEATHERED VINCENTOWN FORMATION
	2					
70	3			SM	MOTTLED GREEN AND BROWN FINE TO MEDIUM SILTY SAND, OCCASIONAL CEMENTED POCKETS	
					GREENISH GRAY DENSE TO VERY DENSE FINE TO MEDIUM SILTY SAND, OCCASIONAL CEMENTED POCKETS	VINCENTOWN FORMATION
	4		45			
	5		68	SM		
75	6					
	7		54			
80	8		21			
					BORING COMPLETED AT 81½ FEET ON 10/30/81	
85						

REFERENCE FOR BORINGS B1-B8, D2, V1-V6;
PSE&G, JANUARY 29, 1982, SUPPLEMENTARY SOILS INVESTIGATION SERVICE WATER INTAKE STRUCTURE AREA, HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

ELEVATION: +100
COORDINATES: N 1169
W 1509
DATUM: SITE DATUM

BORING B2

Location: HOPE CREEK
Completion Date: 10/31/81

Water Level	NA		
Time			
Date			
Casing Depth			

SAMPLE Depth Feet	No.	SYM	Blows/Ft.	UNIFIED SOIL CLASSIFICATION SYM.	Description	GEOLOGIC FORMATION
65					WATER DREDGED DEPTH (31)	
70	1		20	SM	REDDISH BROWN MEDIUM DENSE FINE TO MEDIUM SILTY SAND	WEATHERED VINCENTOWN FORMATION
	2					
	3		40	SM	MOTTLED BROWN AND GREEN DENSE FINE TO MEDIUM SILTY SAND	
75	4		38	SM	GREENISH GRAY VERY DENSE FINE TO MEDIUM SILTY SAND, OCCASIONAL CEMENTED POCKETS	VINCENTOWN FORMATION
	5		51			
80	6		83			
85					BORING COMPLETED AT 81 FEET ON 10/31/81	

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

FIGURE 2.5-50

BORING B3

ELEVATION: +100
COORDINATES: N 1106
W 1423
DATUM: SITE DATUM

Location: HOPE CREEK
Completion Date: 10/31/81

Water Level	NA		
Time			
Date			
Casing Depth	-		

SAMPLE Depth Feet	No.	SYM	Blows Ft.	UNIFIED SOIL CLASSIFICATION SYM.	Description	GEOLOGIC FORMATION
65					WATER	
					DREDGED DEPTH (30)	
70	1		130	SM	REDDISH BROWN VERY DENSE FINE TO MEDIUM SILTY SAND WITH SOME BANDS OF SILT AND OCCASIONAL	WEATHERED VINCENTOWN FORMATION
	2				MOTTLED BROWN AND GREEN DENSE TO MEDIUM DENSE FINE TO MEDIUM SILTY SAND	
	3		41	SM		
75	4		17			
	5		65	SM	GREENISH GRAY VERY DENSE FINE TO MEDIUM SILTY SAND, OCCASIONAL CEMENTED POCKETS	VINCENTOWN FORMATION
	6		63			
80	7		69			
	8		64			
85					BORING COMPLETED AT 85 FEET ON 10/31/81	
90						

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

FIGURE 2.5-50

ELEVATION: +100
COORDINATES: N 1102
W. 1486
DATUM: SITE DATUM

BORING B4

Location: HOPE CREEK
Completion Date: 11/1/81

Water Level	NA		
Time			
Date			
Casing Depth			

SAMPLE Depth Feet	No.	SYM	Blows Ft.	UNIFIED SOIL CLASSIFICATION SYM.	Description	GEOLOGIC FORMATION
					WATER	
65					DREDGED DEPTH (33)	
	1		100	SM	REDDISH BROWN VERY DENSE FINE TO MEDIUM SILTY SAND GRADES FINE TO COARSE SILTY SAND	WEATHERED VINCENTOWN FORMATION
	2					
70	3		43	SM	MOTTLED BROWN AND GREEN DENSE TO VERY DENSE FINE TO MEDIUM SILTY SAND OCCASIONAL CEMENTED POCKETS	
	4		38			
	5		56			
75						
	6		49	SM	GREENISH GRAY VERY DENSE FINE TO MEDIUM SILTY SAND, OCCASIONAL CEMENTED POCKETS	VINCENTOWN FORMATION
	7		101/9"			
80					BORING COMPLETED AT 80 FEET ON 11/1/81	
85						

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 76 of 189
FIGURE 2.5-50

ELEVATION: +100
COORDINATES: N 1152
W 1430
DATUM: SITE DATUM

BORING B5

Location: HOPE CREEK
Completion Date: 11/1/81

Water Level	-NA		
Time			
Date			
Casing Depth			

Depth Feet	Sample No.	SYM	Blows Ft.	USC SYM.	Description	GEOLOGIC FORMATION
					WATER	
65					DREDGED DEPTH (34)	
	1		21	SM	REDDISH BROWN DENSE TO VERY DENSE FINE TO MEDIUM SILTY SAND WITH SOME BANDS OF SILT, AND OCCASIONAL CEMENTED POCKETS	WEATHERED VINCENTOWN FORMATION
	2		100/8"			
70	3		39	SM	MOTTLED GREEN AND BROWN DENSE FINE TO MEDIUM SILTY SAND	
	4					
	5		31	SM	GREENISH GRAY DENSE TO VERY DENSE FINE TO MEDIUM SILTY SAND, OCCASIONAL CEMENTED POCKETS	VINCENTOWN FORMATION
75	6		153/9"			
	7		98/9"			
80						
					BORING COMPLETED AT 81 FEET ON 11/1/81	
85						

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 77 of 189
FIGURE 2.5-50

BORING B6

ELEVATION: +100
COORDINATES: N 1127
W 1501
DATUM: SITE DATUM

Location: HOPE CREEK
Completion Date: 11/3/81

Water Level	NA		
Time			
Date			
Casing Depth			

SAMPLE: Depth Feet	No.	SYM	Blows Ft.	UNIFIED SOIL CLASSIFICATION: SYM.	Description	GEOLOGIC FORMATION
65					WATER DREDGED DEPTH (33)	
	1		141/9"	SM	REDDISH BROWN DENSE TO VERY DENSE FINE TO MEDIUM SILTY SAND, OCCASIONAL CEMENTED POCKETS	WEATHERED VINCENTOWN FORMATION
70	2		43			
	3		26	SM	MOTTLED GREEN AND BROWN MEDIUM DENSE TO VERY DENSE FINE TO MEDIUM SILTY SAND, OCCASIONAL CEMENTED POCKETS	
	4		36			
75	5 6		100/5"			
	7		50	SM	GREENISH GRAY DENSE FINE TO MEDIUM SILTY SAND, OCCASIONAL CEMENTED POCKETS	VINCENTOWN FORMATION
80					BORING COMPLETED AT 80 FEET ON 11/3/81	
85						

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

FIGURE 2.5-50

ELEVATION: +100
COORDINATES: N 1096
W 1447
DATUM: SITE DATUM

BORING B7

Location: HOPE CREEK
Completion Date: 11/4/81

Water Level	NA		
Time			
Date			
Casing Depth			

SAMPLE Depth Feet	No.	SYM	Blows Ft.	UNIFIED SOIL CLASSIFICATION SYM.	Description	GEOLOGIC FORMATION
65					WATER	
					DREDGED DEPTH (32)	
	1		157/9"	SM	REDDISH BROWN DENSE FINE TO MEDIUM SILTY SAND WITH SOME BANDS OF SILT	
	2					
70	3		36	SM	MOTTLED GREEN AND BROWN DENSE FINE TO MEDIUM SILTY SAND, OCCASIONAL CEMENTED POCKETS MEDIUM DENSE TO DENSE	WEATHERED VINCENTOWN FORMATION
	4		26			
75	5		32			
	6		52	SM	GREENISH GRAY DENSE TO VERY DENSE FINE TO MEDIUM SILTY SAND, OCCASIONAL CEMENTED POCKETS	VINCENTOWN FORMATION
			41			
80					BORING COMPLETED AT 80 FEET ON 11/4/81	
85						

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 79 of 189
FIGURE 2.5-50

BORING B8

ELEVATION: +100
COORDINATES: N 1182
W 1478
DATUM: SITE DATUM

Location: HOPE CREEK
Completion Date: 11/4/81

Water Level			
Time			
Date			
Casing Depth			

SAMPLE Depth Feet	No.	SYM	Blows Ft.	UNIFIED SOIL CLASSIFICATION SYM.	Description	GEOLOGIC FORMATION
65					WATER	
					DREDGED DEPTH (31)	
70	1		43	SM	REDDISH BROWN DENSE FINE TO MEDIUM SILTY SAND OCCASIONALLY CEMENTED	WEATHERED VINCENTOWN FORMATION
	2			SM		
	3		44			
	4			SM	MOTTLED GREEN AND BROWN DENSE FINE TO MEDIUM SILTY SAND, OCCASIONAL CEMENTED POCKETS	VINCENTOWN FORMATION
	5		50			
75	6		100/6"		GREENISH GRAY DENSE TO VERY DENSE FINE TO MEDIUM SILTY SAND, OCCASIONAL CEMENTED POCKETS	
	7		50			
80					BORING COMPLETED AT 80 FEET ON 11/4/81	
85						

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

FIGURE 2.5-50

ELEVATION: +101
COORDINATES: N 1154
W 533
DATUM: SITE DATUM

BORING D2

Location: HOPE CREEK
Completion Date: 11/18/81

Water Level	NA		
Time			
Date			
Casing Depth			

SAMPLE Depth Feet	No.	SYM	Blows Ft.	UNIFIED SOIL CLASSIFICATION SYM.	Description	GEOLOGIC FORMATION
25						
30						
					RIVER BED	
35					DRILLED WITHOUT SAMPLING 33-39 FEET	
40	1		20	CL	GRAY MOTTLED SILTY CLAY	
	2					
	3					
45	4			GC	BROWN MOTTLED FINE TO COARSE VERY CLAYEY SILTY SAND AND FINE GRAVEL	
	5		9	SC	GREEN FINE TO MEDIUM CLAYEY SILTY GLAUCONITIC SAND	
50	6		9	GP	FINE GRAVEL	
	7				RED BROWN WITH TRACE GREEN GRADING GREEN WITH TRACE RED BROWN FINE TO MEDIUM CLAYEY SILTY GLAUCONITIC SAND GRADING SILTY GLAUCONITIC SAND WITH DEPTH	WEATHERED VINCENTOWN
55	8			SM		
	9					
	10					
60	11					

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

FIGURE 2.5-50

BORING D2 CONT.

Location:
Completion Date:

Water Level			
Time			
Date			
Casing Depth			

SAMPLE Depth Feet	No.	SYM	Blows Ft.	USC SYM.	Description	GEOLOGIC FORMATION
	11					WEATHERED VINCENTOWN
	12			SM		
	13					
65	14				GRAY FINE TO MEDIUM SILTY SAND, OCCASIONALLY CEMENTED	VINCENTOWN
	15			SM		
70	16					
	17					
	18					
75	19					
	20					
80	21					
	22					
	23					
85	24					
	25					
90	26					
	27					
95	28					
					BORING COMPLETED AT 96 FEET ON 11/18/81	

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 82 of 189
FIGURE 2.5-50

ELEVATION: +101
COORDINATES: N 1184
W 1545
DATUM: SITE DATUM

BORING V1

Location: HOPE CREEK
Completion Date: 11/12/81

Water Level	NA		
Time			
Date			
Casing Depth			

SAMPLE Depth Feet	No.	SYM	Blows Ft.	UNIFIED SOIL CLASSIFICATION SYM.	Description	GEOLOGIC FORMATION
25					RIVER BED	
30					BOULDERS 28-30 FEET	
					DRILLED WITHOUT SAMPLING 30-42 FEET	
35						
40						
	1		12	CL	GRAY MOTTLED SILTY CLAY, STIFF	
45	2		25	GC	BROWN MOTTLED FINE TO COARSE VERY CLAYEY SILTY SAND AND FINE GRAVEL, MEDIUM DENSE BECOMING LOOSE WITH DEPTH	
	3		9	SC	GREEN FINE TO MEDIUM CLAYEY/SILTY GLAUCONITIC SAND, LOOSE	
	4		23	GP	FINE GRAVEL, MEDIUM DENSE	
50	5		12	SC	RED BROWN WITH TRACE GREEN GRADING GREEN WITH TRACE RED BROWN FINE TO MEDIUM CLAYEY/SILTY SAND GRADING SILTY GLAUCONITIC SAND WITH DEPTH, LOOSE TO VERY DENSE DENSE 51-52 FEET LOOSE 51-58 FEET	WEATHERED VINCENTOWN
	6		6			
55	7		1			
	8		10	SM	DENSE TO VERY DENSE BELOW 58 FEET	
	9		54			
60						

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

BORING V1 CONT.

Location:
Completion Date:

Water Level			
Time			
Date			
Casing Depth			

SAMPLE Depth Feet	No.	SYM	Blows Ft.	UNIFIED SOIL CLASSIFICATION SYM.	Description	GEOLOGIC FORMATION
	10		21	SM / SM	GREEN AND WHITE MOTTLED CALCAREOUS GLAUCONITIC SAND 61-63 FEET	WEATHERED VINCENTOWN
	11		32			
65	12		35	SM		
	13		56			
	14		200			
70	15		50	SM	GREEN WITH TRACE RED BROWN AT 70 FEET	
	16		50			
75	17		226	SM	GRAY FINE TO MEDIUM SILTY SAND OCCASIONALLY CEMENTED, VERY DENSE	VINCENTOWN
	18		97			
80	19		100/6"			
	20		102			
85					BORING COMPLETED AT 85 FEET ON 11/12/81	
90						

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 84 of 189
FIGURE 2.5-50

ELEVATION: +101
COORDINATES: N 1126
W 1522
DATUM: SITE DATUM

BORING V2

Location: HOPE CREEK
Completion Date: 11/14/81

Water Level	NA		
Time			
Date			
Casing Depth			

SAMPLE Depth Feet	No.	SYM	Blows Ft.	UNIFIED SOIL CLASSIFICATION SYM.	Description	GEOLOGIC FORMATION
25						
30					RIVER BED	
35						
40						
	1		16	CL	GRAY MOTTLED SILTY CLAY, STIFF	
45	2		39	GC	BROWN MOTTLED FINE TO COARSE VERY CLAYEY SILTY SAND AND FINE GRAVEL, DENSE	
	3		34	SC	GREEN FINE TO MEDIUM CLAYEYE SILTY GLAUCONITIC SAND, LOOSE	
	4		6			
50	5		3	GP	FINE GRAVEL, LOOSE	
	6		10	SM	RED BROWN WITH TRACE GREEN GRADING GREEN WITH TRACE RED BROWN FINE TO MEDIUM SILTY GLAUCONITIC SAND, LOOSE TO VERY DENSE LOOSE 51.5-55.6 FEET THEN DENSE TO VERY DENSE	WEATHERED VINCENTOWN
55	7		8			
	8		40			
	9		58			
60						

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 85 of 189
FIGURE 2.5-50

BORING V2 CONT.

Location:
Completion Date:

Water Level			
Time			
Date			
Casing Depth			

SAMPLE Depth Feet	No.	SYM	Blows/Ft.	SYM.	UNIFIED SOIL CLASSIFICATION Description	GEOLOGIC FORMATION
	10		51	SM		WEATHERED VINCENTOWN
	11		33	SM	GREEN AND WHITE MOTTLED CALCAREOUS GLAUCONITIC SAND 62.5-63.5 AND 64.5-65.0 FEET	
	12		43	SM		
65	13		35			
	14		57	SM		
70	15		100/6"		GREEN WTIH TRACE RED BROWN AT 71 FEET	
	16		39			
	17		51	SM		
75	18		100/5"			
	19		100/3"		GRAY FINE TO MEDIUM SILTY SAND OCCASIONALLY CEMENTED, VERY DENSE	VINCENTOWN
80	20		70	SM		
	21		47			
85	22		100/6"			
					BORING COMPLETED AT 86 FEET ON 11/14/81	

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 86 of 189
FIGURE 2.5-50

BORING V3

ELEVATION: + 101
COORDINATES: N 1105.11
W 1515.58
DATUM: SITE DATUM

Location: HOPE CREEK
Completion Date: 11/19/81

Water Level	NA		
Time			
Date			
Casing Depth			

Depth Feet	No.	SYM	Blows Ft.	SYM	Description	GEOLOGIC FORMATION
25, 30					RIVER BED	
35, 40					DRILLED WITHOUT SAMPLING 33-42 FEET	
	1		10	CL	GRAY MOTTLED SILTY CLAY, STIFF	
45	2		115	GC	BROWN MOTTLED FINE TO COARSE VERY CLAYEY SILTY SAND AND FINE GRAVEL, VERY DENSE	
	3		12	SC	GREEN FINE TO MEDIUM CLAYEY SILTY GLAUCONITIC SAND, MEDIUM DENSE	
	4		8	GP	FINE GRAVEL, LOOSE	
50	5		53	SC	RED BROWN WITH TRACE GREEN GRADING GREEN WITH TRACE RED BROWN FINE TO MEDIUM CLAYEY SILTY GLAUCONITIC SAND GRADING SILTY GLAYCONITIC SAND WITH DEPTH, LOOSE TO VERY DENSE	WEATHERED VINCENTOWN
	6		1			
55	7		44		DENSE 50-52 FEET LOOSE 52-54.5 FEET THEN DENSE TO VERY DENSE	
	8		44	SM		
	9		63			
60				SM		

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

FIGURE 2.5-50

BORING V3 CONT.

Location:
Completion Date:

Water Level			
Time			
Date			
Casing Depth			

Depth Feet	No.	SYM	Blows Ft.	SYM.	Description (Unified Soil Classification)	Geologic Formation
	10		59	SM	GREEN AND WHITE MOTTLED CALCAREOUS GLAUCONITIC SAND 60-60.5 AND 61.5-63.5 FEET	WEATHERED VINCENTOWN
				SM		
	11		22			
65	12		35	SM		
	13		51			
	14		100/4"			
70	15		35			
				SM	GREEN WITH TRACE RED BROWN AT 71.5 FEET	
	16		36			
75	17		37			
	18		107	SM	GRAY FINE TO MEDIUM SILTY SAND OCCASIONALLY CEMENTED, VERY DENSE	VINCENTOWN
80	19		116			
	20		100/5"			
	21		100/5"			
	22		100/1"			
85	23		100/1"			
					BORING COMPLETED AT 85'7" ON 11/19/81	
90						

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 88 of 189
FIGURE 2.5-50

BORING V4

ELEVATION: +101
COORDINATES: N 1055
W 1498
DATUM: SITE DATUM

Location: HOPE CREEK
Completion Date: 11/17/81

Water Level	NA		
Time			
Date			
Casing Depth			

SAMPLE Depth Feet	No.	SYM	Blows/Ft.	UNIFIED SOIL CLASSIFICATION SYM.	Description	GEOLOGIC FORMATION
25						
					RIVER BED	
30					BOULDERS 28-30 FEET	
					DRILLED WITHOUT SAMPLING 28-41 FEET	
35						
40						
	1		5	CL	GRAY MOTTLED SILTY CLAY, MEDIUM STIFF GRADING STIFF	
	2		12			
45	3		60	GC	BROWN MOTTLED FINE TO COARSE VERY CLAYEY SILTY SAND AND FINE GRAVEL, VERY DENSE	
	4		15	SC	GREEN FINE TO MEDIUM CLAYEY SILT GLAUCONITIC SAND, MEDIUM DENSE	
50	5		9			
				GP	FINE GRAVEL, LOOSE	
	6		14	SC	RED BROWN WITH TRACE GREEN GRADING GREEN WITH TRACE RED BROWN FINE TO MEDIUM CLAYEY SILTY GLAUCONITIC SAND, GRADING SILTY GLAUCONITIC SAND WITH DEPTH, LOOSE TO VERY DENSE	WEATHERED VINCENTOWN
55	7		0			WEATHERED VINCENTOWN
	8		46	SM	MEDIUM DENSE 51.5 TO 53 FEET, LOOSE 53-55.5 THEN DENSE TO VERY DENSE	WEATHERED VINCENTOWN
	9		33			WEATHERED VINCENTOWN
60						

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

FIGURE 2.5-50

BORING V4 CONT.

Location: HOPE CREEK
Completion Date:

Water Level			
Time			
Date			
Casing Depth			

SAMPLE Depth Feet	No.	SYM	Blows Ft.	UNIFIED SOIL CLASSIFICATION SYM.	Description	GEOLOGIC FORMATION
	10		109	SM	GREEN AND WHITE MOTTLED CALCAREOUS GLAUCONITIC SAND	WEATHERED VINCENTOWN
	11		35		65.5-66.5 FEET	
65	12		37			
				SM		
	13		40			
	14		130	SM		
70	15		300			
	16		38	SM	GREEN WITH TRACE RED BROWN AT 71.5 FEET	
75	17		44			
	18		251	SM	GRAY FINE TO MEDIUM SILTY SAND, OCCASIONALLY CEMENTED, VERY DENSE	VINCENTOWN
	19		100/3"			
80	20		71			
	21		100/5"			
	22		100/1"			
85			100/1"			
	23		300		BORING COMPLETED AT 86 FEET ON 11/17/81	
90						

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 90 of 189
FIGURE 2.5-50

ELEVATION: +98
COORDINATES: N 1137
W 1393
DATUM: SITE DATUM

BORING V5

Location: HOPE CREEK
Completion Date: 11/20/81

Water Level	NA		
Time			
Date			
Casing Depth			

SAMPLE Depth Feet	No.	SYM	Blows Ft.	UNIFIED SOIL CLASSIFICATION SYM.	Description	GEOLOGIC FORMATION
25						
					DRILLED WITHOUT SAMPLING 0-29 FEET	
30	1		10	OH	DARK GRAY THICKLY LAMINATED ORGANIC VERY SILTY CLAY INTERLAMINATED WITH THIN PARTINGS OF FINE TO MEDIUM SILTY SAND, SOFT TO MEDIUM STIFF	
	2		29			
	3		6			
35	4		26	OH	DARK GRAY ORGANIC SILTY CLAY, STIFF	
	5		18	SM	POSSIBLY GRAY SAND	
40	6		11	CL	DARK GRAY OCCASIONALLY MOTTLED SANDY SILTY CLAY WITH FINE GRAVEL, MEDIUM STIFF	
	7		9			
	8		14			
45					BORING TERMINATED DUE TO OBSTRUCTION AT 45 FEET ON 11/20/81	
50						

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 91 of 189
FIGURE 2.5-50

ELEVATION: +98
COORDINATES: N 1134
W 1398
DATUM: SITE DATUM

BORING V6

Location: HOPE CREEK
Completion Date: 11/22/81

Water Level	NA		
Time			
Date			
Casing Depth			

Depth Feet	No.	SYM	Blows Ft.	SYM.	Description (Unified Soil Classification)	Geologic Formation
25					DRILLED WITHOUT SAMPLING 0-29 FEET	
30	1		8	OH	DARK GRAY THICKLY LAMINATED ORGANIC VERY SILTY CLAY INTERLAMINATED WITH THIN PARTINGS OF FINE TO MEDIUM SILTY SAND, SOFT TO MEDIUM STIFF	
	2		2			
	3		9			
35	4		14		4" BAND OF SAND AT BASE	
				OH	DARK GRAY ORGANIC SILTY CLAY, MEDIUM STIFF TO STIFF	
	5		13	CL	GRAY MOTTLED SANDY SILTY CLAY WITH TRACE FINE GRAVEL, MEDIUM STIFF BECOMING SOFT	
40	6		11			
	7		3			
	8		8			
45	9		10	SP/SC	GREEN BROWN MOTTLED FINE TO COARSE VERY CLAYEY SILTY GLAUCONITIC SAND, MEDIUM DENSE SOME GRAVEL 45-45.5 FEET	
	10		15	GC	FINE CLAYEY GRAVEL	
50	11		1	SC	RED BROWN WITH TRACE GREEN GRADING GREEN WITH TRACE RED BROWN FINE TO MEDIUM SILTY GLAUCONITIC SAND GRADING SILTY GLAUCONITIC SAND WITH DEPTH, LOOSE TO VERY DENSE LOOSE 49-51 FEET THEN DENSE	WEATHERED VINCENTOWN
	12		37	SM		
	13		42			
55	14		56			
	15		31	SM	GRADING GREEN WITH TRACE RED BROWN AT 57 FEET	
60	16		17			

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

FIGURE 2.5-50

BORING V6 CONT.

Location:
Completion Date:

Water Level			
Time			
Date			
Casing Depth			

SAMPLE Depth Feet	No.	SYM	Blows Ft.	UNIFIED SOIL CLASSIFICATION SYM.	Description	GEOLOGIC FORMATION
	16		17	SM	GRAY FINE TO MEDIUM SILTY SAND, DENSE	WEATHERED VINCENTOWN
	17		31			
	18		32			
65	19		59	SM	GREEN WITH TRACE RED BROWN FINE TO MEDIUM SILTY SAND, VERY DENSE TO DENSE	
	20		100/5"			
70	21		42			
	22		54	SM	GRAY FINE TO MEDIUM SILTY SAND, OCCASIONALLY CEMENTED, VERY DENSE	VINCENTOWN
	23		75			
75	24		100/4"			
	25		127			
80	26		59			
	27		100/2"			
	28		100/2"			
85	29		100/2"			
90	30		100/2"			
	31		100/1"			
95					BORING COMPLETED AT 94 FEET ON 11/22/81	

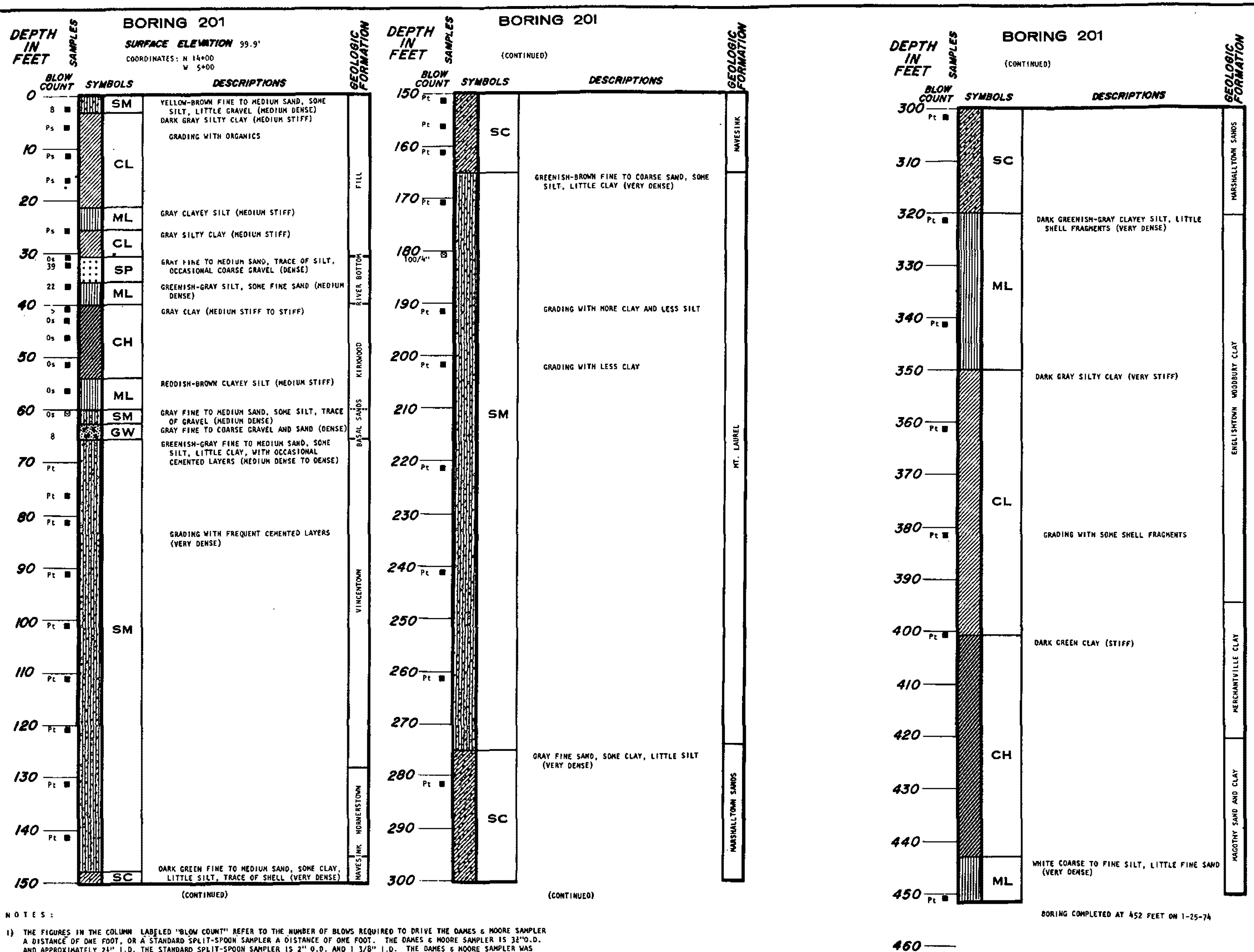
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

FIGURE 2.5-50



NOTES:

1) THE FIGURES IN THE COLUMN LABELED "BLOW COUNT" REFER TO THE NUMBER OF BLOWS REQUIRED TO DRIVE THE DAMES & MOORE SAMPLER A DISTANCE OF ONE FOOT, OR A STANDARD SPLIT-SPOON SAMPLER A DISTANCE OF ONE FOOT. THE DAMES & MOORE SAMPLER IS 3½" O.D. AND APPROXIMATELY 2½" I.D. THE STANDARD SPLIT-SPOON SAMPLER IS 2" O.D. AND 1 3/8" I.D. THE DAMES & MOORE SAMPLER WAS DRIVEN WITH A 300 LB HAMMER AT A 30" DROP AND THE STANDARD SPLIT-SPOON SAMPLER USED A 140 LB HAMMER AT A 30" DROP, UNLESS OTHERWISE STATED ON THE LOGS.

2. THE LETTERS IN THE "BLOW COUNT" COLUMN INDICATES THE FOLLOWING:
"P" INDICATES THAT THE SAMPLER WAS ADVANCED BY THE WEIGHT OF THE DRILL ROD AND DRIVE WEIGHT WITHOUT DRIVING.

"Os" INDICATES THAT THE OSTERBERG SAMPLER WAS USED FOR UNDISTURBED SAMPLING.

"Pt" INDICATES THAT THE PITCHER SAMPLER WAS USED FOR UNDISTURBED SAMPLING.

"Ps" INDICATES THAT THE DAMES & MOORE PISTON SAMPLER WAS USED FOR UNDISTURBED SAMPLING.

"Tw" INDICATES DAMES & MOORE THIN WALL SAMPLE.

3. ELEVATIONS REFER TO THE PUBLIC SERVICE PLANT DATUM.

4. THE DISCUSSION IN THE TEXT OF THE REPORT IS NECESSARY FOR A PROPER UNDERSTANDING OF THE NATURE OF THE SUBSURFACE MATERIALS.

REFERENCE FOR BORINGS 201-274:
PSE&G, MAY 23, 1974, REPORT, FOUNDATION STUDIES, PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

REVISION 0
APRIL 11, 1988

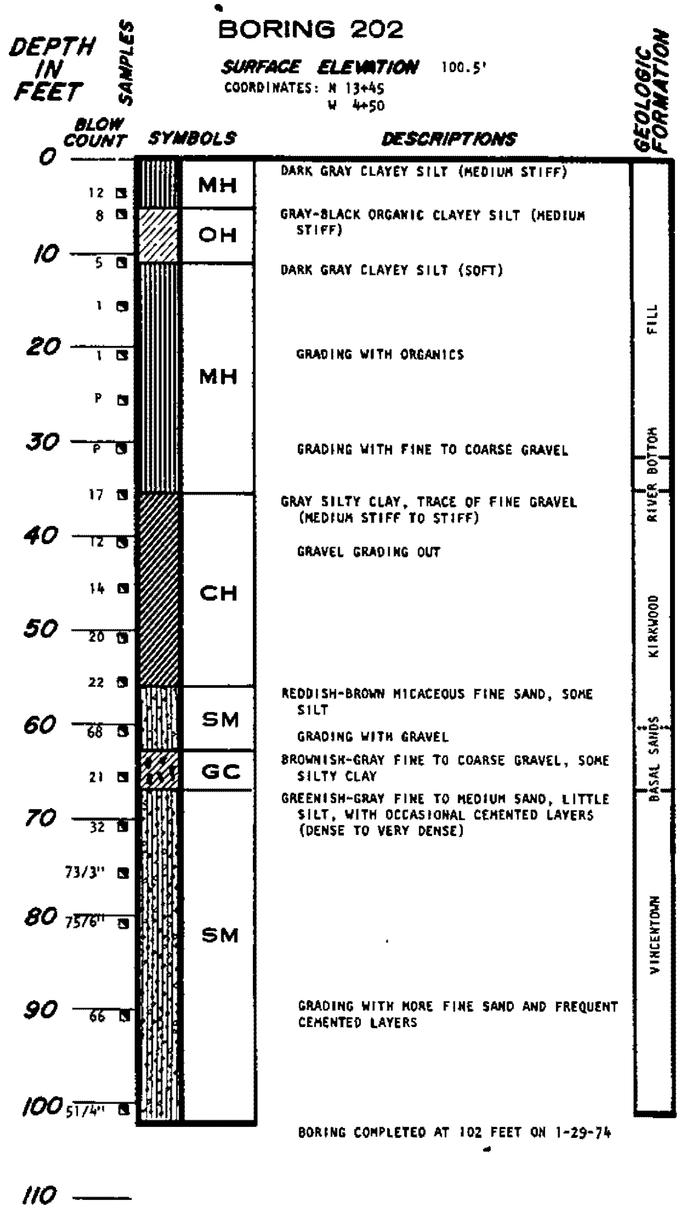
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 94 of 189

FIGURE 2.5-50



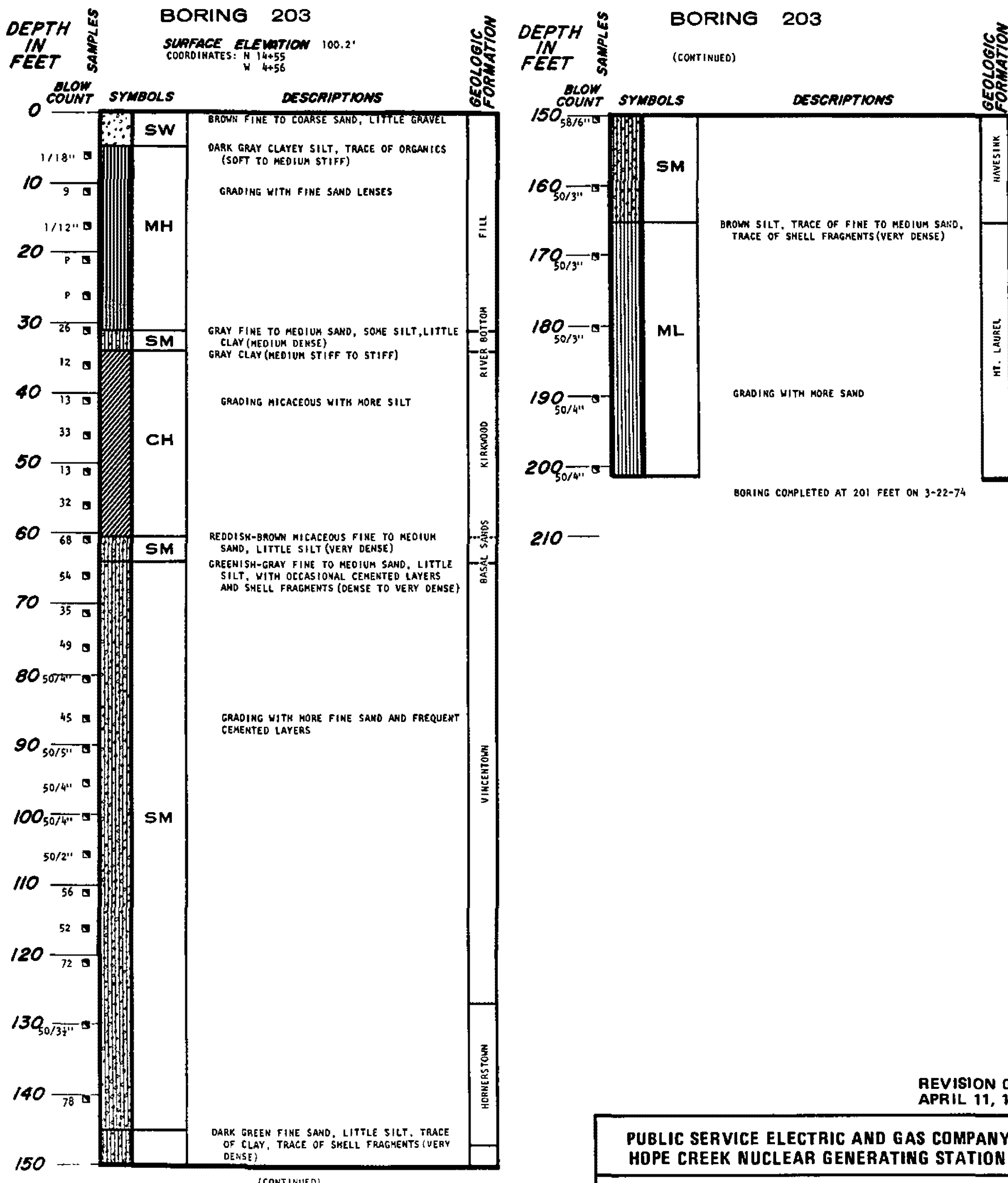
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 95 of 189
FIGURE 2.5-50



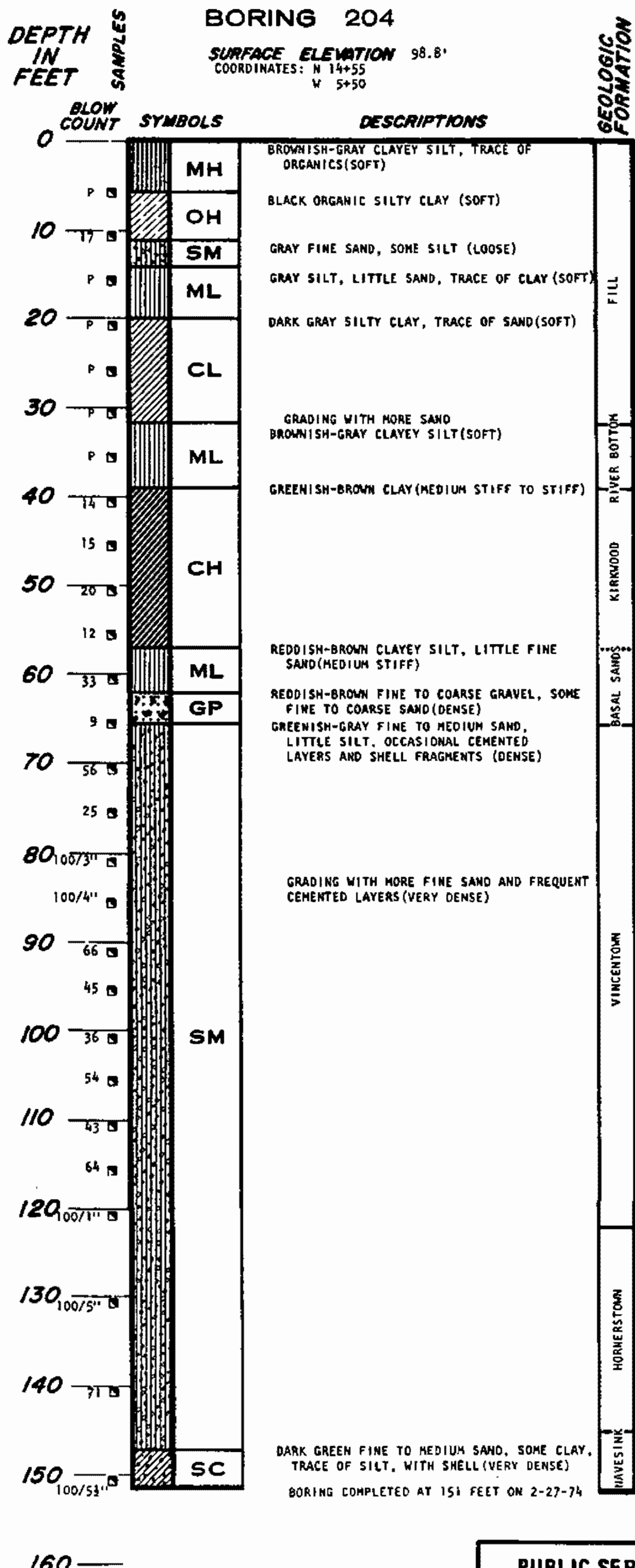
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 96 of 189
FIGURE 2.5-50



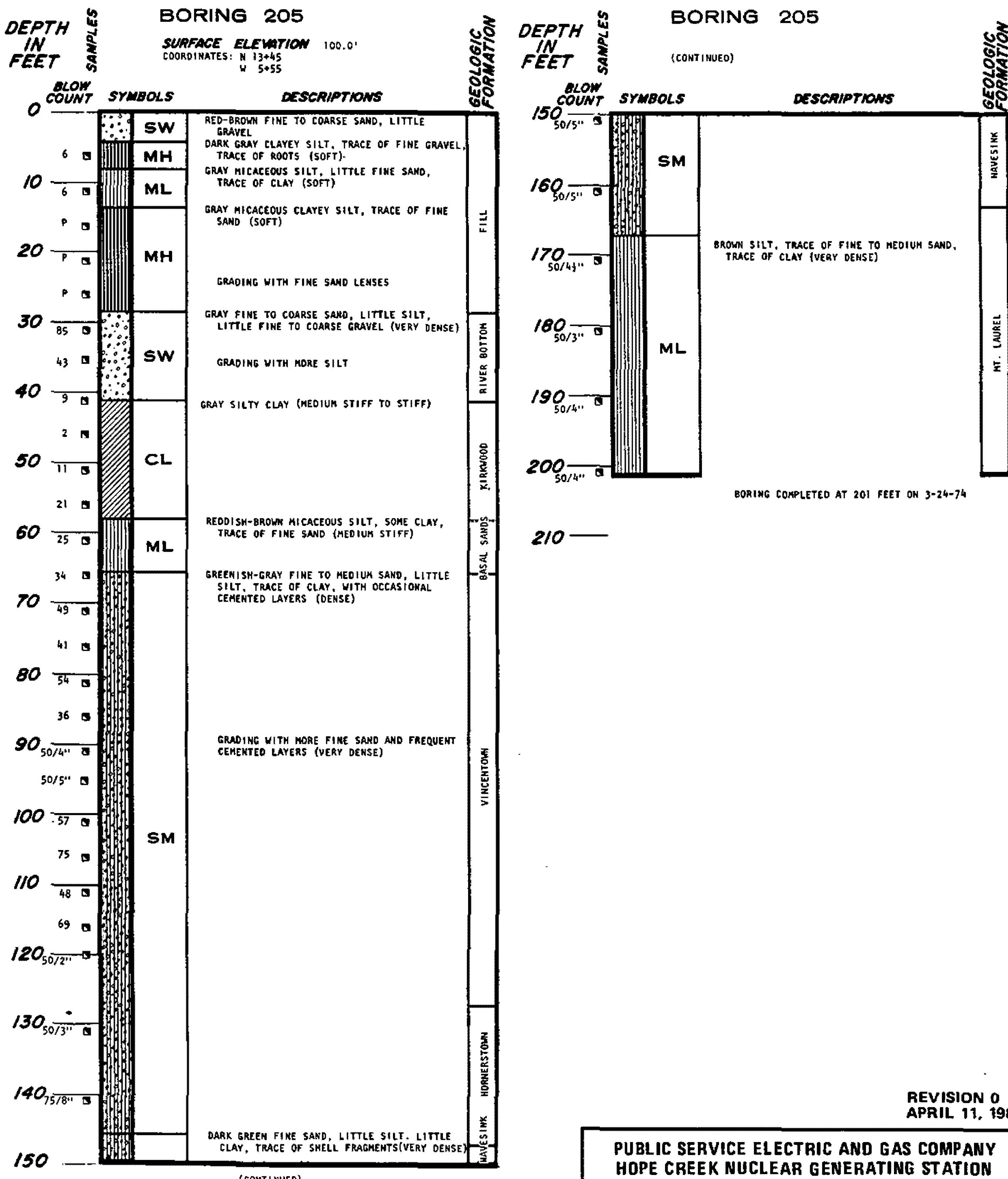
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 97 of 189
FIGURE 2.5-50



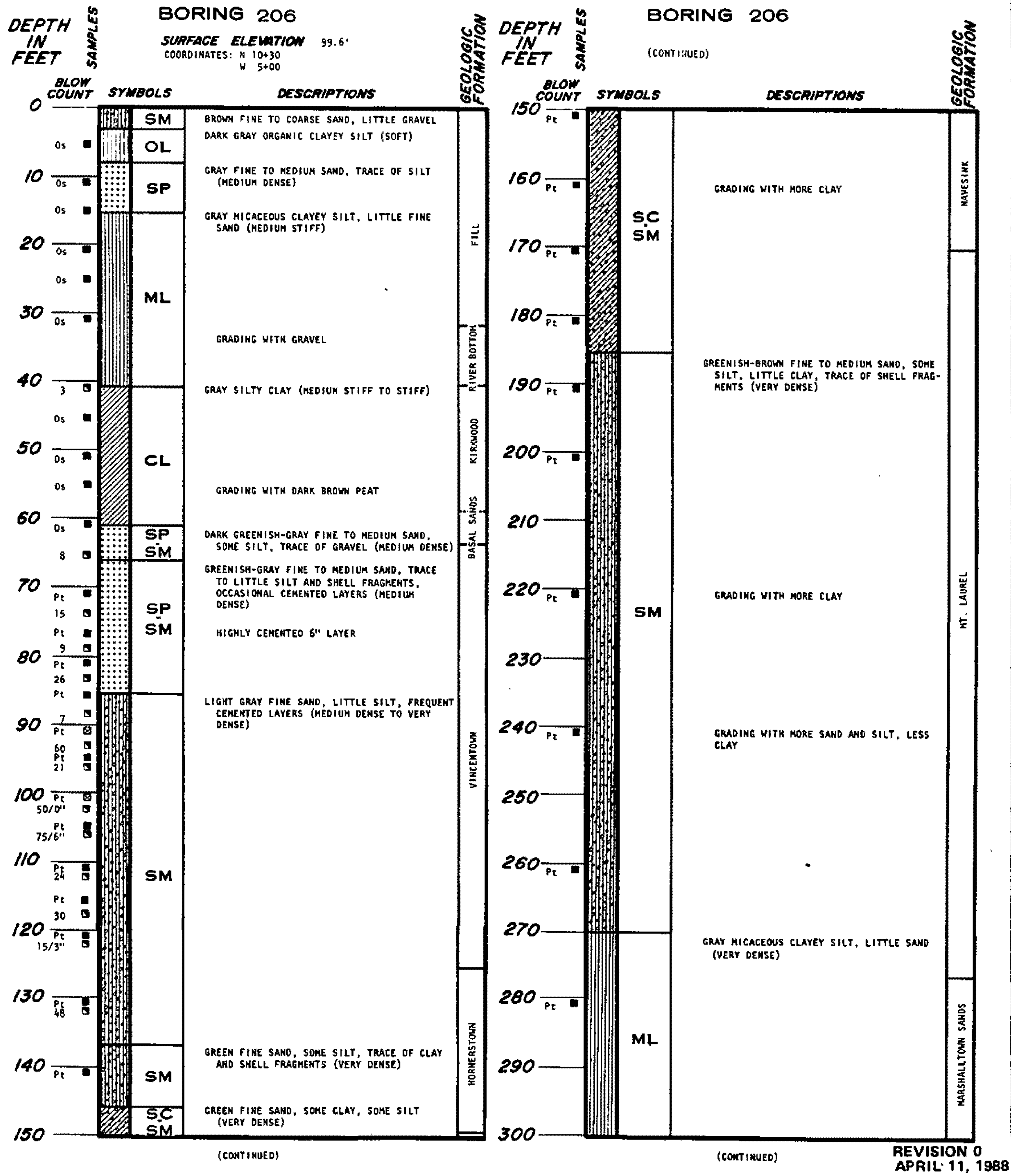
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

FIGURE 2.5-50



REVISION 0
APRIL 11, 1988

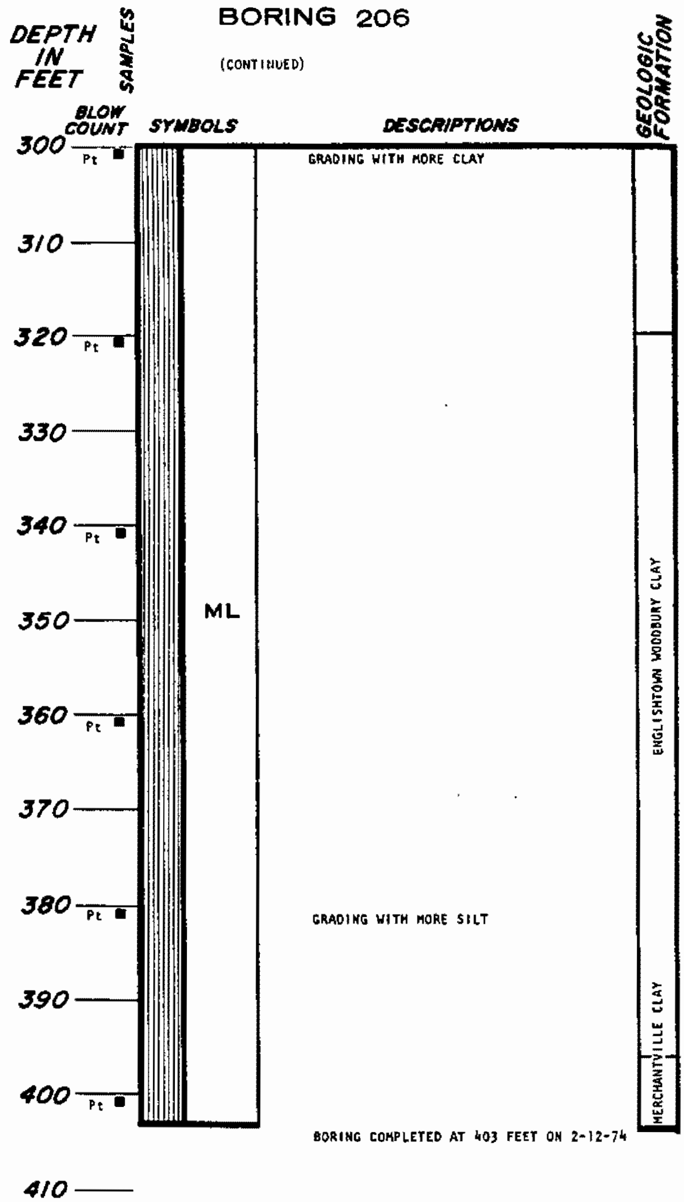
NOTE: THE STANDARD SPLIT SPOON SAMPLER WAS DRIVEN WITH A 300 LB. HAMMER AT A 24" DROP.

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 99 of 189
FIGURE 2.5-50



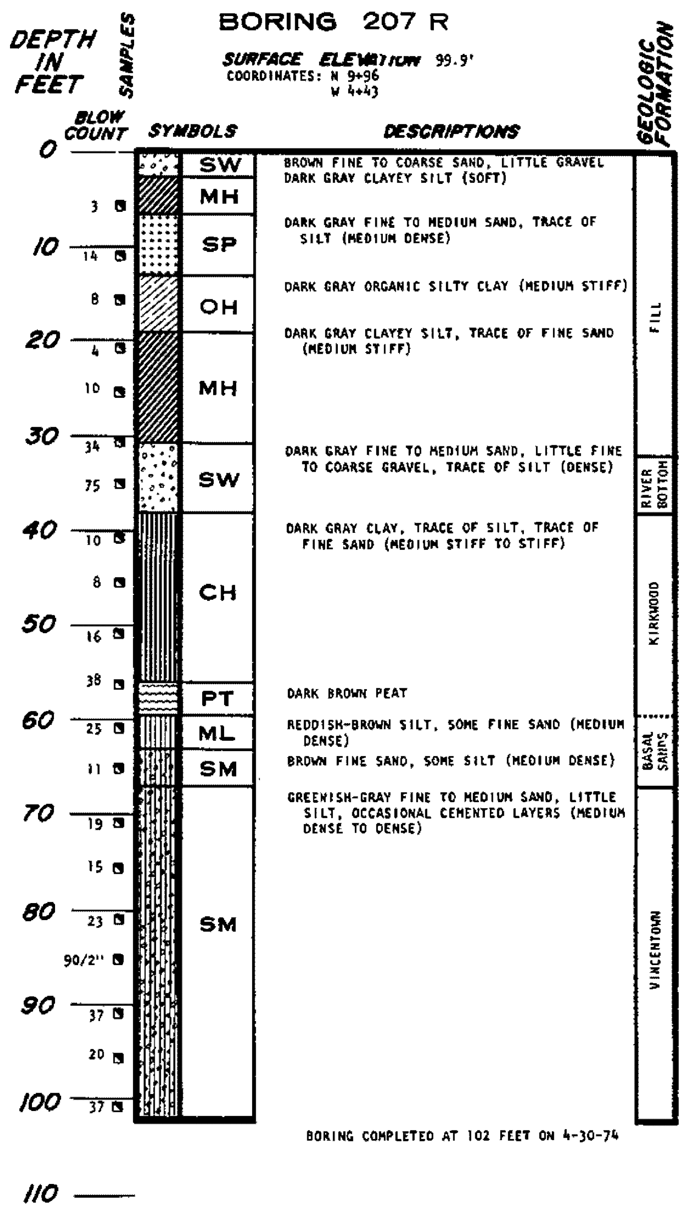
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 100 of 189
FIGURE 2.5-50



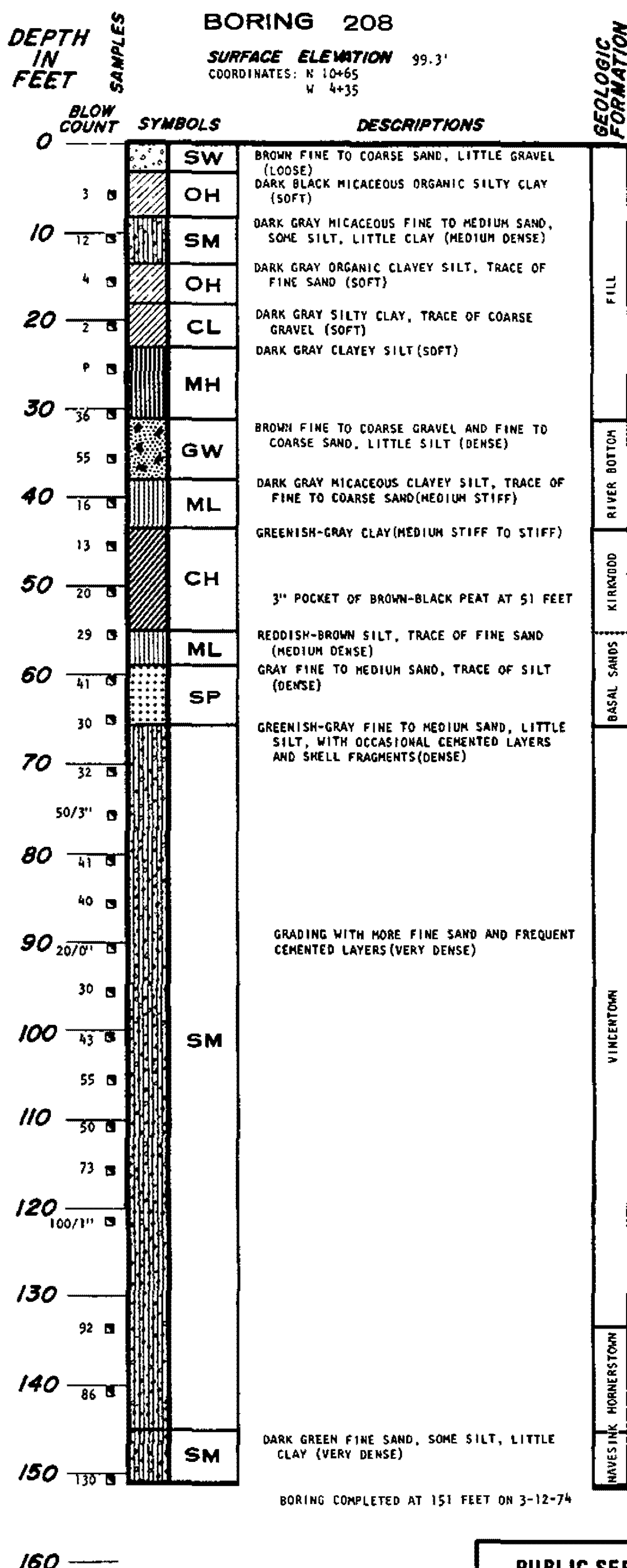
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 101 of 189
FIGURE 2.5-50



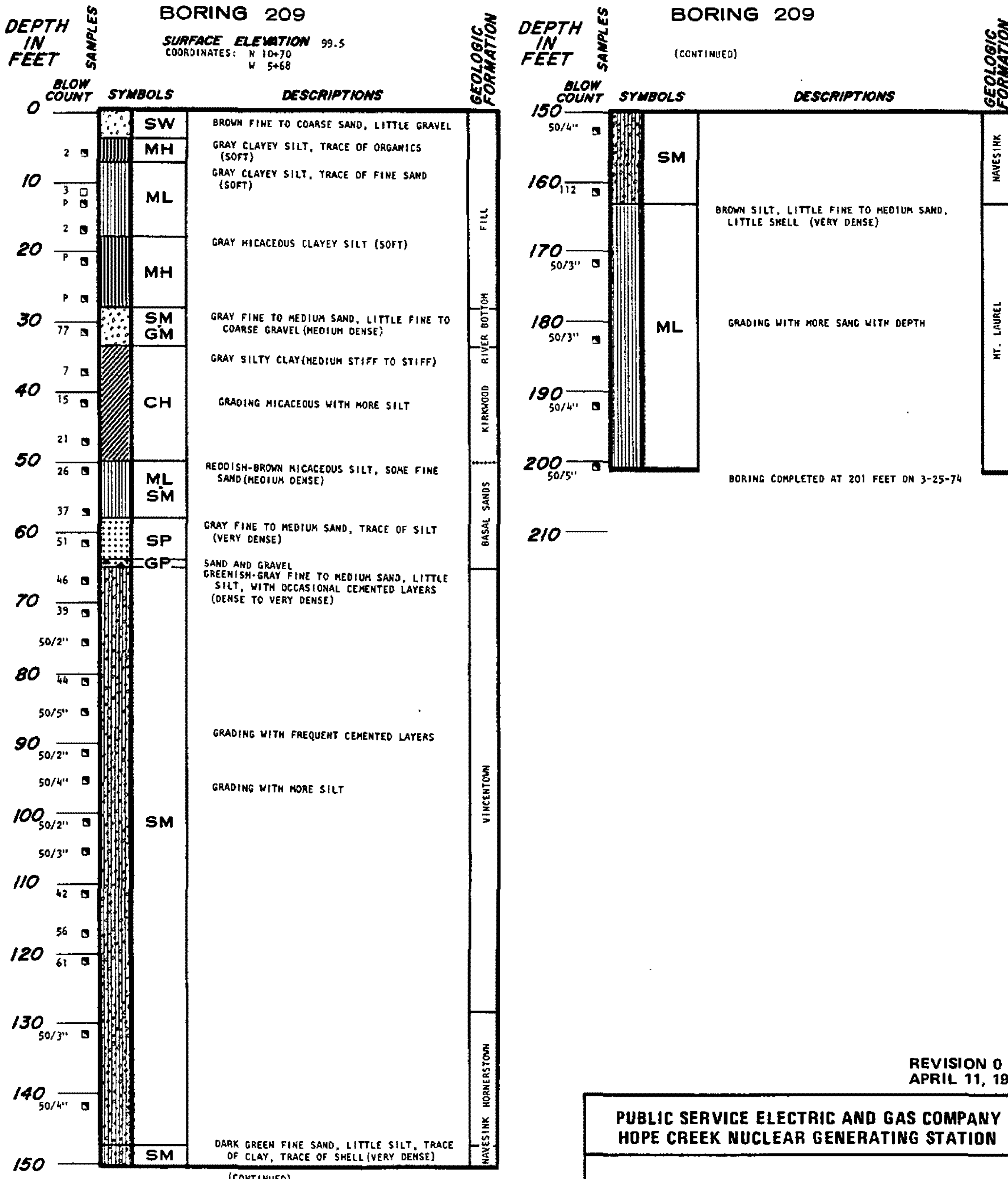
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 102 of 189
FIGURE 2.5-50



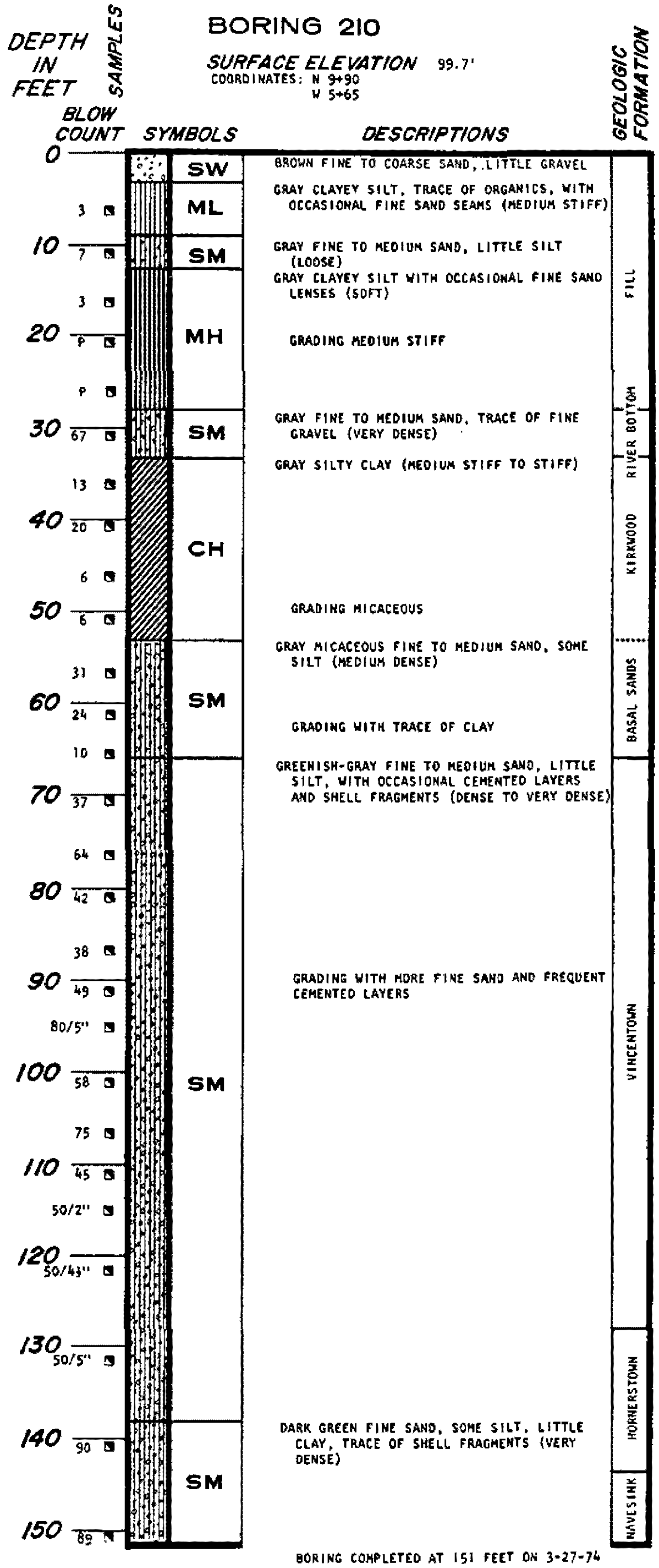
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 103 of 189
FIGURE 2.5-50



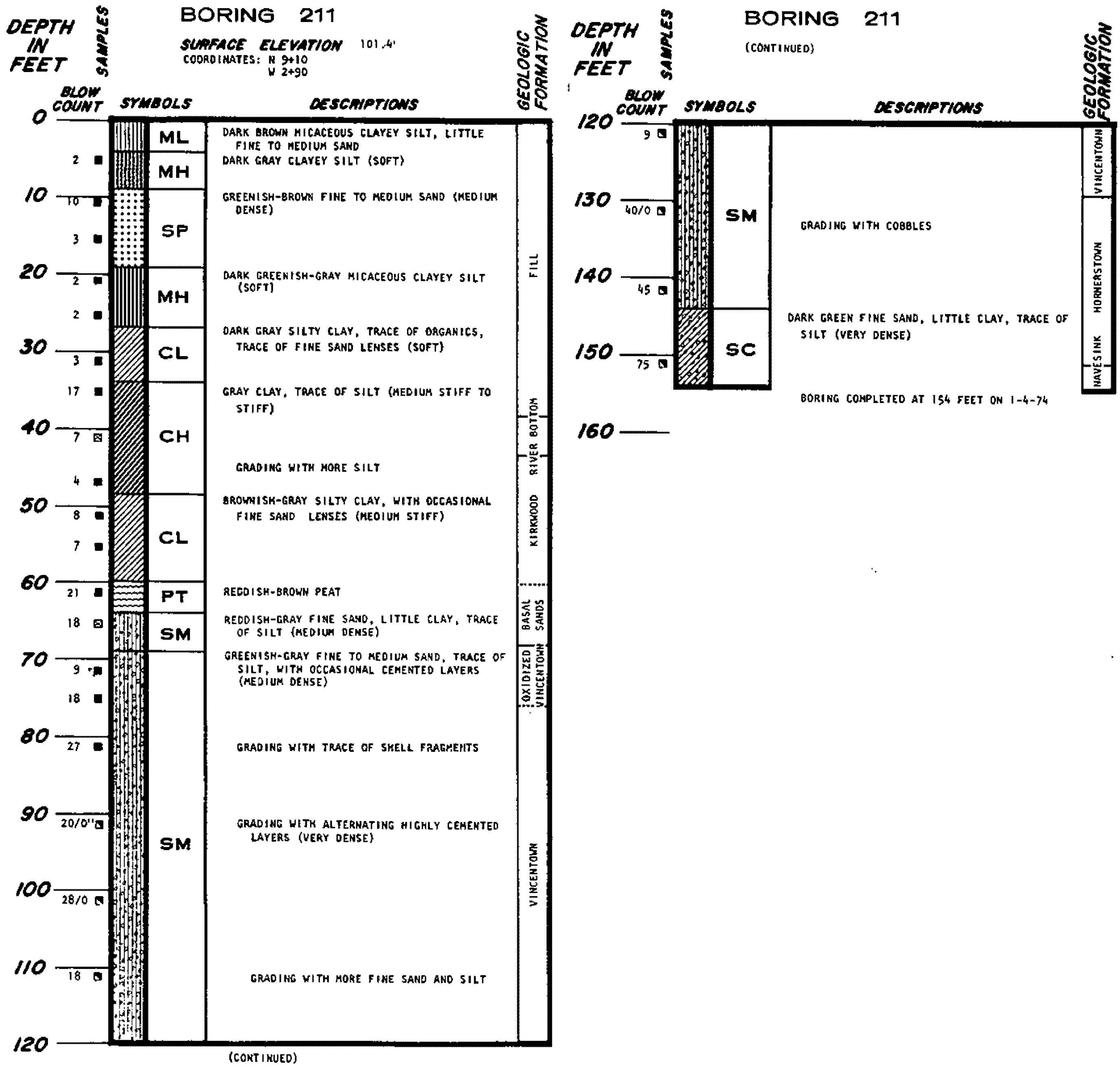
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 104 of 189
FIGURE 2.5-50



NOTE: THE STANDARD SPLIT SPOON SAMPLER WAS DRIVEN WITH A 300 LB. HAMMER AT A 30" DROP

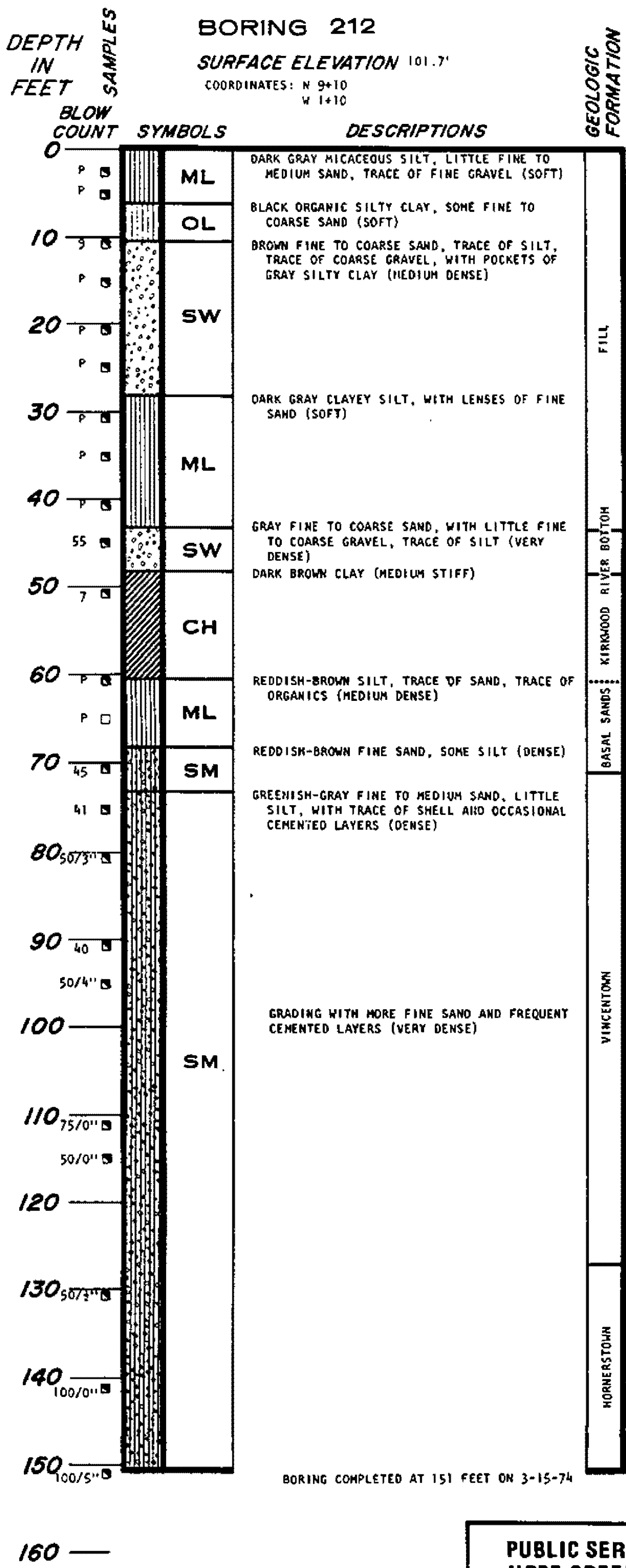
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 105 of 189
FIGURE 2.5-50



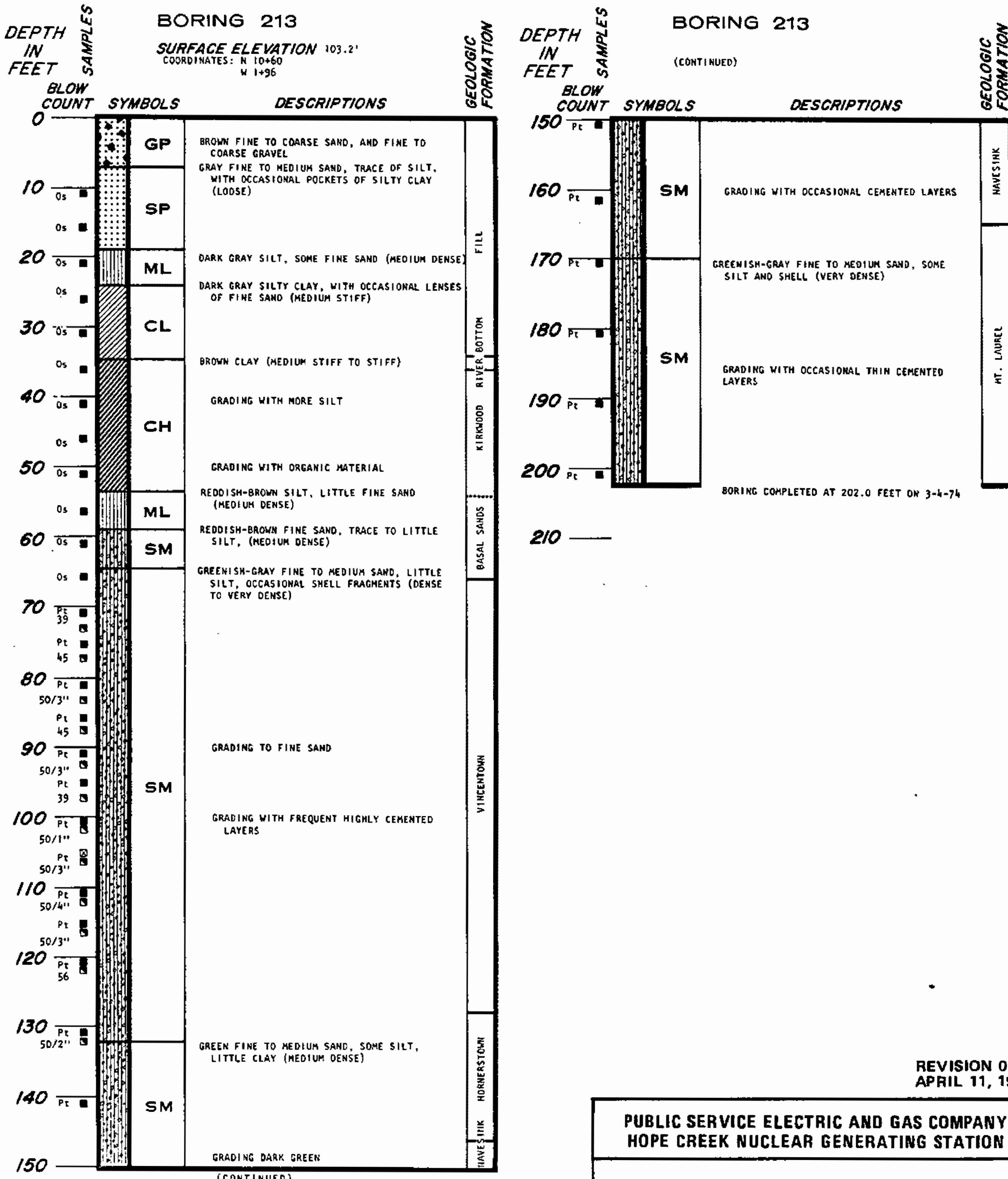
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 106 of 189
FIGURE 2.5-50



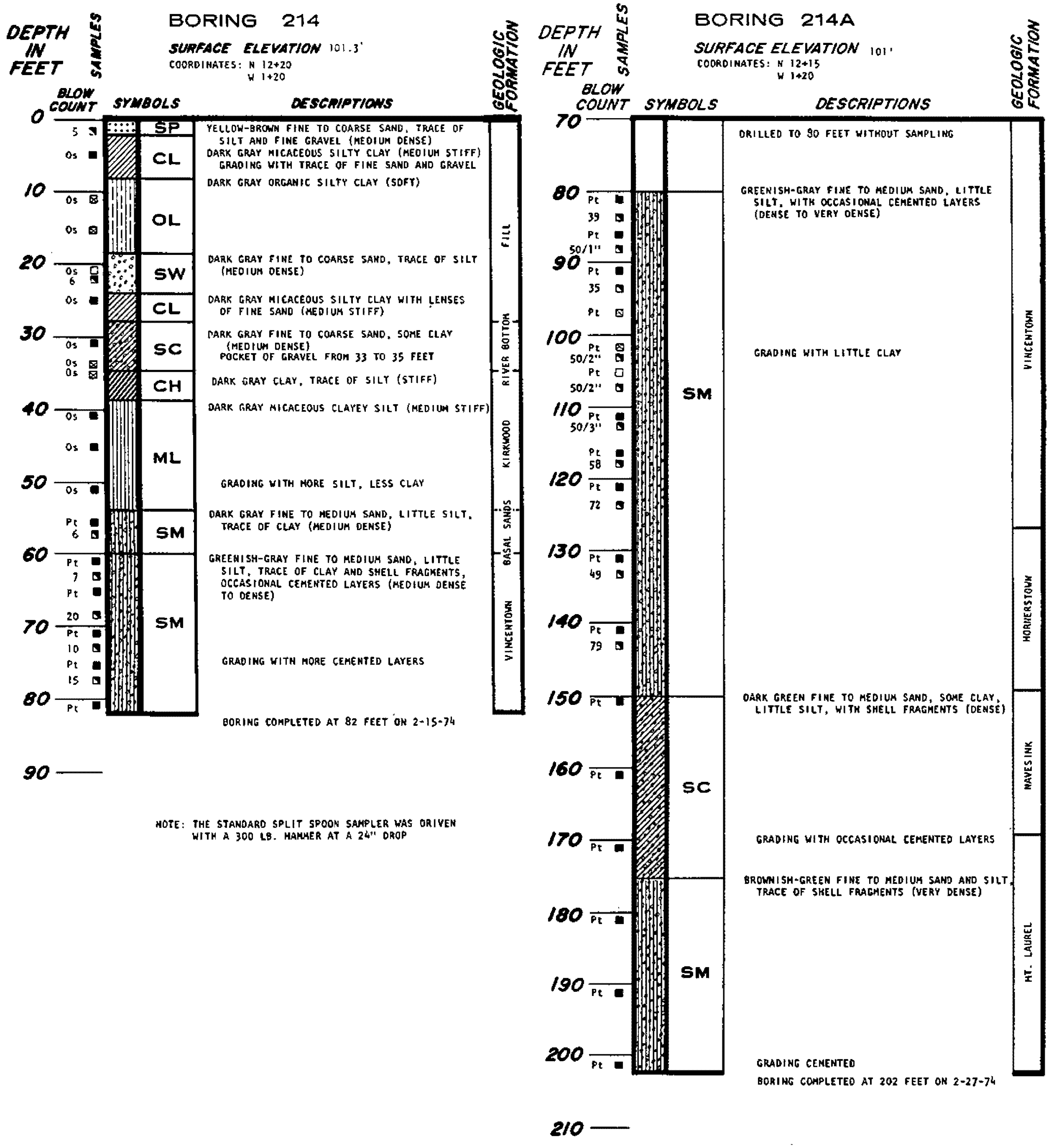
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 107 of 189
FIGURE 2.5-50



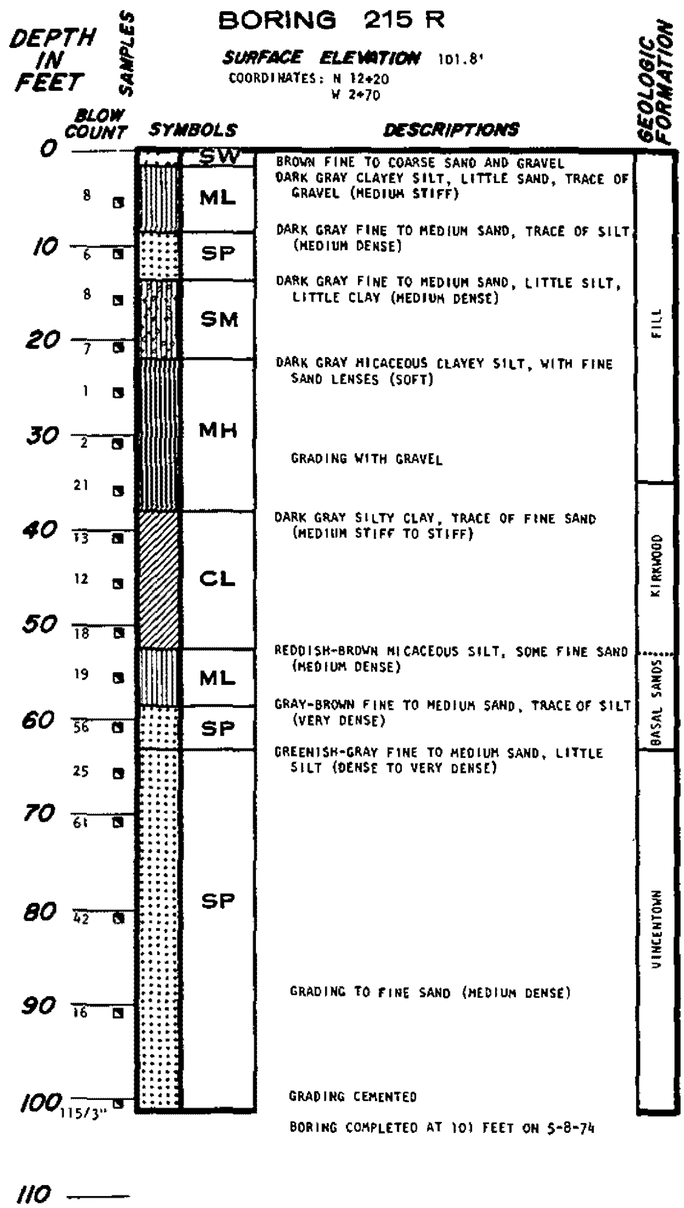
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 108 of 189
FIGURE 2.5-50



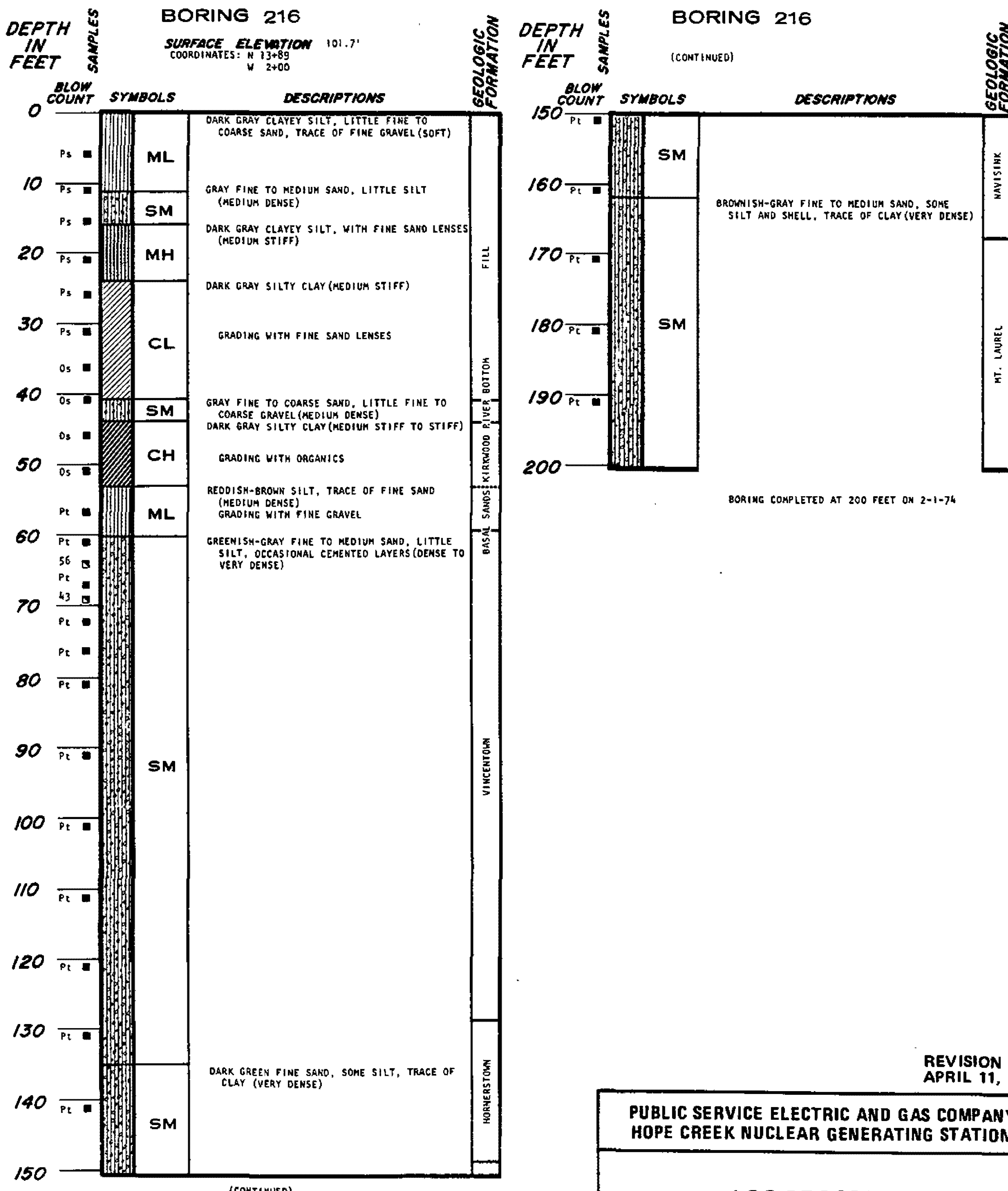
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 109 of 189
FIGURE 2.5-50



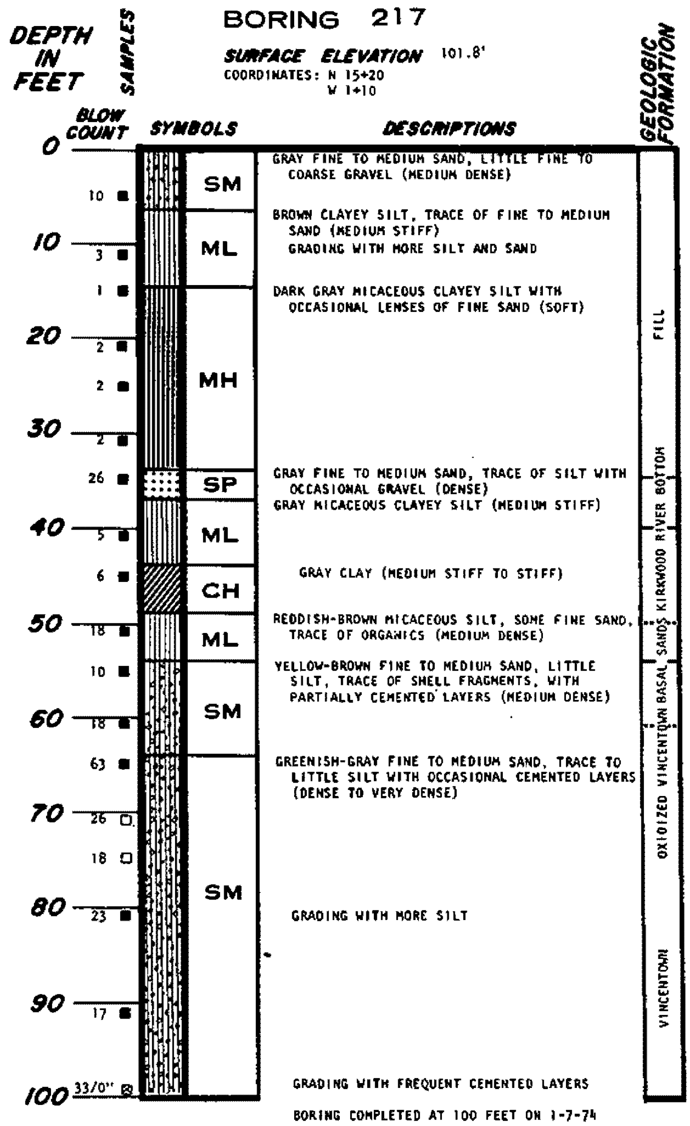
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 110 of 189
FIGURE 2.5-50



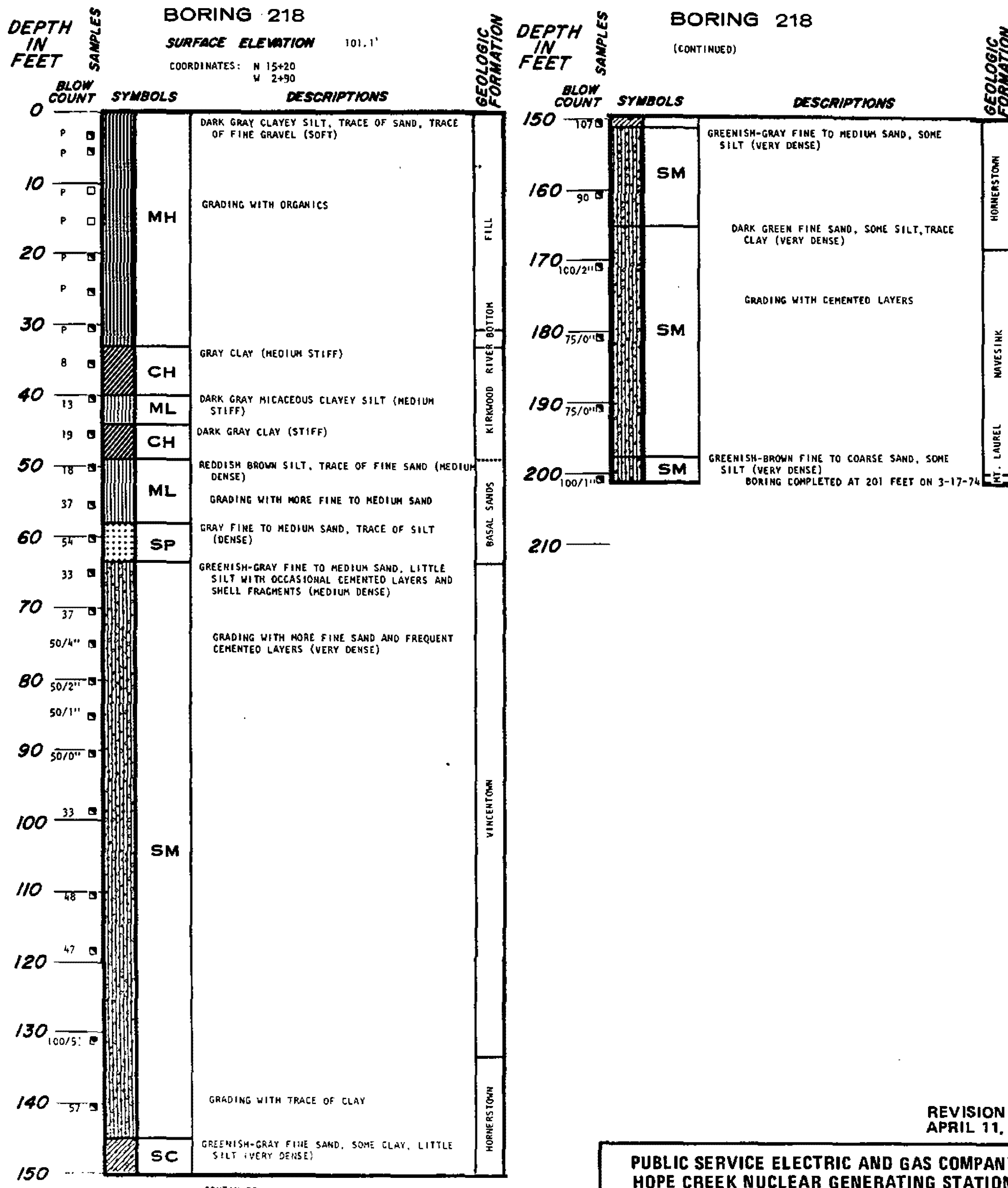
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 111 of 189
FIGURE 2.5-50



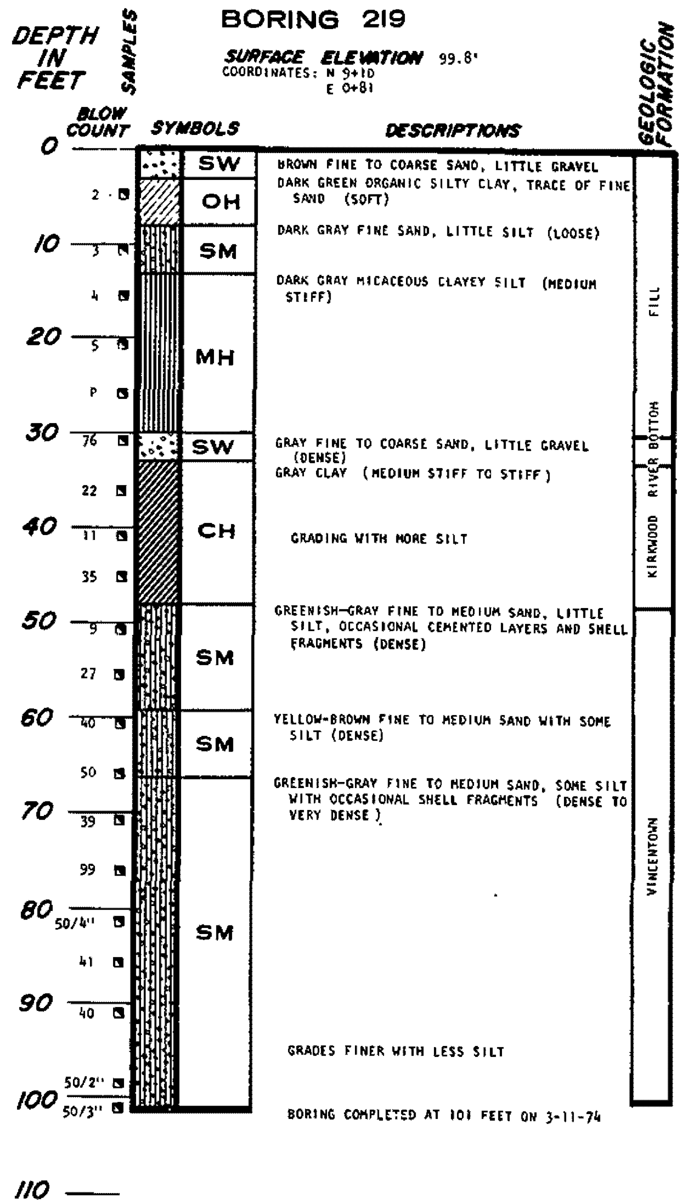
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 112 of 189
FIGURE 2.5-50



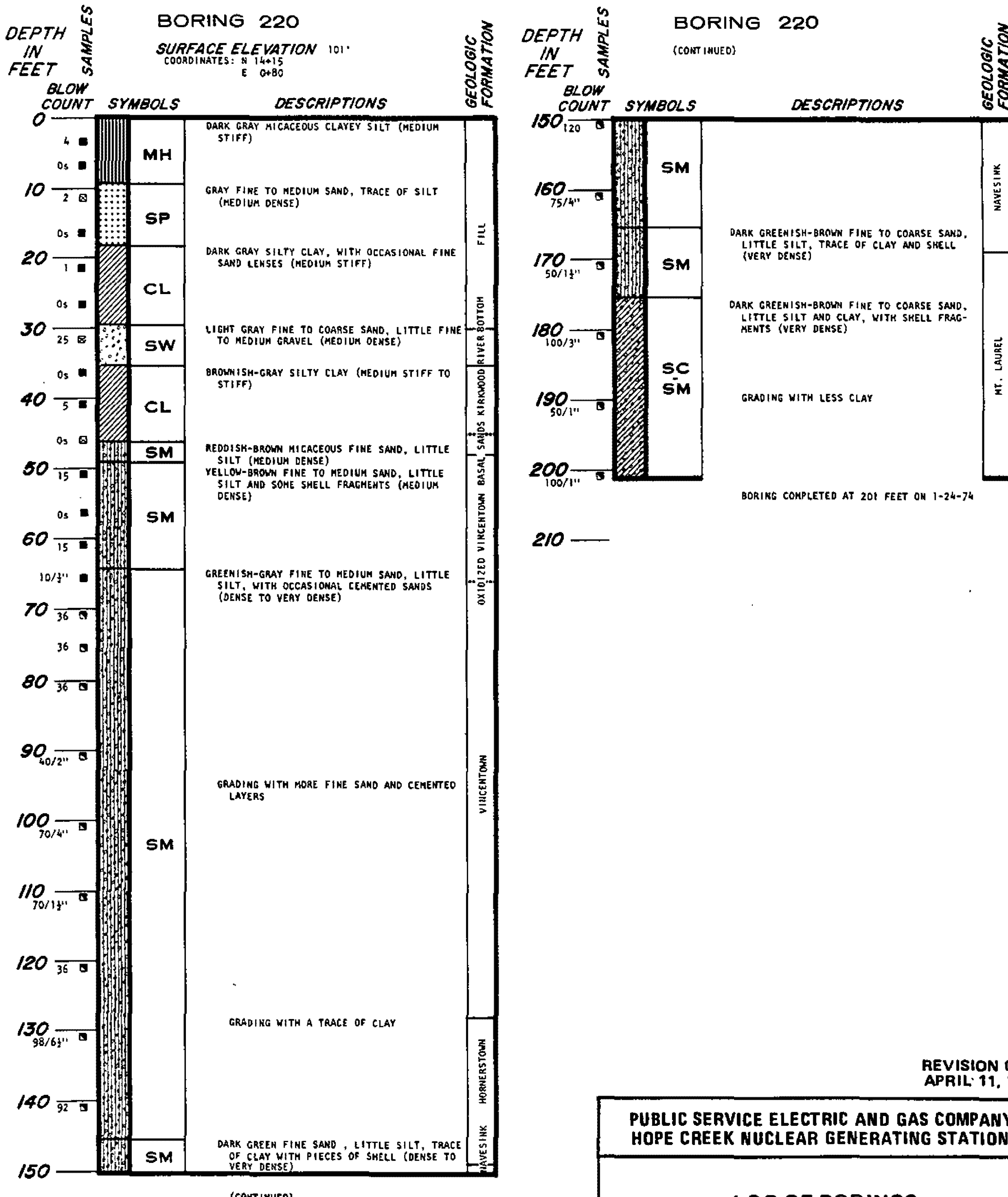
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 113 of 189
FIGURE 2.5-50



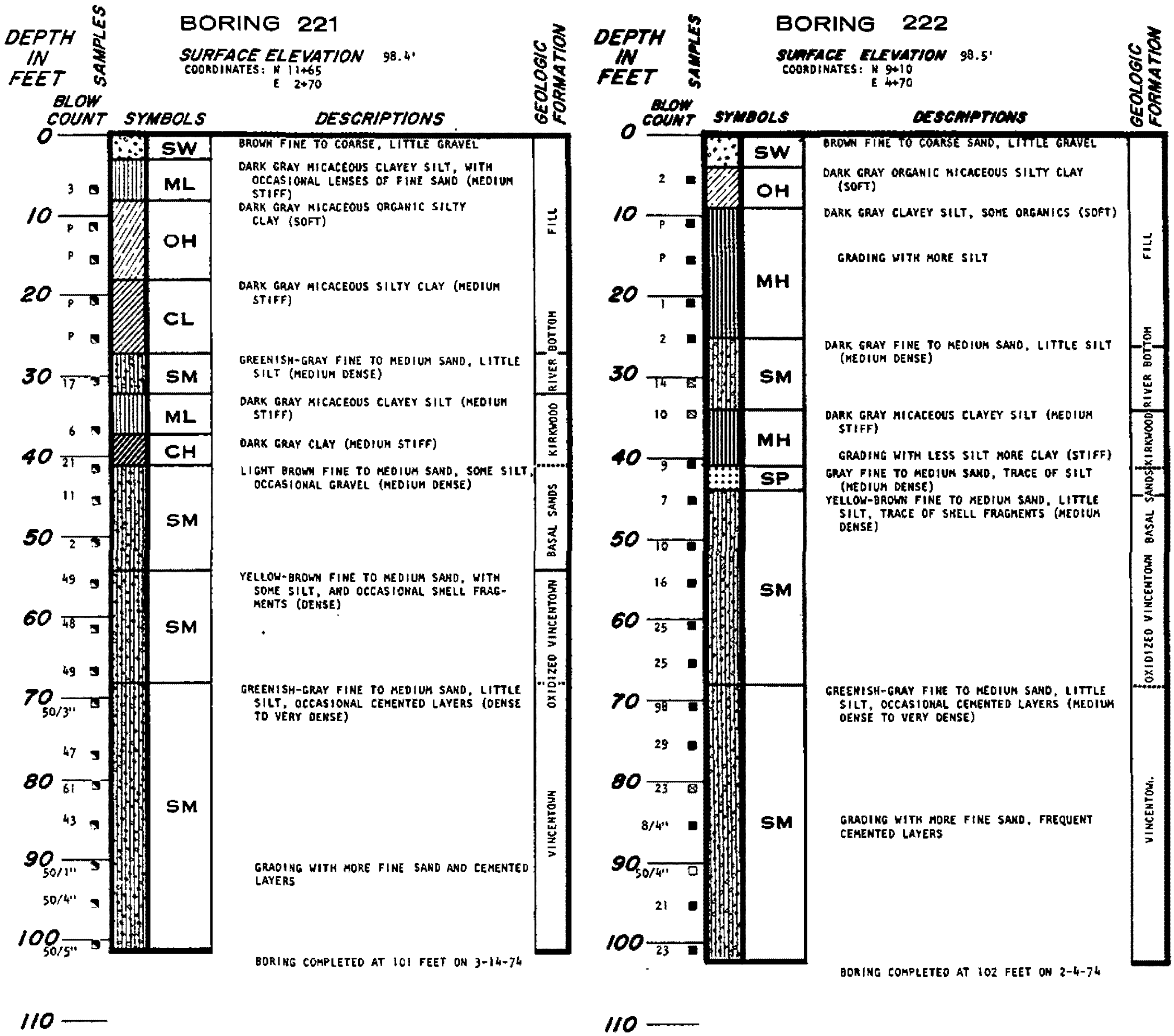
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 114 of 189
FIGURE 2.5-50



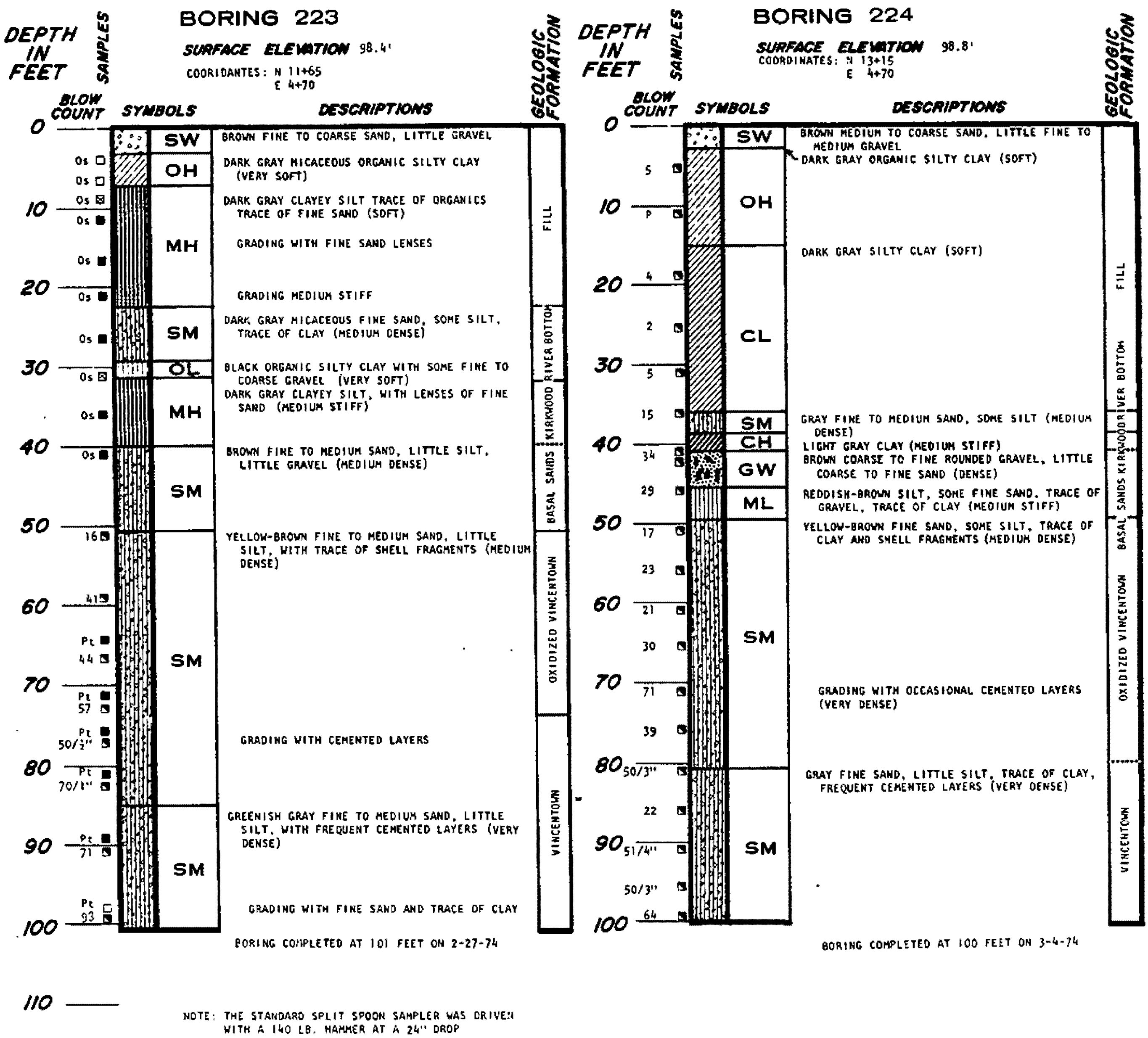
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 115 of 189
FIGURE 2.5-50



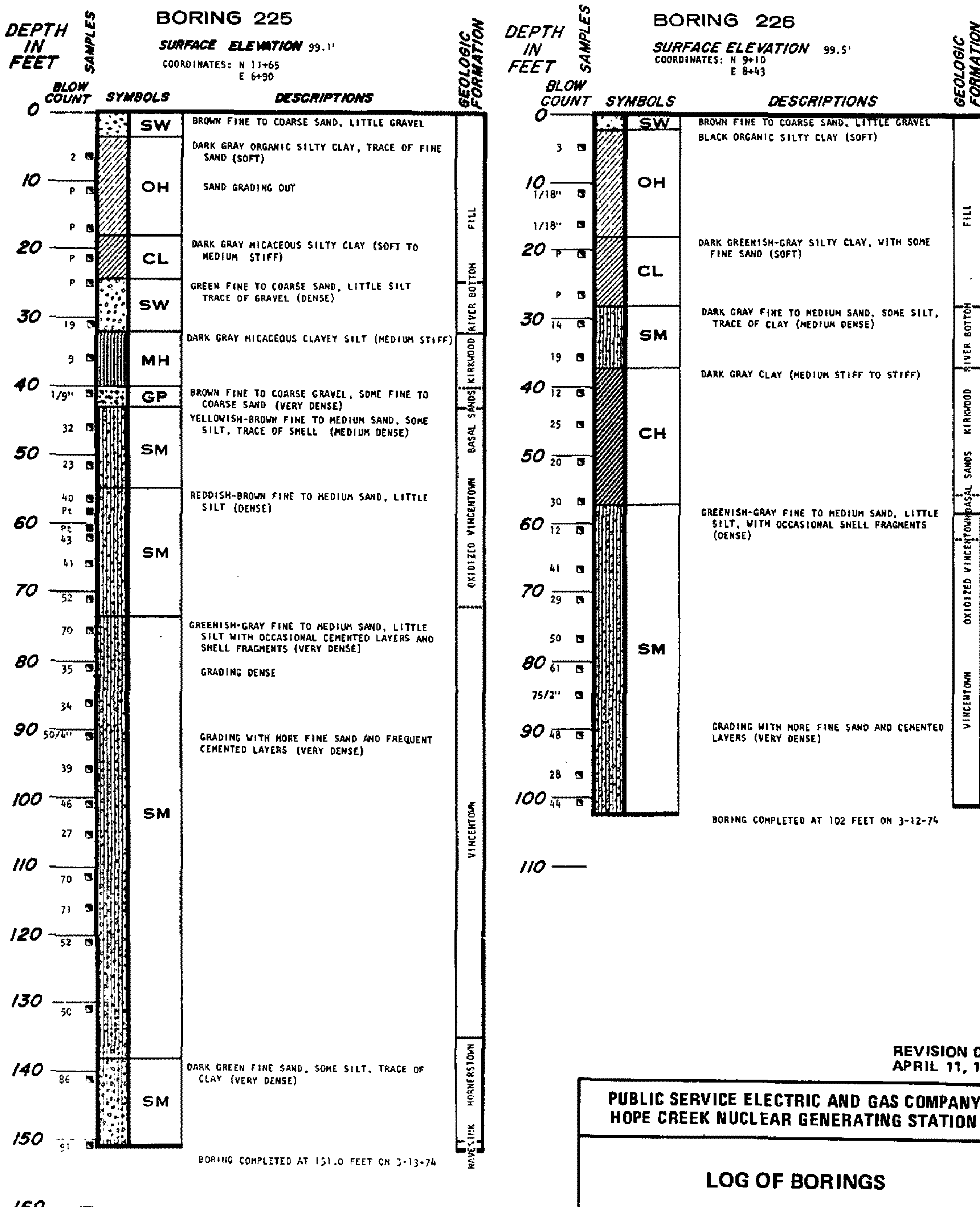
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 116 of 189
FIGURE 2.5-50



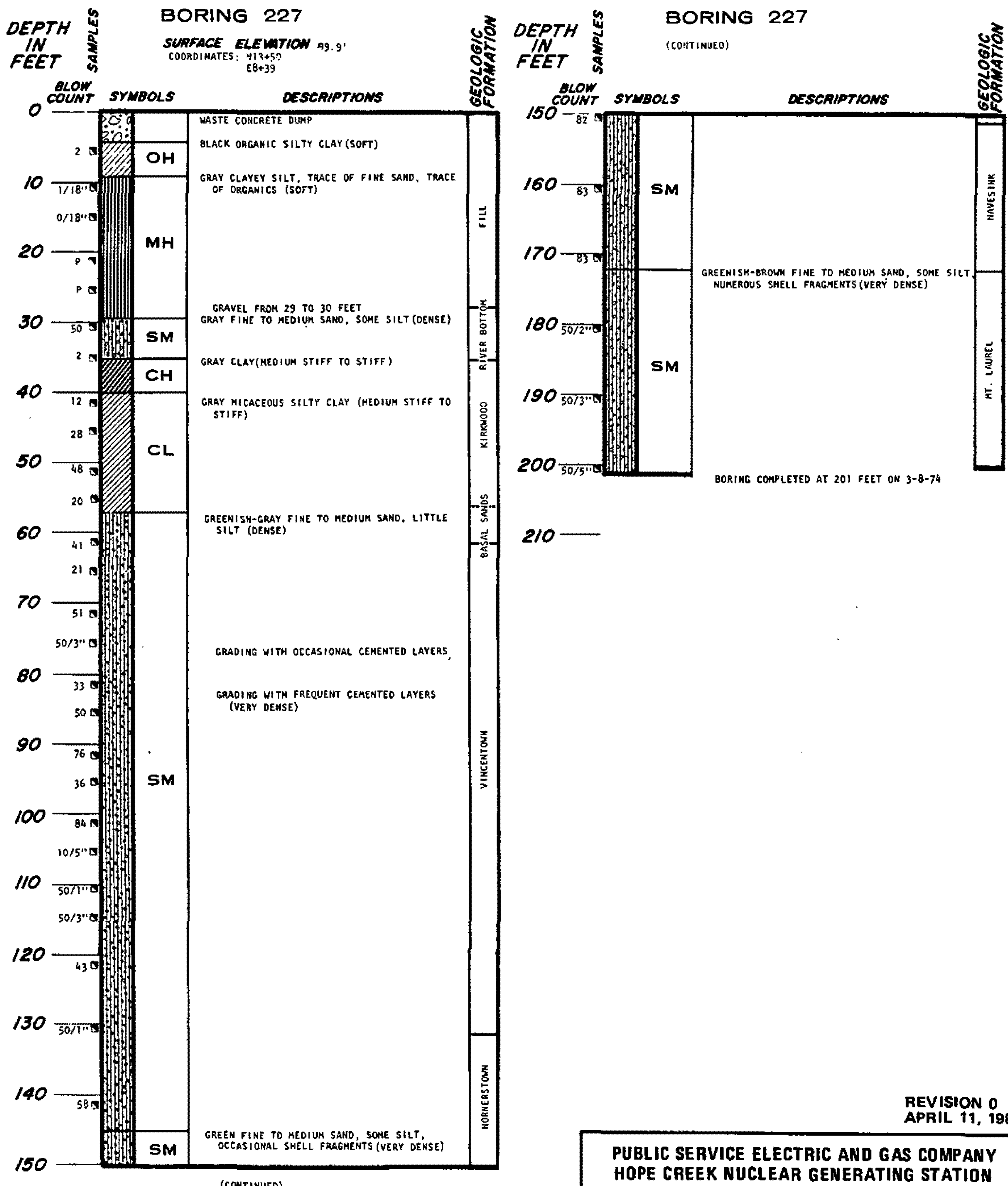
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 117 of 189
FIGURE 2.5-50



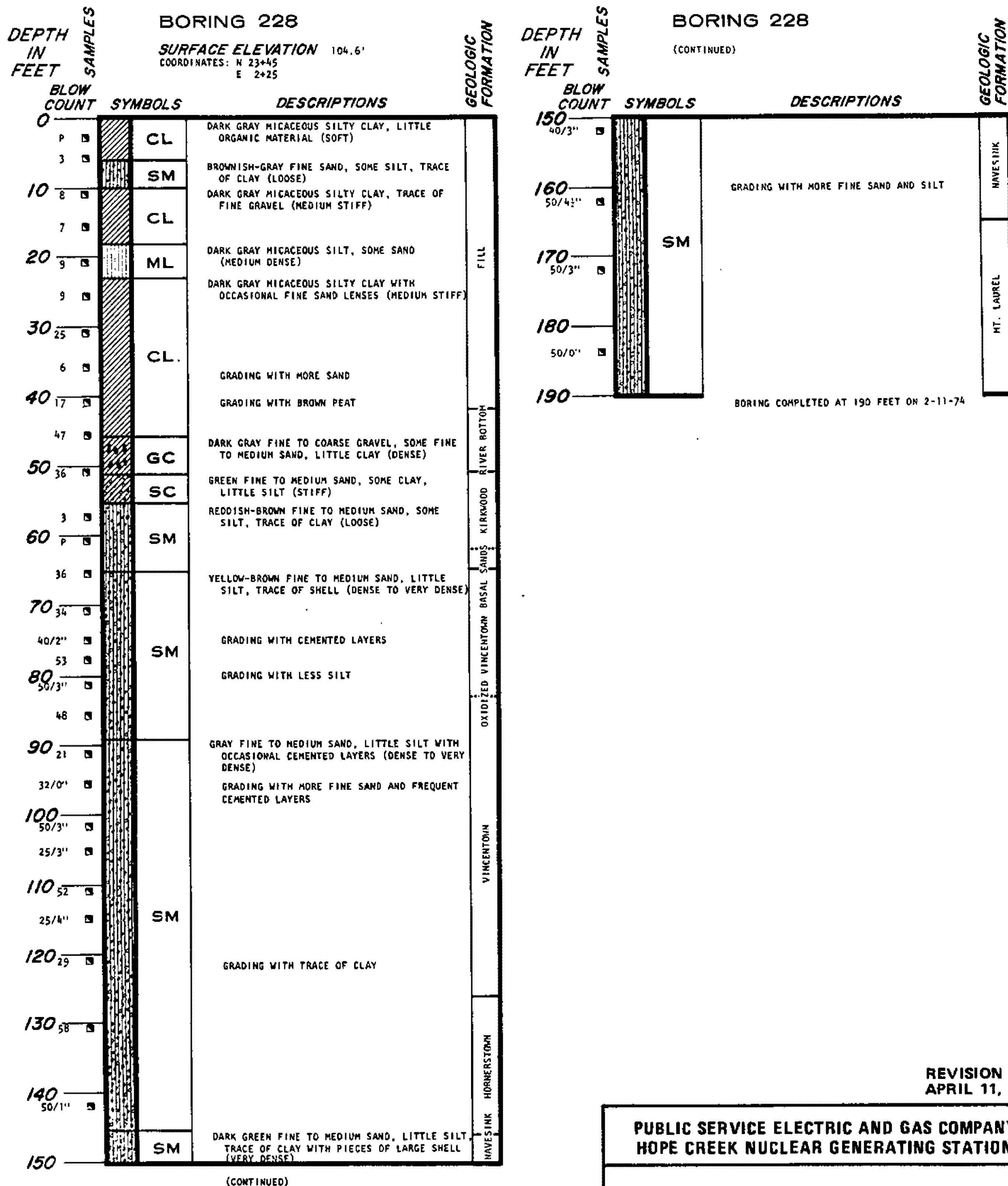
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 118 of 189
FIGURE 2.5-50



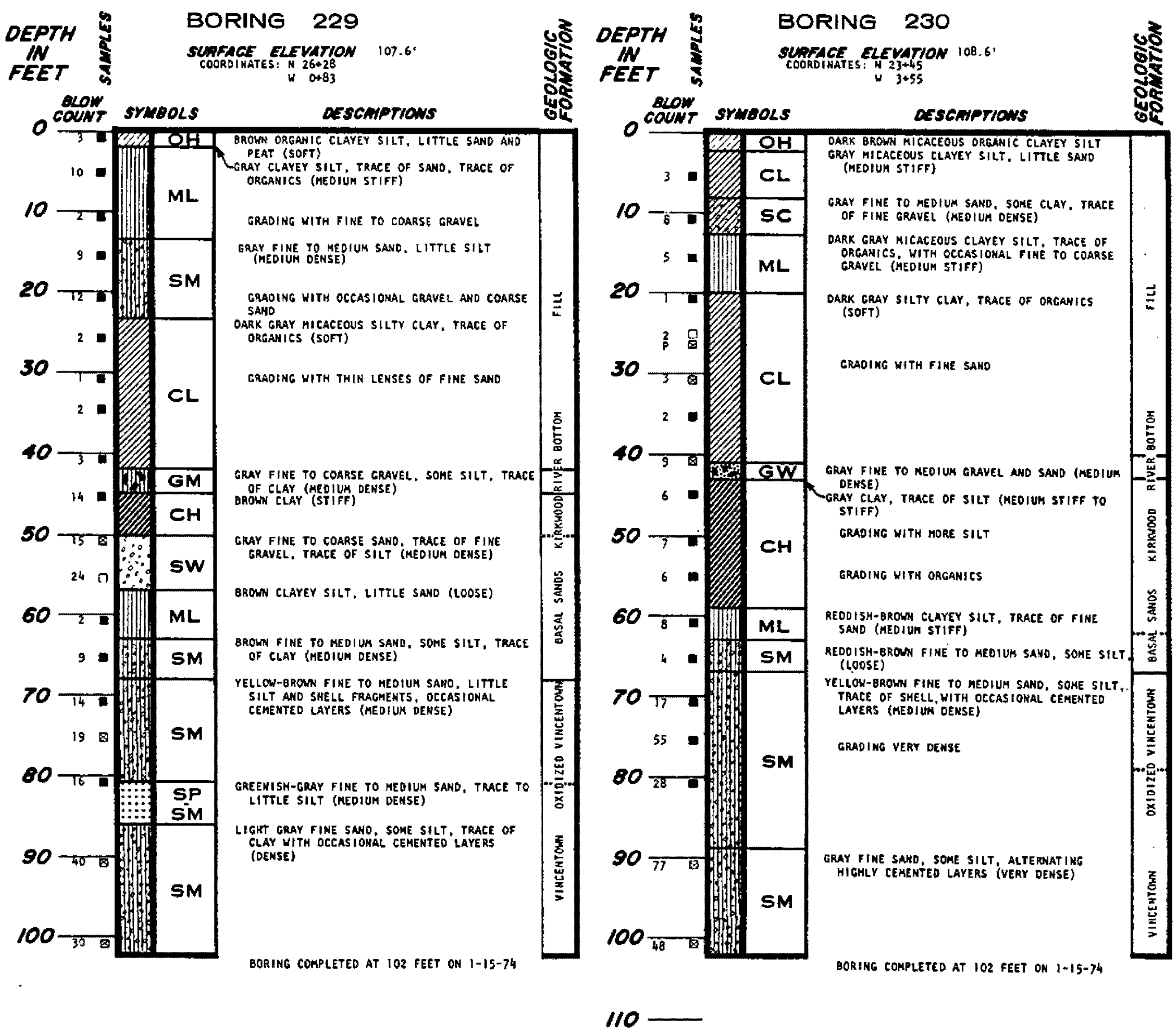
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

FIGURE 2.5-50



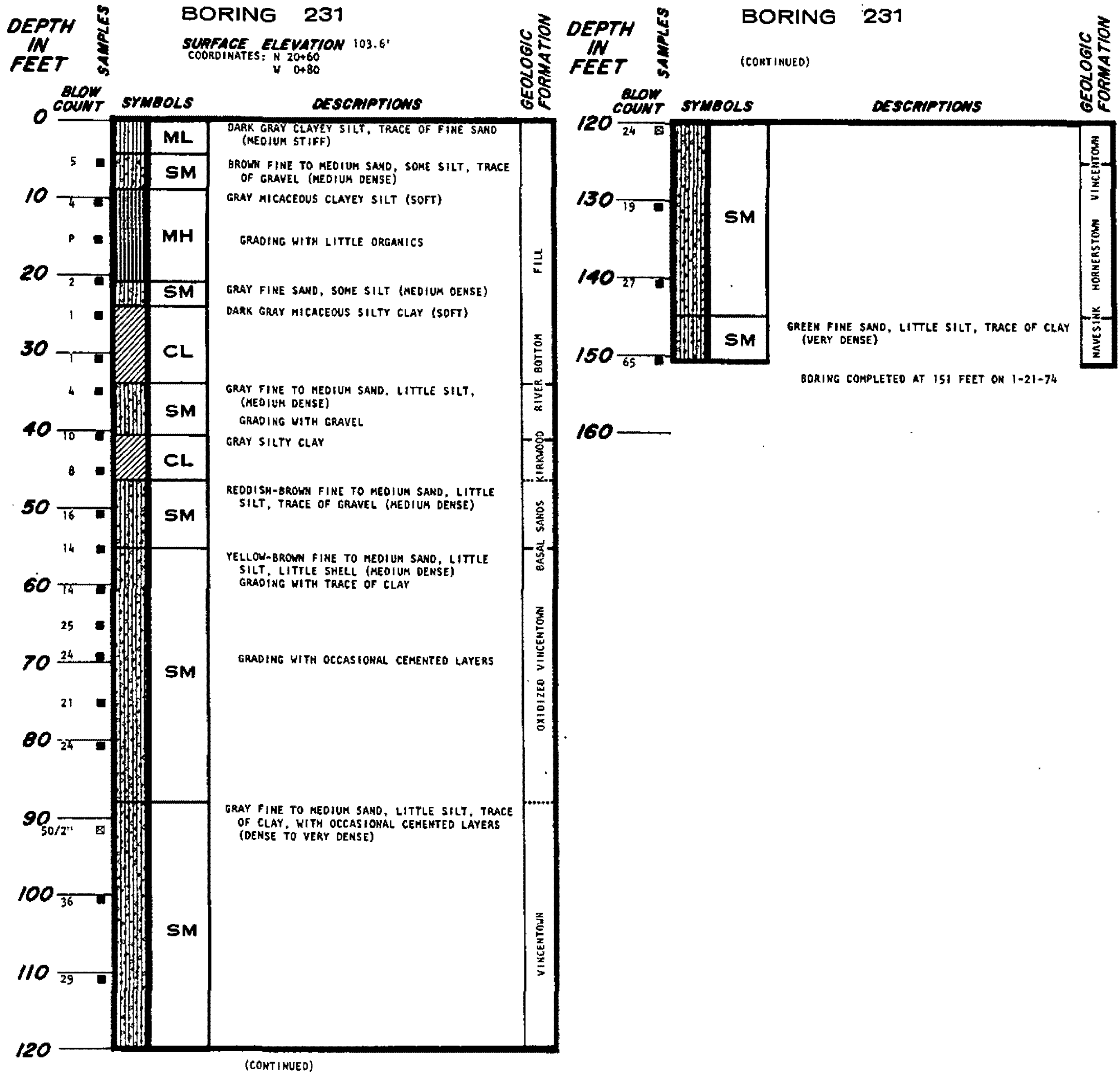
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 120 of 189
FIGURE 2.5-50



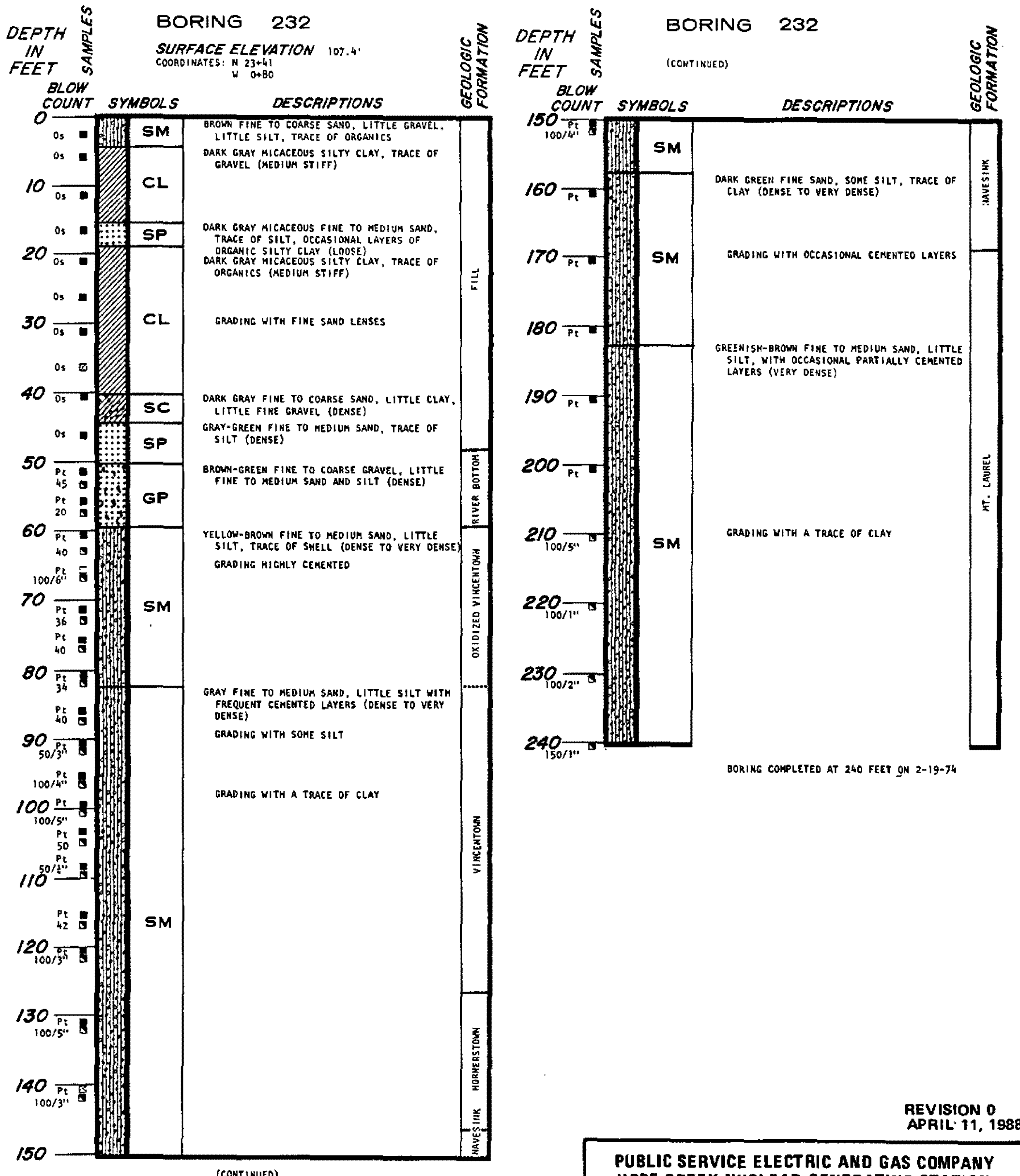
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 121 of 189
FIGURE 2.5-50



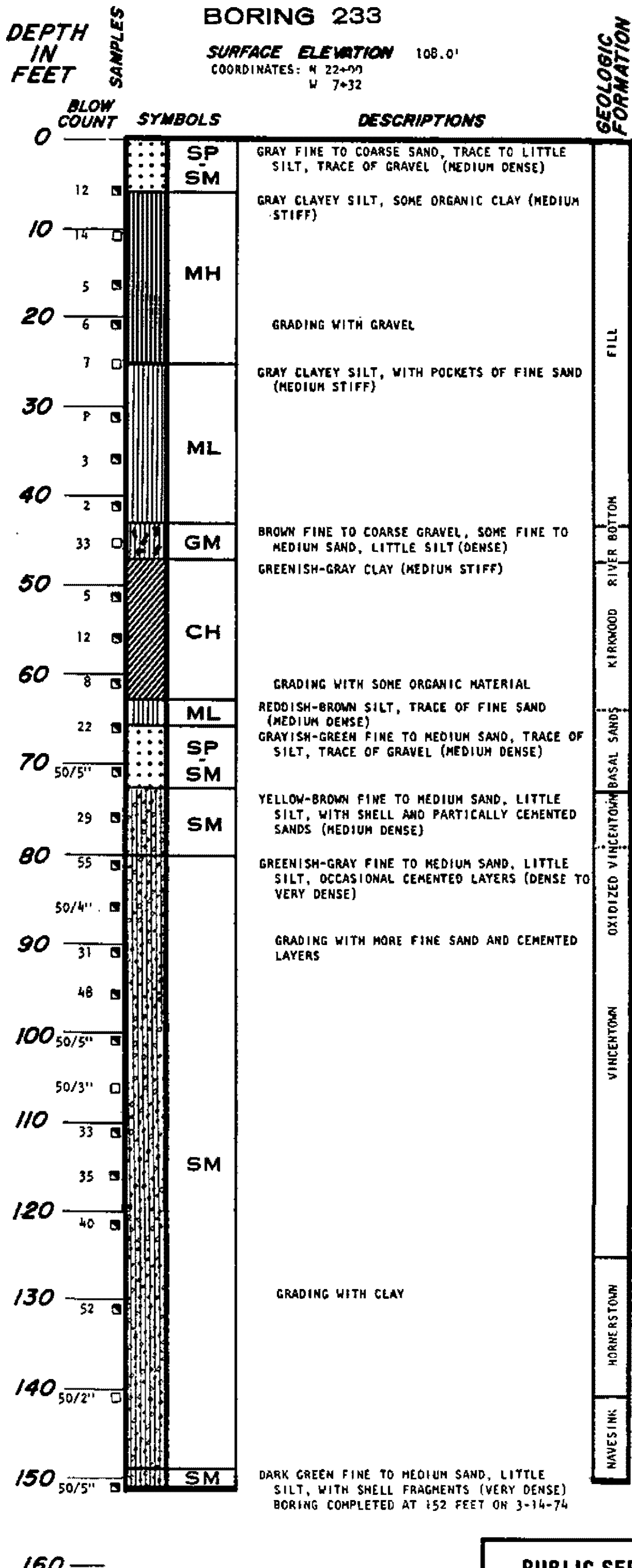
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 122 of 189
FIGURE 2.5-50



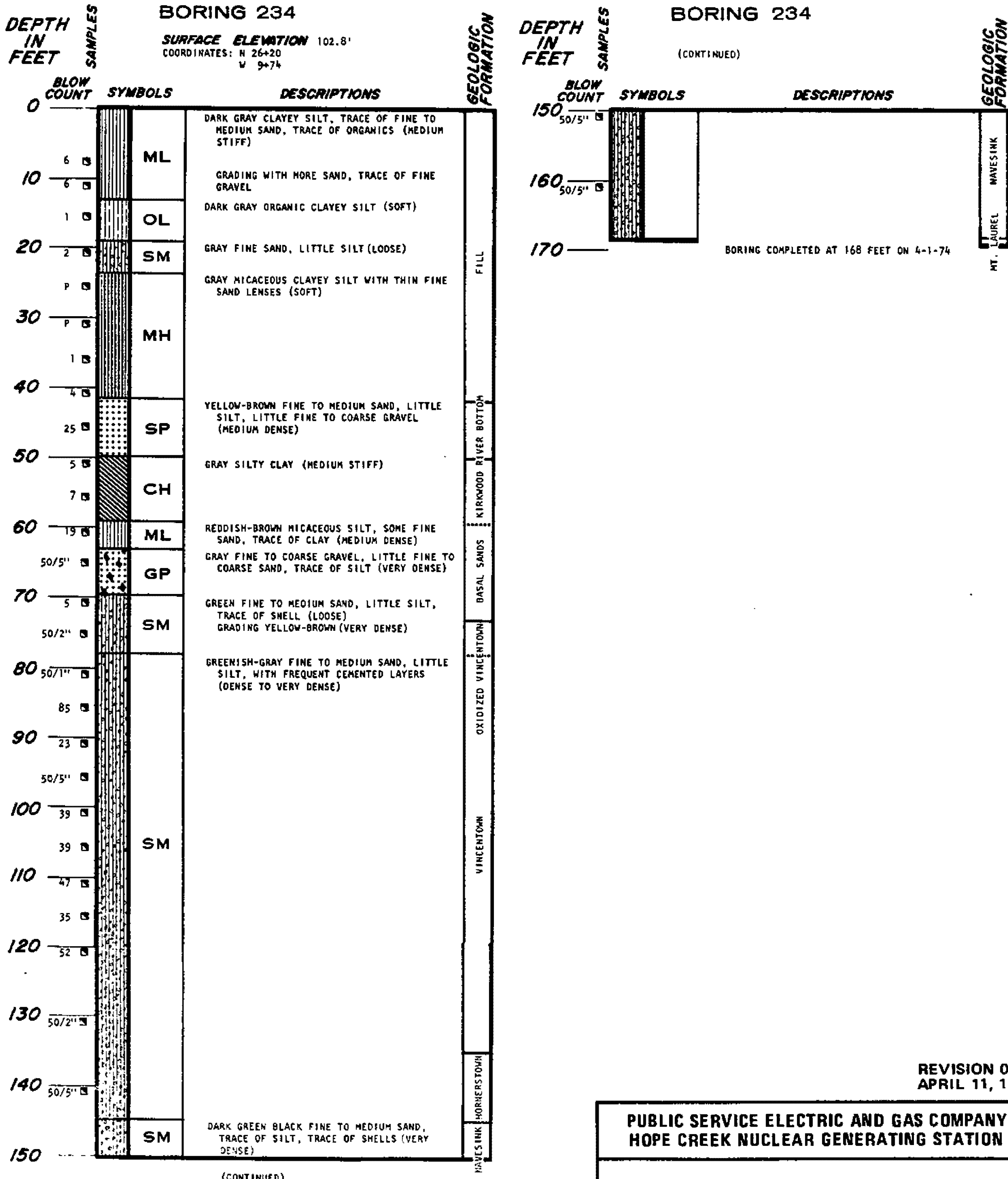
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 123 of 189
FIGURE 2.5-50



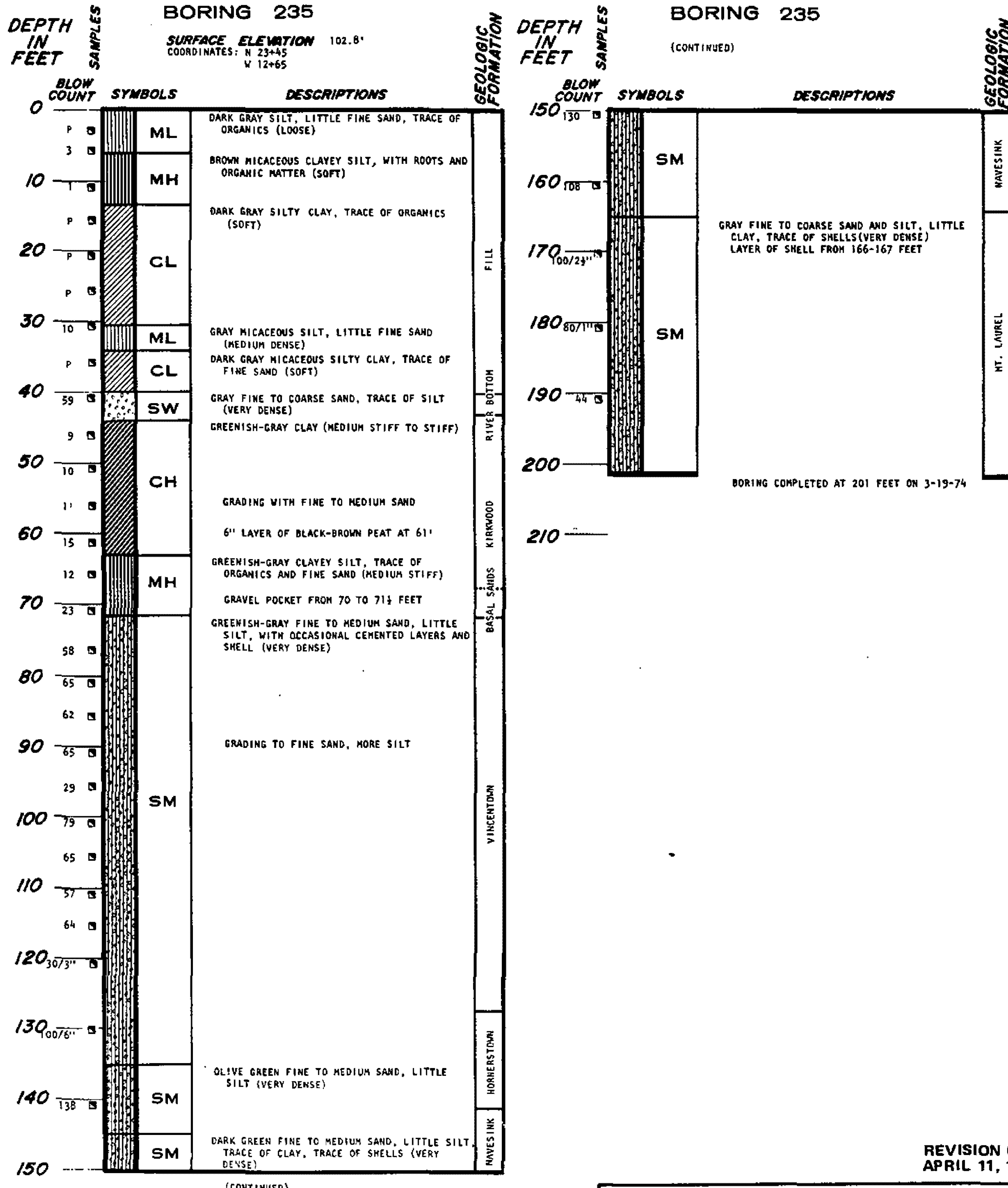
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 124 of 189
FIGURE 2.5-50



REVISION 0
APRIL 11, 1988

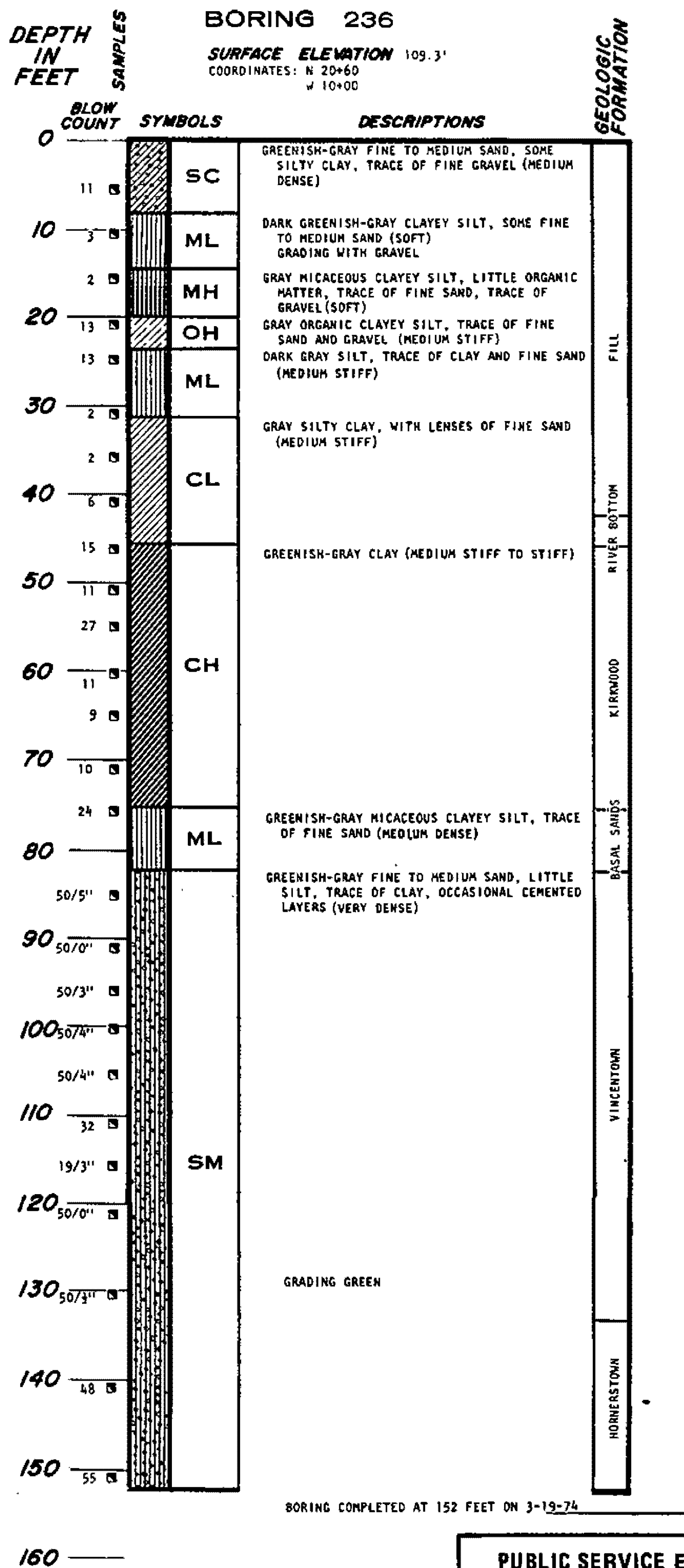
NOTE: THE STANDARD SPLIT SPOON SAMPLER WAS DRIVEN WITH A 300 LB. HAMMER AT A 30" DROP FOR SAMPLES AT 180 AND 190 FEET.

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 125 of 189
FIGURE 2.5-50



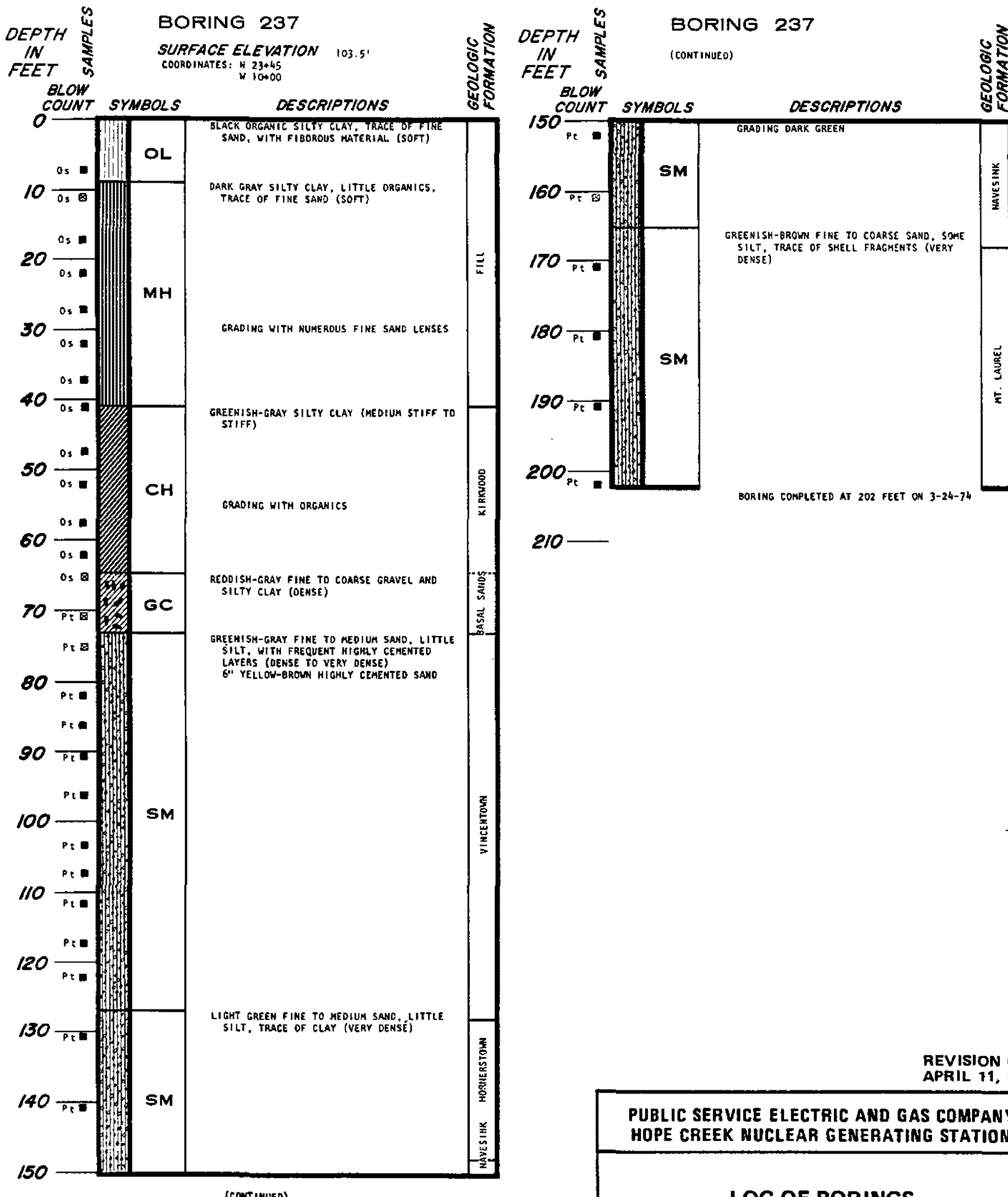
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 126 of 189
FIGURE 2.5-50



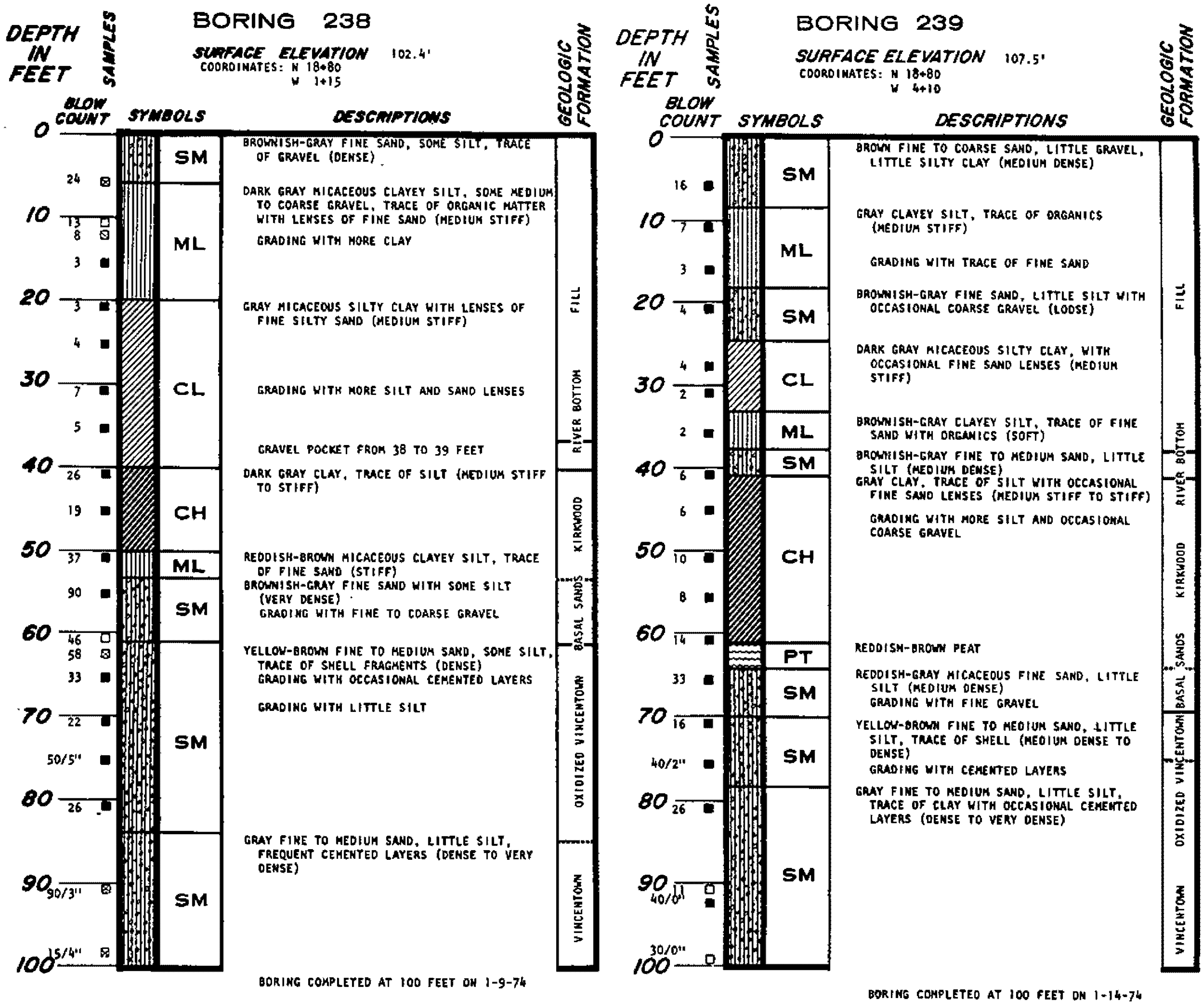
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 127 of 189
FIGURE 2.5-50



NOTE: THE DAMES & MOORE 'U' TYPE SAMPLER WAS DRIVEN WITH A 140 LB. HAMMER AT A 30" DROP FOR SAMPLES FROM 0 TO 60 FEET.

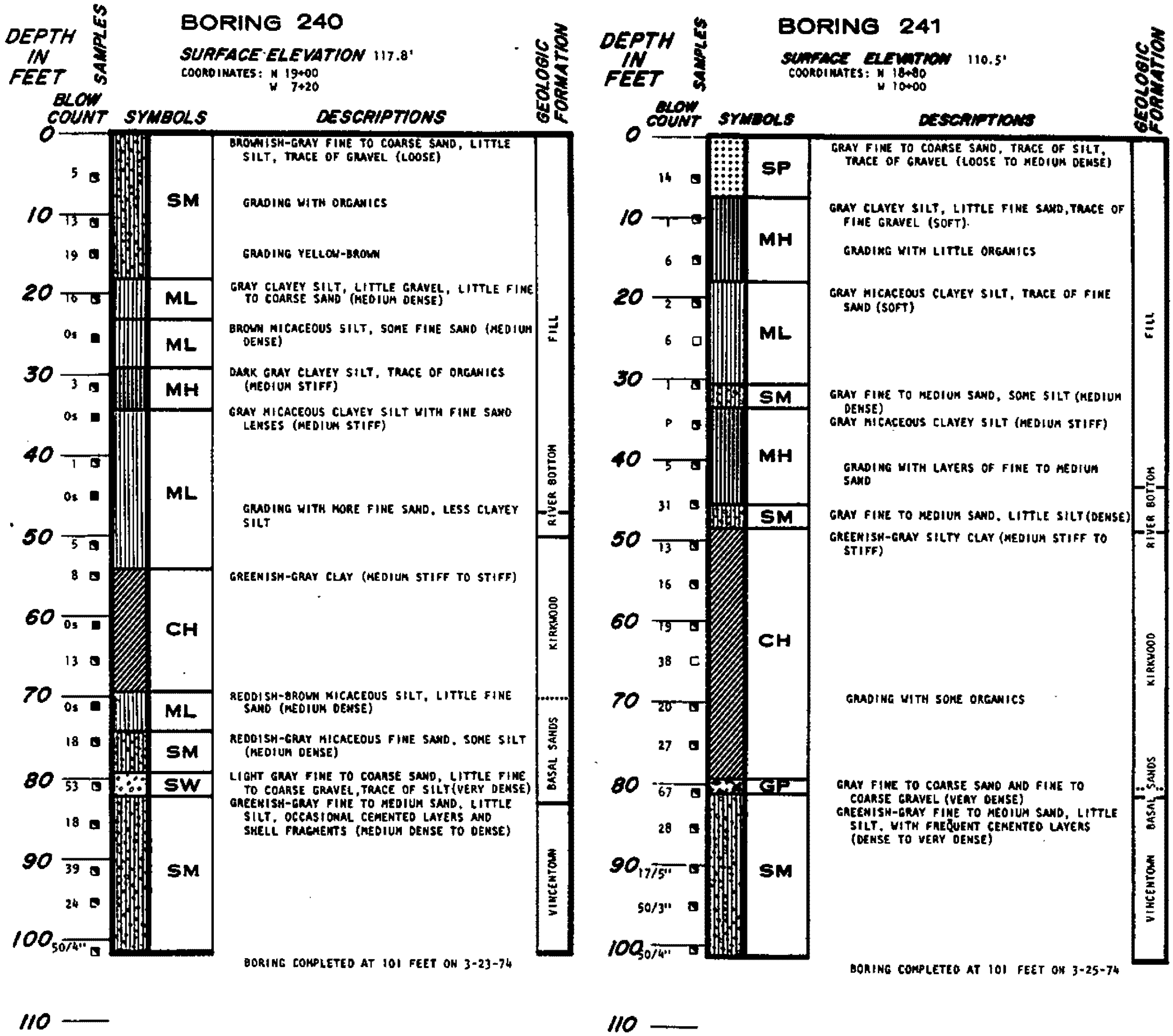
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 128 of 189
FIGURE 2.5-50



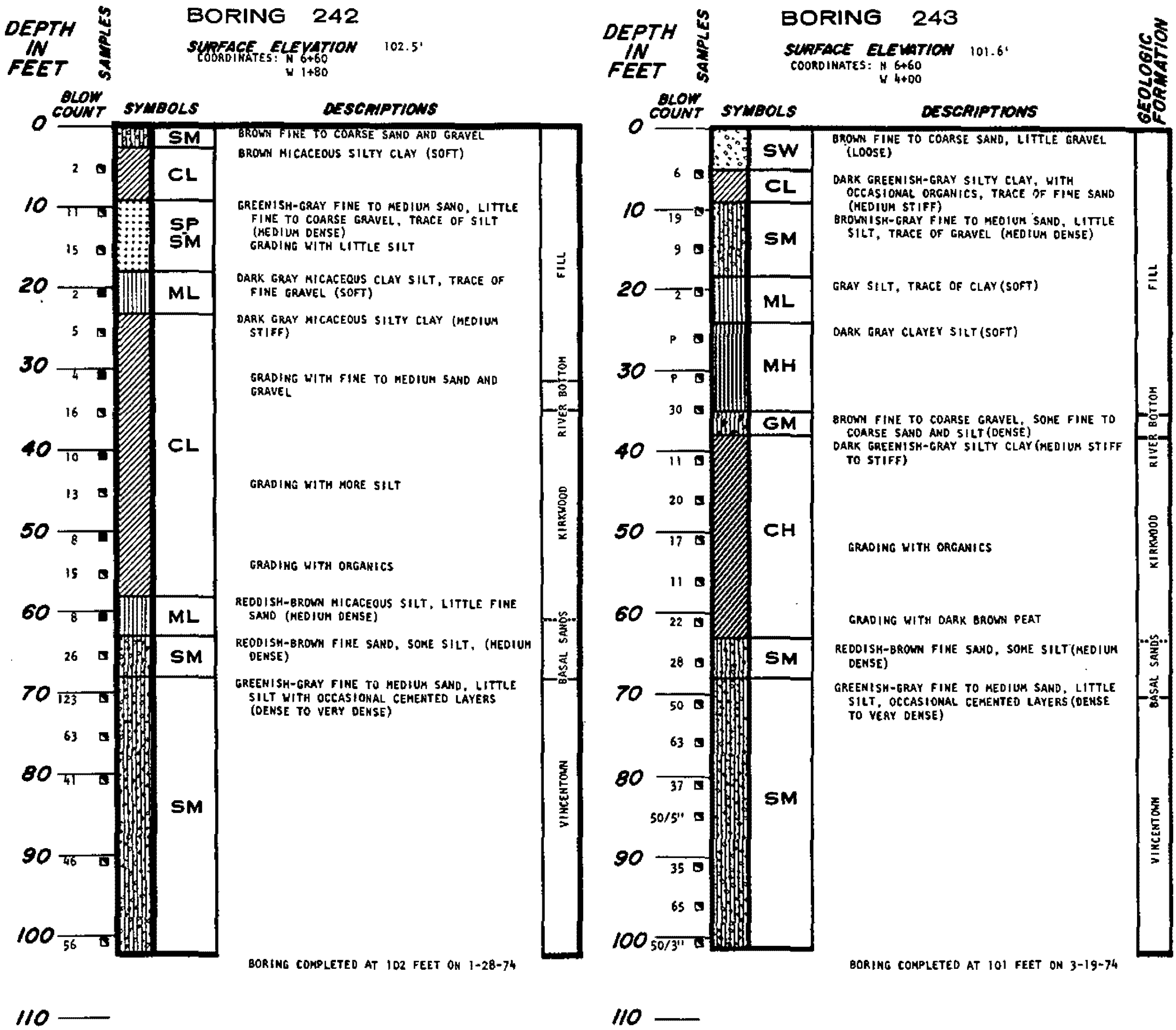
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 129 of 189
FIGURE 2.5-50



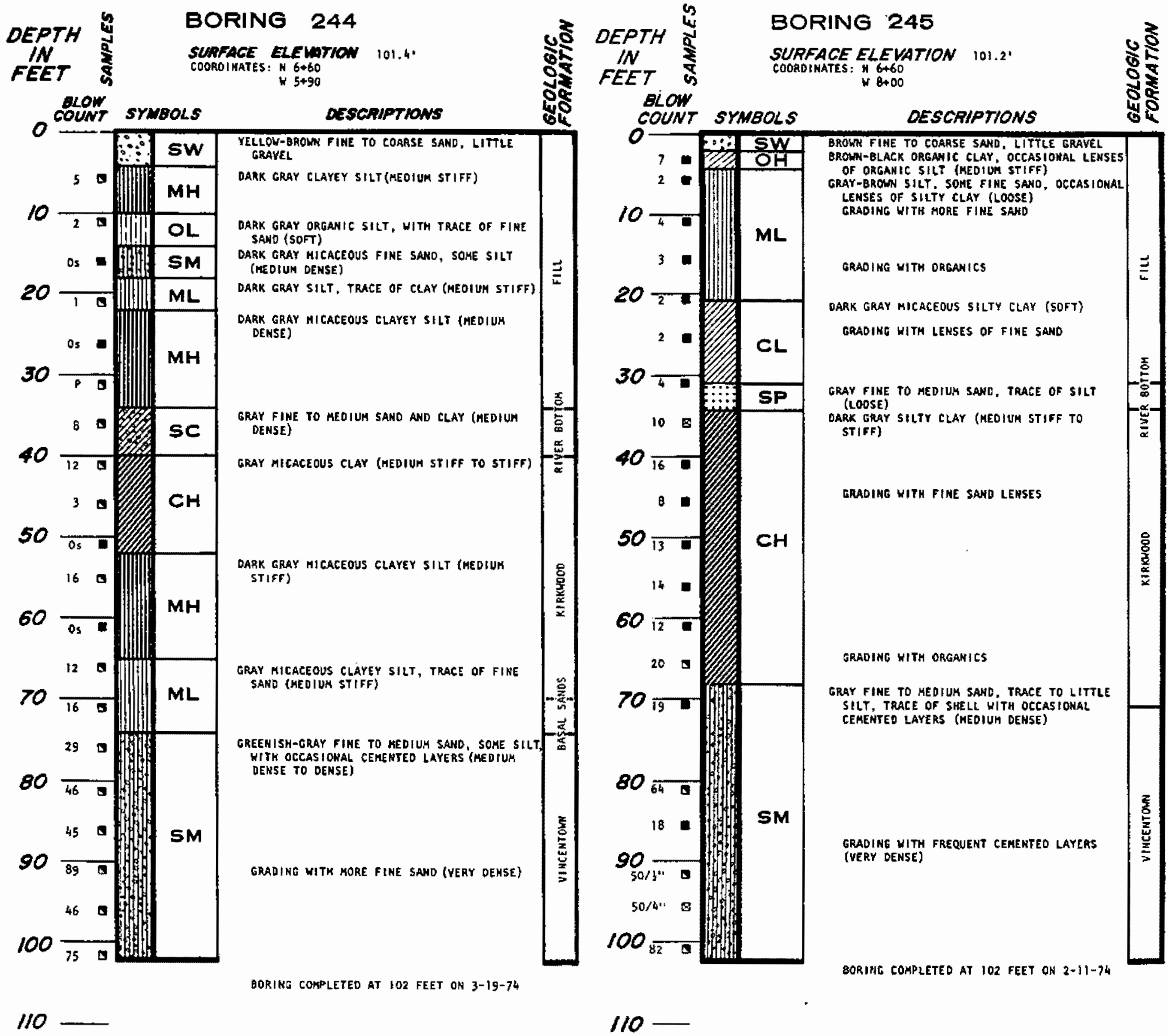
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 130 of 189
FIGURE 2.5-50



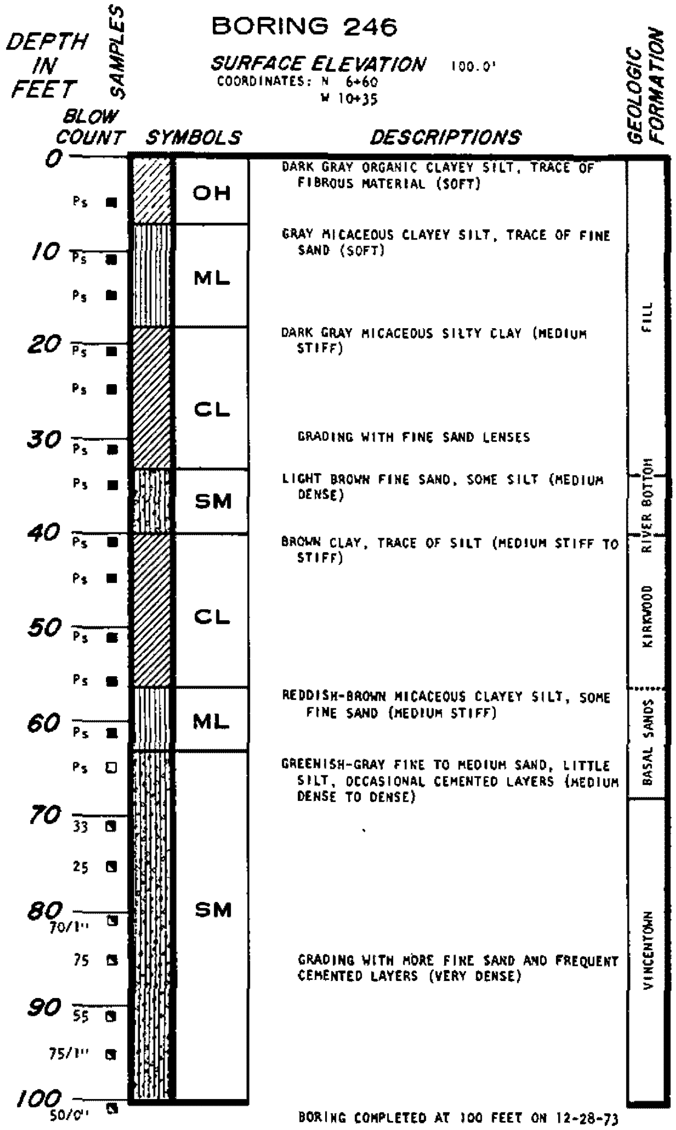
REVISION 0
APRIL 11, 1988

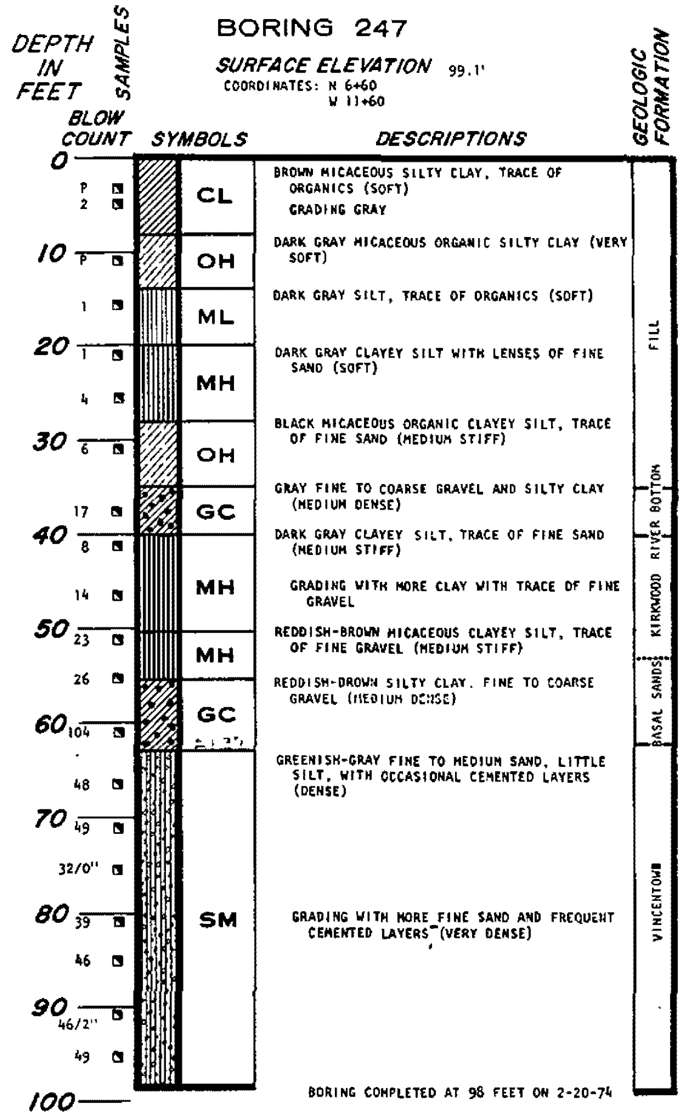
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 131 of 189
FIGURE 2.5-50





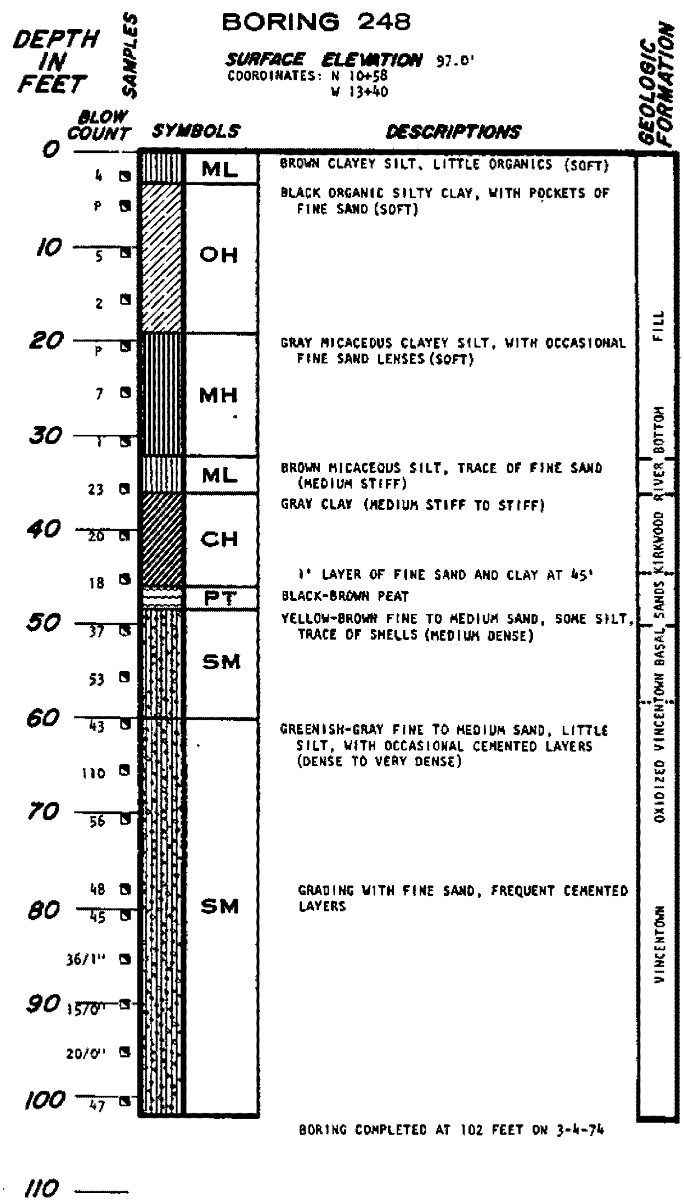
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG OF BORINGS

UPDATED FSAR

Sheet 132 of 189
FIGURE 2.5-50



REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

FIGURE 2.5-50

BORING 249

SURFACE ELEVATION 98.1'

COORDINATES: N 11+60
W 13+63

DEPTH IN FEET	BLOW COUNT	SYMBOLS	DESCRIPTIONS	GEOLOGIC FORMATION
0–9	3, 4	ML	BROWN MICACEOUS CLAYEY SILT (SOFT) GRADING WITH ORGANIC SILTY CLAY	FILL
9–26	T, 1, P, 4	CL	DARK GRAY MICACEOUS SILTY CLAY, TRACE OF ORGANICS (SOFT) GRADING WITH LENSES OF FINE SAND AND BLACK ORGANIC SILTY CLAY	FILL
26–31		OL	DARK GRAY ORGANIC SILTY CLAY (SOFT)	RIVER BOTTOM
31–36	Tw, 11	SP	DARK GRAY FINE SAND, TRACE OF SILT WITH OCCASIONAL POCKETS OF CLAYEY SILT (LOOSE) GRADING WITH FINE TO COARSE GRAVEL	RIVER BOTTOM
36–46	Tw, Tw	CH	DARK BROWN CLAY, TRACE OF SILT (MEDIUM STIFF TO STIFF) GRADING GRAY	KIRKWOOD
46–54	50/5", 30/1"	SM	BROWN FINE TO MEDIUM SAND, LITTLE SILT (MEDIUM DENSE) GRADING WITH COARSE GRAVEL	BASAL SANDS
54–60	13	SM	YELLOW-BROWN FINE TO MEDIUM SAND, LITTLE SILT, TRACE OF SHELL FRAGMENTS (MEDIUM DENSE)	OXIDIZED VINCENTOWN
60–80	26, 28, 50, 18	SP SM	GREENISH-GRAY FINE TO MEDIUM SAND, TRACE TO LITTLE SILT, TRACE OF SHELL FRAGMENTS (MEDIUM DENSE) GRADDING WITH OCCASIONAL CEMENTED LAYERS	OXIDIZED VINCENTOWN
80–149	33, 25/4", 10/1", 23, 35/5", 35/0", 50/4", 32, 62, 74, 88	SM	LIGHT GRAY FINE SAND, SOME SILT, LITTLE CLAY WITH FREQUENT CEMENTED LAYERS (DENSE TO VERY DENSE) GRADING GREEN	VINCENTOWN / HORNERSTOWN
149–150		SM	DARK GREEN FINE SAND, SOME SILT, LITTLE CLAY, SHELL FRAGMENTS (VERY DENSE)	NAVESINK

(CONTINUED)

BORING 249

(CONTINUED)

DEPTH IN FEET	BLOW COUNT	SYMBOLS	DESCRIPTIONS	GEOLOGIC FORMATION
150–167	60, 60	SM		NAVESINK
167–201	50/2", 100/3", 30/2", 104	SM SC	GREENISH-BROWN FINE TO COARSE SAND, SOME SILT, LITTLE CLAY WITH SHELL FRAGMENTS (VERY DENSE) BORING COMPLETED AT 201 FEET ON 2-15-74	MT. LAUREL

210

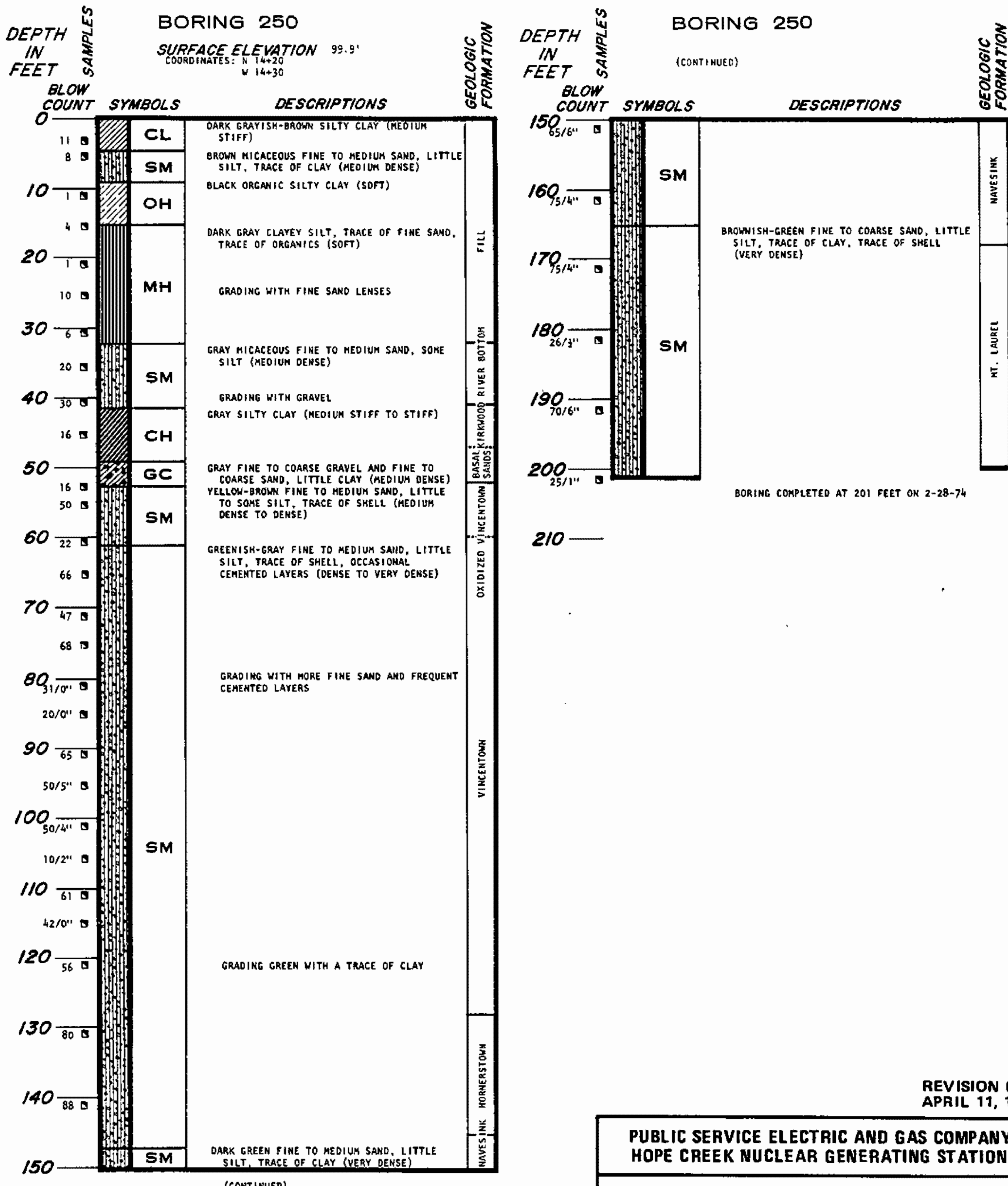
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

FIGURE 2.5-50



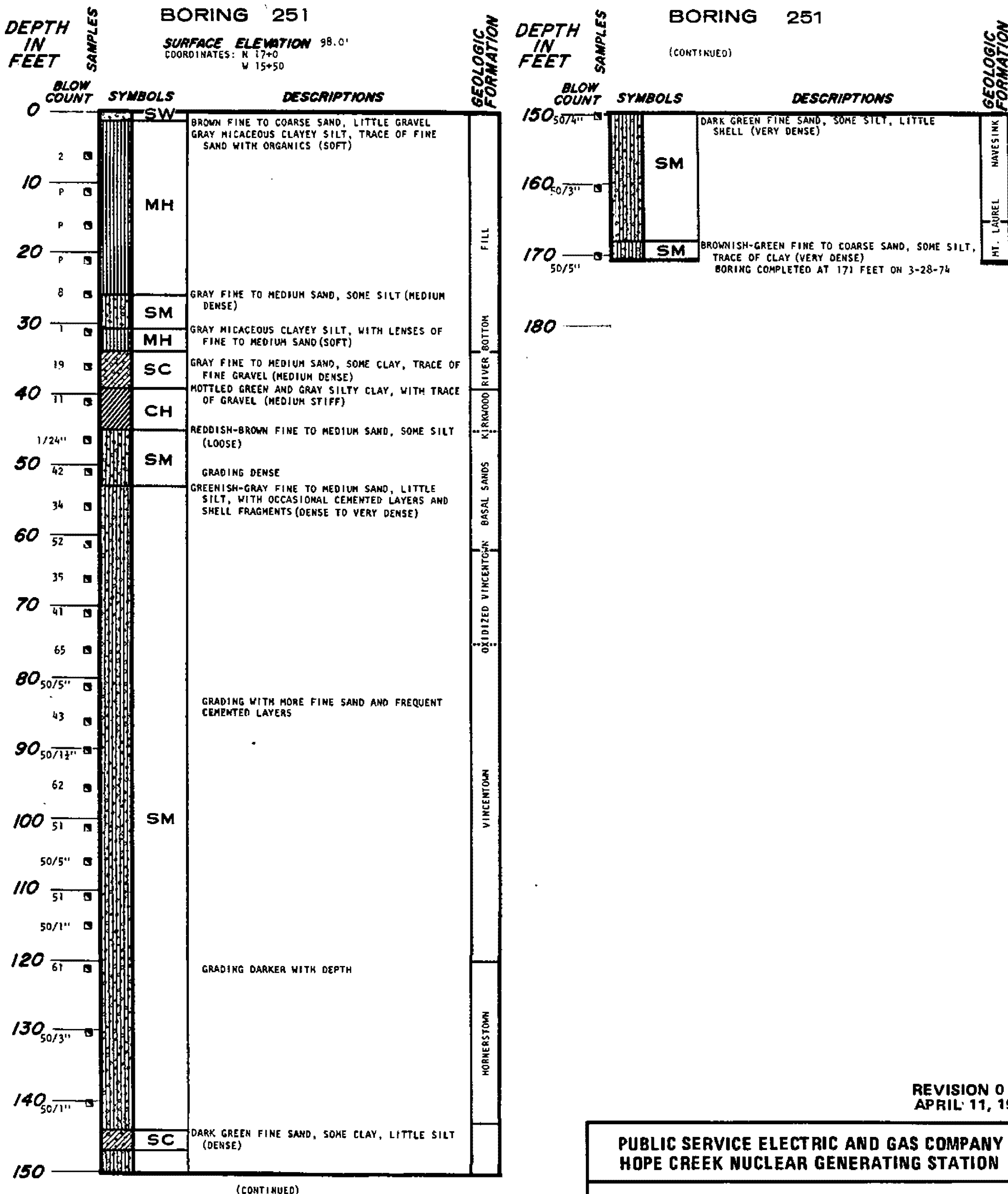
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 135 of 189
FIGURE 2.5-50



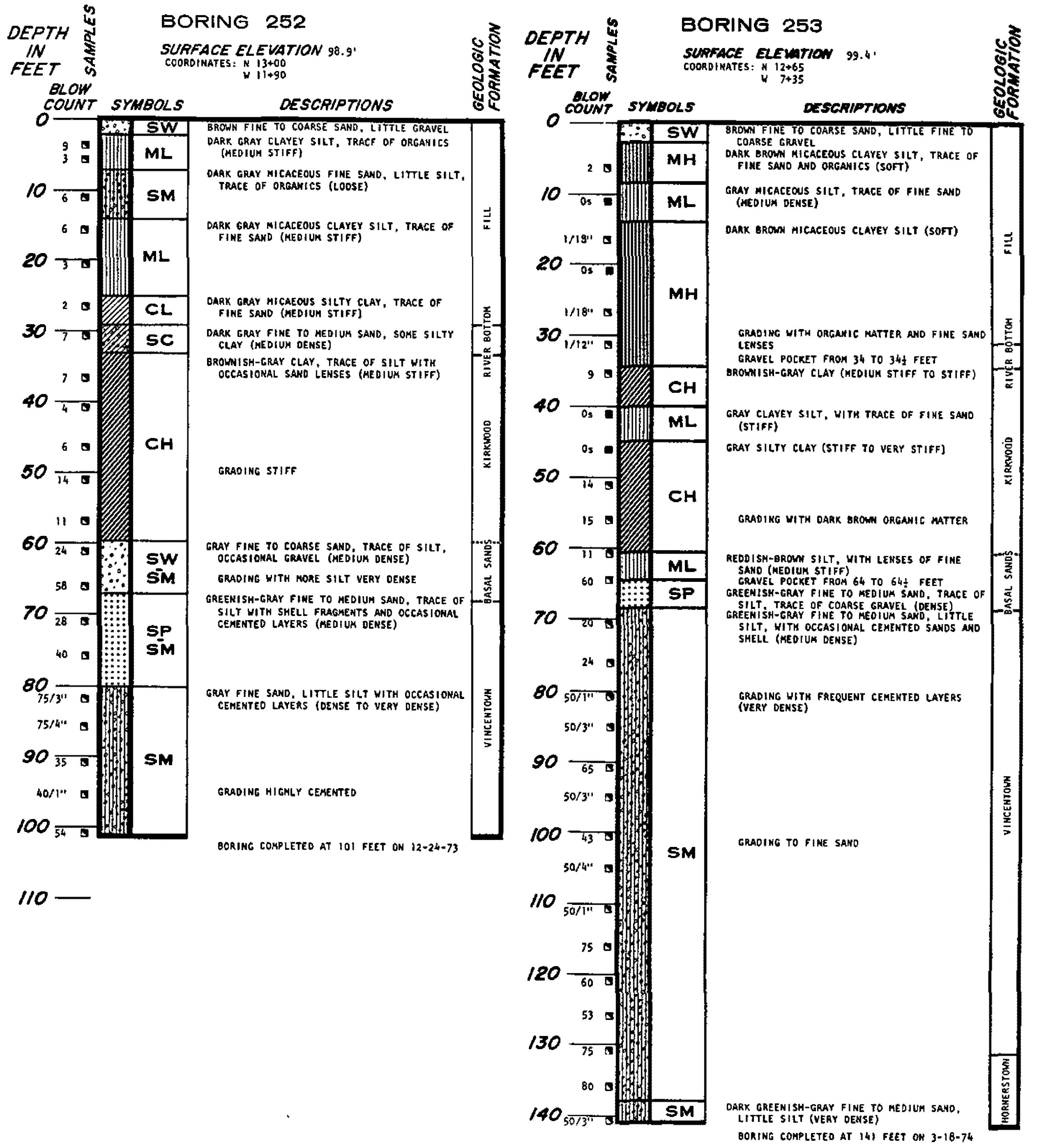
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 136 of 189
FIGURE 2.5-50



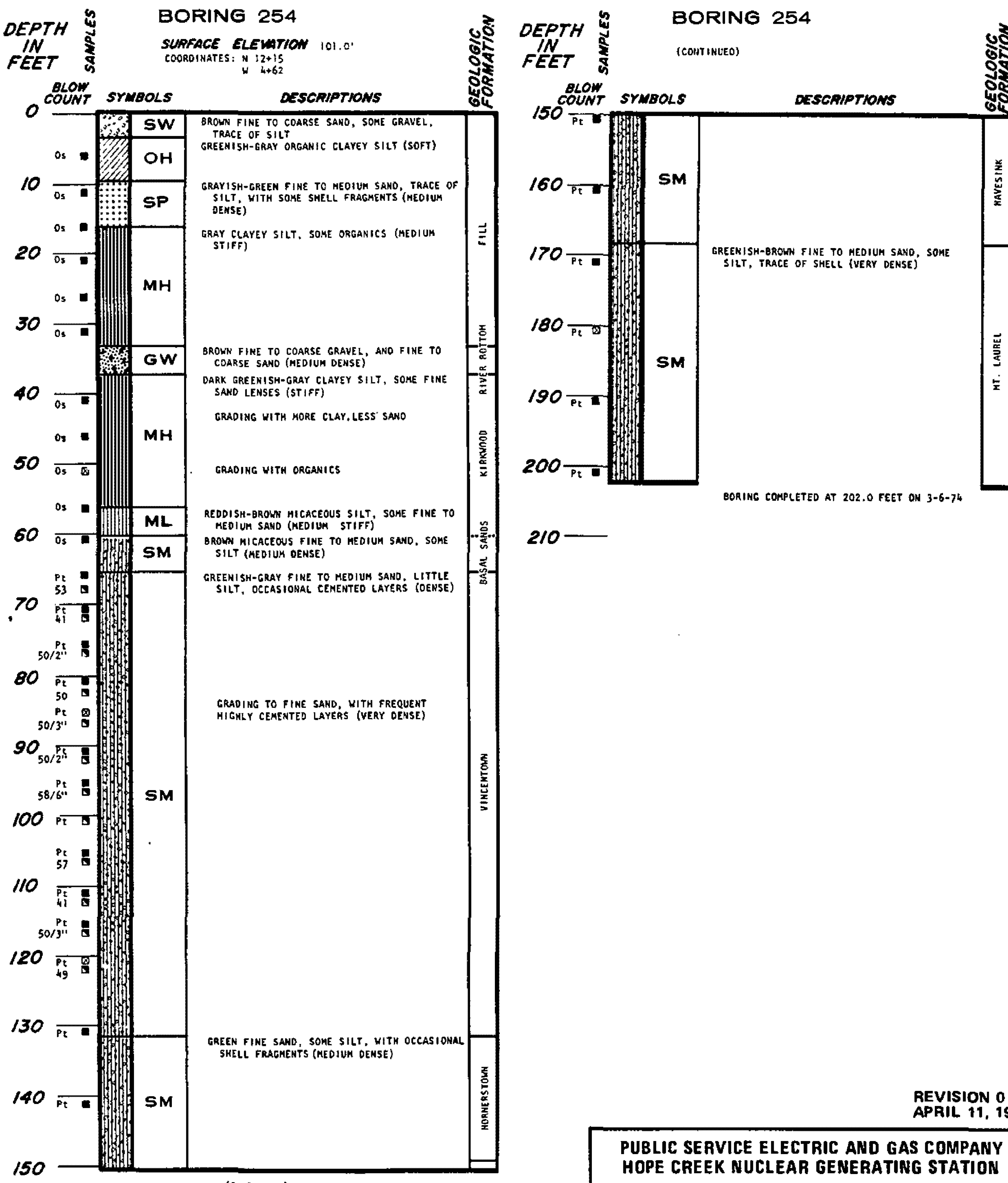
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 137 of 189
FIGURE 2.5-50



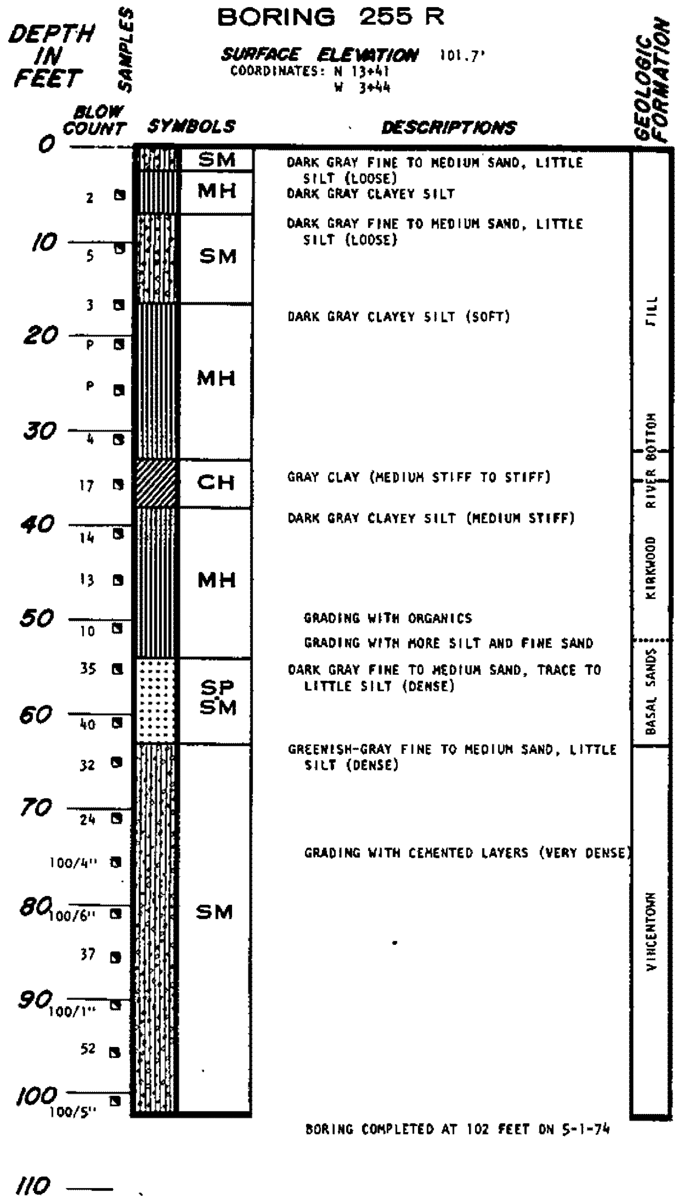
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 138 of 189
FIGURE 2.5-50



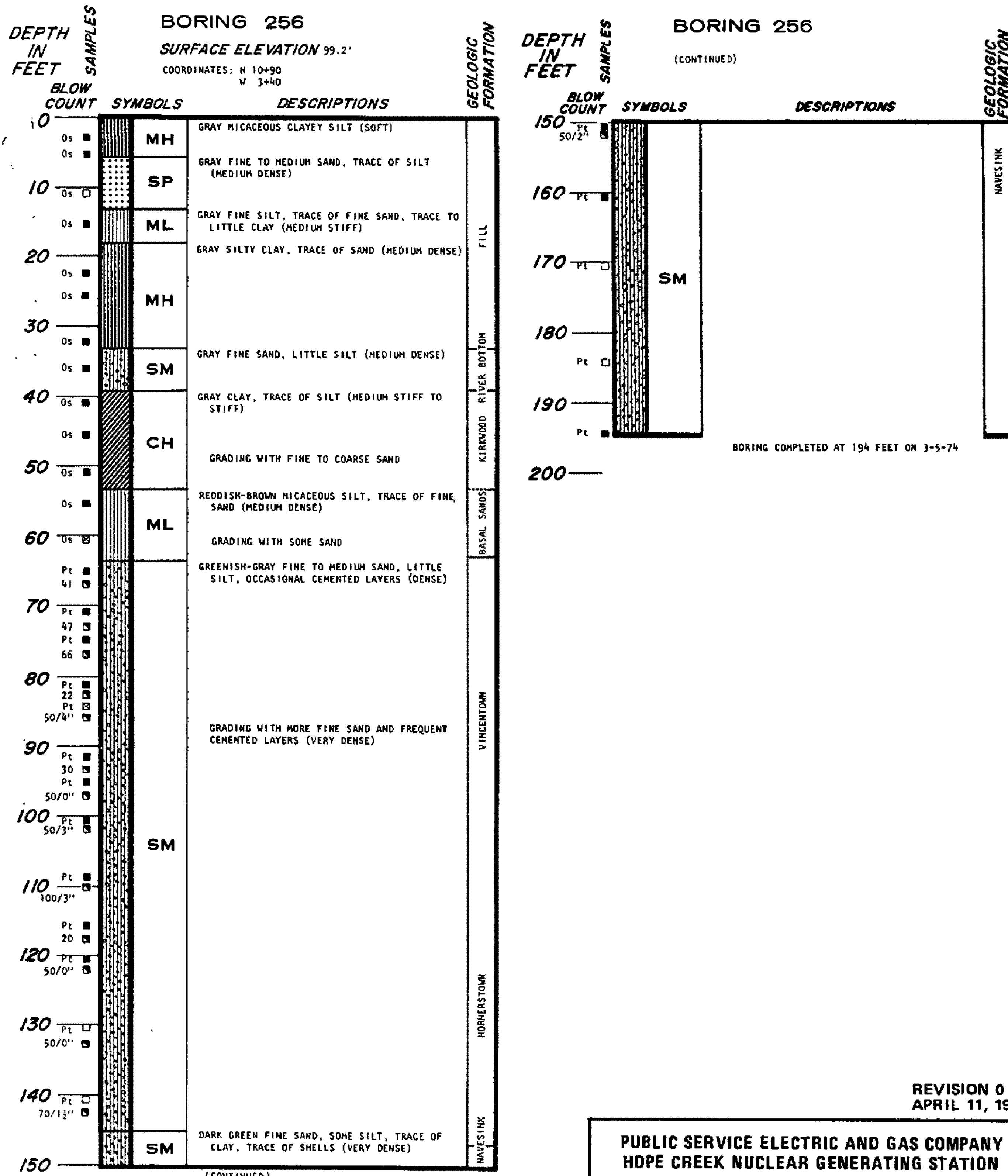
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 139 of 189
FIGURE 2.5-50



NOTE: THE STANDARD SPLIT SPOON SAMPLER WAS DRIVEN WITH A 140 LB. HAMMER AT A 24'' DROP.

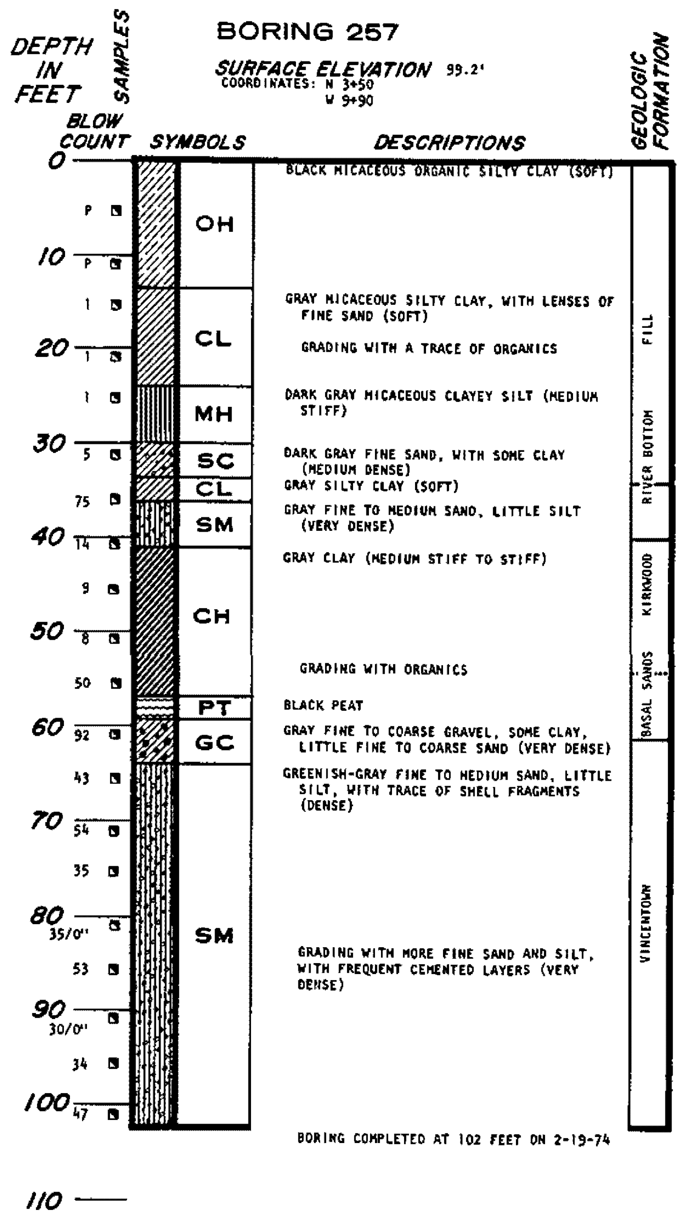
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 140 of 189
FIGURE 2.5-50



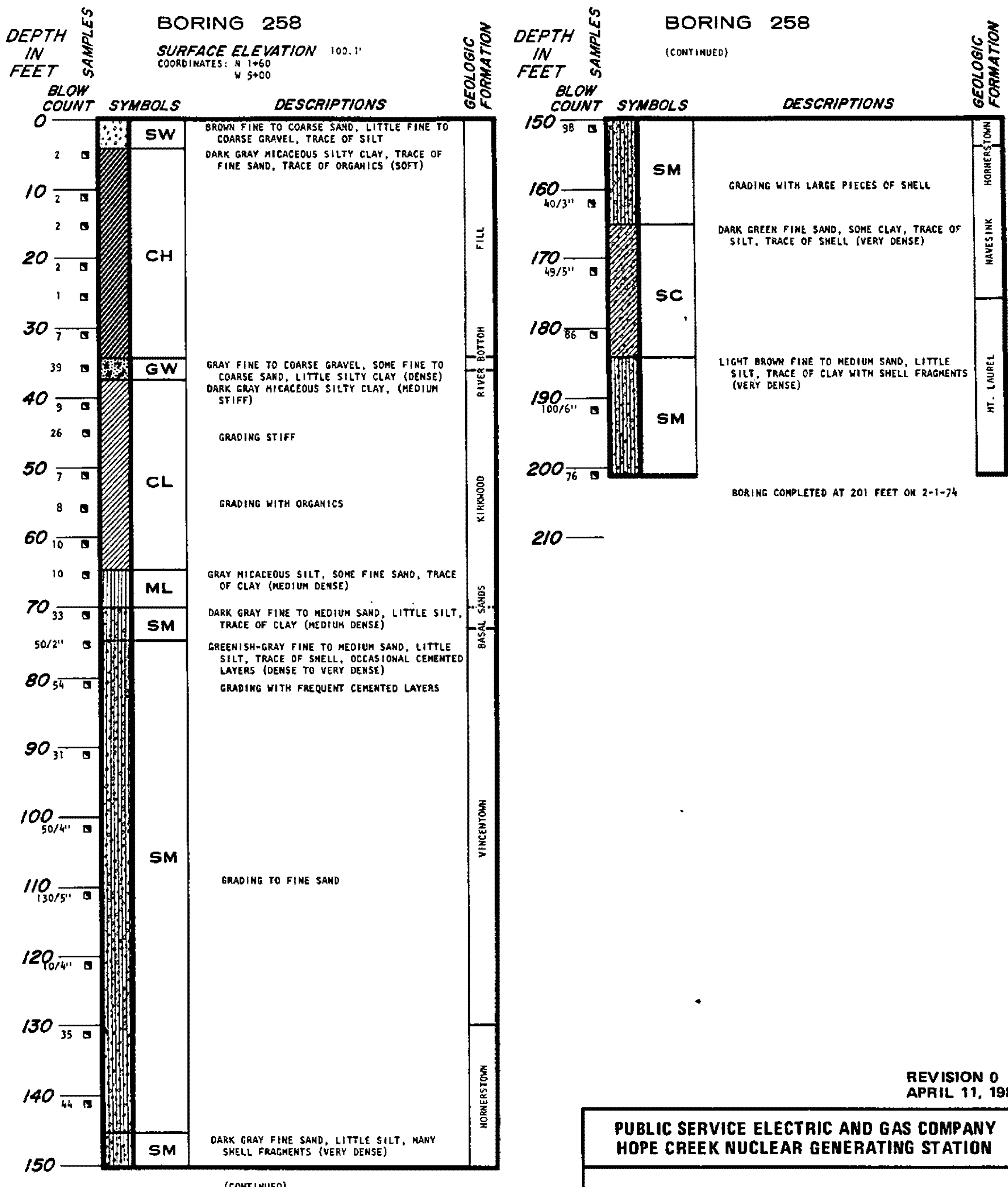
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 141 of 189
FIGURE 2.5-50



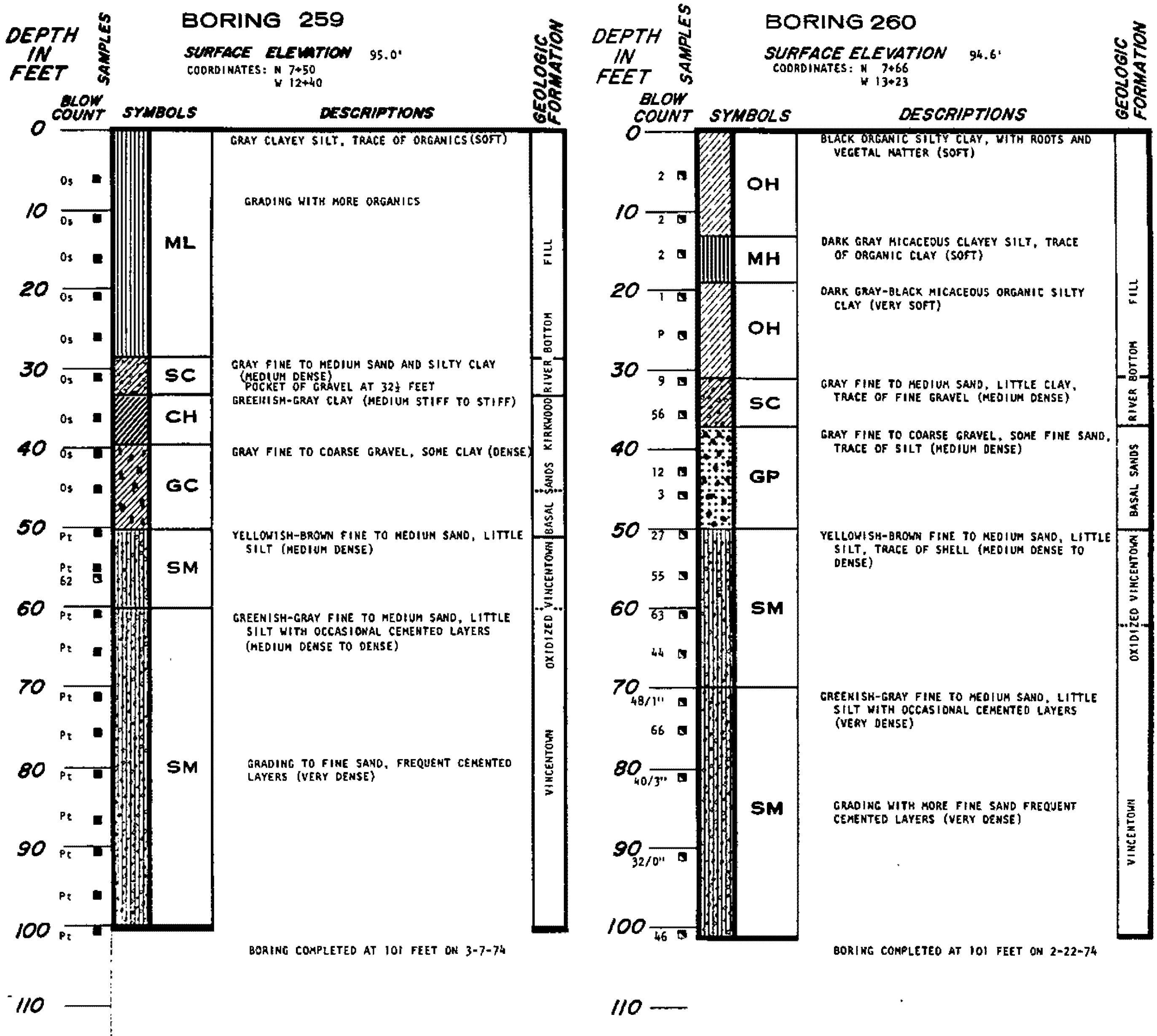
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 142 of 189
FIGURE 2.5-50



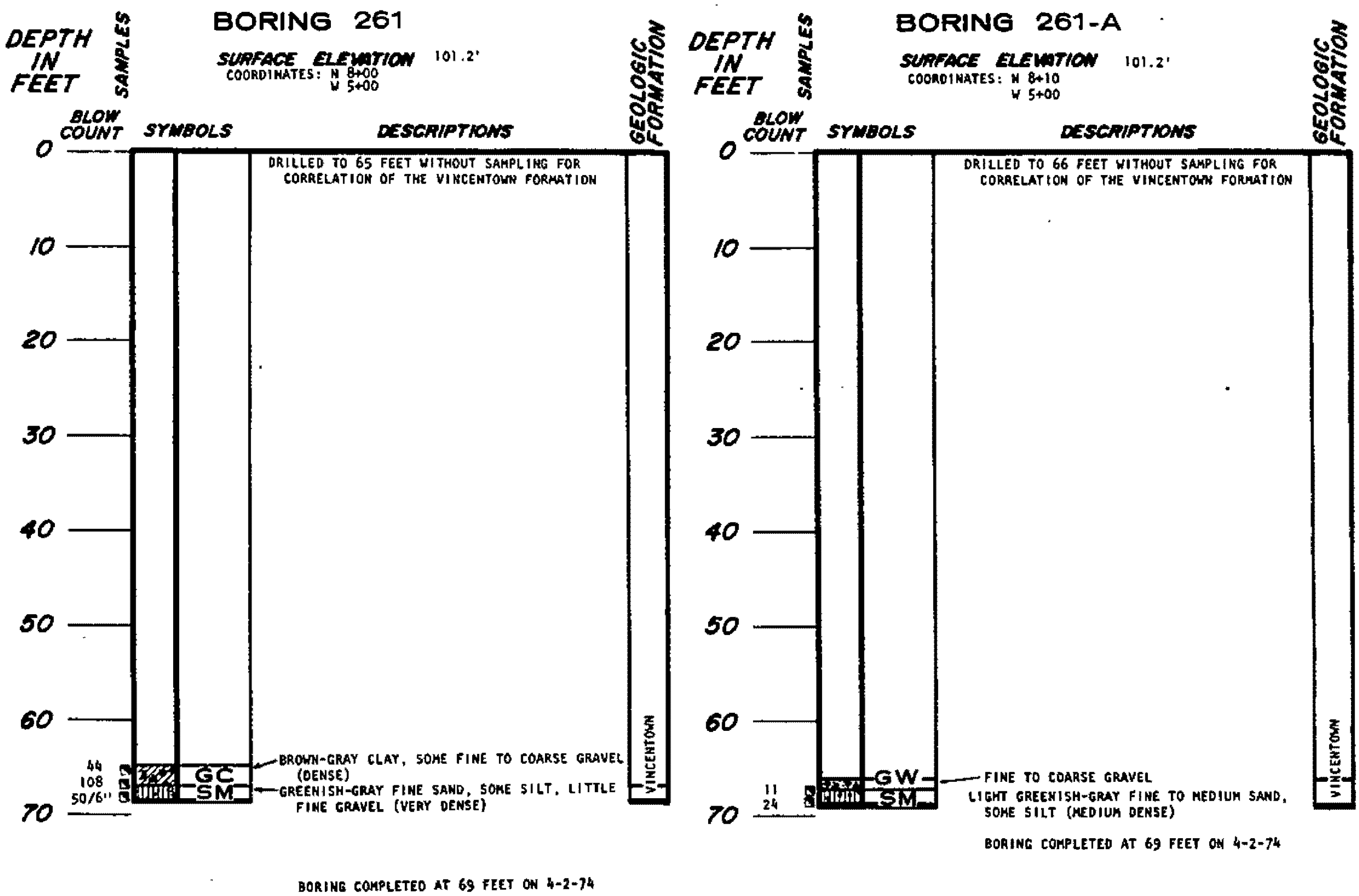
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 143 of 189
FIGURE 2.5-50



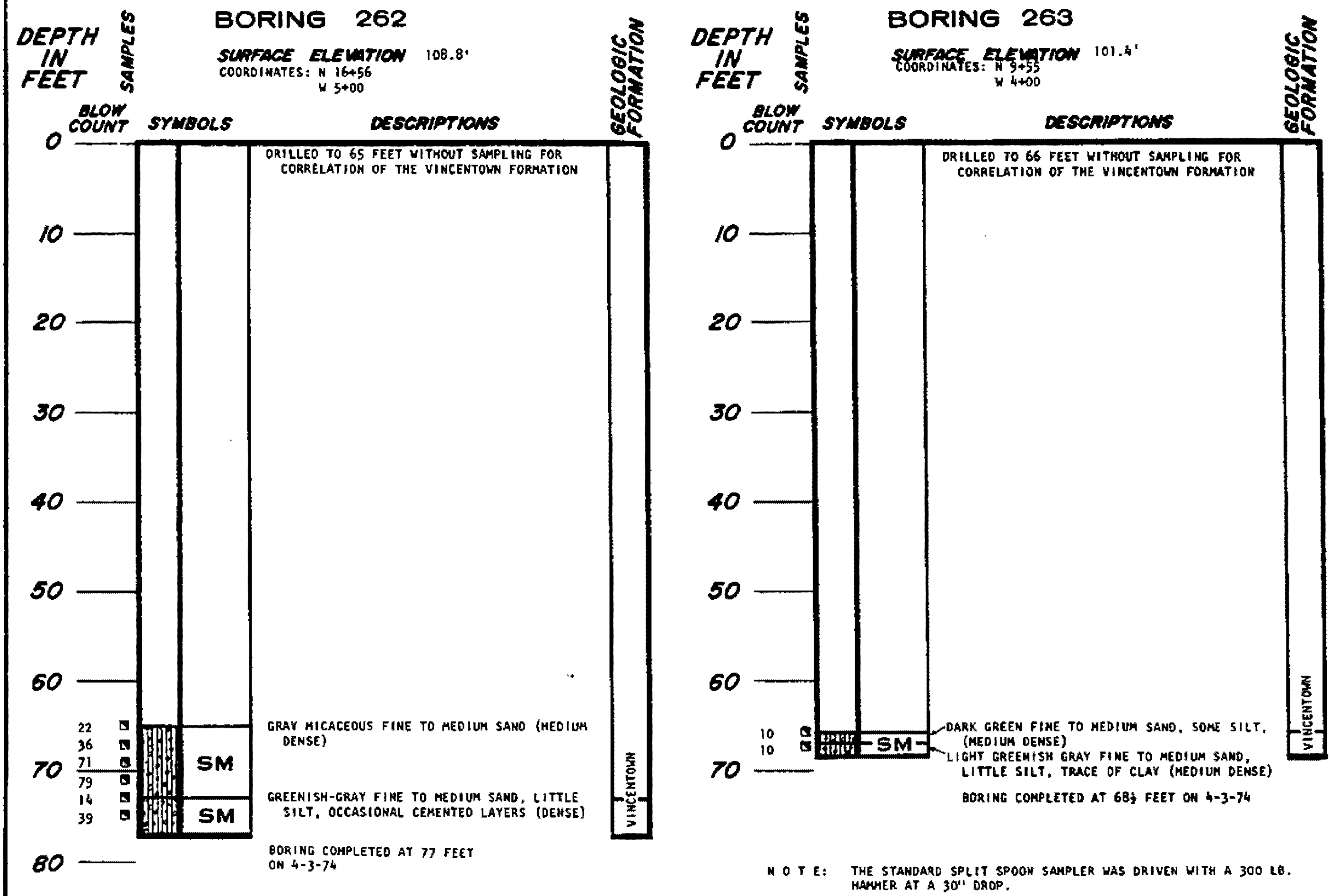
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

FIGURE 2.5-50



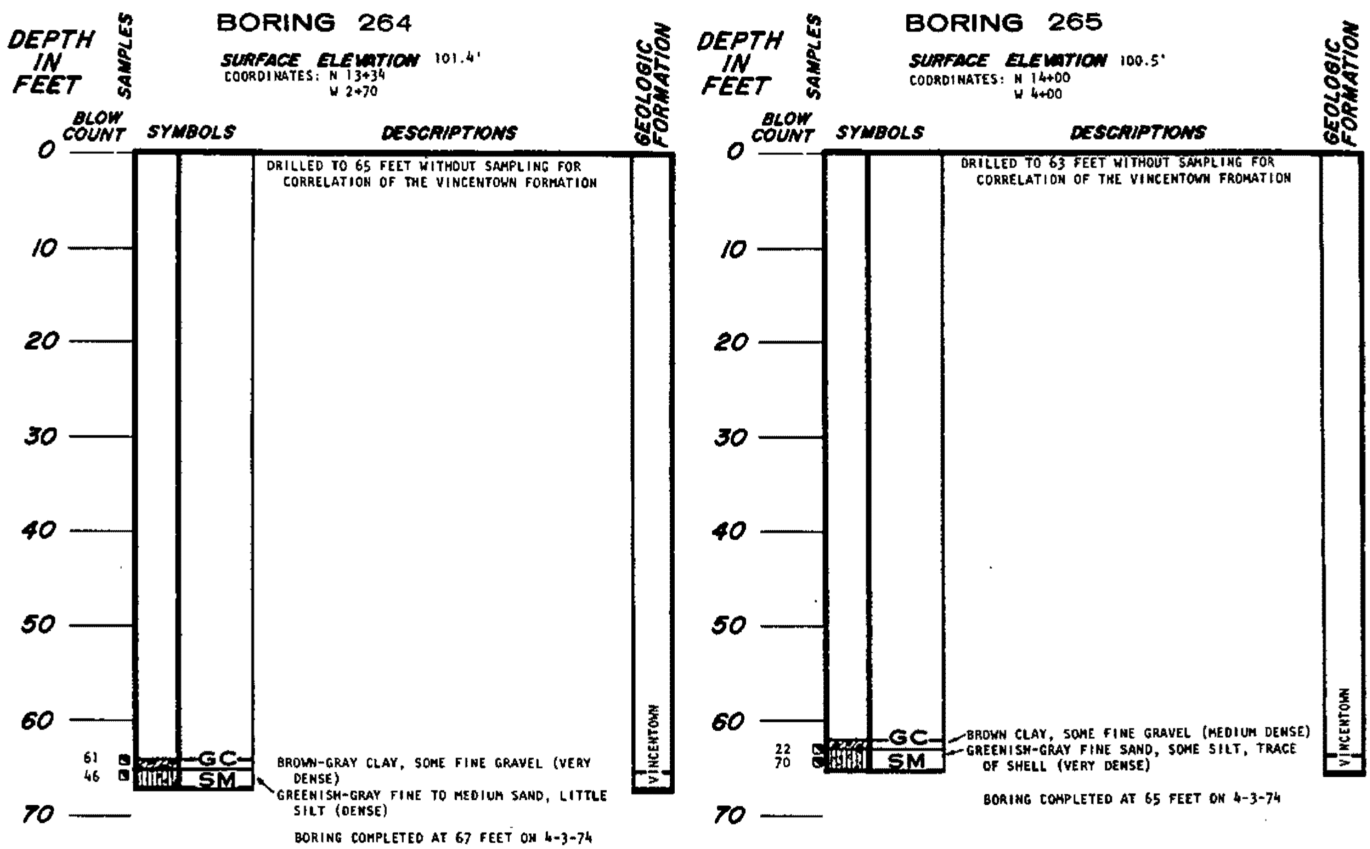
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 145 of 189
FIGURE 2.5-50



REVISION 0
APRIL 11, 1988

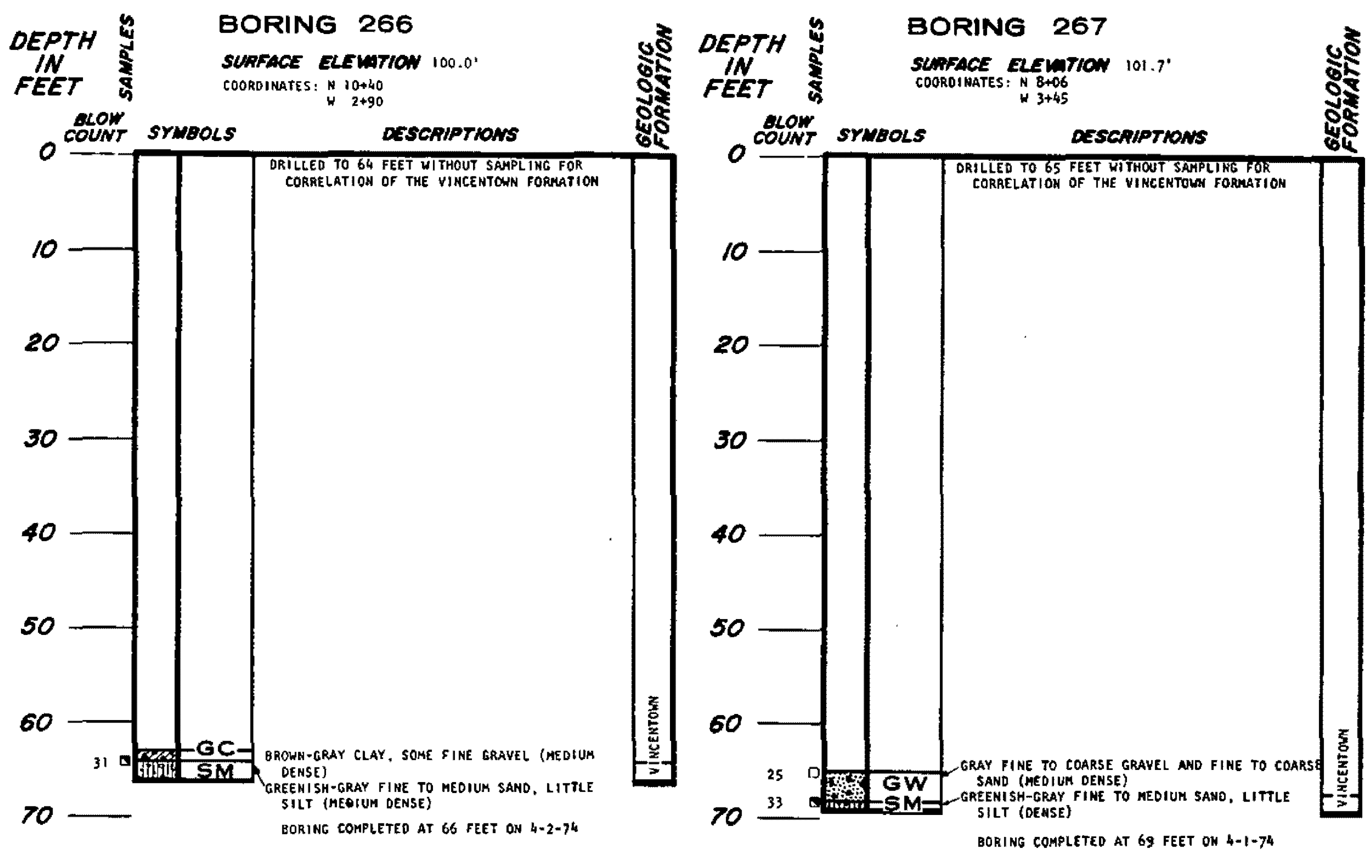
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 146 of 189

FIGURE 2.5-50



REVISION 0
APRIL 11, 1988

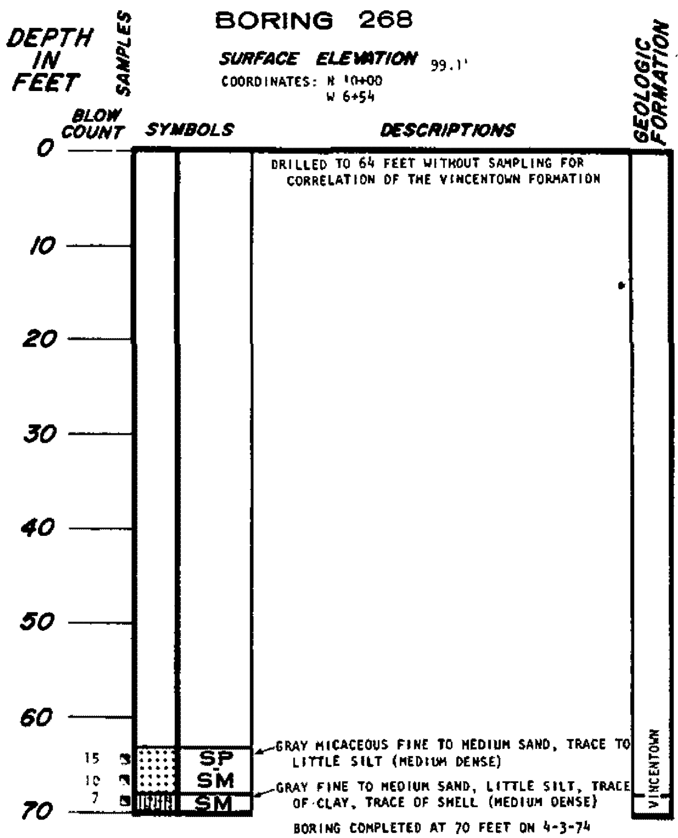
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

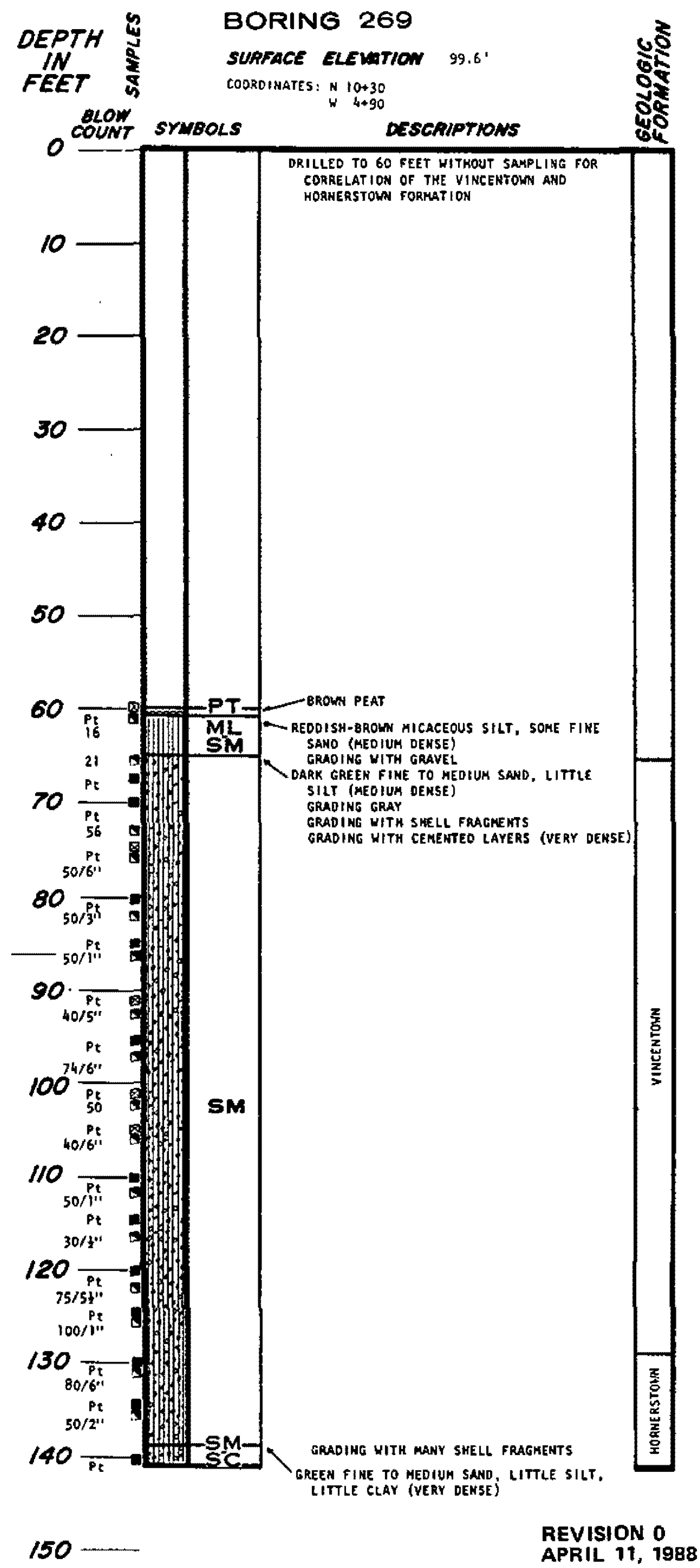
UPDATED FSAR

Sheet 147 of 189

FIGURE 2.5-50



NOTE: THE STANDARD SPLIT SPOON SAMPLER WAS DRIVEN WITH A 300 LB. HAMMER AT A 30" DROP.



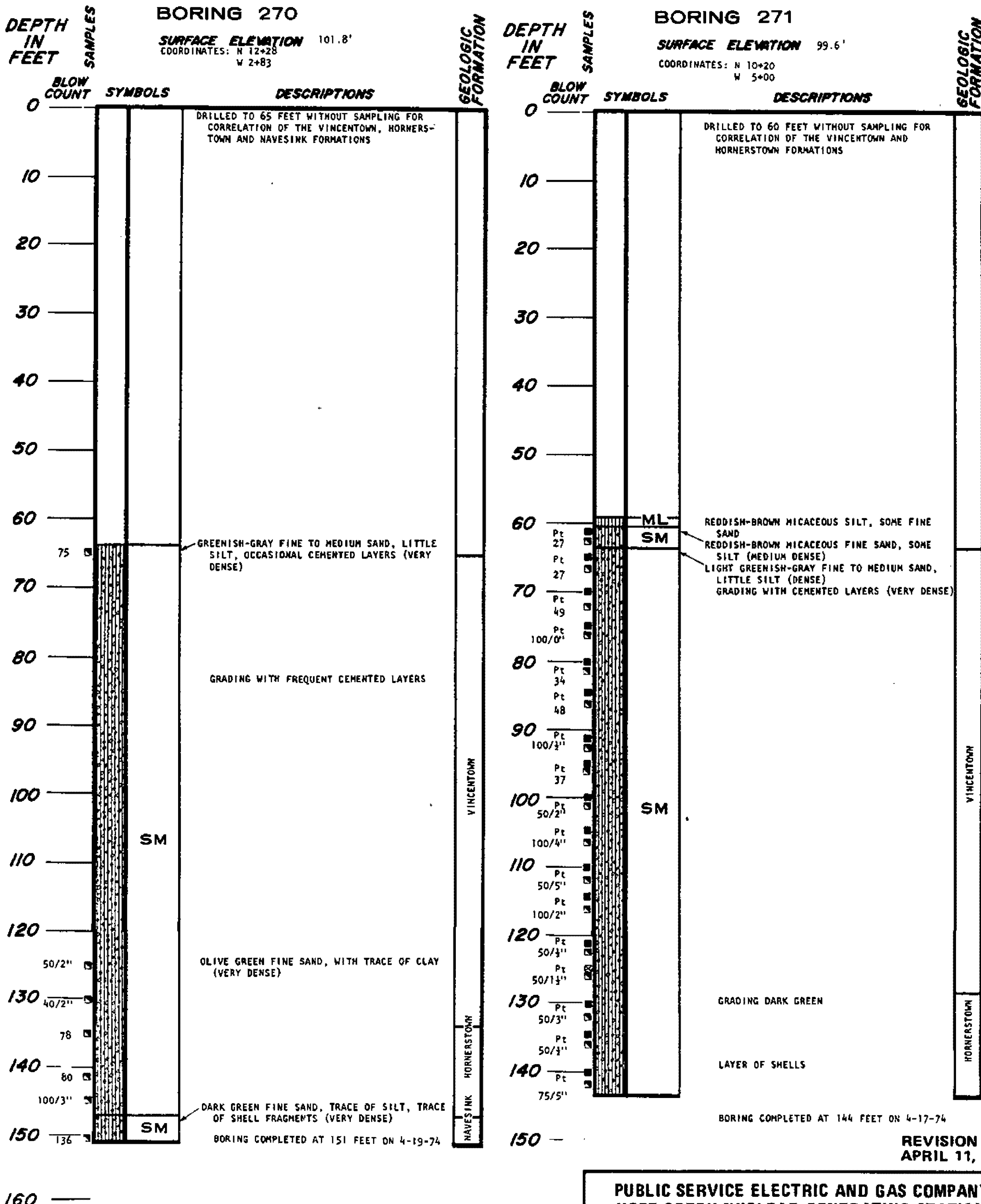
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 148 of 189
FIGURE 2.5-50



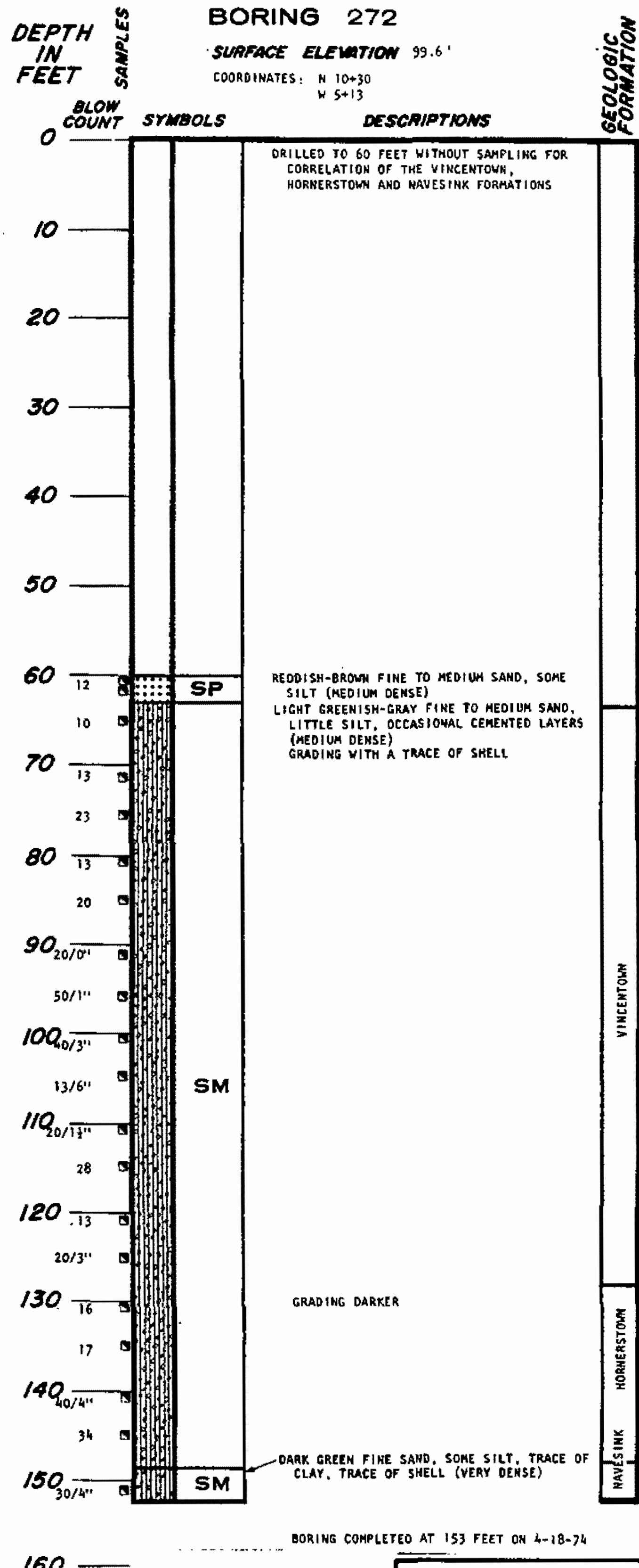
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 149 of 189
FIGURE 2.5-50



NOTE: THE STANDARD SPLIT SPOON SAMPLER WAS DRIVEN WITH A 300 LB. HAMMER AT A 24" DROP.

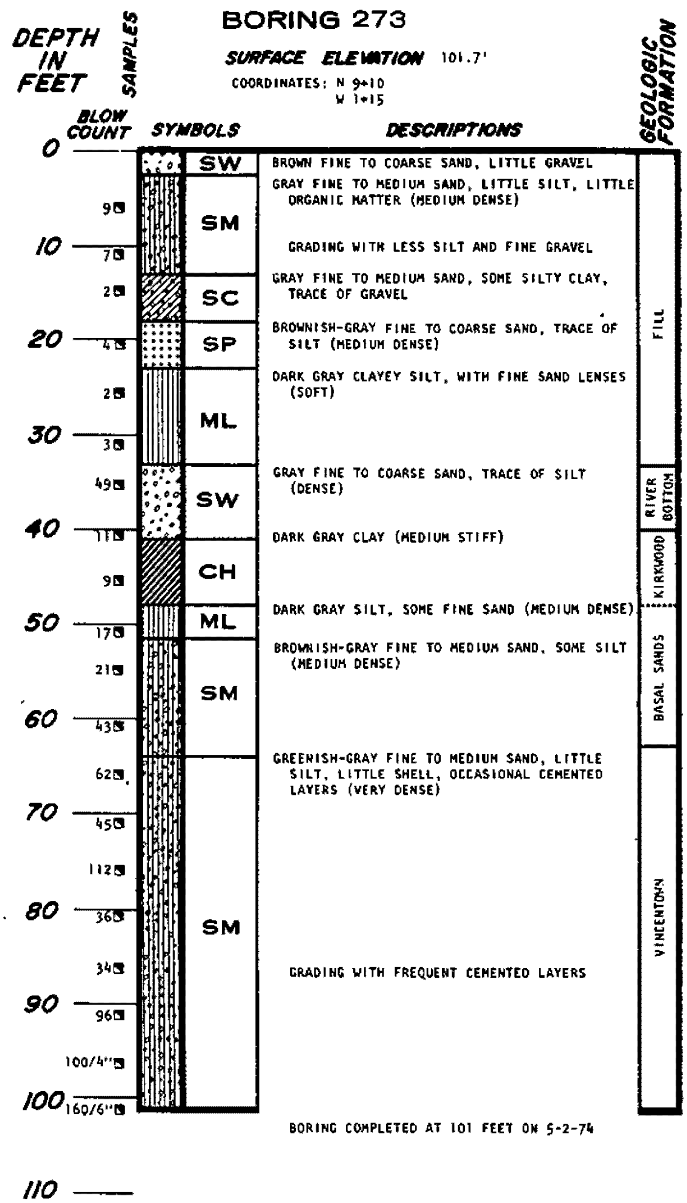
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 150 of 189
FIGURE 2.5-50



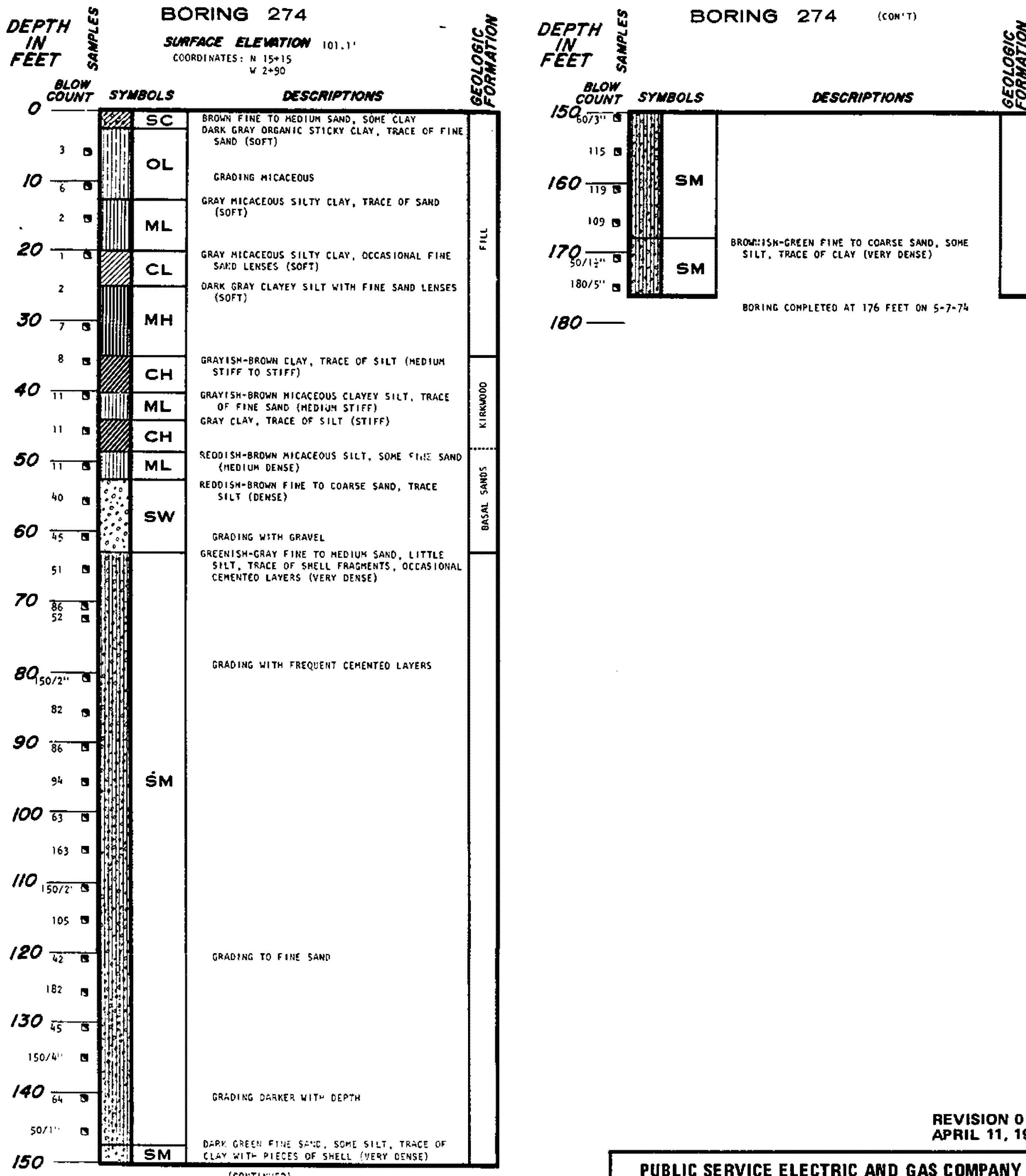
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 151 of 189
FIGURE 2.5-50



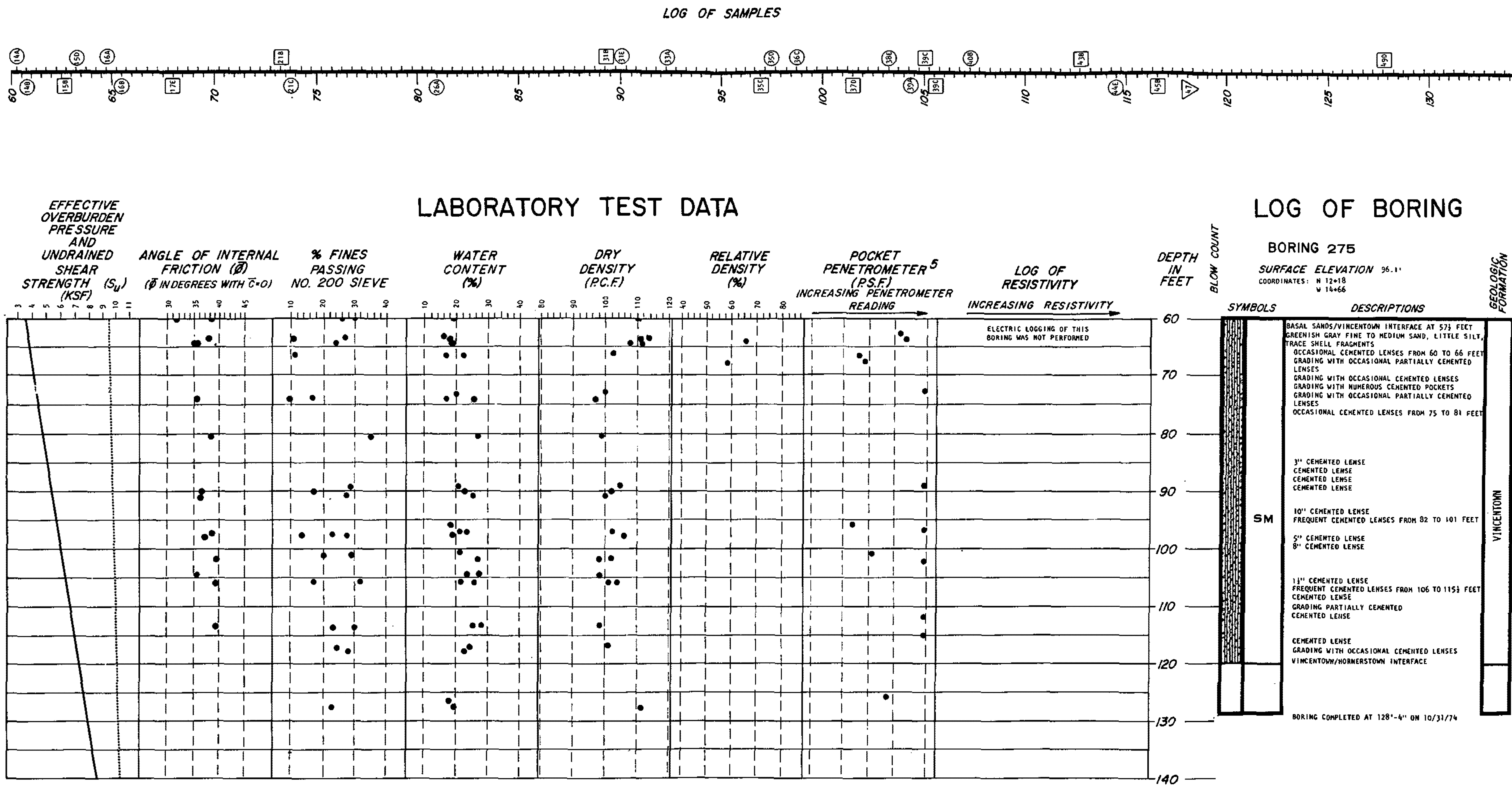
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 152 of 189
FIGURE 2.5-50



NOTES:

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2. ELEVATIONS REFER TO THE PUBLIC SERVICE DATUM.
3. THE DISCUSSION IN THE TEXT OF THE REPORT IS NECESSARY FOR A PROPER UNDERSTANDING OF THE NATURE AND ENGINEERING PROPERTIES OF THE SUBSURFACE MATERIALS.
4. THE SAMPLING PROCEDURE USED DID NOT ALLOW THE OBSERVANCE OF ALL CEMENTED LENSES. ADDITIONAL CEMENTED LENSES THAN THOSE INDICATED CAN BE EXPECTED.
5. POCKET PENETROMETER READINGS WERE USED TO QUALITATIVELY EVALUATE THE VARIATION IN THE DEGREE OF CEMENTATION. NO ATTEMPT WAS MADE NOR SHOULD BE MADE TO USE THESE READINGS TO QUANTITATIVELY EVALUATE THE SHEAR STRENGTH OF THE SAMPLES.
6. INDEX PROPERTY TESTS WERE MADE IN CONJUNCTION WITH ALL CONSOLIDATED ISOTROPIC UNDRAINED (CIU) TRIAXIAL TESTS AND STRESS-CONTROLLED CYCLIC TRIAXIAL TESTS.
7. RELATIVE DENSITIES WERE DETERMINED ACCORDING TO THE AKROYD METHOD DESCRIBED IN REF. 1 IN THE TEXT.

8) THE EFFECTIVE OVERBURDEN PRESSURE WAS COMPUTED BASED ON THE FOLLOWING ASSUMPTIONS:
   A) THE GROUND-WATER TABLE WAS CONSIDERED TO BE AT THE GROUND SURFACE.
   B) THE GROUND SURFACE WAS CONSIDERED TO BE AT AN ELEVATION +100 FEET.
   C) AN AVERAGE SATURATED DENSITY OF 123 PCF AND A BUOYANT UNIT WEIGHT OF 60.6 PCF WAS USED

9) THE UNDRAINED SHEAR STRENGTH WAS COMPUTED BY THE USE OF FIGURE 4.34, NORMALIZED UNDRAINED SHEAR STRENGTH VS CONSOLIDATION PRESSURE FOR VINCENTOWN SANDS.

10) THE PRESSURE DISTRIBUTION AND UNDRAINED SHEAR STRENGTHS SHOWN REPRESENT A GENERALIZED CONDITION OF THE VINCENTOWN FORMATION BASED UPON INTERPRETATION OF PRESENTLY AVAILABLE DATA. SOME VARIATION FROM THESE PLOTS MUST BE EXPECTED.

REFERENCE FOR BORINGS 275-280A, B2-B3A:
PSE&G, JULY 1975, SUPPLEMENTARY FOUNDATION STUDIES, PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

KEY:

(N) INDICATES A CONSOLIDATED ISOTROPIC UNDRAINED (CIU) TRIAXIAL TEST WHERE N REPRESENTS THE SAMPLE NUMBER.

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▽N A SUFFICIENT LENGTH OF SAMPLE FOR TESTING COULD NOT BE OBTAINED BECAUSE OF ALTERNATING ZONES OF HIGHLY CEMENTED AND LESS CEMENTED MATERIAL.

—1— AVERAGE PRESSURE DISTRIBUTION EXCLUDING THE BUILDING LOAD

---2--- AVERAGE UNDRAINED SHEAR STRENGTH DISTRIBUTION EXCLUDING THE BUILDING LOAD

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

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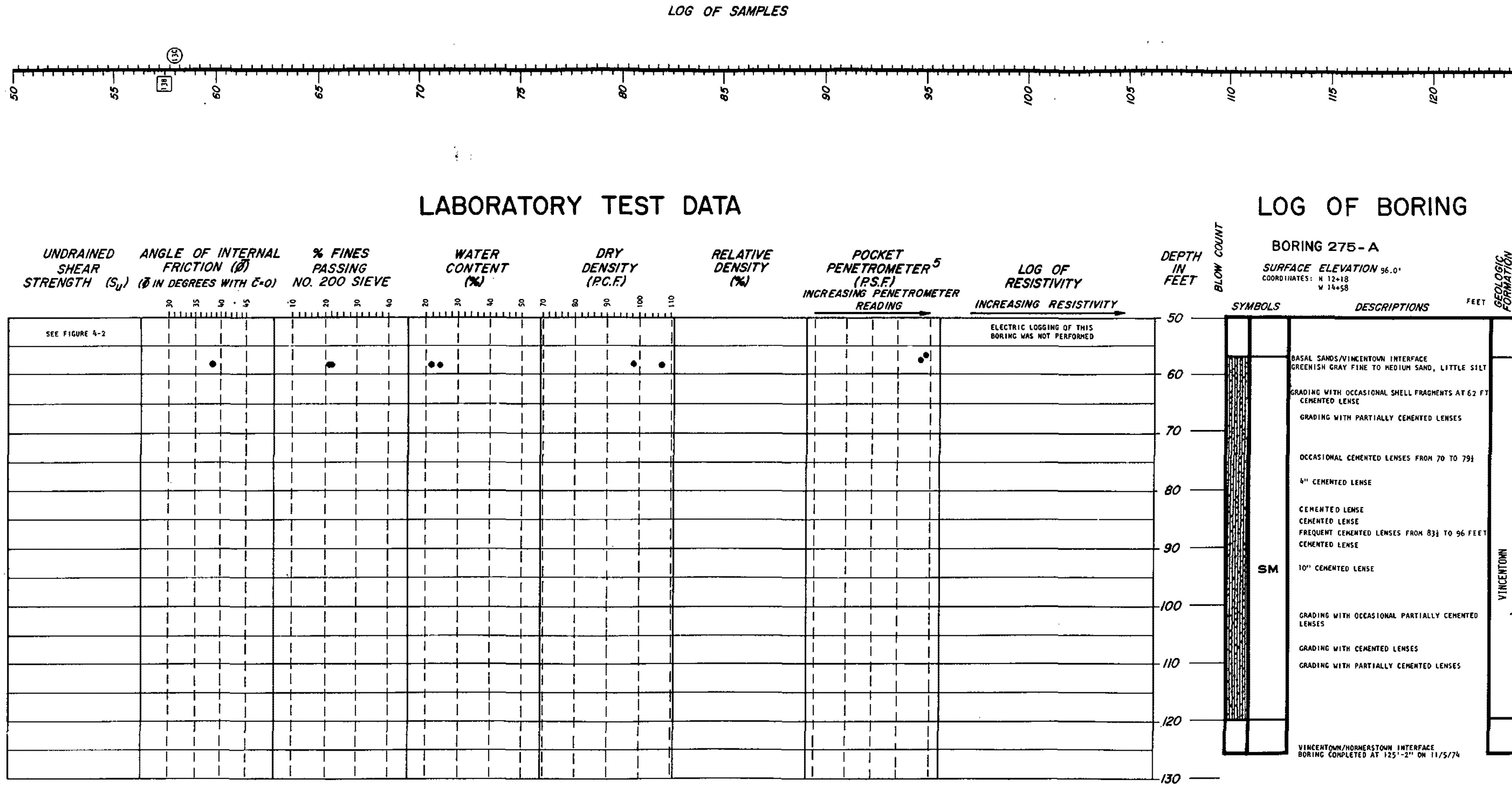
FIGURE 2.5-50



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REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

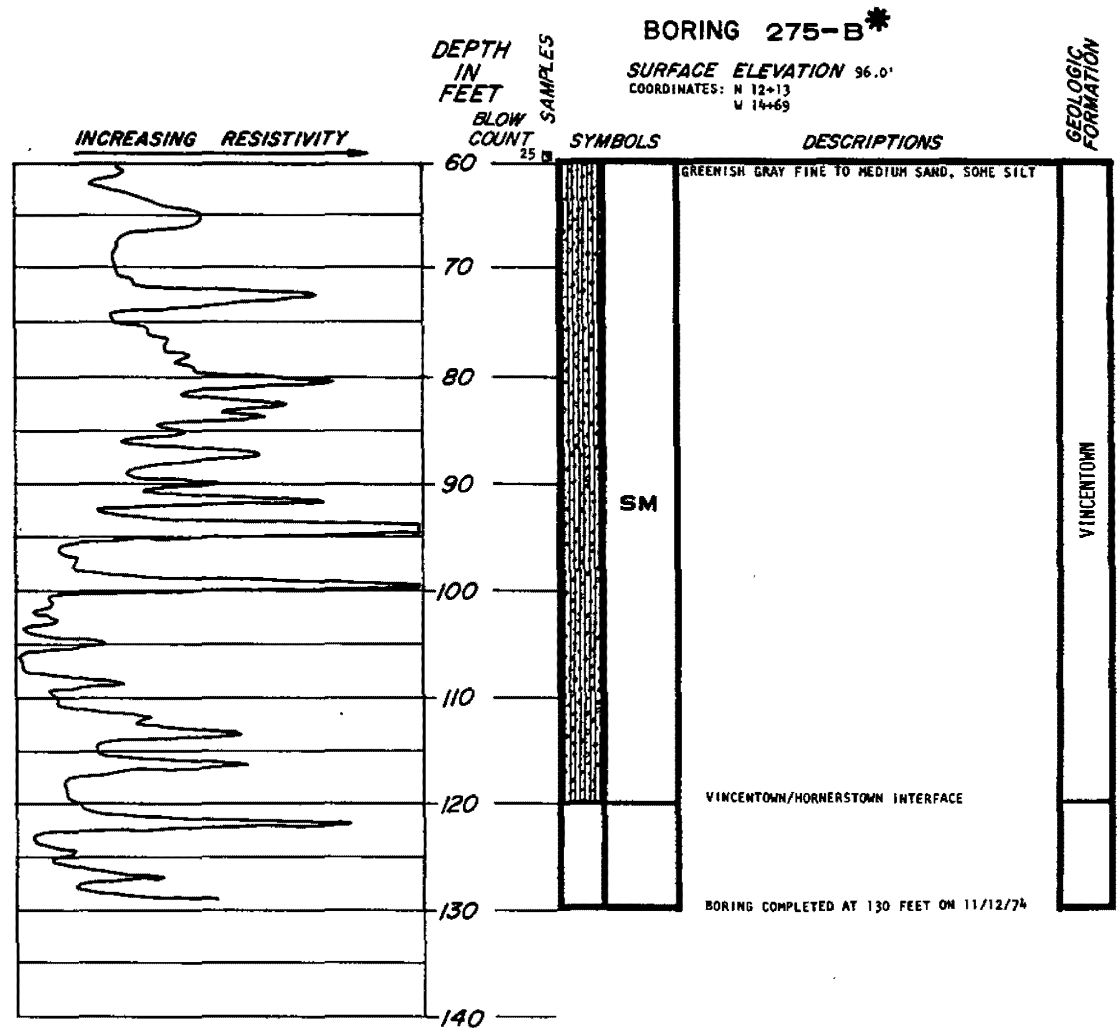
LOG BORINGS

UPDATED FSAR

Sheet 154 of 189
FIGURE 2.5-50

LOG OF RESISTIVITY

LOG OF BORING



*NO TESTING WAS PERFORMED ON SAMPLES FROM THIS BORING BECAUSE SUFFICIENT SAMPLES INDICATING THE LEAST DEGREE OF CEMENTATION WERE OBTAINED IN THE IMMEDIATELY ADJACENT BORINGS. (B-275 AND B-275A)

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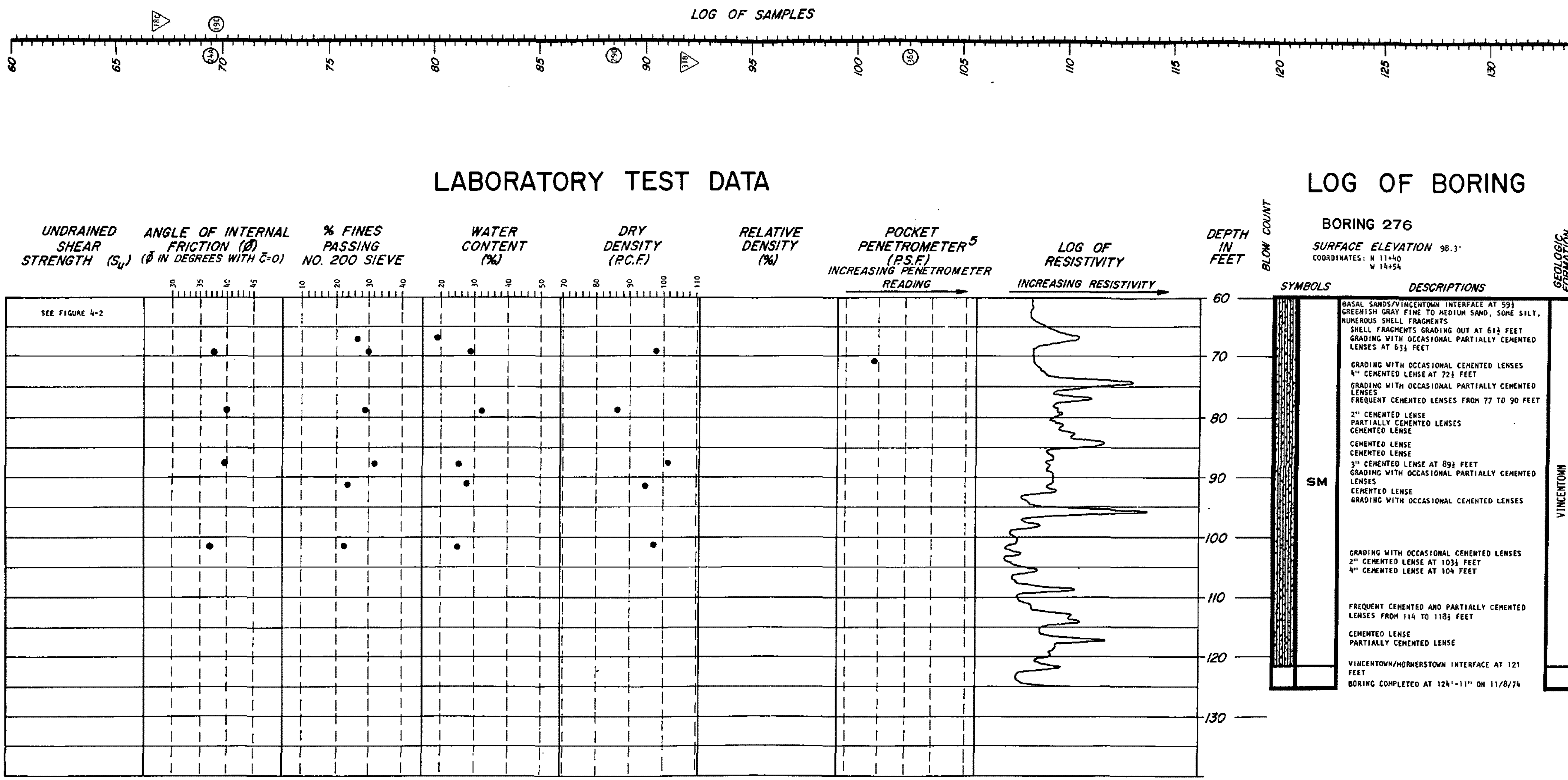
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

FIGURE 2.5-50



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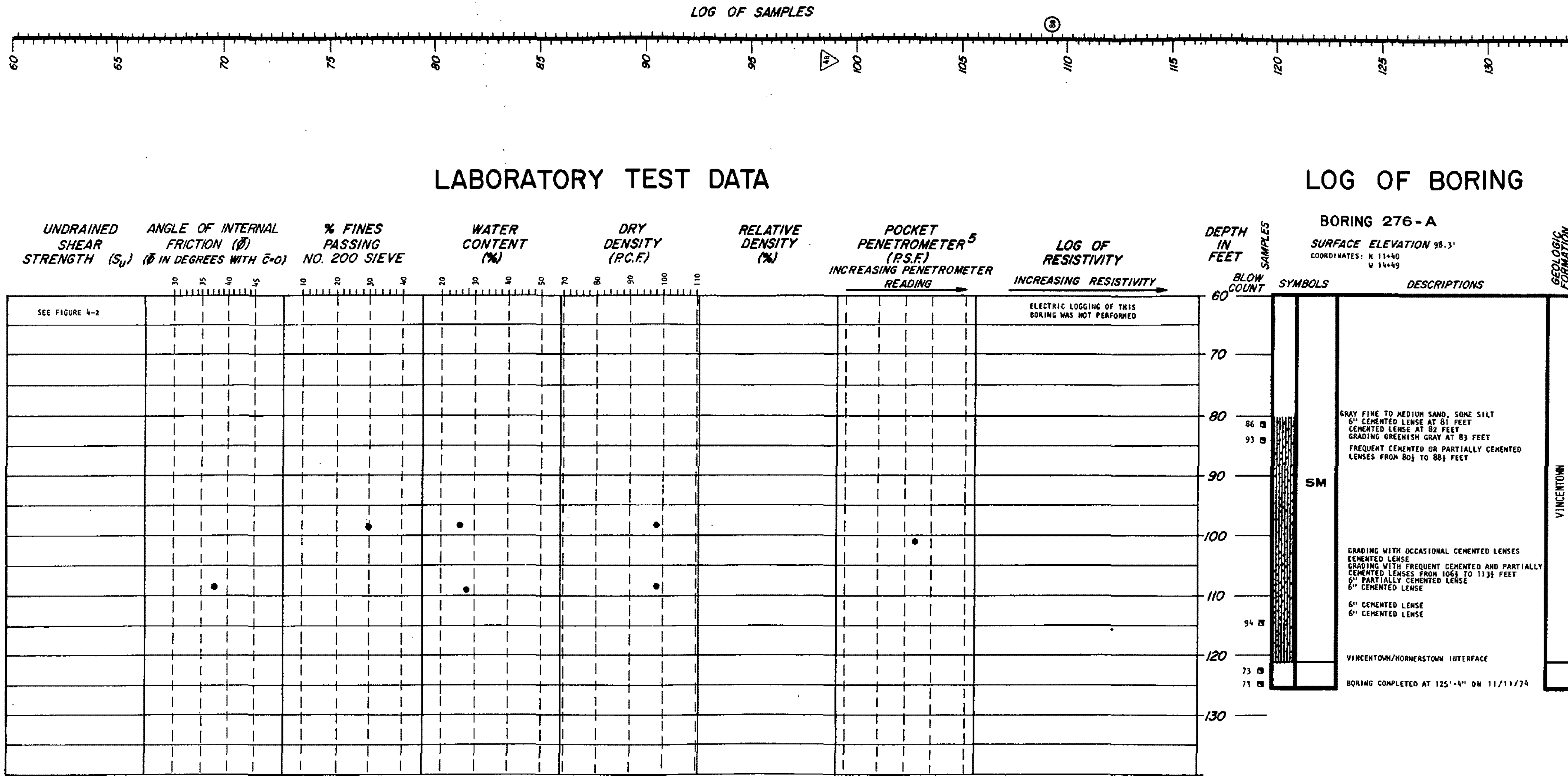
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 156 of 189
FIGURE 2.5-50



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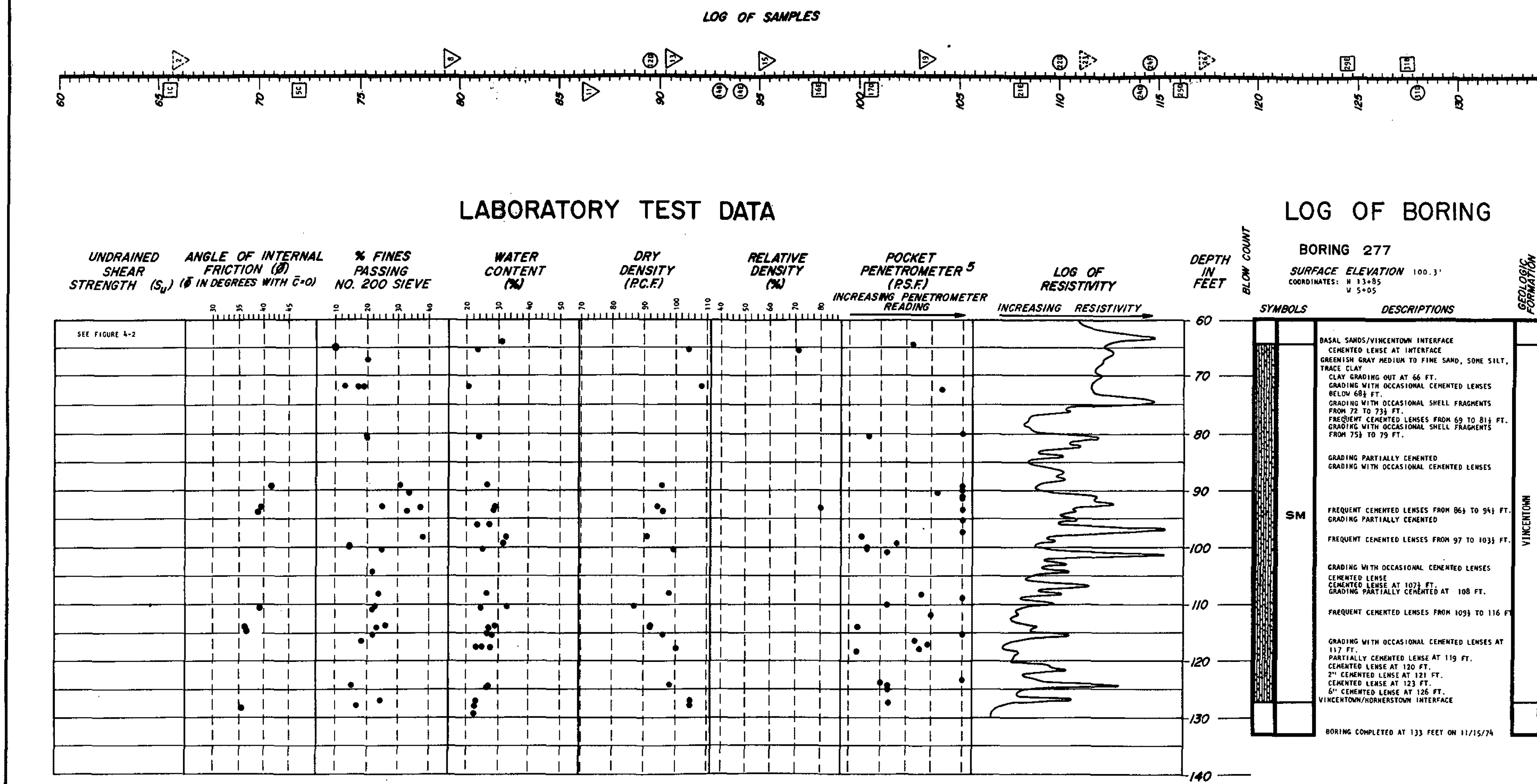
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 157 of 189
FIGURE 2.5-50



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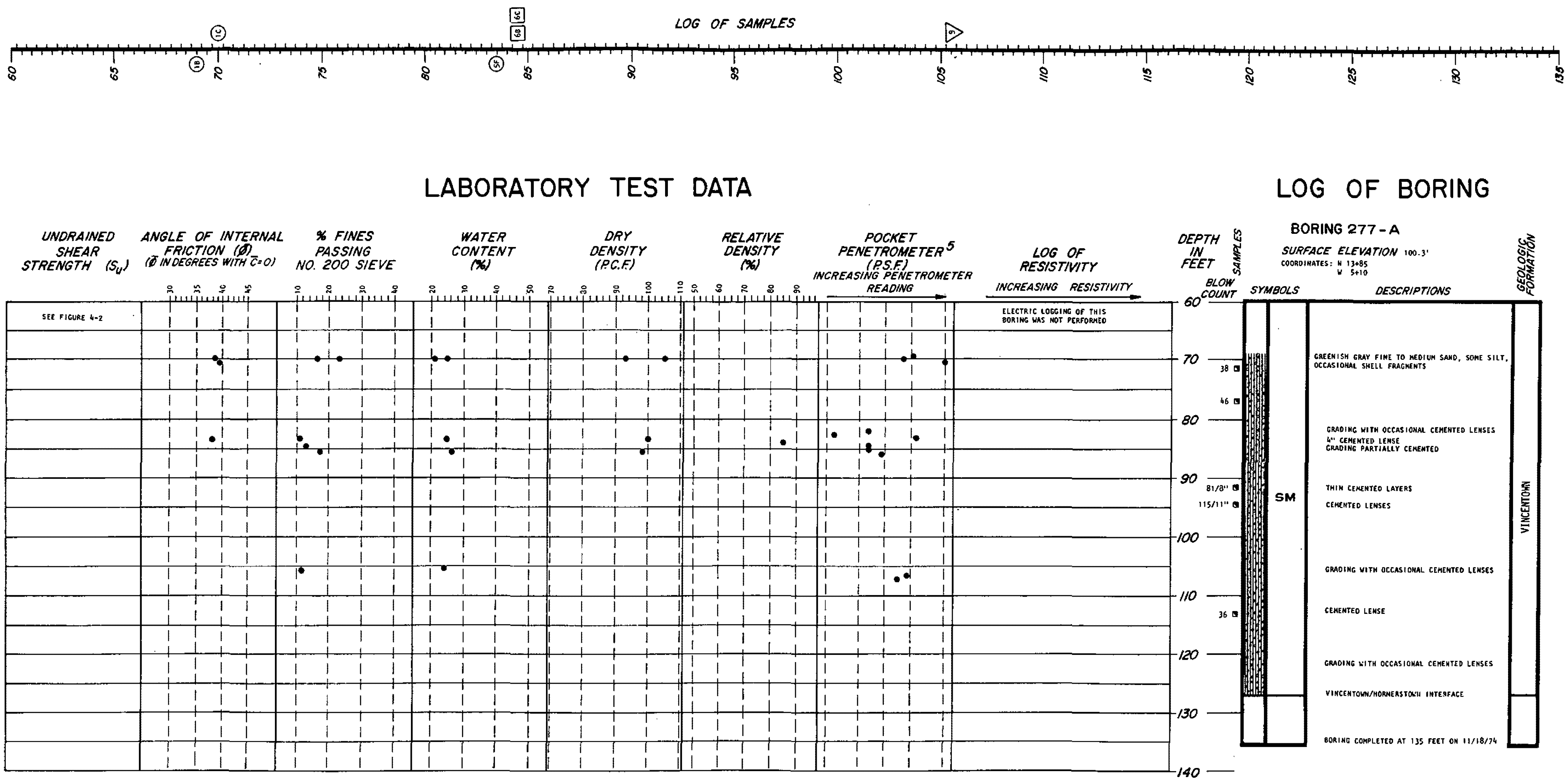
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 158 of 189
FIGURE 2.5-50



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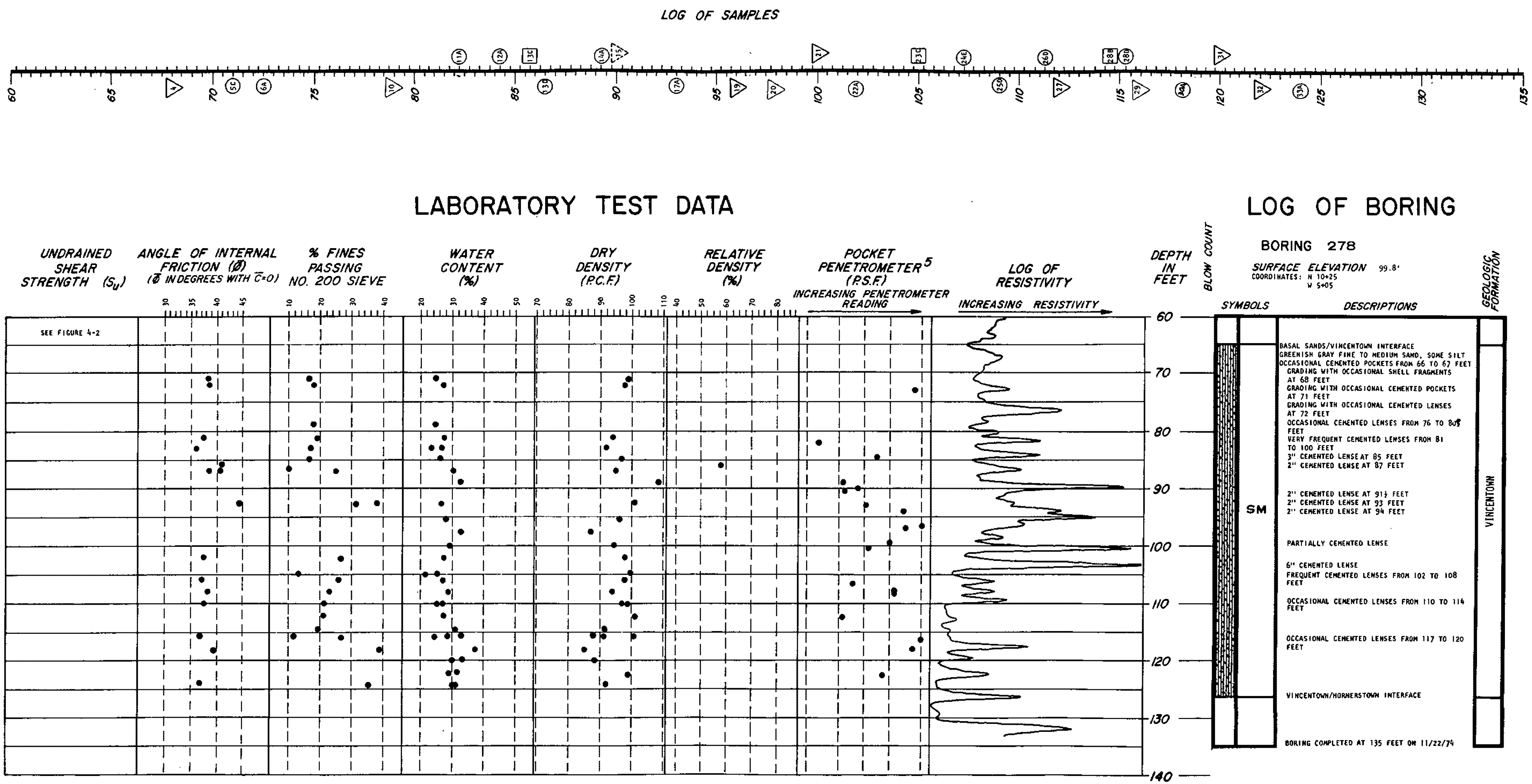
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 159 of 189
FIGURE 2.5-50



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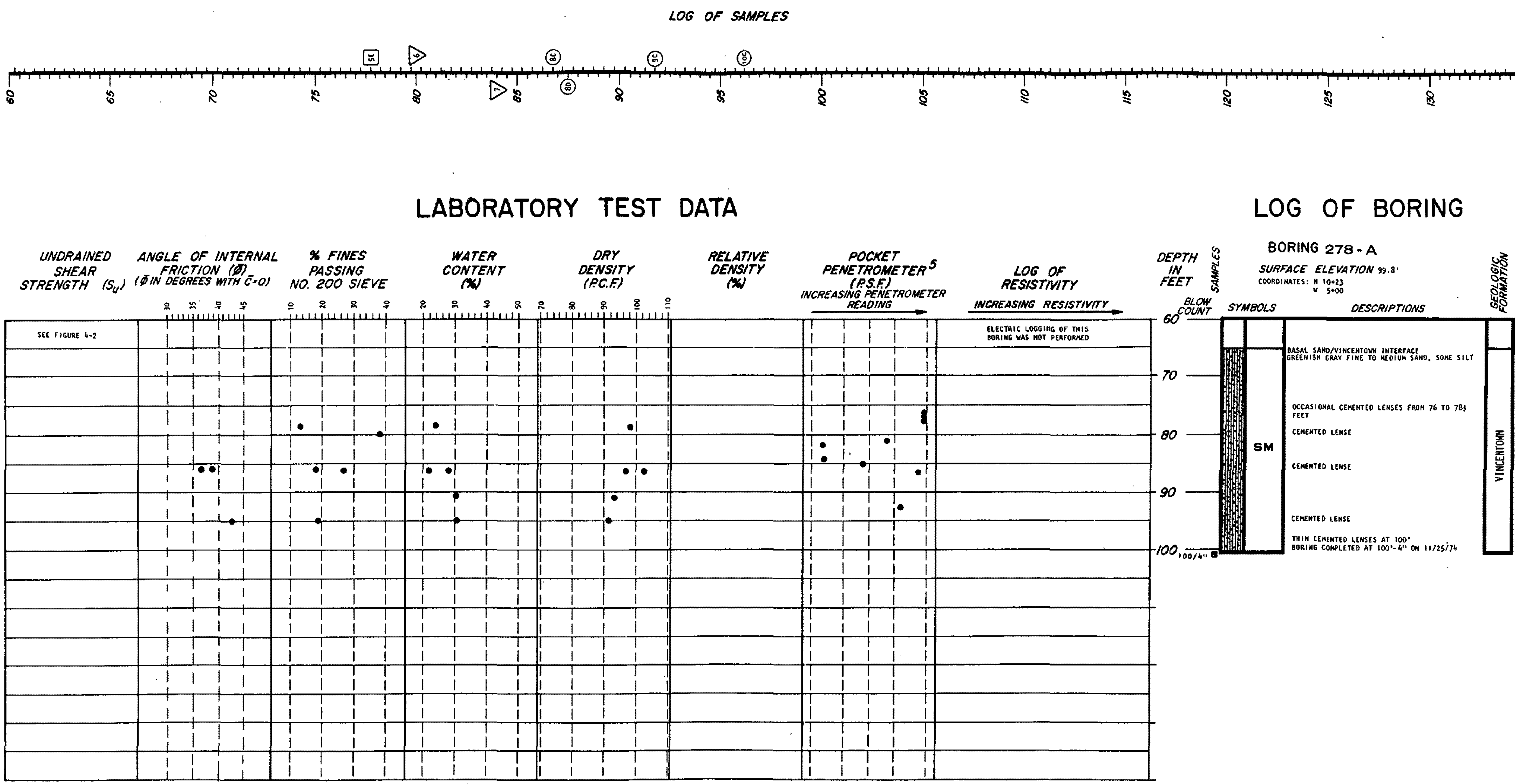
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 160 of 189
FIGURE 2.5-50



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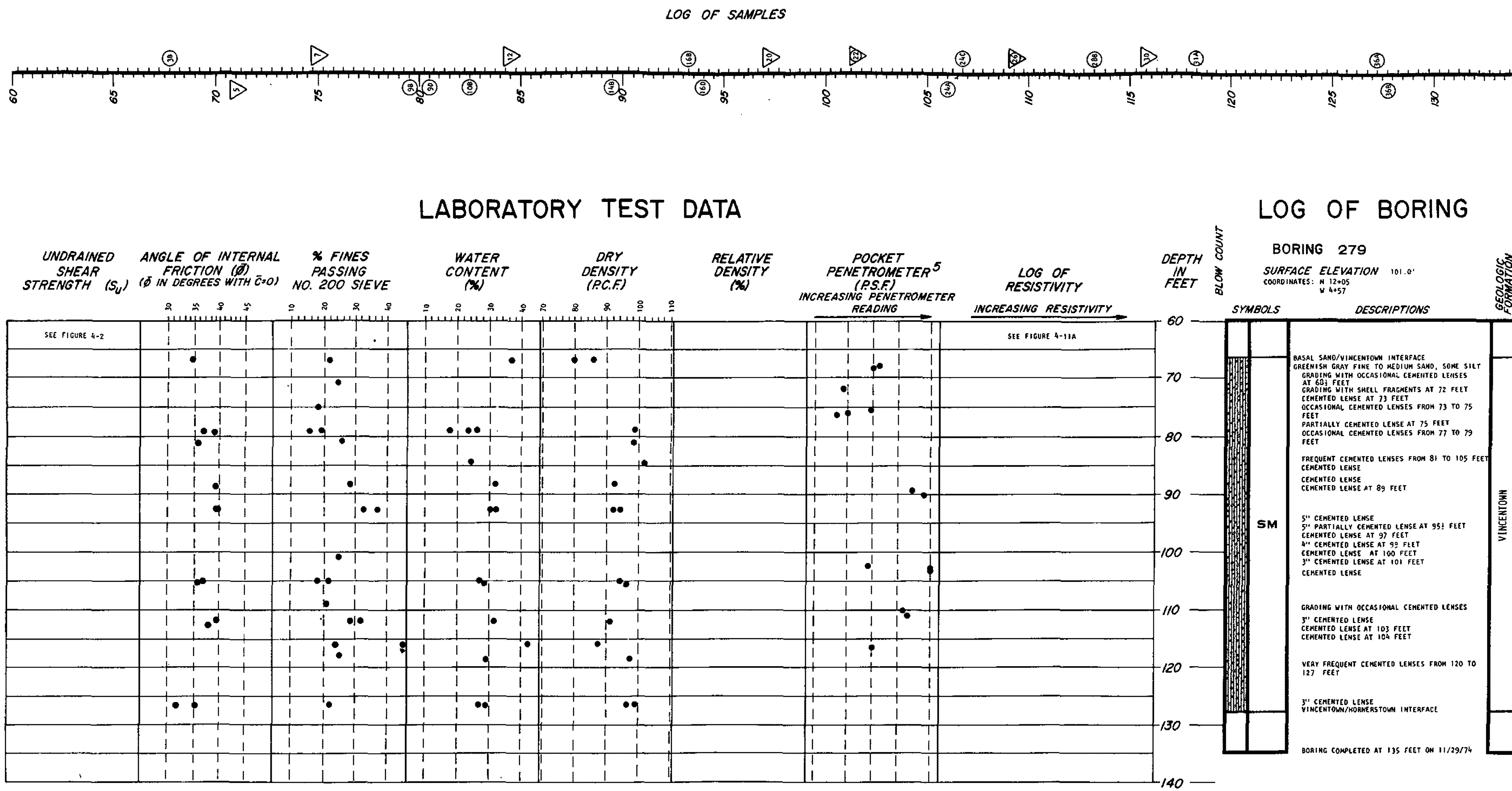
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 161 of 189
FIGURE 2.5-50



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REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

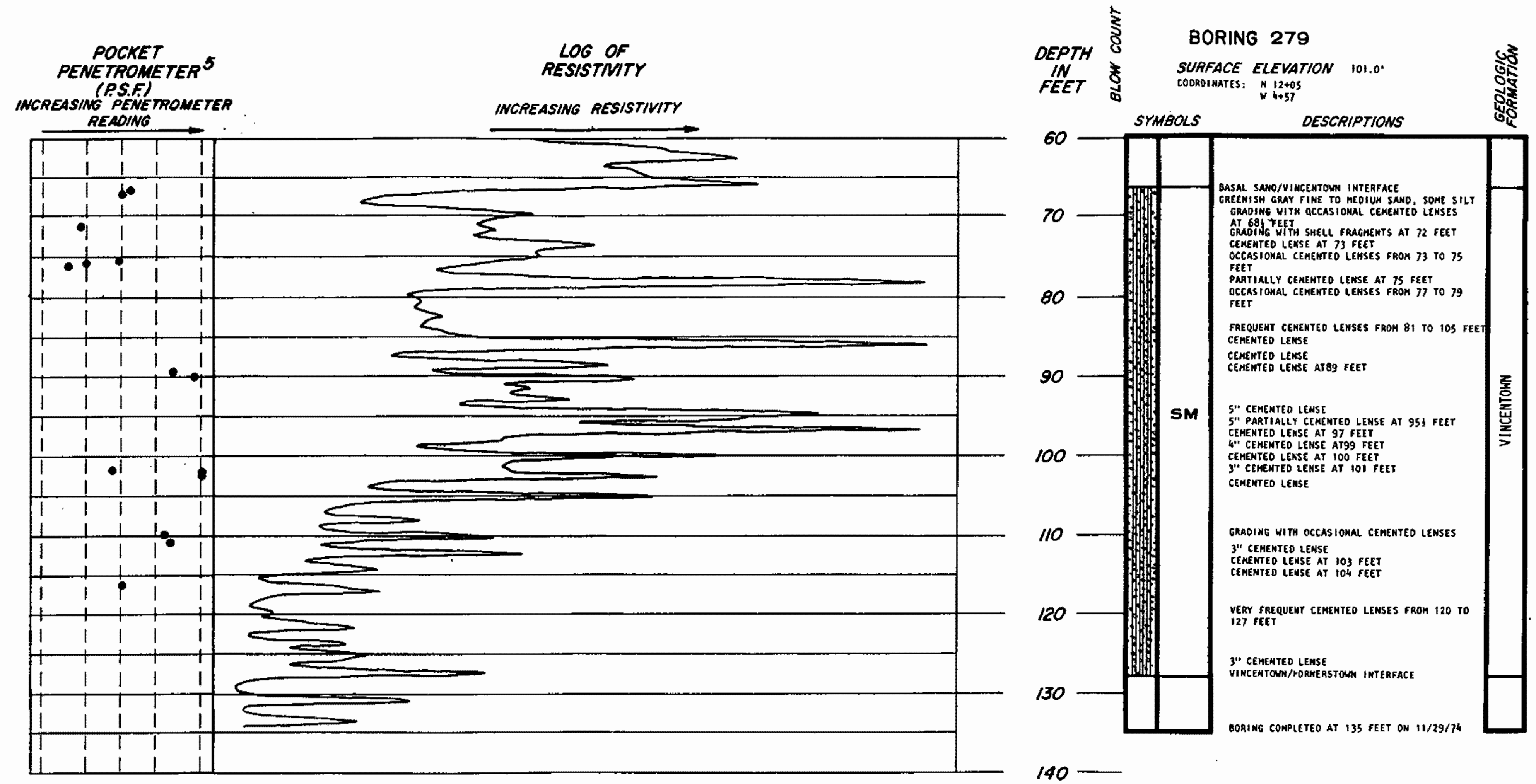
LOG BORINGS

UPDATED FSAR

Sheet 162 of 189
FIGURE 2.5-50

LABORATORY TEST DATA

LOG OF BORING



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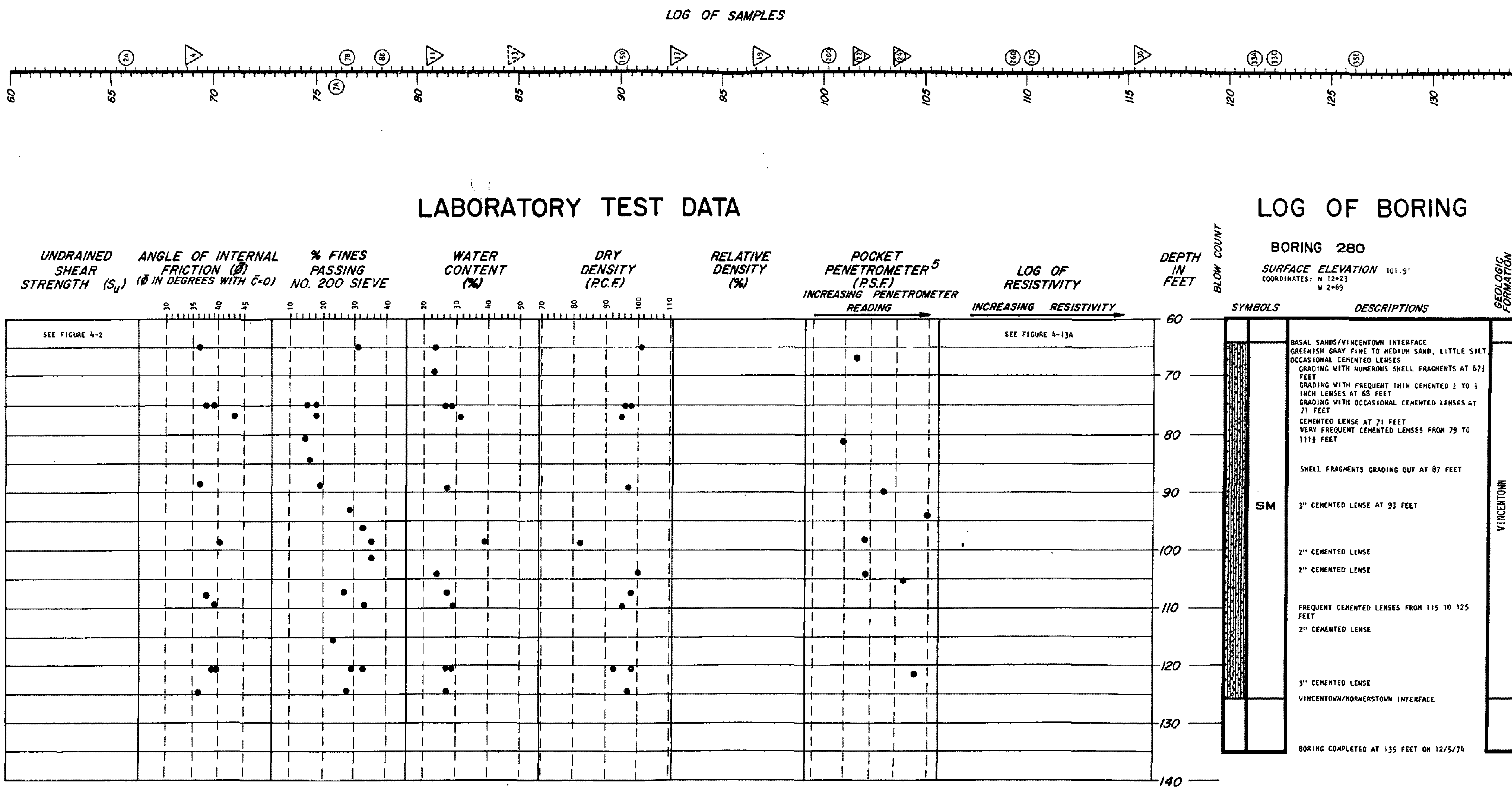
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 163 of 189
FIGURE 2.5-50



NOTES:

1. THE FIGURES IN THE COLUMN LABELED "BLOW COUNT" REFER TO THE NUMBER OF BLOWS REQUIRED TO DRIVE A STANDARD SPLIT-SPOON SAMPLER A DISTANCE OF ONE FOOT USING A 140 POUND DRIVE WEIGHT FALLING 30 INCHES. THE STANDARD SPLIT-SPOON SAMPLER IS 2" O.D. AND 1-3/8" I.D. UNLESS OTHERWISE STATED. THE DENISON SAMPLER WAS USED FOR UNDISTURBED SAMPLING.
2. ELEVATIONS REFER TO THE PUBLIC SERVICE DATUM.
3. THE DISCUSSION IN THE TEXT OF THE REPORT IS NECESSARY FOR A PROPER UNDERSTANDING OF THE NATURE AND ENGINEERING PROPERTIES OF THE SUBSURFACE MATERIALS.
4. THE SAMPLING PROCEDURE USED DID NOT ALLOW THE OBSERVANCE OF ALL CEMENTED LENSES. ADDITIONAL CEMENTED LENSES THAN THOSE INDICATED CAN BE EXPECTED.
5. POCKET PENETROMETER READINGS WERE USED TO QUALITATIVELY EVALUATE THE VARIATION IN THE DEGREE OF CEMENTATION. NO ATTEMPT WAS MADE NOR SHOULD BE MADE TO USE THESE READINGS TO QUANTITATIVELY EVALUATE THE SHEAR STRENGTH OF THE SAMPLES.
6. INDEX PROPERTY TESTS WERE MADE IN CONJUNCTION WITH ALL CONSOLIDATED ISOTROPIC UNDRAINED (CIU) TRIAXIAL TESTS AND STRESS-CONTROLLED CYCLIC TRIAXIAL TESTS.

KEY:

(N) INDICATES A CONSOLIDATED ISOTROPIC UNDRAINED (CIU) TRIAXIAL TEST WHERE N REPRESENTS THE SAMPLE NUMBER.

[N] INDICATES A STRESS-CONTROLLED CYCLIC TRIAXIAL TEST WHERE N REPRESENTS THE SAMPLE NUMBER.

▽N A SUFFICIENT LENGTH OF SAMPLE FOR TESTING COULD NOT BE OBTAINED BECAUSE OF ALTERNATING ZONES OF HIGHLY CEMENTED AND LESS CEMENTED MATERIAL.

▽N A SUFFICIENT LENGTH OF SAMPLE FOR TESTING COULD NOT BE OBTAINED BECAUSE OF A HIGH DEGREE OF CEMENTATION.

▽N A SUFFICIENT LENGTH OF SAMPLE FOR TESTING COULD NOT BE OBTAINED BECAUSE OF DISTURBANCE.

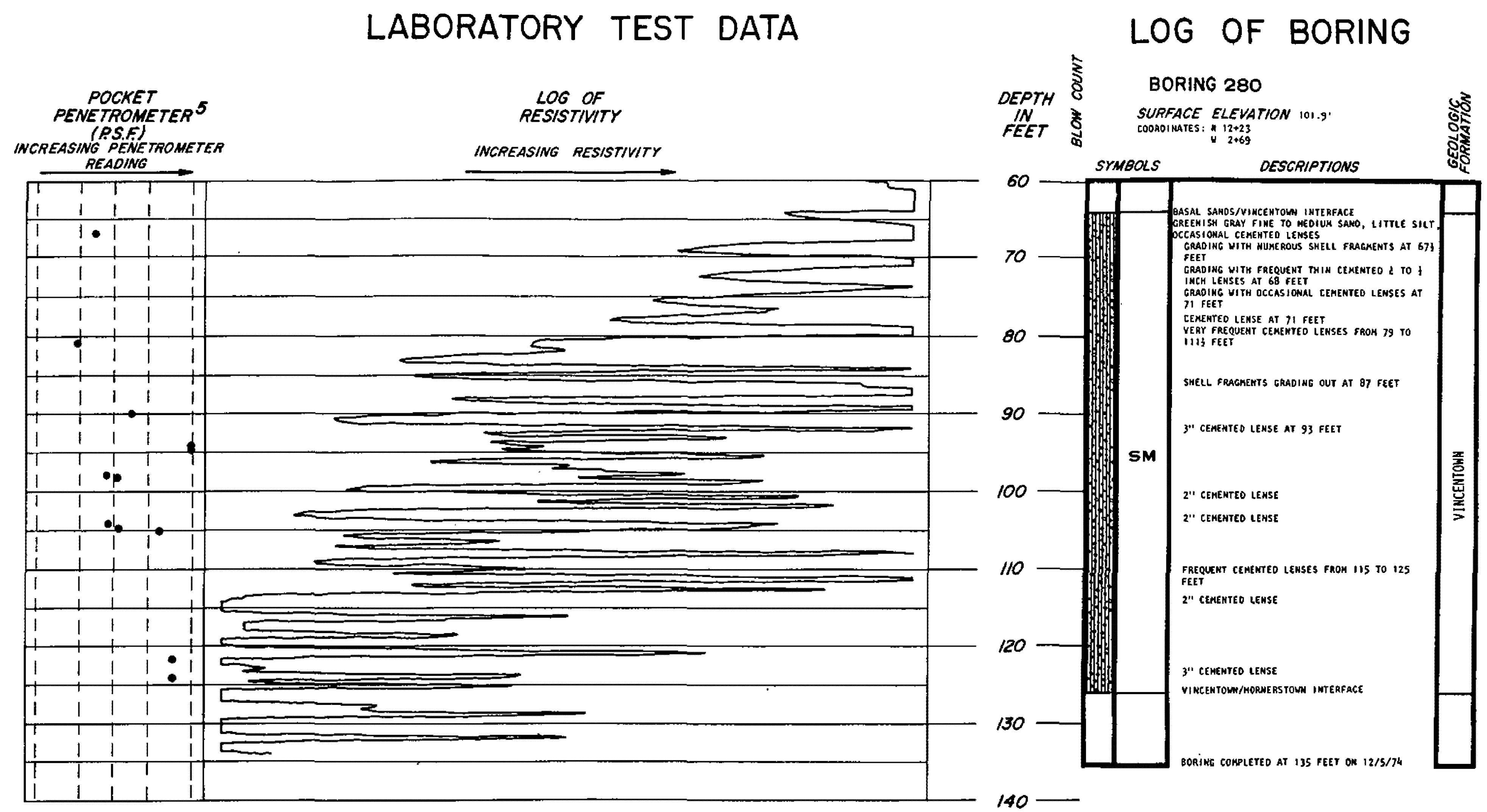
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 164 of 189
FIGURE 2.5-50



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3. THE DISCUSSION IN THE TEXT OF THE REPORT IS NECESSARY FOR A PROPER UNDERSTANDING OF THE NATURE AND ENGINEERING PROPERTIES OF THE SUBSURFACE MATERIALS.
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REVISION 0
APRIL 11, 1988

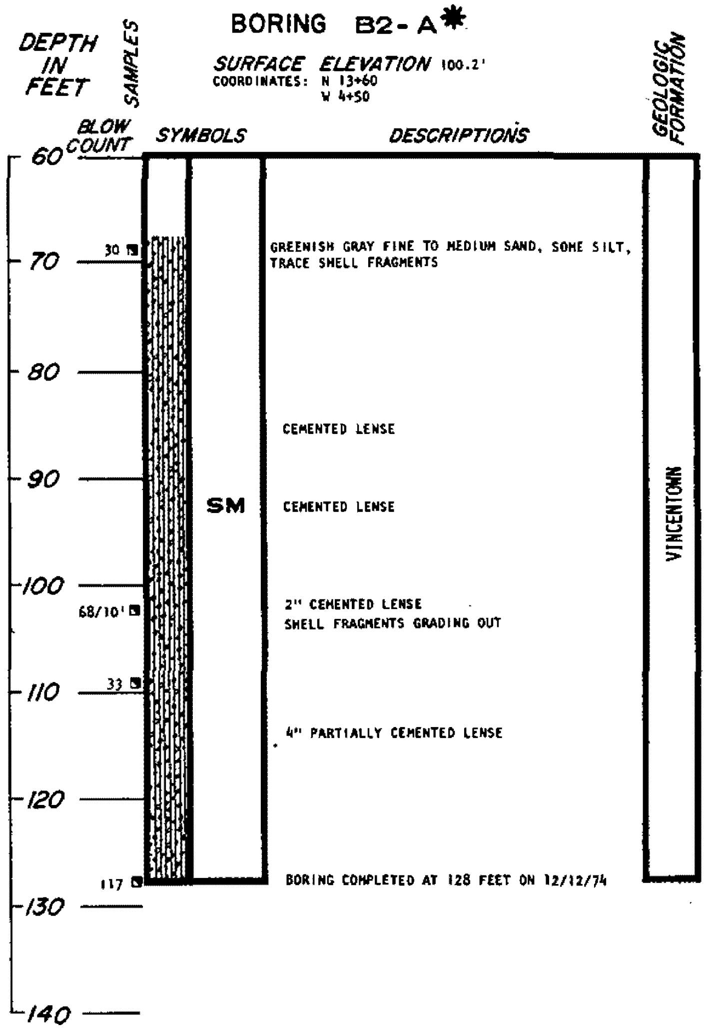
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 165 of 189
FIGURE 2.5-50

LOG OF BORING



*NO TESTING WAS PERFORMED ON SAMPLES FROM THIS BORING BECAUSE SUFFICIENT SAMPLES INDICATING THE LEAST DEGREE OF CEMENTATION WERE TESTED IN THE IMMEDIATELY ADJACENT BORING. (B-2)

NOTES:

1. THE FIGURES IN THE COLUMN LABELED "BLOW COUNT" REFER TO THE NUMBER OF BLOWS REQUIRED TO DRIVE A STANDARD SPLIT-SPOON SAMPLER A DISTANCE OF ONE FOOT USING A 140 POUND DRIVE WEIGHT FALLING 30 INCHES. THE STANDARD SPLIT-SPOON SAMPLER IS 2" O.D. AND 1-3/8" I.D. UNLESS OTHERWISE STATED THE DENISON SAMPLER WAS USED FOR UNDISTURBED SAMPLING.
2. ELEVATIONS REFER TO THE PUBLIC SERVICE DATUM.
3. THE DISCUSSION IN THE TEXT OF THE REPORT IS NECESSARY FOR A PROPER UNDERSTANDING OF THE NATURE AND ENGINEERING PROPERTIES OF THE SUBSURFACE MATERIALS.
4. THE SAMPLING PROCEDURE USED DID NOT ALLOW THE OBSERVANCE OF ALL CEMENTED LENSES. ADDITIONAL CEMENTED LENSES THAN THOSE INDICATED CAN BE EXPECTED.

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

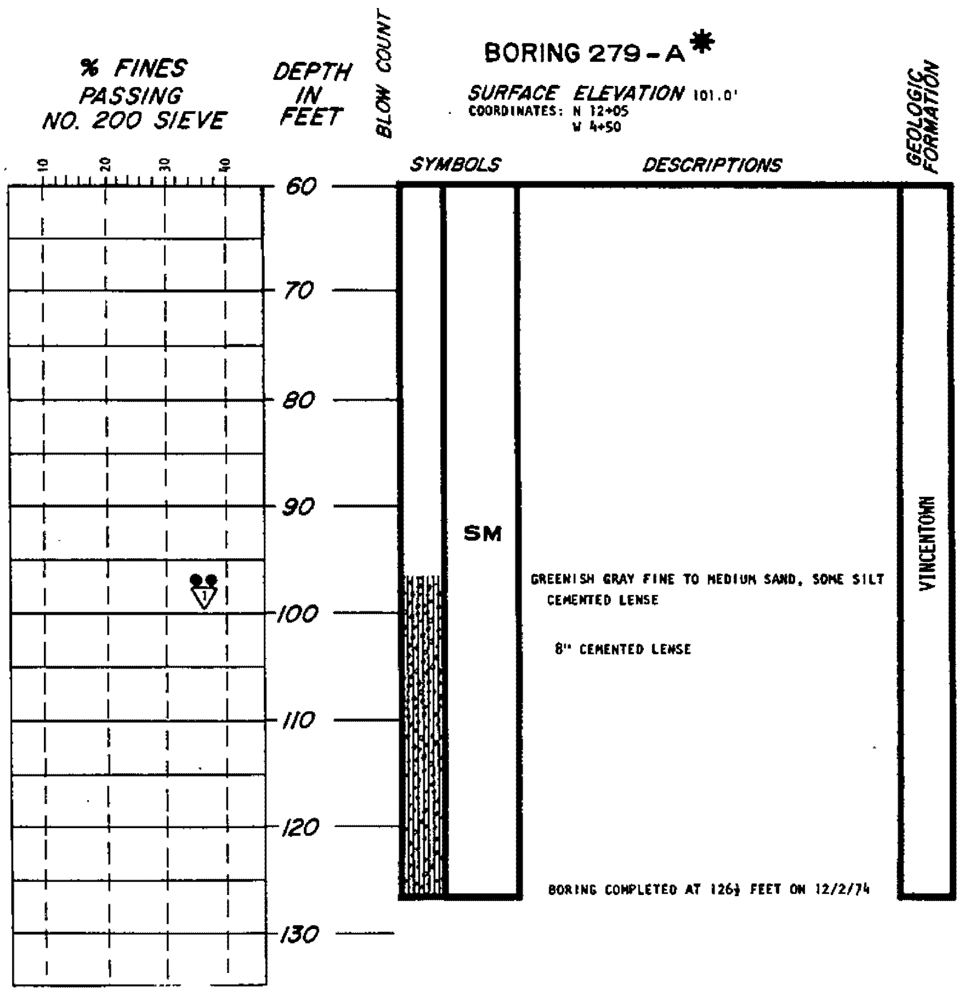
LOG BORINGS

UPDATED FSAR

FIGURE 2.5-50

LABORATORY TEST DATA

LOG OF BORING



(1) A SUFFICIENT LENGTH OF SAMPLE FOR TESTING COULD NOT BE OBTAINED BECAUSE OF ALTERNATING ZONES OF HIGHLY CEMENTED AND LESS CEMENTED MATERIAL.

* NO OTHER TESTING WAS PERFORMED ON SAMPLES FROM THIS BORING BECAUSE SUFFICIENT SAMPLES INDICATING THE LEAST DEGREE OF CEMENTATION WERE TESTED IN THE IMMEDIATELY ADJACENT BORING (B-279)

NOTES:

1. THE FIGURES IN THE COLUMN LABELED "BLOW COUNT" REFER TO THE NUMBER OF BLOWS REQUIRED TO DRIVE A STANDARD SPLIT-SPOON SAMPLER A DISTANCE OF ONE FOOT USING A 140 POUND DRIVE WEIGHT FALLING 30 INCHES. THE STANDARD SPLIT-SPOON SAMPLER IS 2" O.D. AND 1-3/8" I.D. UNLESS OTHERWISE STATED THE DENISON SAMPLER WAS USED FOR UNDISTURBED SAMPLING.
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REVISION 0
APRIL 11, 1988

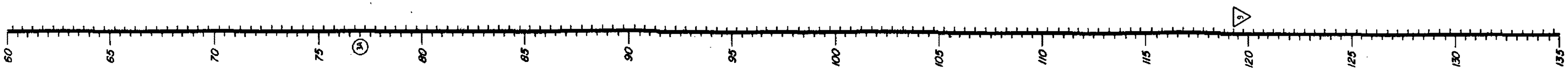
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

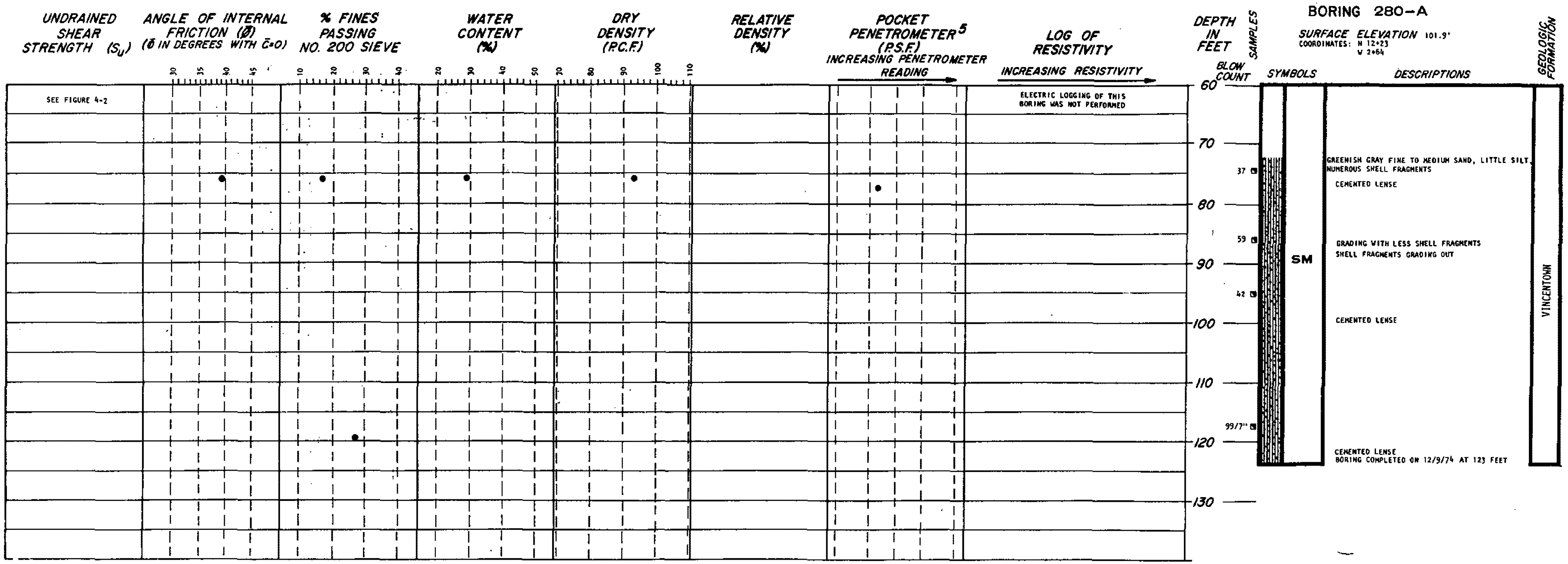
FIGURE 2.5-50

LOG OF SAMPLES



LABORATORY TEST DATA

LOG OF BORING



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5. POCKET PENETROMETER READINGS WERE USED TO QUALITATIVELY EVALUATE THE VARIATION IN THE DEGREE OF CEMENTATION. NO ATTEMPT WAS MADE NOR SHOULD BE MADE TO USE THESE READINGS TO QUANTITATIVELY EVALUATE THE SHEAR STRENGTH OF THE SAMPLES.
6. INDEX PROPERTY TESTS WERE MADE IN CONJUNCTION WITH ALL CONSOLIDATED ISOTROPIC UNDRAINED (CIU) TRIAXIAL TESTS AND STRESS-CONTROLLED CYCLIC TRIAXIAL TESTS.

KEY:

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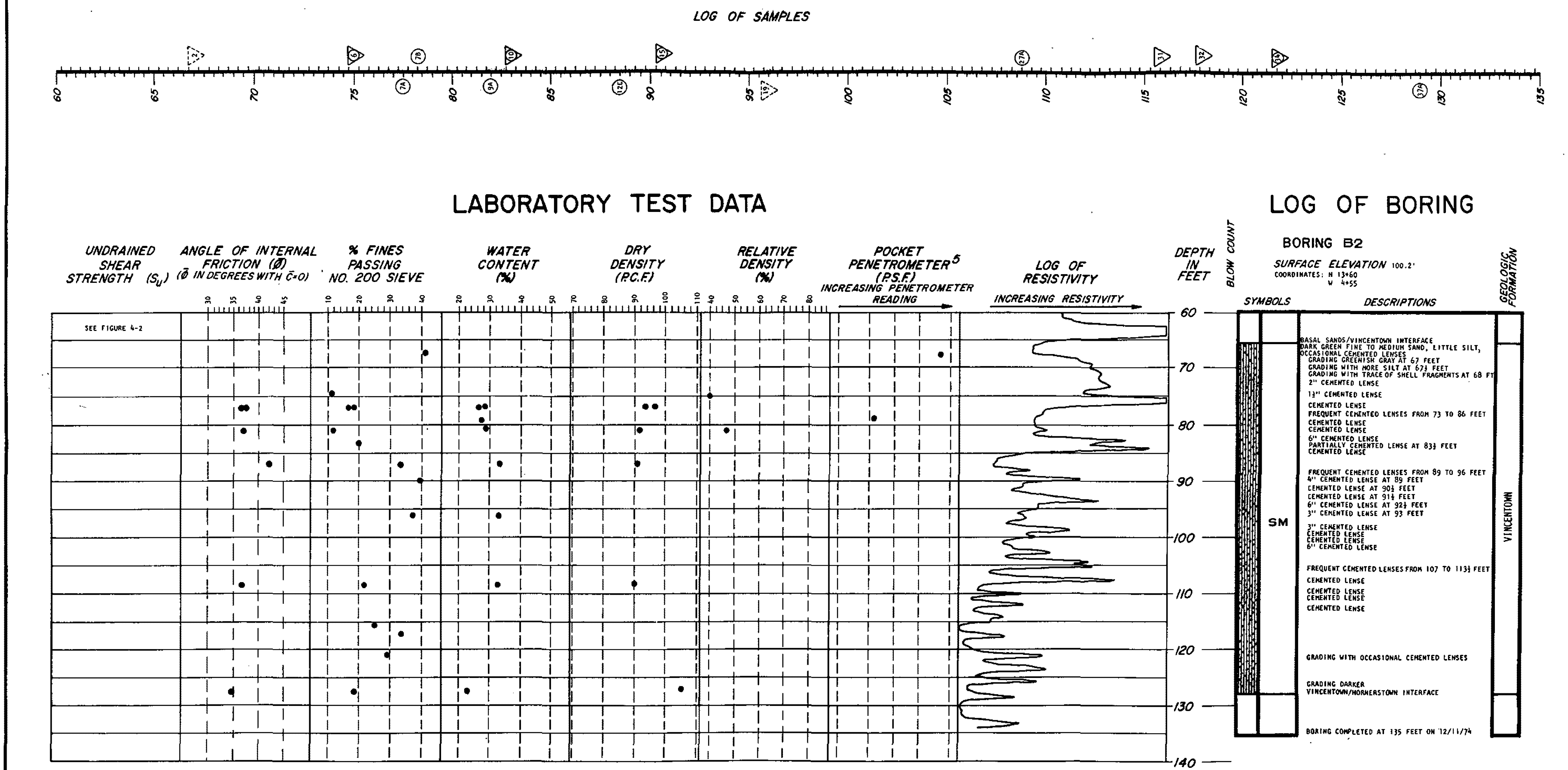
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 168 of 189
FIGURE 2.5-50



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7. RELATIVE DENSITIES WERE DETERMINED ACCORDING TO THE ARROYO METHOD DESCRIBED IN REF. 1 IN THE TEXT

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REVISION 0
APRIL 11, 1988

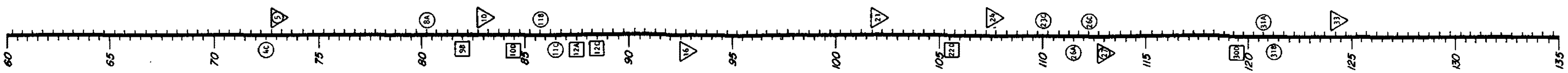
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

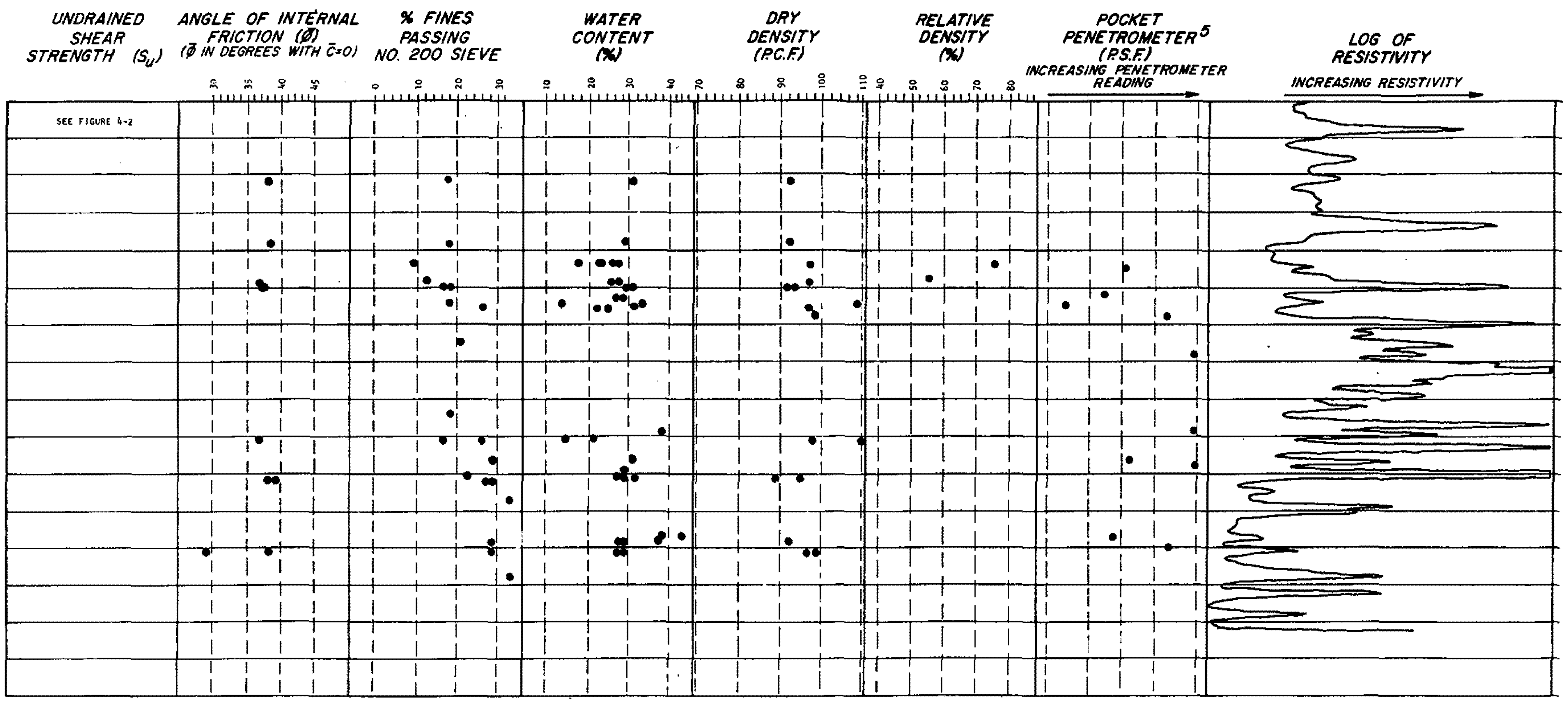
UPDATED FSAR

FIGURE 2.5-50

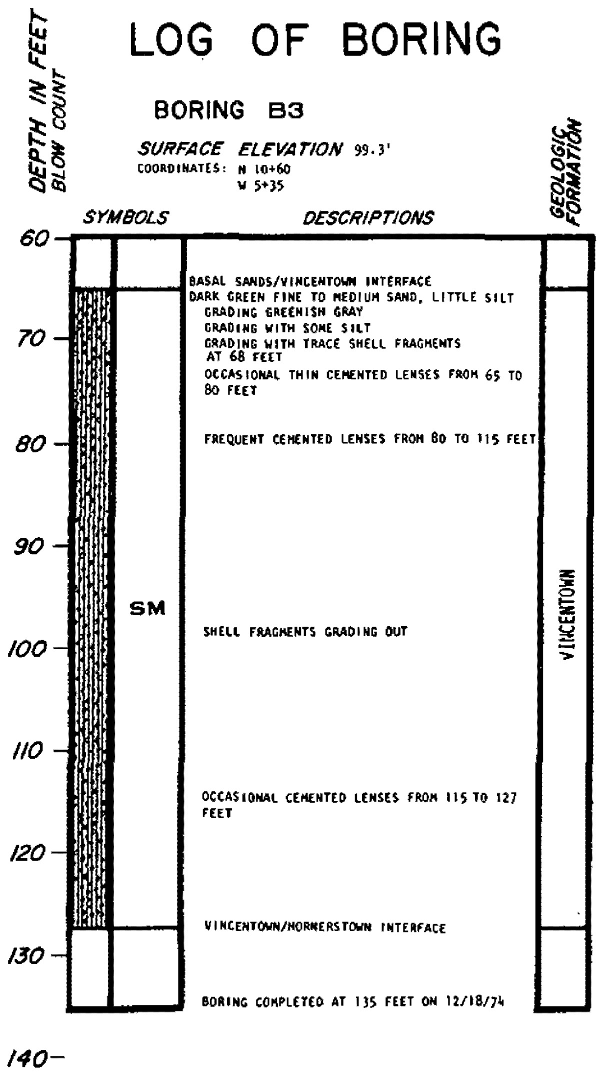
LOG OF SAMPLES



LABORATORY TEST DATA



LOG OF BORING



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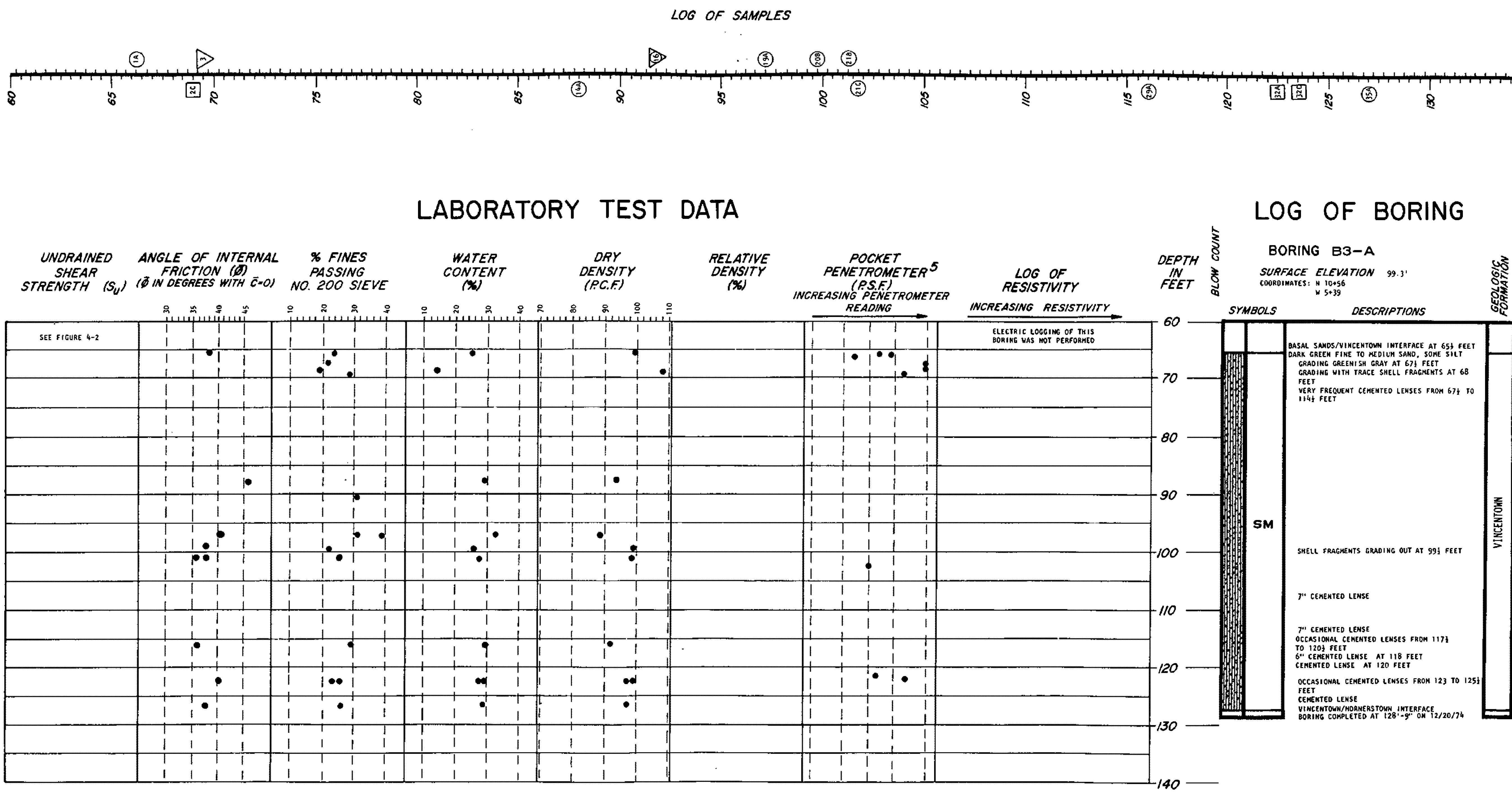
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

FIGURE 2.5-50



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REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 171 of 189
FIGURE 2.5-50

Surface Elevation: 100.0
Drilling Method: Rotary Wash with Mud
Sampling Method: 3" diameter Osterberg
Ft. of Casing Used

BORING NO. SS-1 (HOPE CREEK)

Completion Date: 11/25/74
Location of Boring:
N 11+97
E 0+20

DRILLER:
SUPERVISED BY:

Depth Feet	Sample No.	Blows / Ft.	SYM.	Description
			GW/SW	Fine gravel with silty clay and fine sand
	1		CL	Dark gray silty clay
5	2			
				Seam of reddish brown fine sand
	3		SP	Gray fine sand
	4			
10	5			Gray black medium-fine sand
				Clay lense
	6			Trace of gravel
15	7			
	8		GW	Black coarse to fine gravel and coarse to fine sand
20	9		SW	Black gray coarse to fine sand, trace gravel
	10			
	11			
	12		CL	Gray silty clay
25	13			
	14			
30	15			Coarse to fine sand, some fine gravel
				Boring terminated at 31'
5				
0				

REFERENCE FOR BORINGS SS-1 - SS-4:
PSE&G, FEBRUARY 1975, SLOPE STABILITY PROGRAM, PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

FIGURE 2.5-50

Surface Elevation: 101.5

BORING NO. SS-2 (HOPE CREEK)

Drilling Method: Rotary Wash with Mud

Sampling Method: 3" diameter Osterberg

Completion Date: 11/27/74

Location of Boring:

N 7+04
W 3+78

Ft. of Casing Used

SAMPLE: Depth Feet	No.	SYM	Blows Ft.	UNIFIED SOIL CLASSIFICATION: SYM.	Description
	1			GW/SW	Medium-fine sand with silt and gravel
5	2			CL	Dark gray clay, some silt
	3				
	4				
10	5			SP/SM	Gray fine sand with trace of silt and gravel
	6				Lense of silty clay
15	7				
	8			ML	Gray silt, little fine sand
	9				
20	10				
	11				Dark gray clayey silt
25	12				
	13				
	14				
30	15				
	16				Seams of fine sand
35	17			SP/SM	Fine gray sand, little silt
					Boring terminated at 36'
0					

DRILLER:

SUPERVISED BY:

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 173 of 189
FIGURE 2.5-50

Surface Elevation: 99.1 BORING NO. SS-3 (HOPE CREEK)

Drilling Method: Rotary Wash with Revert

Sampling Method: 5" diameter Osterberg

Completion Date: 11/29/74

Location of Boring:

N 12+00

W 7+45

Ft. of Casing Used.

Depth Feet	SAMPLE No.	SYM	Blows/Ft.	UNIFIED SOIL CLASSIFICATION SYM.	Description
				SW	Orange brown coarse to fine sand, trace of gravel
	1				Dark gray clayey silt
5	2				
	3			ML/CL	Dark gray silty clay
	4				
10					Lense of fine sand and silt
	5				Gray clayey silt
	6				
15	7			SP/ML	Gray fine sand, little silt
	8			ML	Gray silt
	9				Gray clayey silt
20					Dark gray clayey silt
	10				
	11				
25	12				Seams of fine sand
	13				
	14				
30	15				
	16				
35	17			SM	Brownish gray coarse to fine sand, little silt
					Boring terminated at 36'
0					

DRILLER:

SUPERVISED BY:

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

FIGURE 2.5-50

Surface Elevation: 101.5

Drilling Method: Rotary Wash with Revert

Sampling Method: 3 " diameter Osterberg

BORING NO. SS-4 (HOPE CREEK)

Completion Date: 11/26/74

Location of Boring:

N 16+40
W 3+80

Ft. of Casing Used

DRILLER:

SUPERVISED BY:

SAMPLE				UNIFIED SOIL CLASSIFICATION	
Depth Feet	No.	SYM	Blows / Ft.	SYM.	Description
				GW	Coarse gravel
	1			CL ML	Dark gray silty clay
	2				
5	3			ML	Dark gray clayey silt
	4				trace of organic material
	5				
10					
	6				Dark gray clay
	7				Gray fine sand, little silt
15	8			SM	
	9				
	10				
20					
	11			ML	Dark gray silt, little fine sand
	12				
25	13				Dark gray silty clay, seams of silt
	14				
	15				
30					
	16				
	17				
35	18			SP	Gray fine sand, some clay fines
	19				
				GW	Gravel
40					Boring terminated at 40'

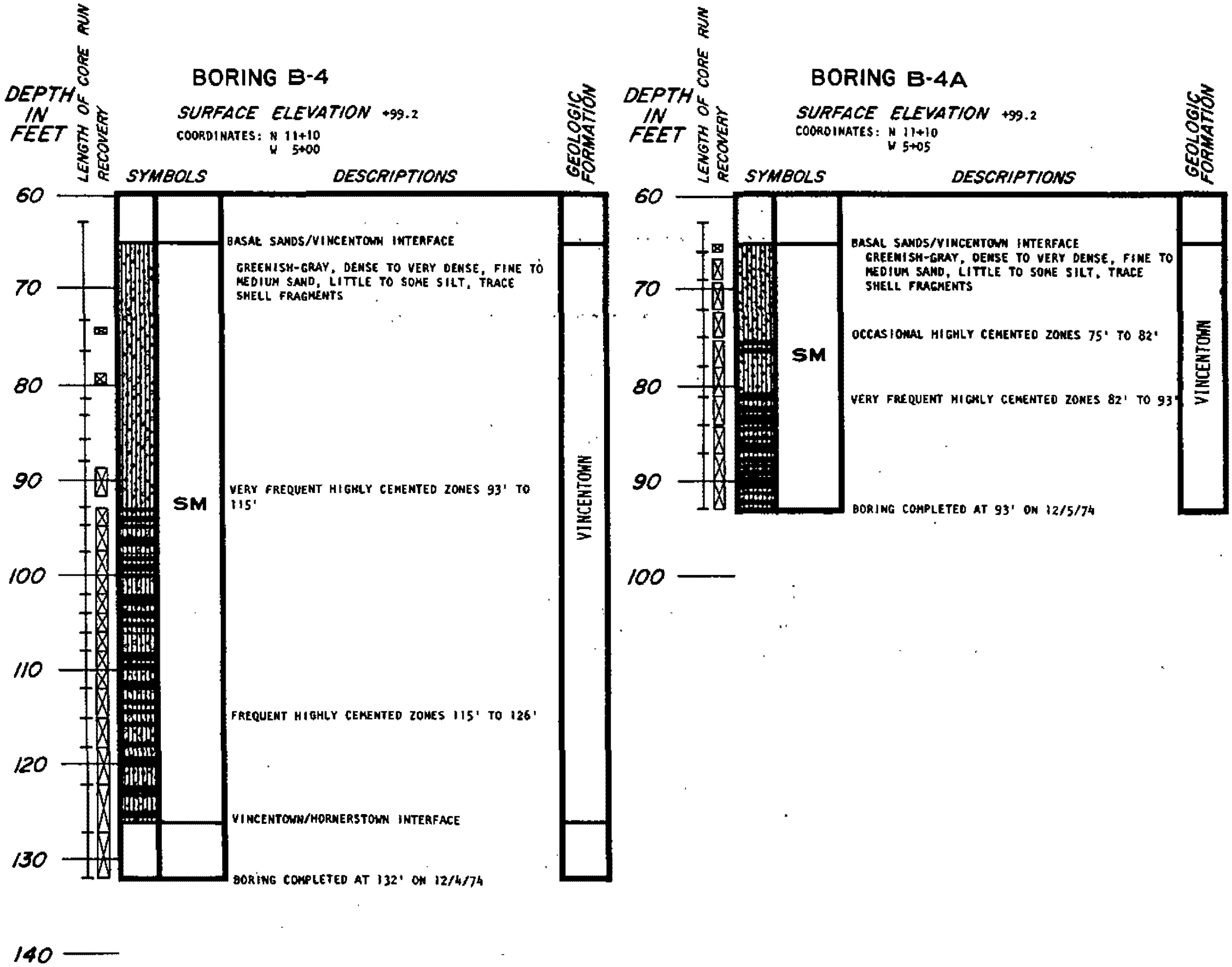
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

FIGURE 2.5-50



KEY:

INDICATES HIGHLY CEMENTED ZONE

NOTES:

1. VINCENTOWN/HORNERSTOWN INTERFACE LOCATED BY ELECTRIC AND GAMMA LOGS
2. ELEVATIONS REFER TO PUBLIC SERVICE DATUM
3. THE DISCUSSION IN THE TEXT OF THE REPORT IS NECESSARY FOR A PROPER UNDERSTANDING OF THE NATURE OF THE SUBSURFACE MATERIALS

REFERENCE FOR BORINGS B-4 - B-7A:
PSE&G, FEBRUARY 18, 1975, SUPPLEMENTARY FOUNDATION STUDIES, PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

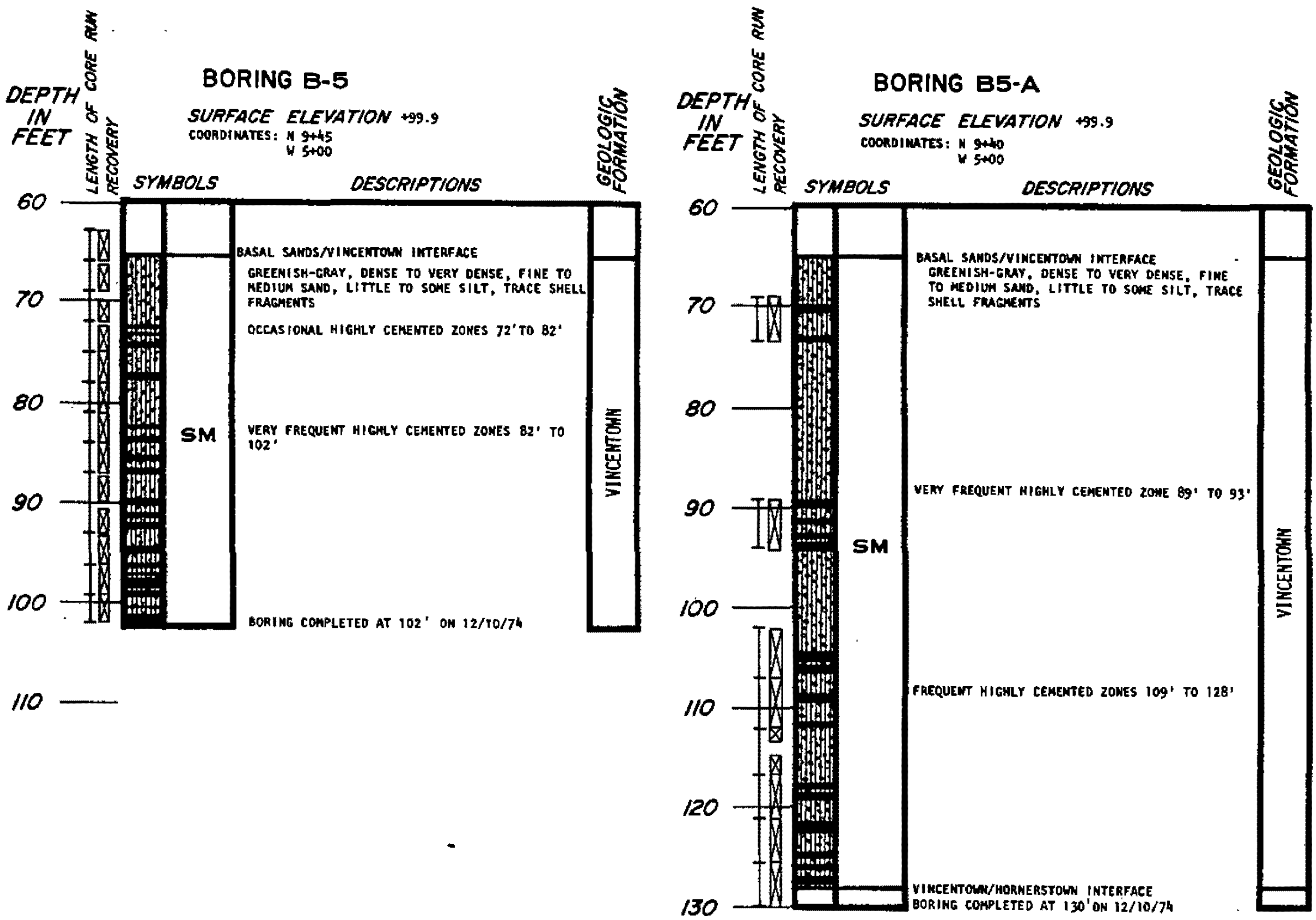
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 176 of 189
FIGURE 2.5-50



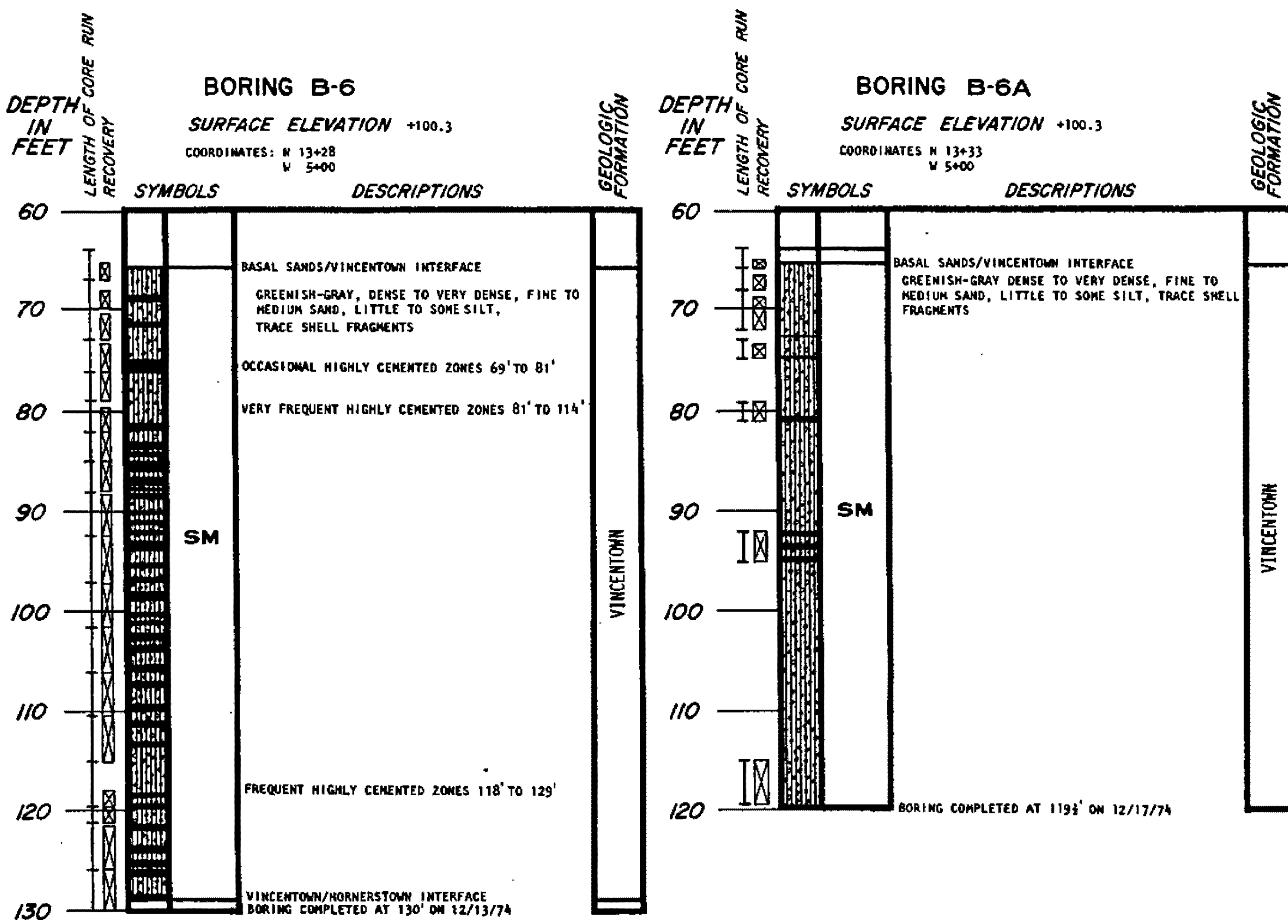
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 177 of 189
FIGURE 2.5-50



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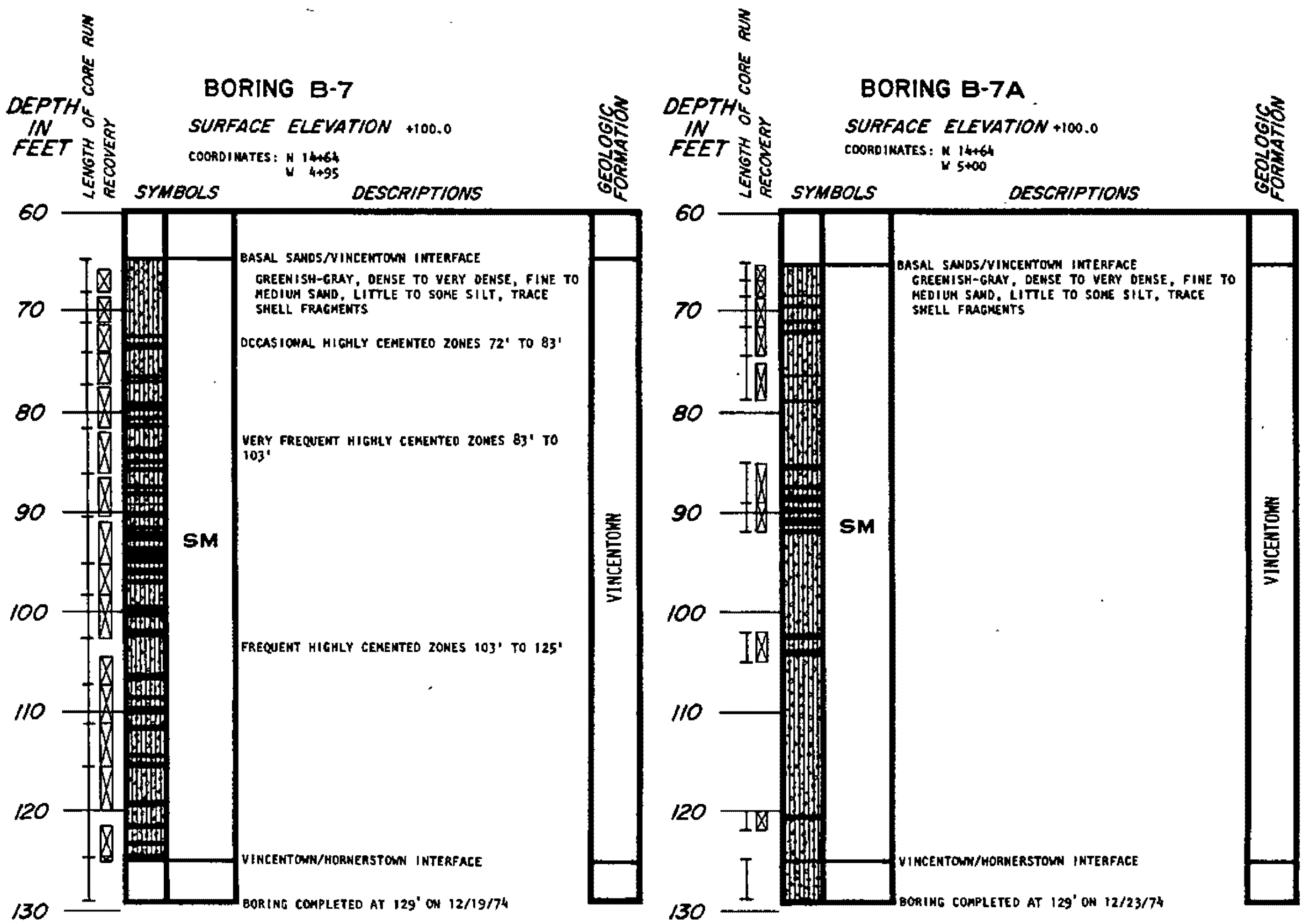
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 178 of 189
FIGURE 2.5-50



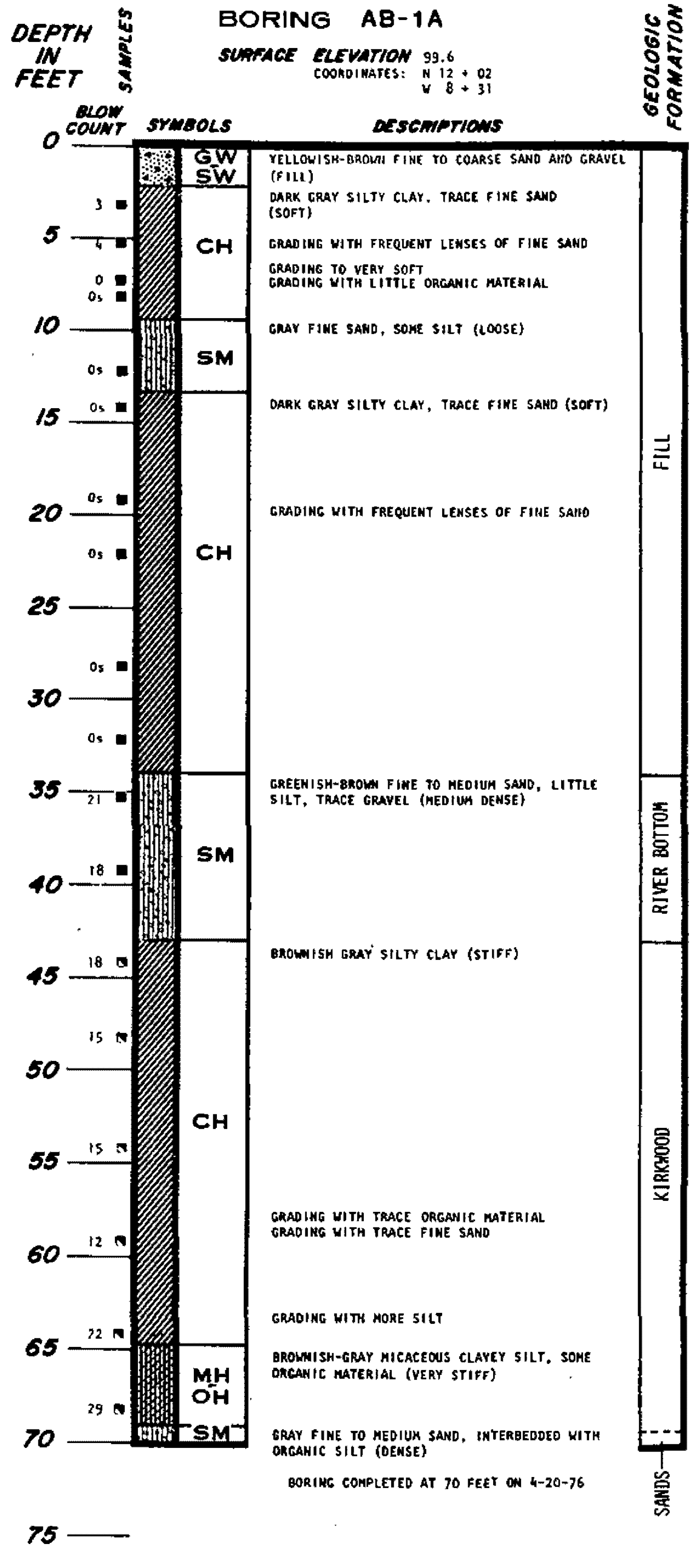
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 179 of 189
FIGURE 2.5-50



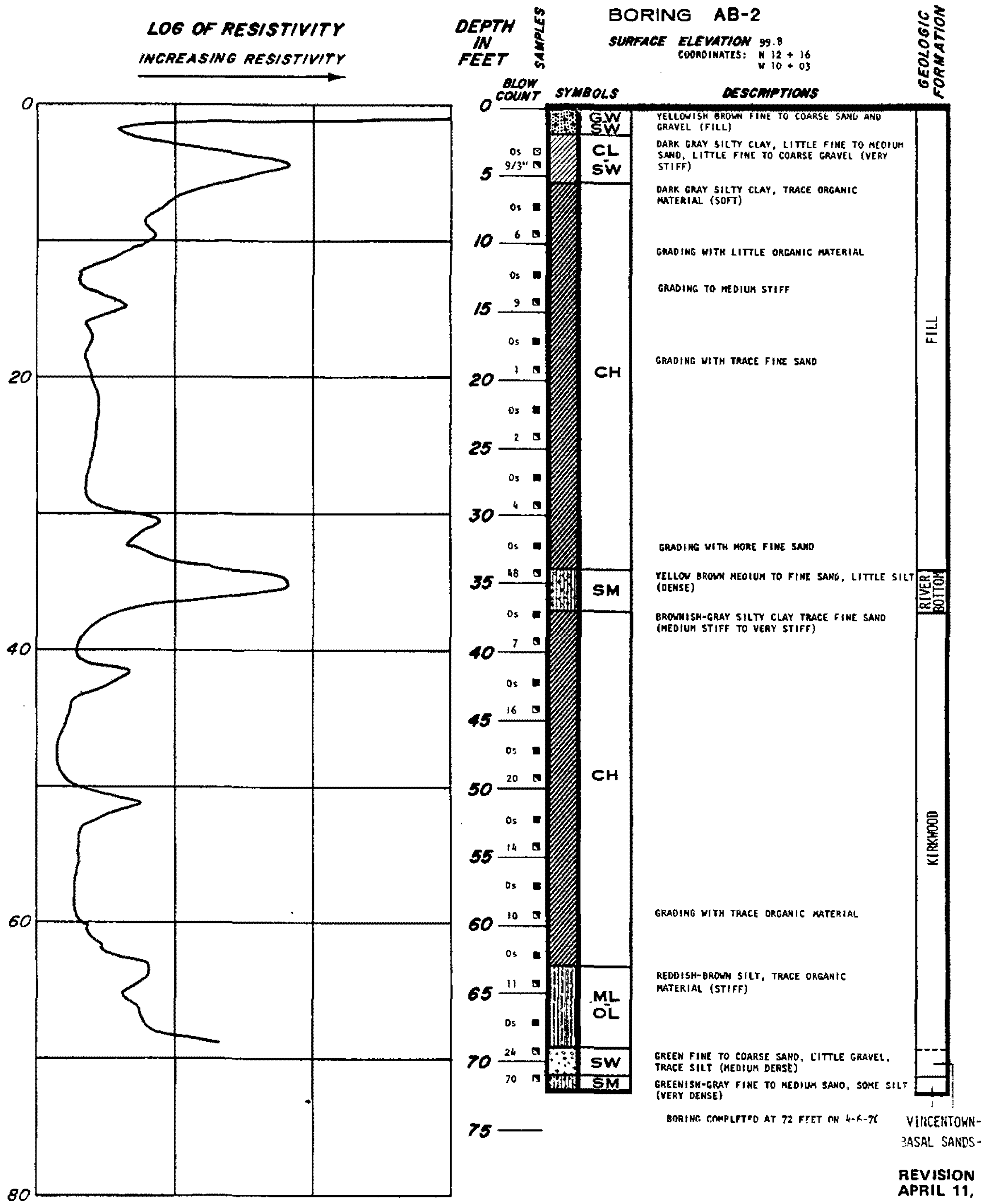
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 180 of 189
FIGURE 2.5-50



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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

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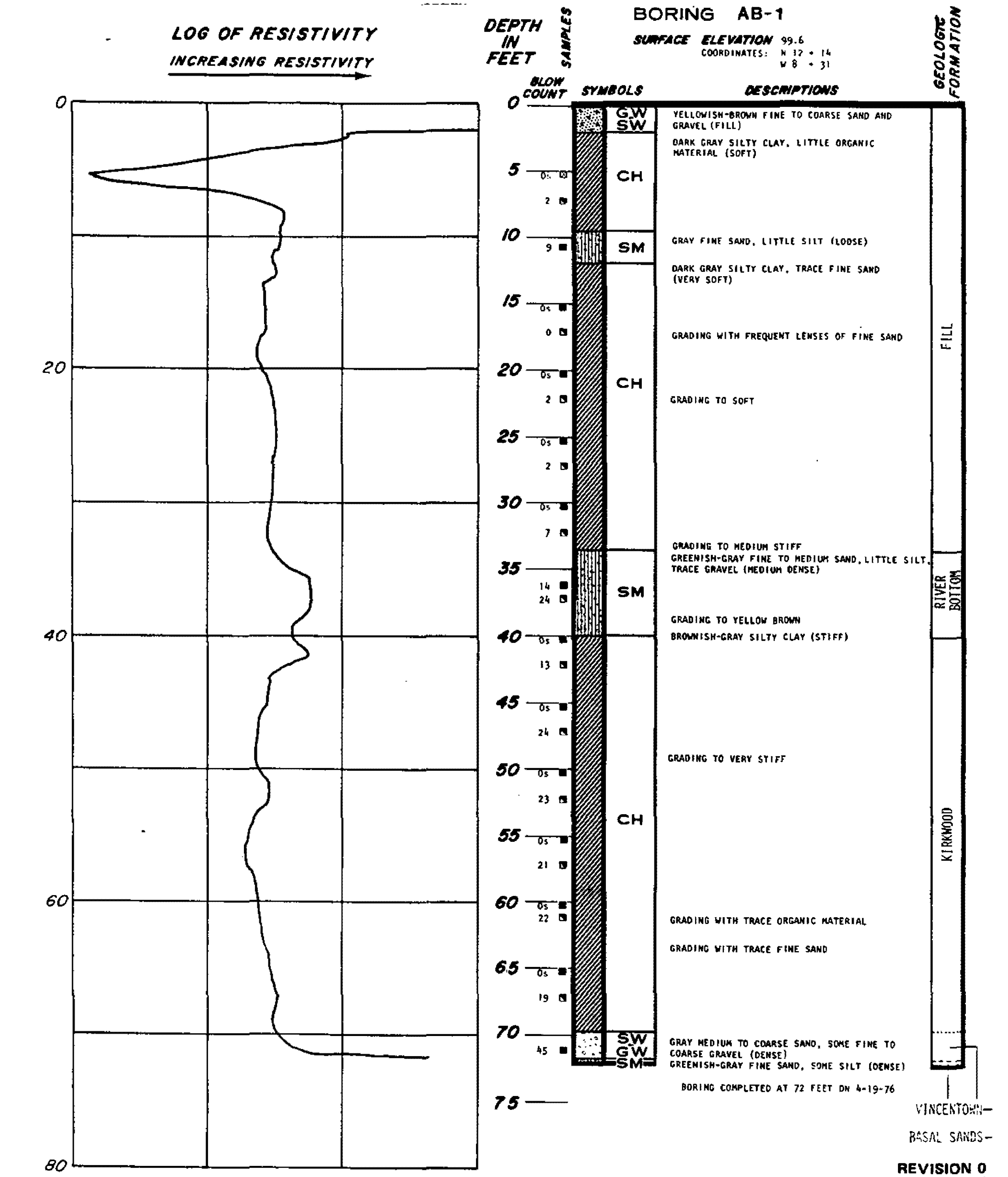
FIGURE 2.5-50



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APRIL 11, 1988

NOTES:

1) THE FIGURES IN THE COLUMN LABELED "BLOW COUNT" REFER TO THE NUMBER OF BLOWS REQUIRED TO DRIVE THE DAMES & MOORE SAMPLER A DISTANCE OF ONE FOOT OR A STANDARD SPLIT-SPOON SAMPLER A DISTANCE OF ONE FOOT. THE DAMES & MOORE SAMPLER IS 3¼" O.D. AND APPROXIMATELY 2½" I.D. THE STANDARD SPLIT-SPOON SAMPLER IS 2" O.D. AND 1 3/8" I.D. THE DAMES & MOORE SAMPLER WAS DRIVEN WITH A 350 LB. HAMMER AT A 30" DROP WHILE THE STANDARD SPLIT-SPOON SAMPLER USED A 140 LB. HAMMER AT A 30" DROP UNLESS OTHERWISE STATED ON THE LOGS
2) THE LETTERS IN THE "BLOW COUNT" COLUMN INDICATE THE FOLLOWING:
"OS" INDICATES THAT THE OSTERBERG SAMPLER WAS USED FOR UNDISTURBED SAMPLING.
3) ELEVATIONS REFER TO THE PUBLIC SERVICE PLANT DATUM.
4) THE DISCUSSION IN THE TEXT OF THE REPORT IS NECESSARY FOR A PROPER UNDERSTANDING OF THE NATURE OF THE SUBSURFACE MATERIALS.
5) THE USE OF THE GEOLOGIC FORMATION DESIGNATION IN THE DESCRIPTION OF SOILS ENCOUNTERED IS USED FOR EASE OF DESCRIPTION AND SHOULD NOT BE INTERPRETED AS A TEXTURAL OR ENGINEERING DESCRIPTION.

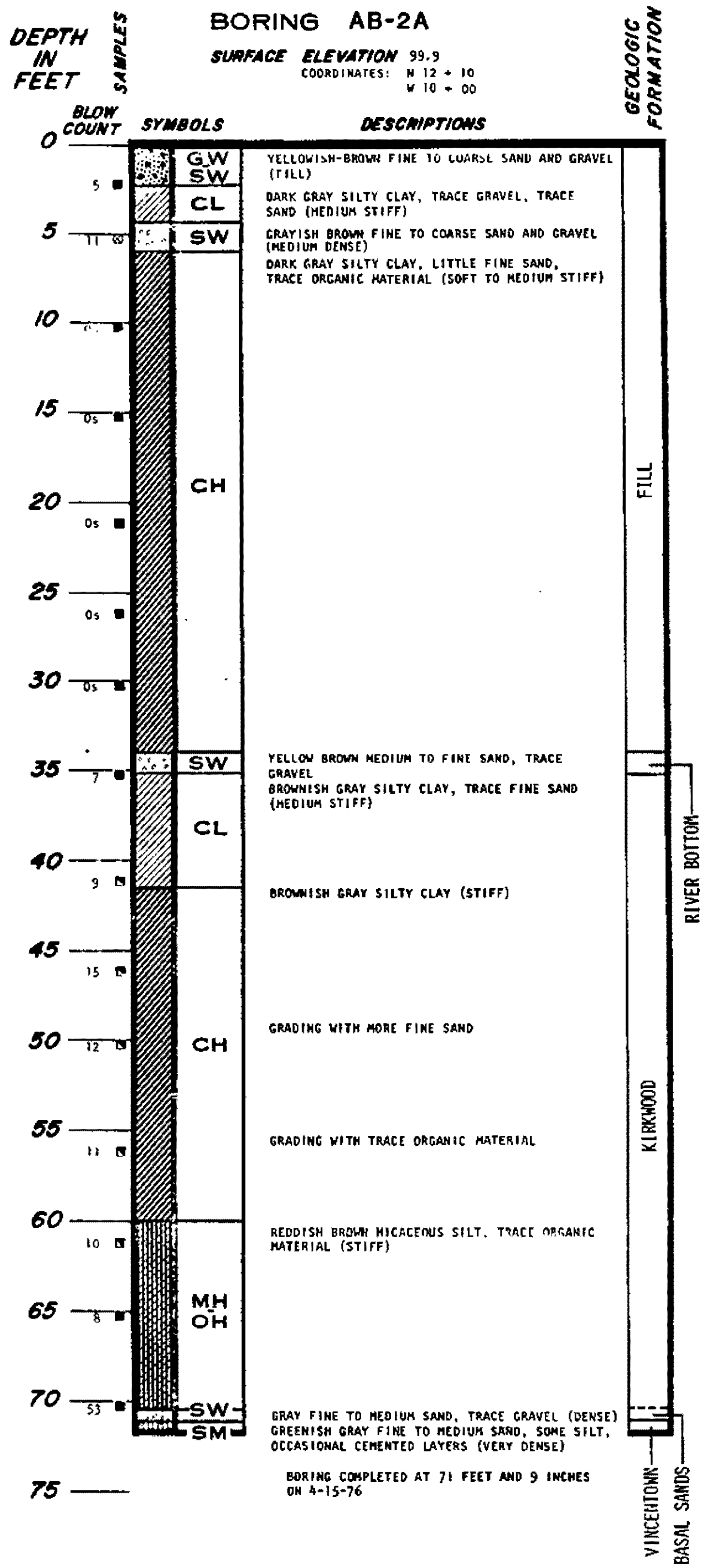
REFERENCE FOR BORINGS AB-1 - AB-5A:
PSE&G, DECEMBER 1976, AUXILIARY BORING PROGRAM ALONG SERVICE WATER SUPPLY LINES, PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 182 of 189
FIGURE 2.5-50



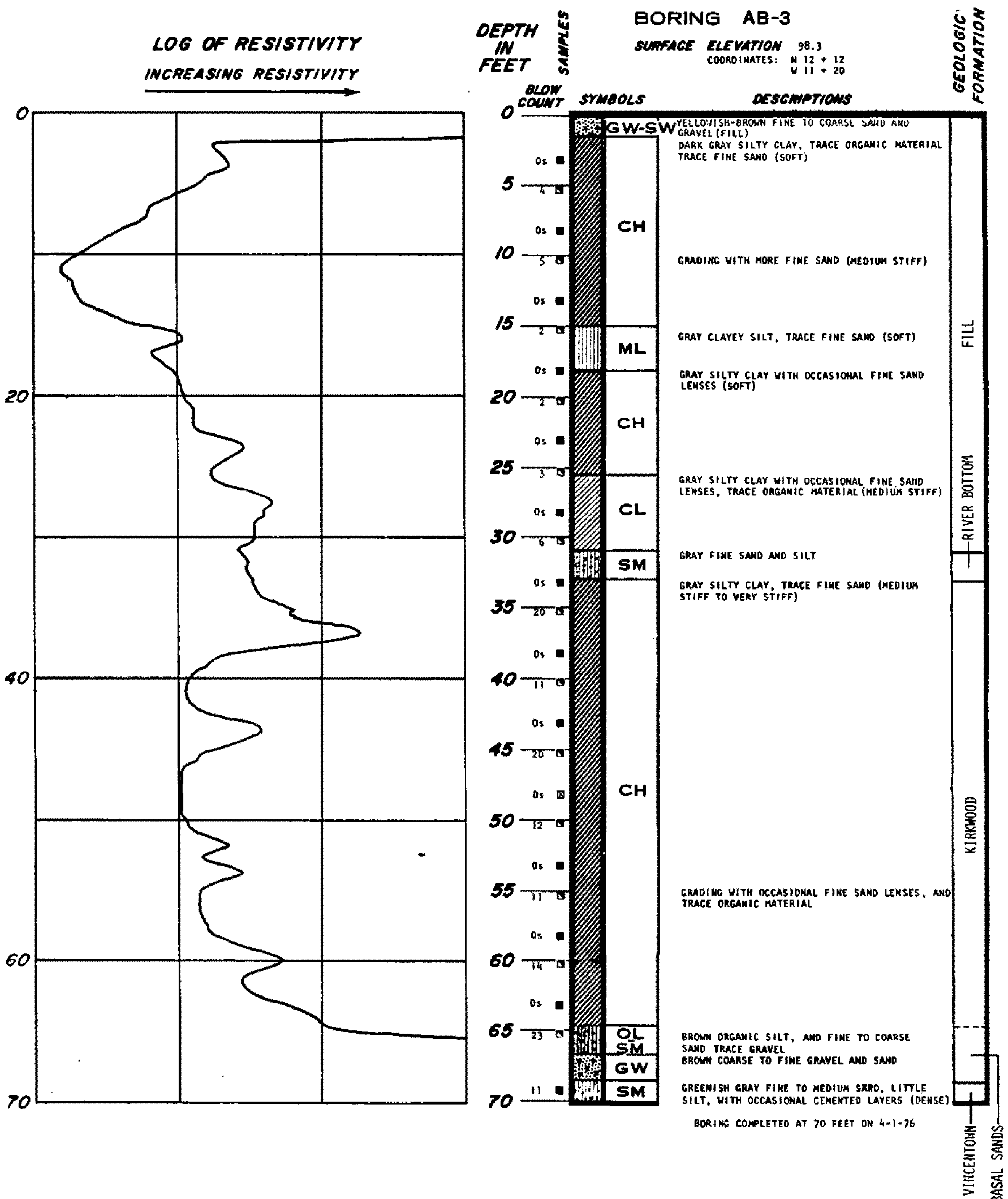
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 183 of 189
FIGURE 2.5-50



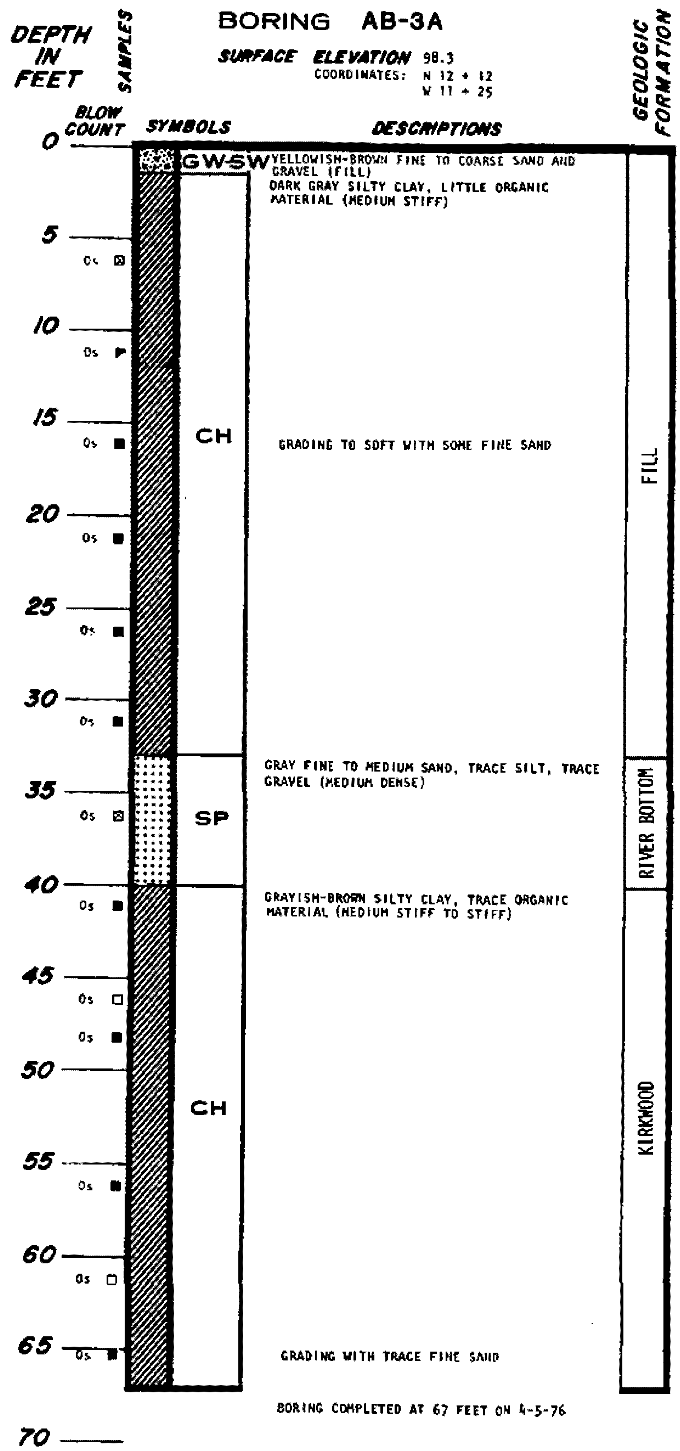
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 184 of 189
FIGURE 2.5-50



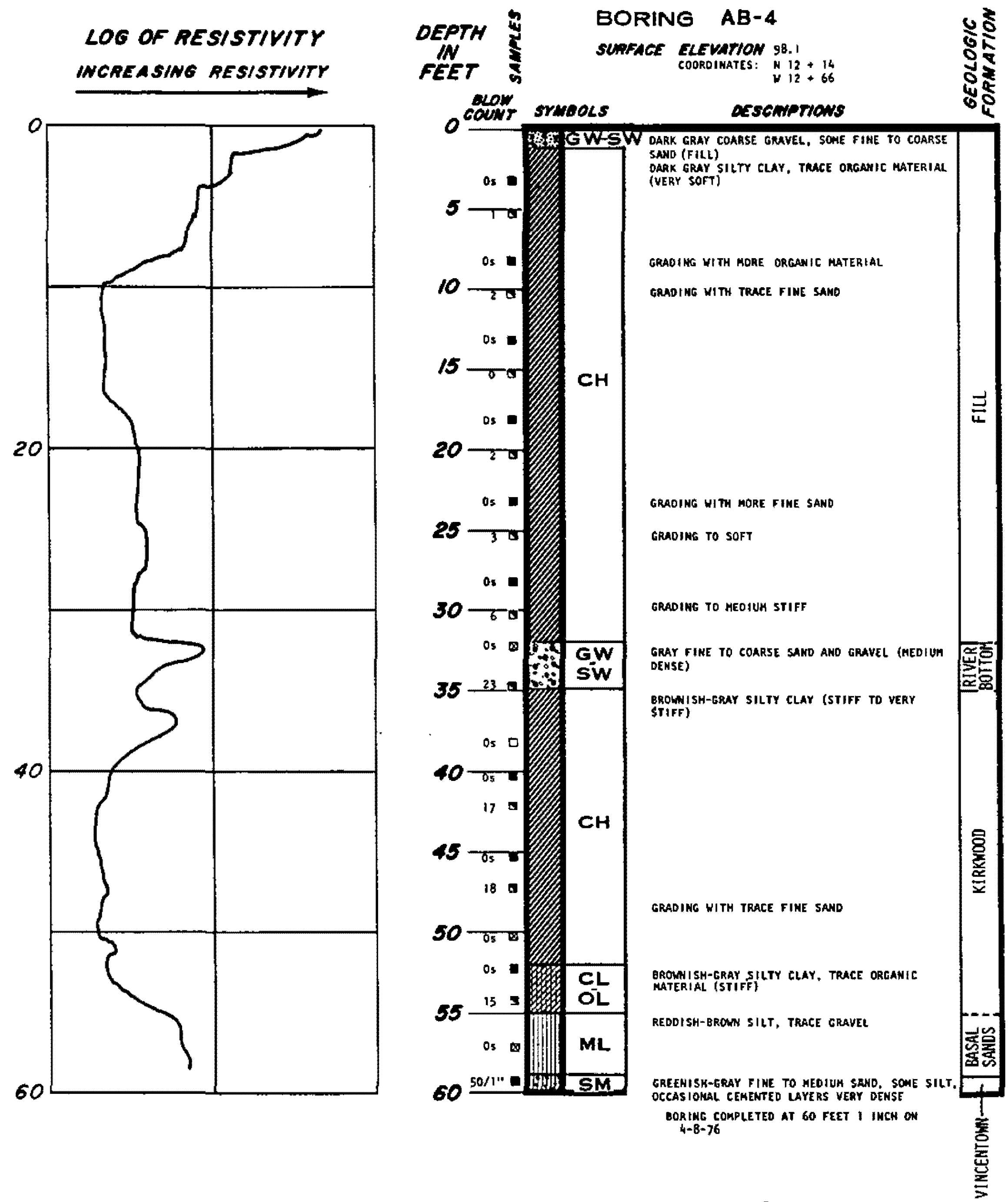
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 185 of 189
FIGURE 2.5-50



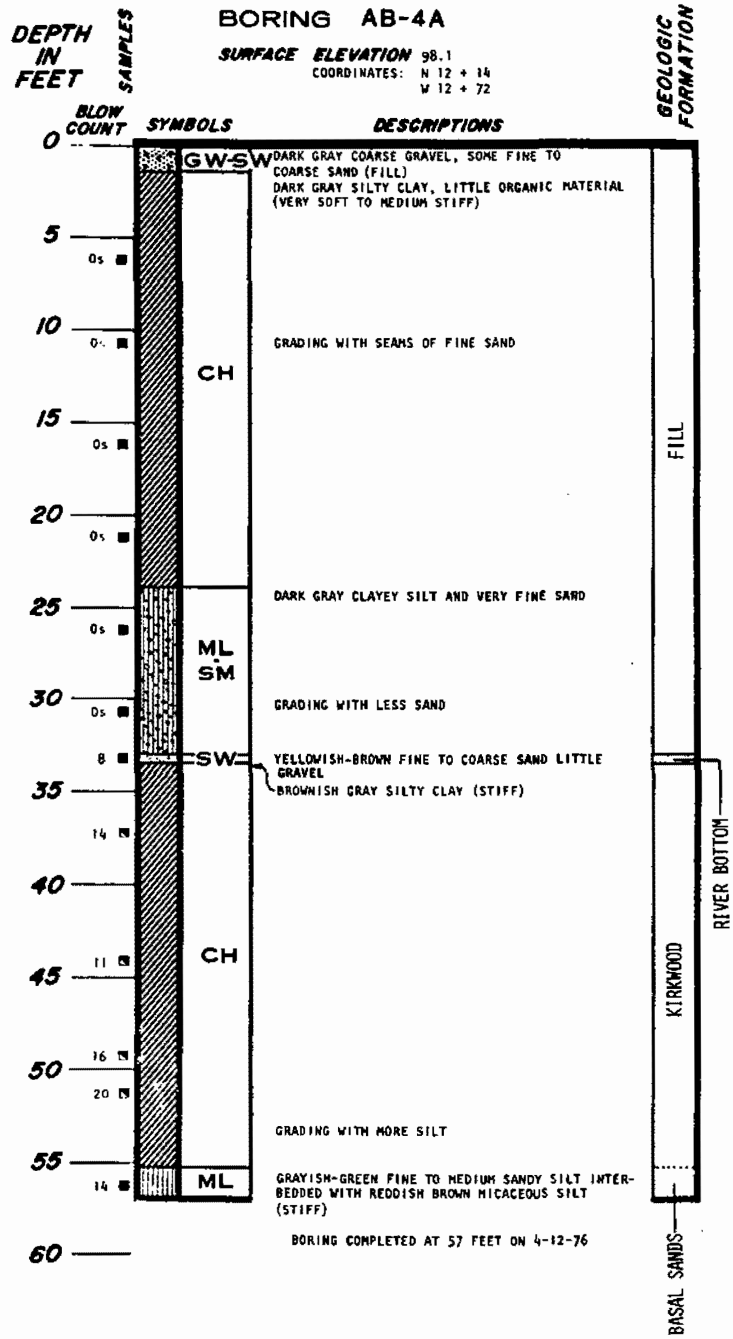
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 186 of 189
FIGURE 2.5-50



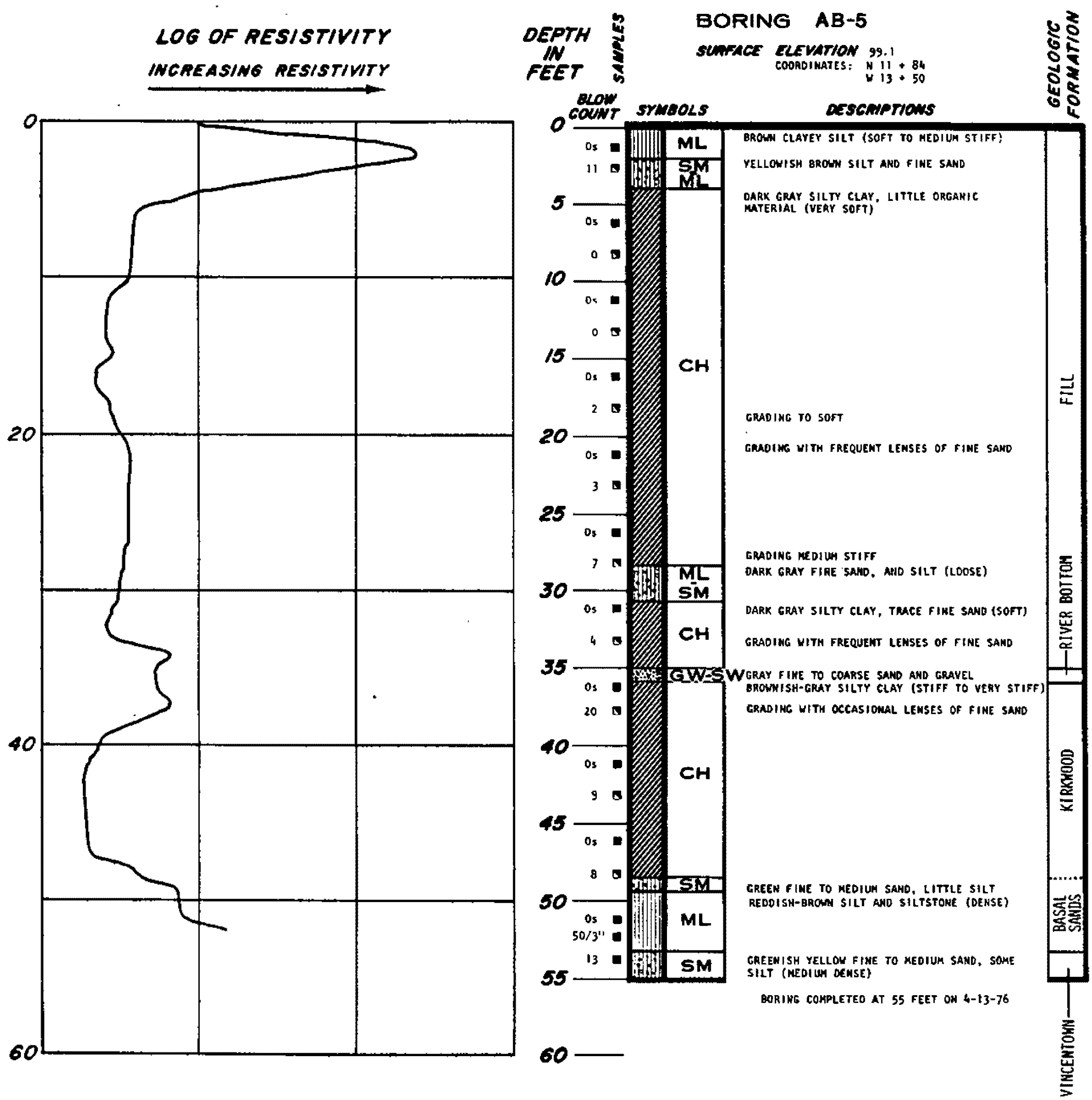
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 187 of 189
FIGURE 2.5-50



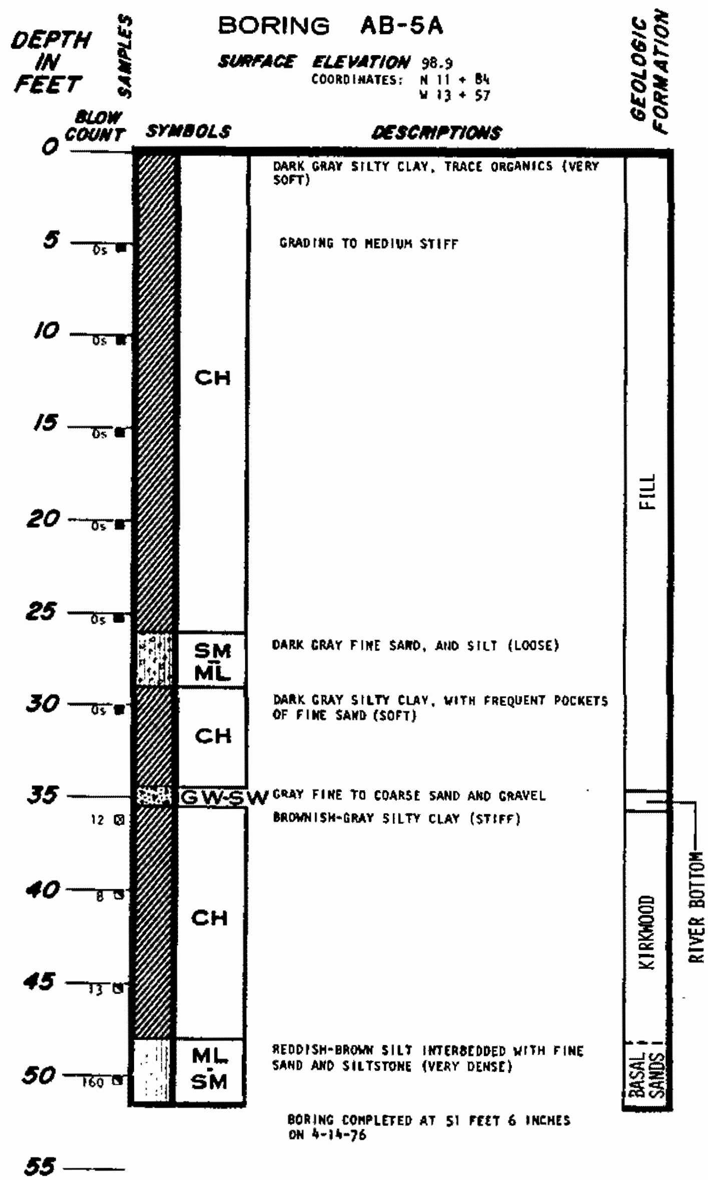
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 188 of 189
FIGURE 2.5-50



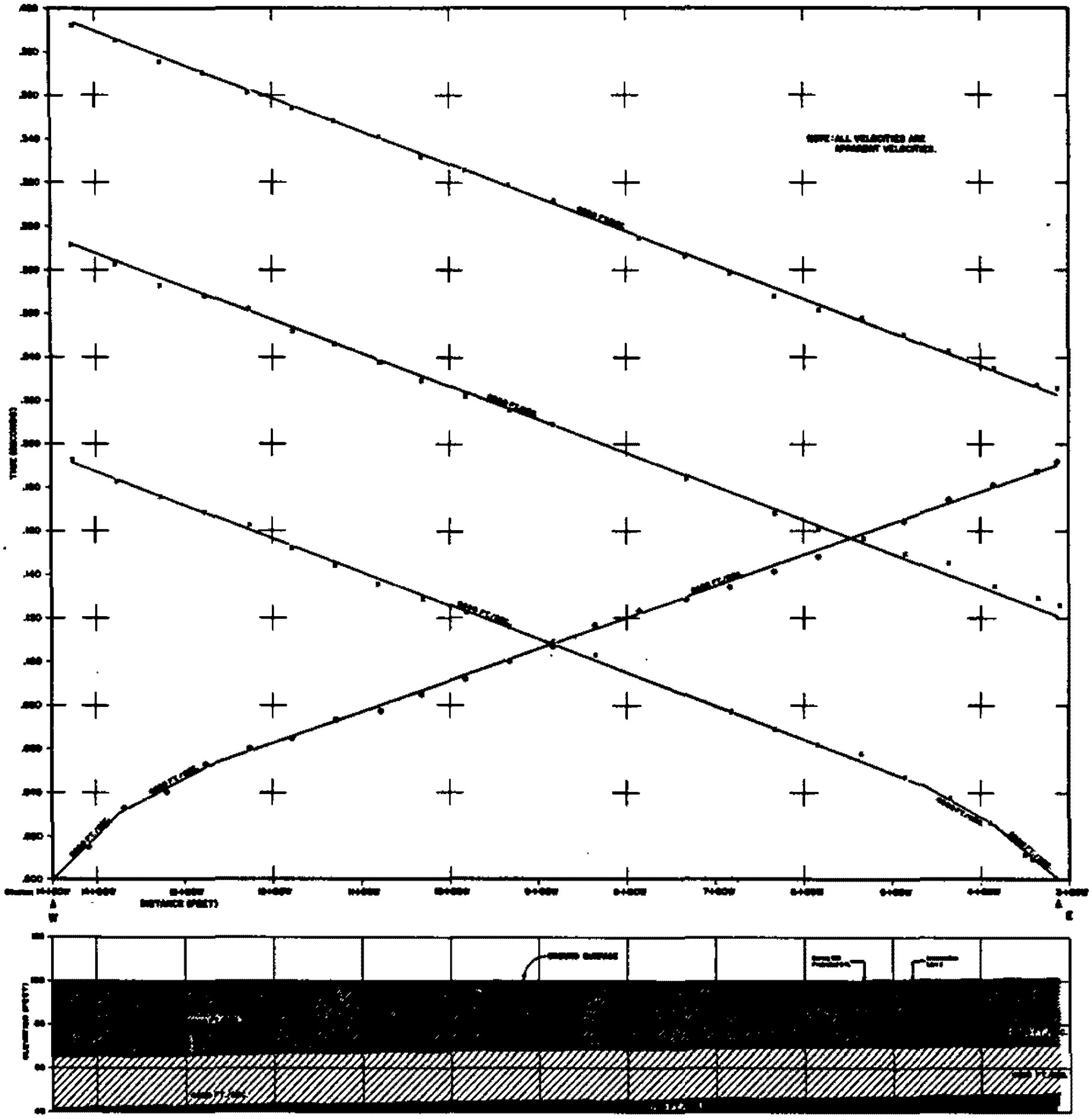
REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOG BORINGS

UPDATED FSAR

Sheet 189 of 189
FIGURE 2.5-50



KEY:

VELOCITY 2500 TO 2900 FT./SEC.

VELOCITY 4600 FT./SEC.

VELOCITY 6700 FT./SEC.

REFERENCE:
PSE&G, MAY 23, 1974, REPORT, FOUNDATION STUDIES, PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

NOTES:

1) Δ= SHOT POINT POSITION.

2) TIME DISTANCE PLOTS SHOW INFORMATION COLLECTED FROM SHOT POINTS MADE AT SEVERAL LOCATIONS ALONG A SEISMIC LINE. FOR CLARIFICATION, TWO PLOT SYMBOLS HAVE BEEN USED TO INDICATE THE ORIGIN OF THE ENERGY ON THE REGULAR REFRACTION LINES, (•) FROM THE LEFT, AND (X) FROM THE RIGHT.

3) THE SUBSURFACE CONDITIONS SHOWN REPRESENT OUR EVALUATION OF THE MOST PROBABLE SUBSURFACE CONDITIONS BASED ON OUR INTERPRETATION OF PRESENTLY AVAILABLE DATA. SOME VARIATIONS FROM THESE CONDITIONS MUST BE EXPECTED.

4) FOR A FULL UNDERSTANDING OF THE INFORMATION PRESENTED ON THIS PLATE, THE TEXT OF THE REPORT MUST BE READ.

5) ALL VELOCITIES SHOWN ARE APPARENT COMPRESSIONAL WAVE VELOCITIES.

6) CROSS-SECTION HAS A VERTICAL EXAGGERATION OF 2½:1.

7) LINE 1 IS APPROXIMATELY ALONG COORDINATE 1400 NORTH, BUT IS ACTUALLY ALONG A LINE N86°W.

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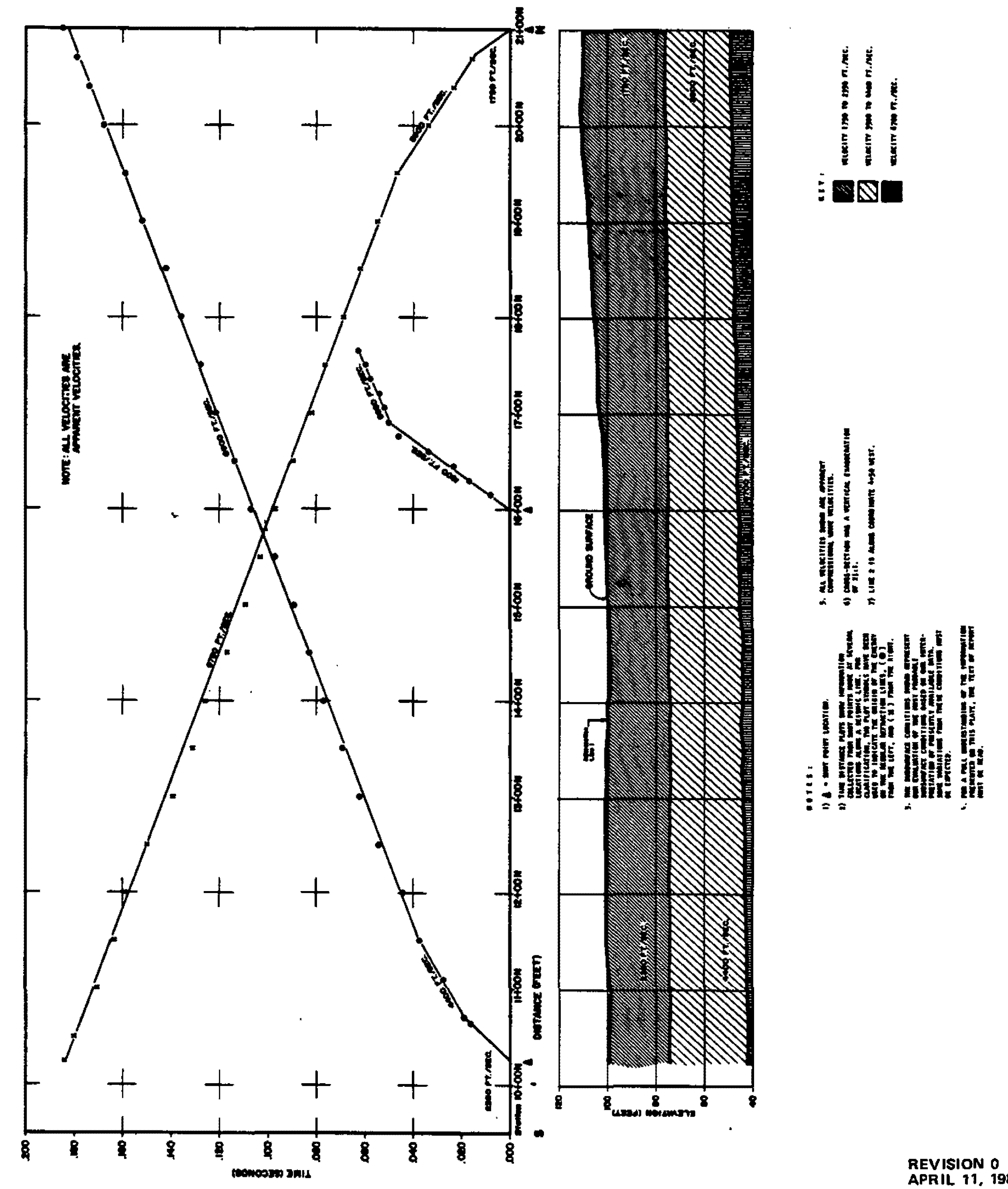
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

SEISMIC REFRACTION SURVEY
SEISMIC LINE 1

UPDATED FSAR

FIGURE 2.5-51



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APRIL 11, 1988

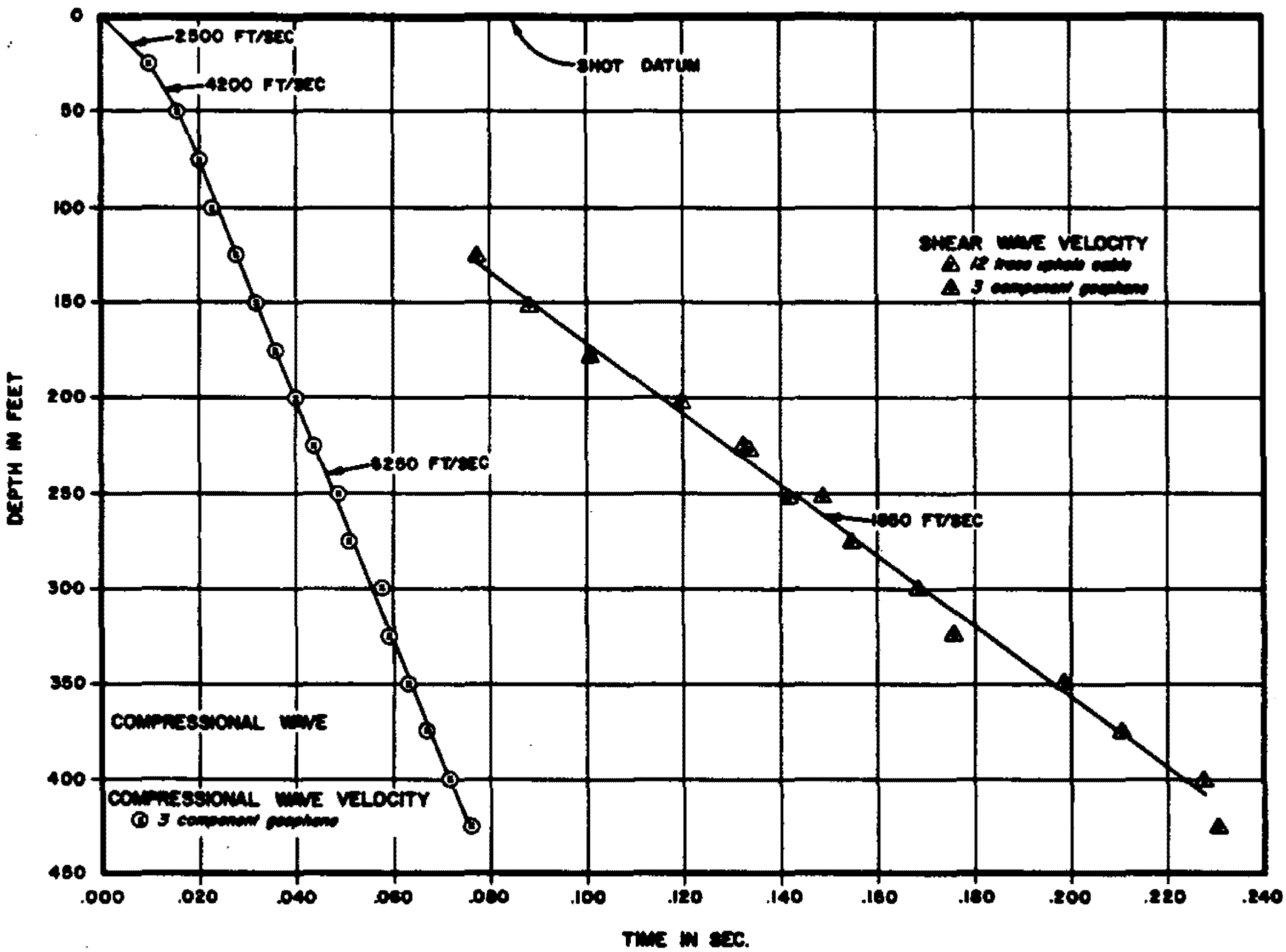
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

SEISMIC REFRACTION SURVEY
SEISMIC LINE 2

REFERENCE:
PSE&G, MAY 23, 1974, REPORT, FOUNDATION STUDIES, PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY, REPORT PREPARED BY DAMES & MOORE.

UPDATED FSAR

FIGURE 2.5-52



REFERENCE:
PSE&G, MAY 23, 1974, REPORT, FOUNDATION STUDIES, PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY, REPORT PREPARED BY DAMES & MOORE.

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

UPHOLE COMPRESSION AND SHEAR WAVE VELOCITY SURVEY

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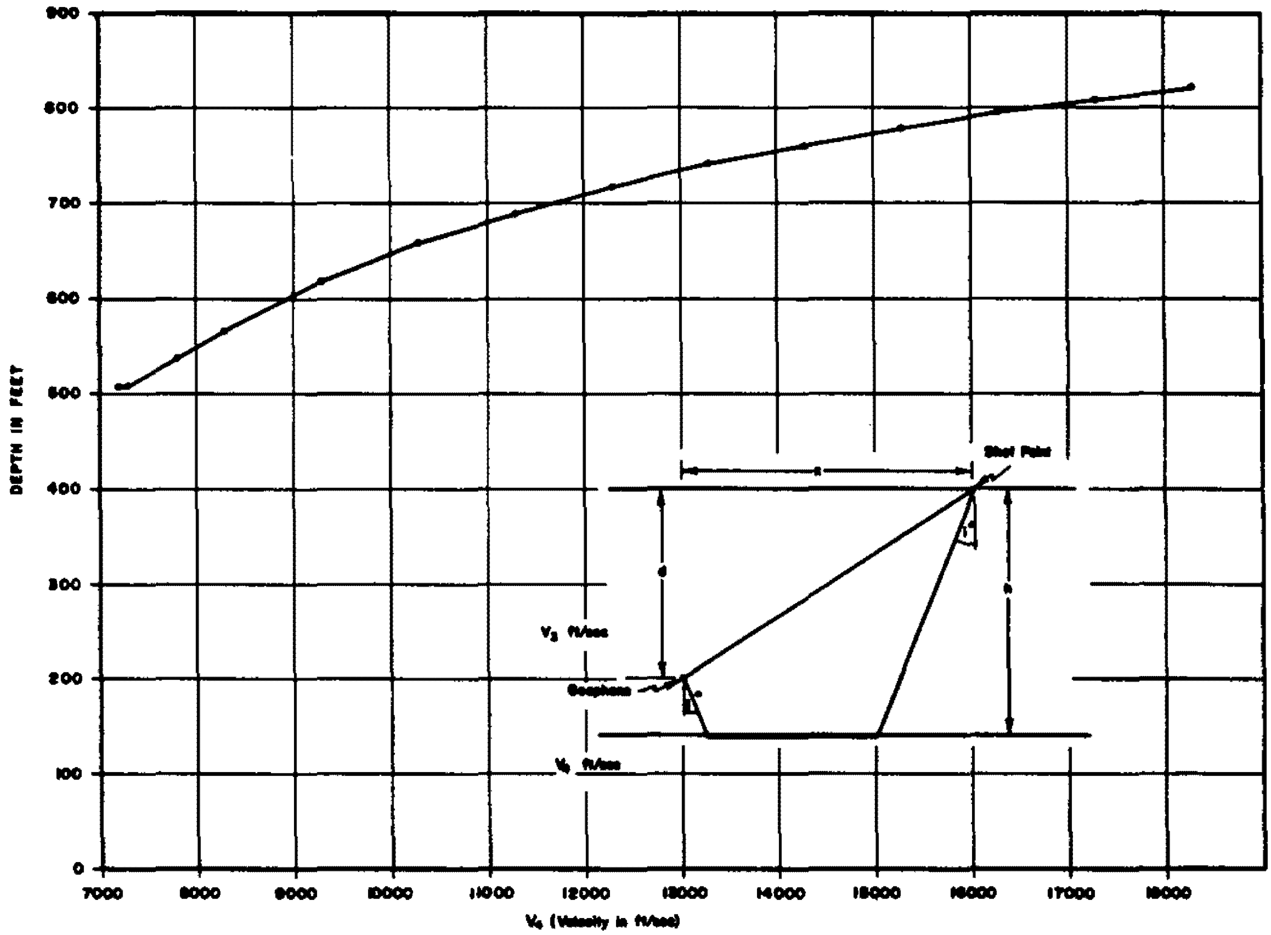
FIGURE 2.5-53



NOTES:

1. APPROXIMATION OF EFFECTIVE DEPTH OF INVESTIGATION FOR GEOPHONE @ 418 FEET IN BORING 201 AND SHOT AT 30 FOOT DEPTH, OFFSET 1580 FEET EAST OF BORING 201
2. ASSUMPTIONS: a) FLAT REFRACTOR
   b) 0.005 SECONDS EARLY ARRIVAL WOULD BE INTERPRETED AS A DEEP REFRACTOR
   FORMULA FOR THIS GEOMETRIC CONFIGURATION:

$$d_1 = \frac{(X^2 + d^2)^{\frac{1}{2}}}{2 \cos i} + \tan i \, (0.0025 - \frac{X}{2}) + \frac{d}{2}$$

3. SEE DIAGRAM FOR PARAMETERS.
4. ADD SHOT POINT DEPTH TO d_1 IF $d_{SHOT\ POINT} > 0$
   V_3 = 6800 FT/SEC

REFERENCE:

PSE&G, MAY 23, 1974, REPORT, FOUNDATION STUDIES, PROPOSED HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY, REPORT PREPARED BY DAMES & MOORE.

REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

THEORETICAL VARIATION OF EFFECTIVE DEPTH OF INVESTIGATION WITH VELOCITY CONTRAST

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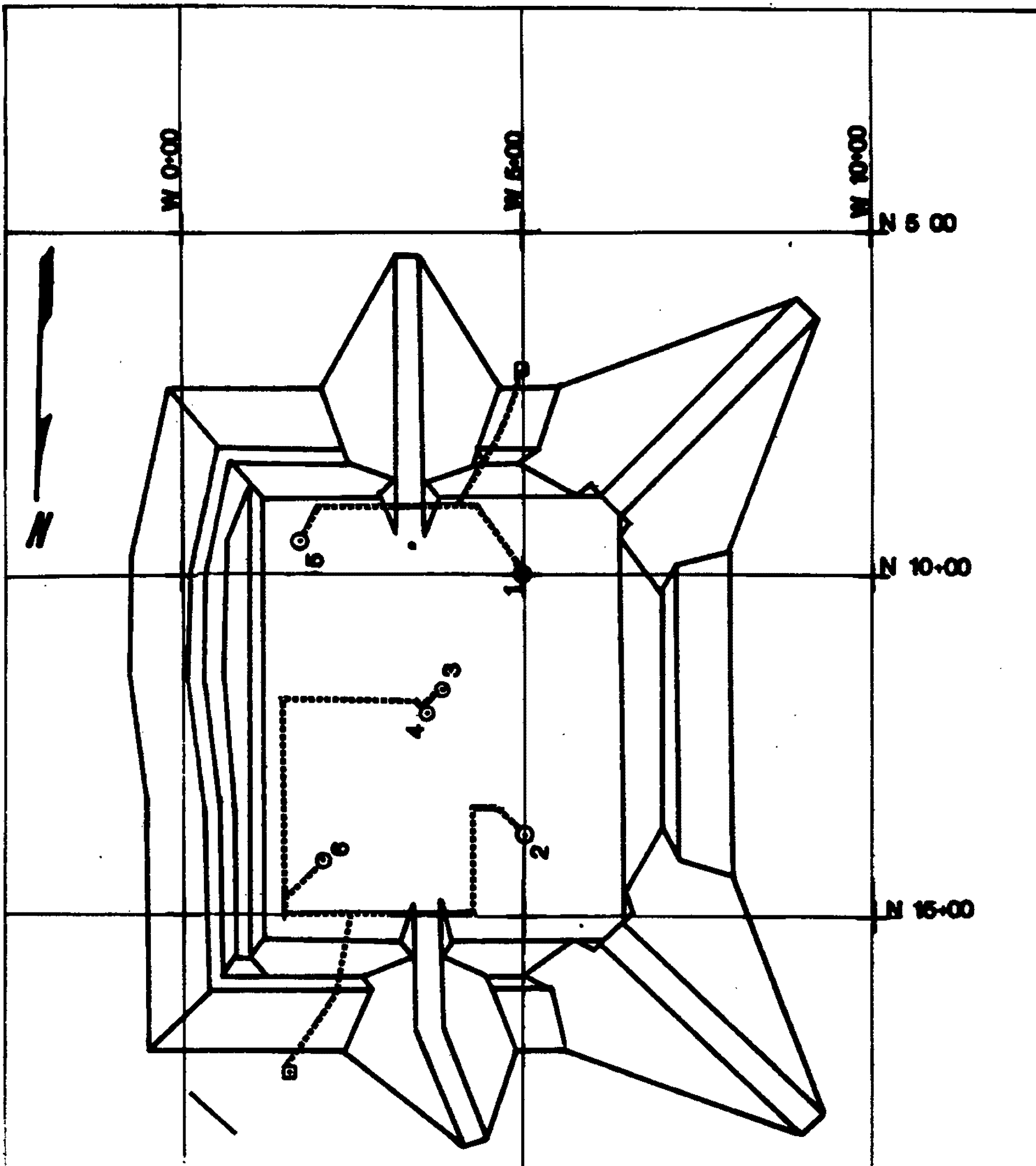
FIGURE 2.5-54



REFERENCE:
PSE&G, AUGUST 1977, FINAL REPORT, HEAVE/SETTLEMENT MEASUREMENT PROGRAM, HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

LEGEND

- ⊙ Location of Extensometer
- ■ Location of Instrumentation Terminal Building
- ----- Location of Extensometer cables.

Note: All locations are approximate only.

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APRIL 11, 1988

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HOPE CREEK NUCLEAR GENERATING STATION

LOCATIONS OF EXTENSOMETERS

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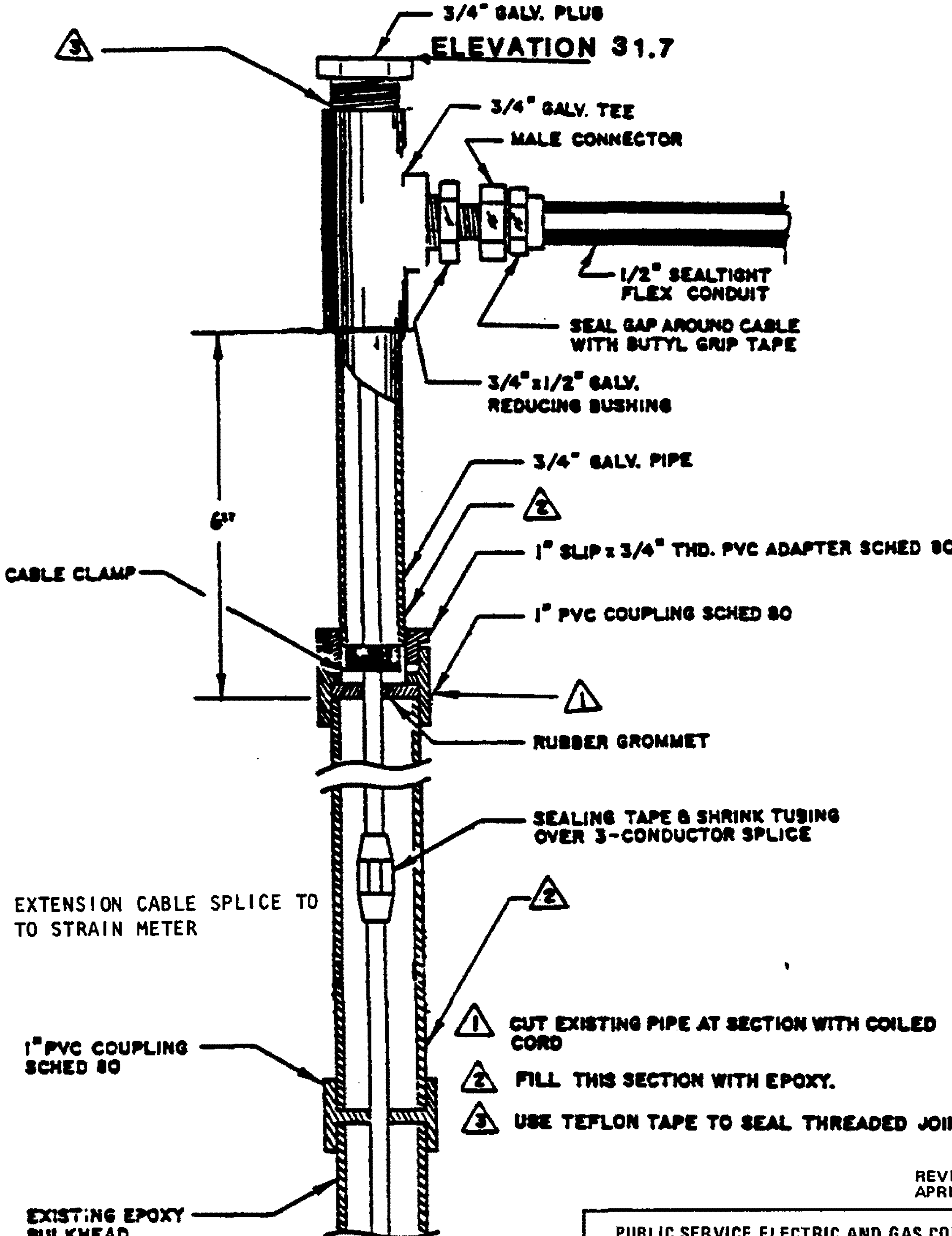
FIGURE 2.5-55



1 CUT EXISTING PIPE AT SECTION WITH COILED CORD

2 FILL THIS SECTION WITH EPOXY.

3 USE TEFLON TAPE TO SEAL THREADED JOINTS.

REFERENCE:
PSE&G, AUGUST 1977, FINAL REPORT, HEAVE/SETTLEMENT MEASUREMENT PROGRAM, HOPE CREEK GENERATING STATION, LOWER ALLOWAYS CREEK TOWNSHIP, NEW JERSEY. REPORT PREPARED BY DAMES & MOORE.

BASED ON SLOPE INDICATOR CO.
DRAWING NO. 51751-1 (1/20/77)
NOT TO SCALE

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

EXTENSOMETER NO. 1 (AS-BUILT)

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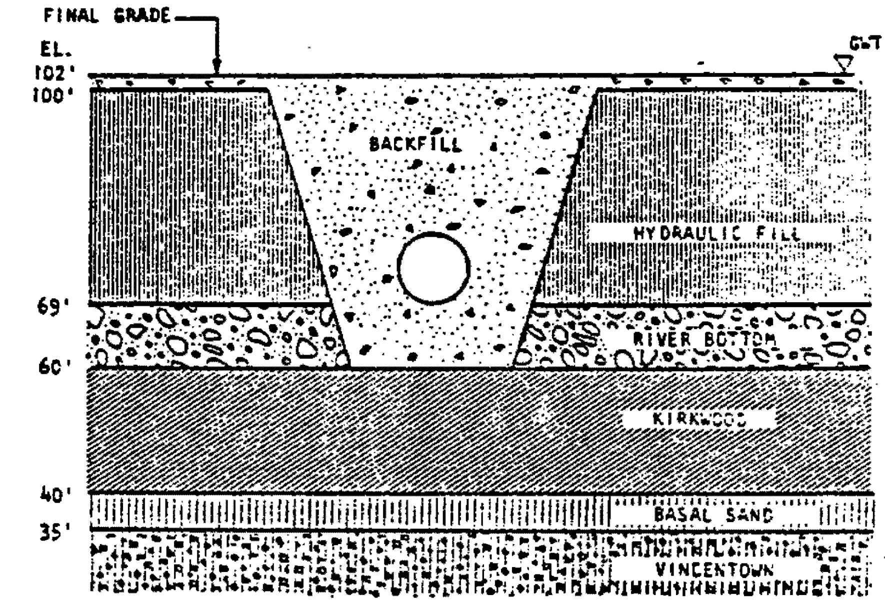
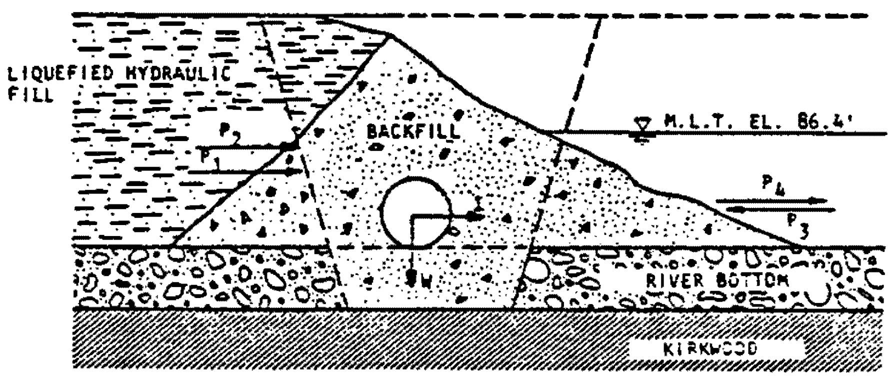
FIGURE 2.5-56



EMBEDDED PIPE



POSTULATED LIQUEFIED HYDRAULIC FILL

KEY:

- I INERTIA FORCE
- P_1, P_3 HYDROSTATIC FORCE
- P_2, P_4 HYDRODYNAMIC FORCE (FROM WESTERGAARD)
- W DEAD LOAD

NOTE:

THE ABOVE CONFIGURATIONS ARE INTENDED TO ILLUSTRATE THE POSTULATED FAILURE MECHANISM AND ARE NOT INTENDED FOR DESIGN

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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

PIPELINE STABILITY DURING EARTHQUAKE EXCITATION

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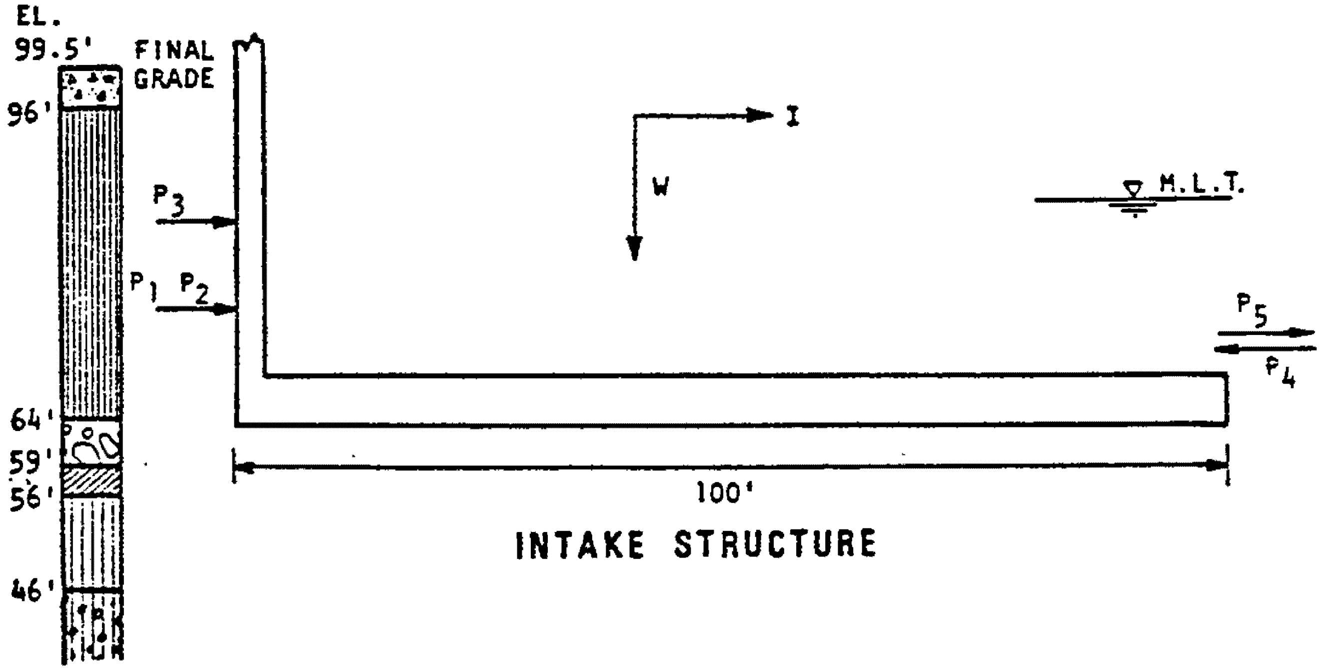
FIGURE 2.5-57



KEY:

	BACKFILL	I	INERTIA FORCE
	HYDRAULIC FILL	P_1, P_4	HYDROSTATIC FORCE
	RIVER BOTTOM SANDS	P_2	STATIC ACTIVE SOIL FORCE
	KIRKWOOD CLAYS	P_3	DYNAMIC INCREMENTAL FORCE FROM MATSUO-OHARA
	BASAL SANDS	P_5	HYDRODYNAMIC FORCE FROM WESTERGAARD
	VINCENTOWN SANDS	W	TOTAL LOAD

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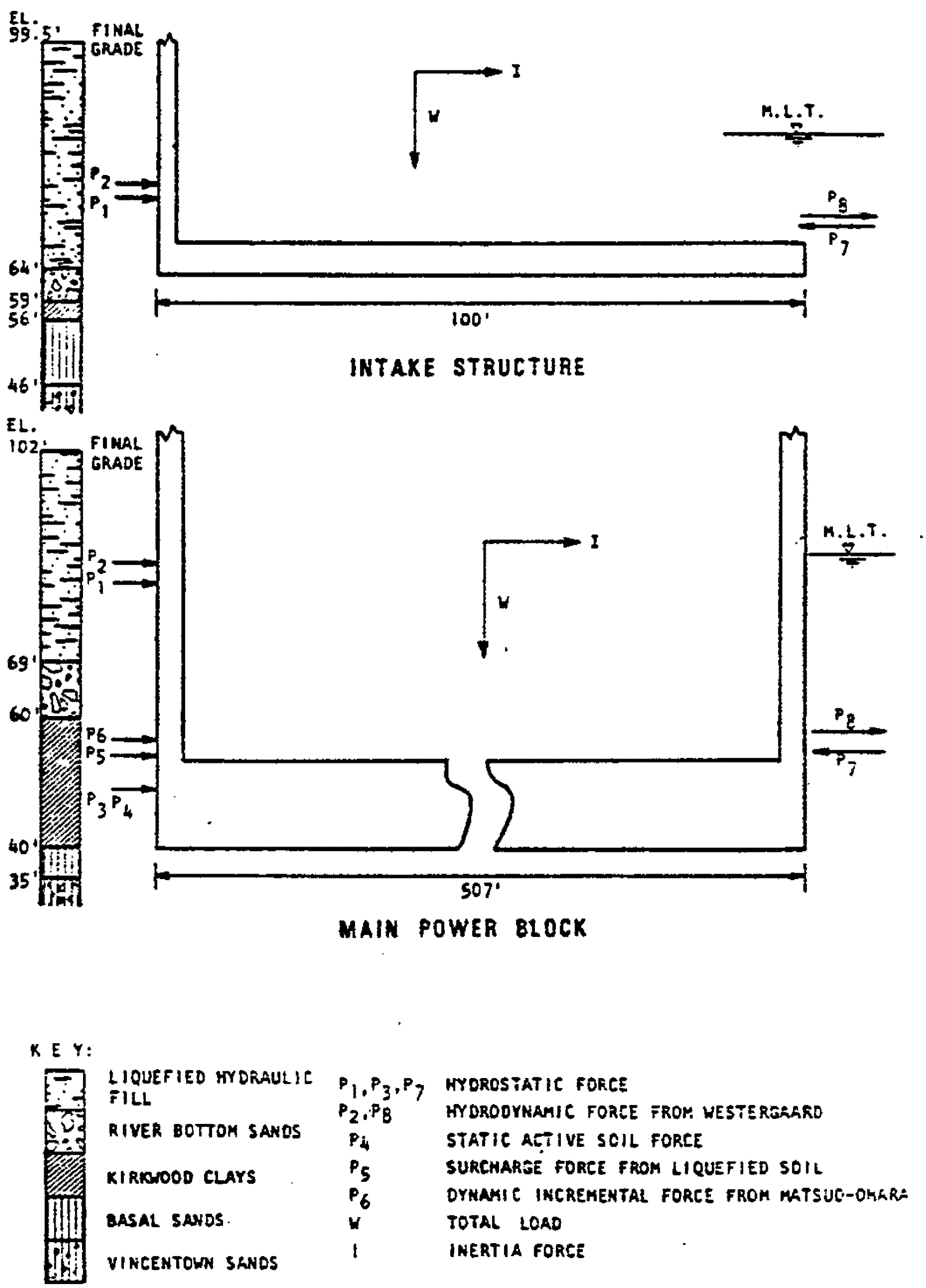
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

MODEL USED IN THE ANALYSIS FOR INTAKE STRUCTURE WITHOUT LIQUEFACTION

UPDATED FSAR

FIGURE 2.5-58



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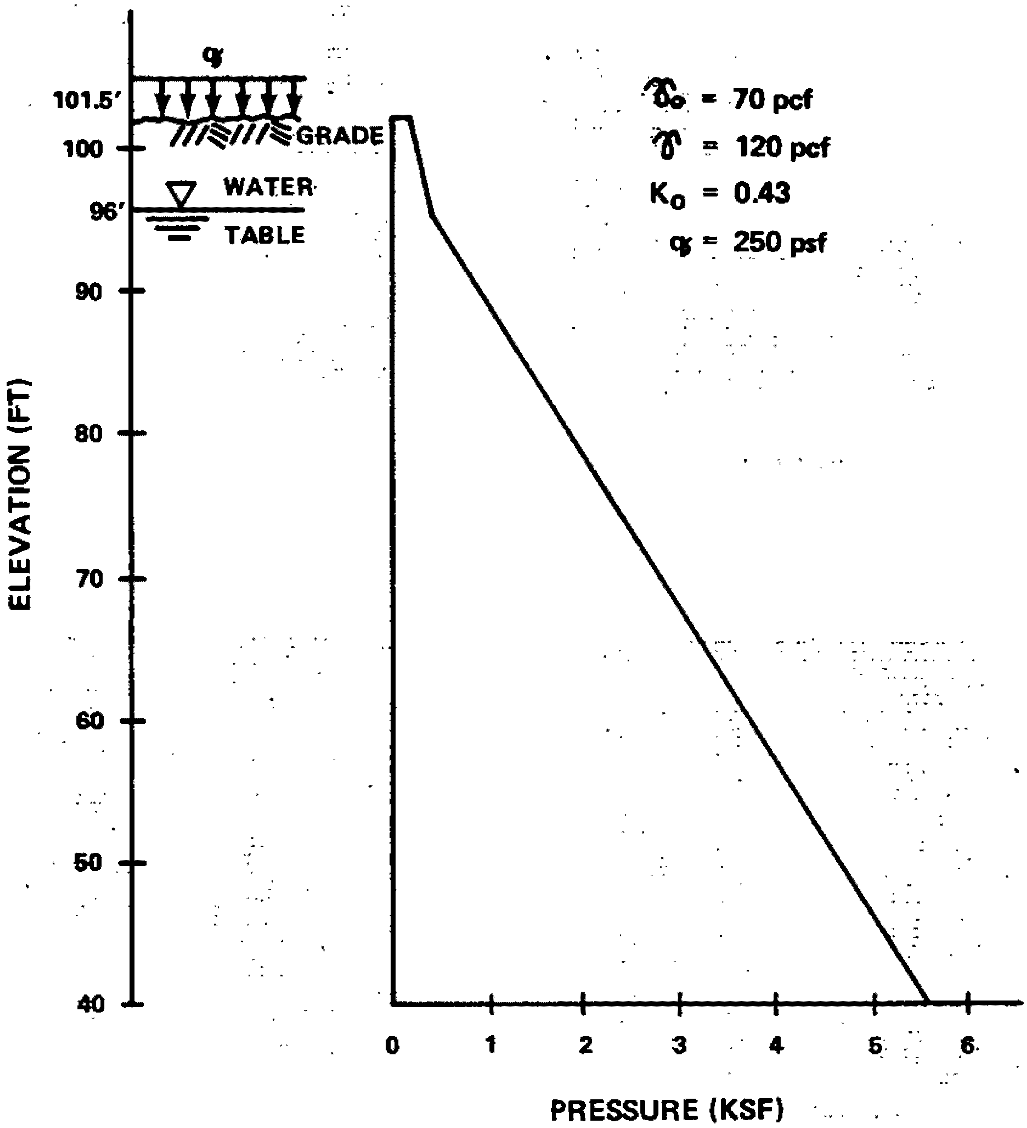
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

MODELS USED IN THE ANALYSIS FOR THE POWER BLOCK AND INTAKE STRUCTURE ASSUMING LIQUEFACTION

UPDATED FSAR

FIGURE 2.5-59



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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

STATIC EARTH PRESSURE DISTRIBUTION

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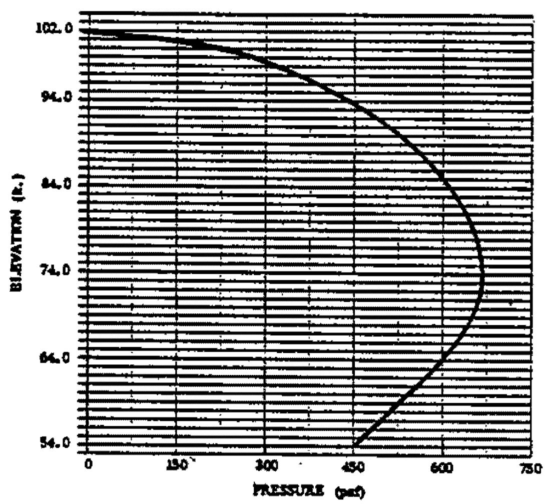
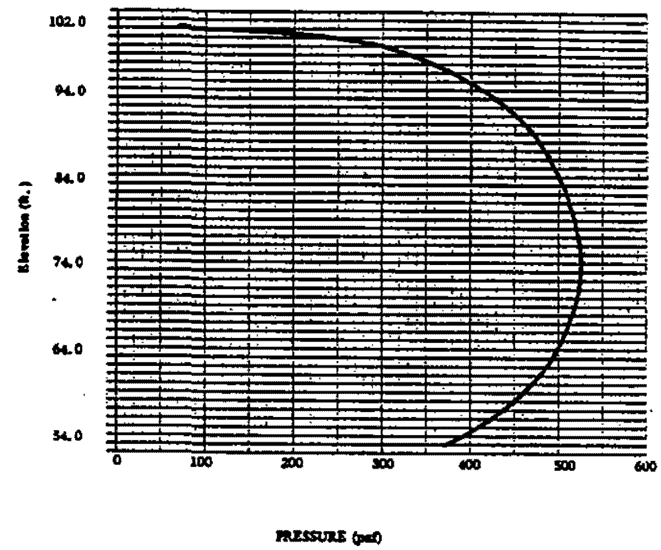
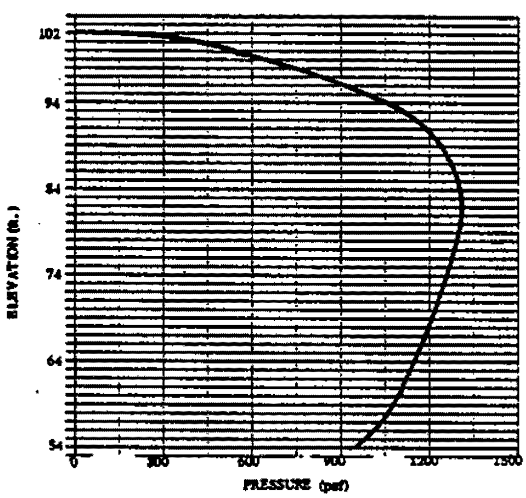
FIGURE 2.5-60



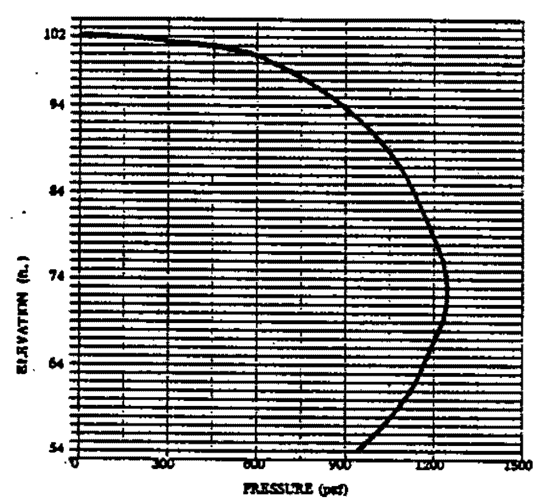
EAST-WEST WALL (OBE)



NORTH-SOUTH WALL (OBE)



EAST-WEST WALL (SSE)



NORTH-SOUTH WALL (SSE)

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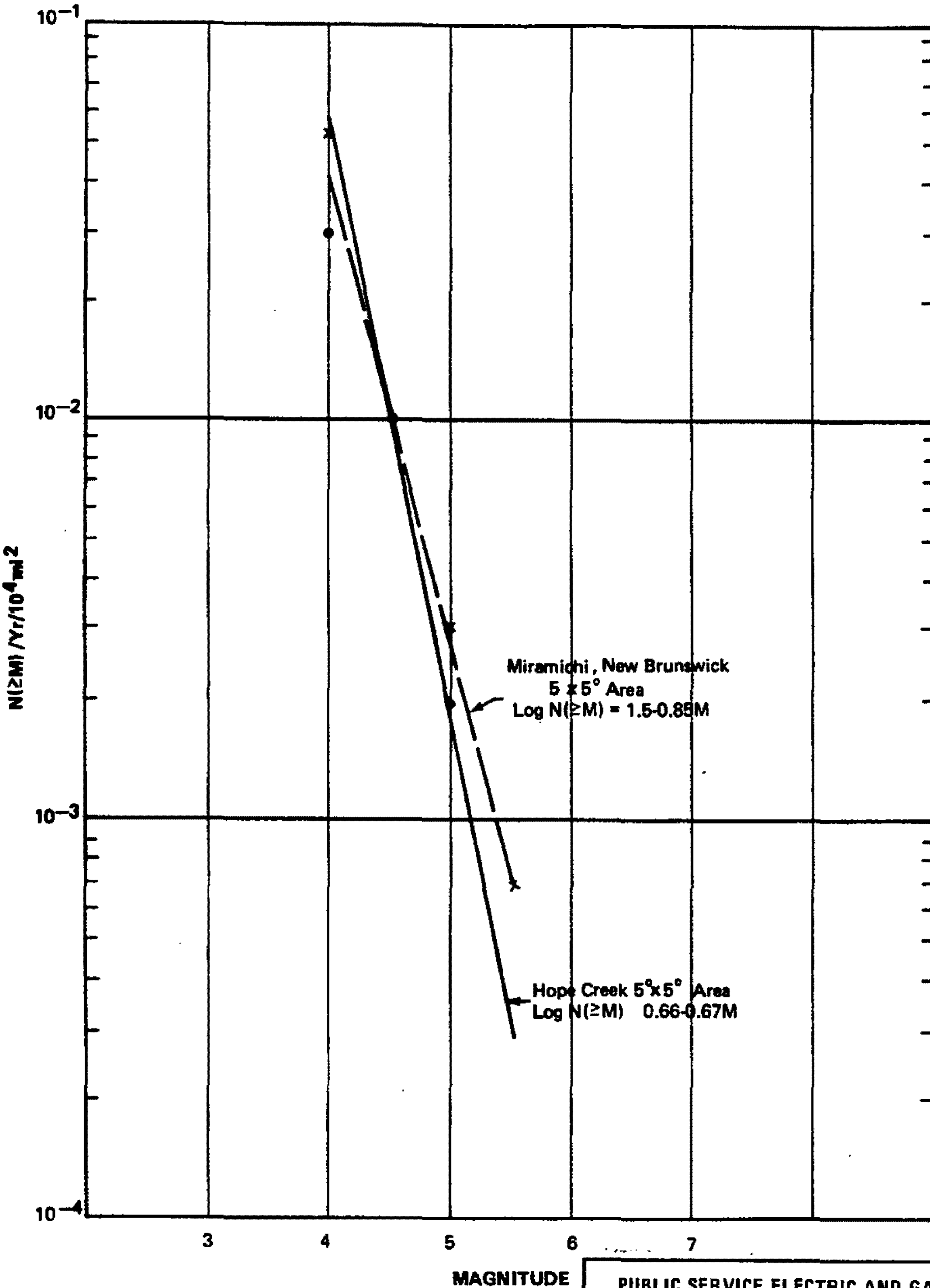
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY HOPE CREEK NUCLEAR GENERATING STATION
DYNAMIC EARTH PRESSURE DISTRIBUTION
UPDATED FSAR FIGURE 2.5-61



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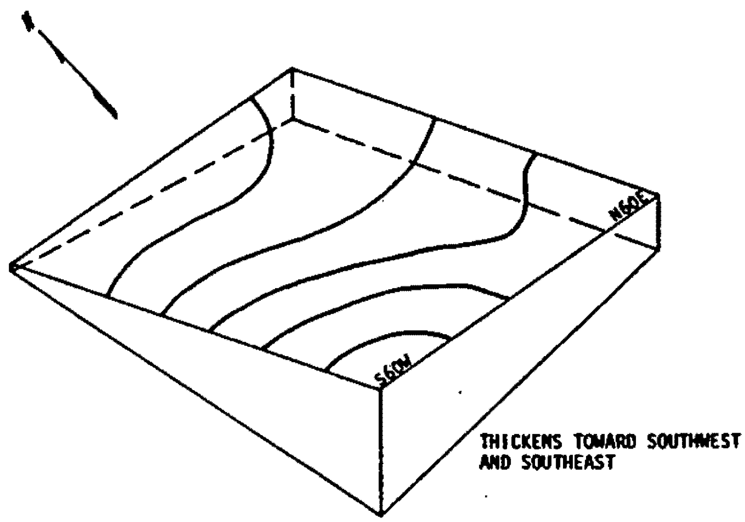
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

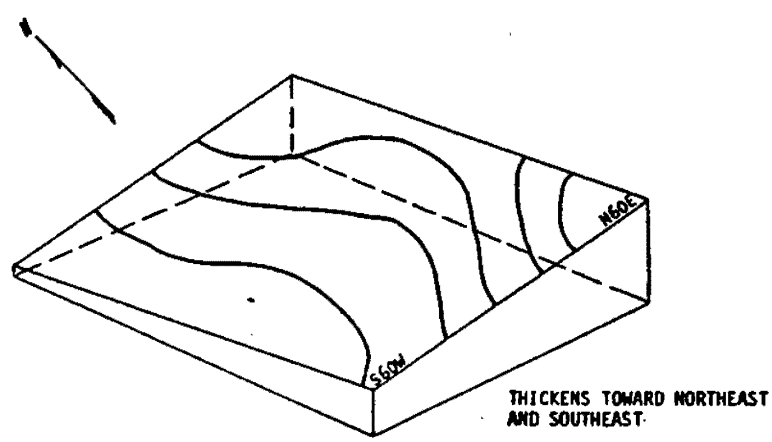
RECURRENCE CURVES

UPDATED FSAR

FIGURE 2.5-62



TERTIARY KIRKWOOD FORMATION



TERTIARY VINCENTOWN FORMATION

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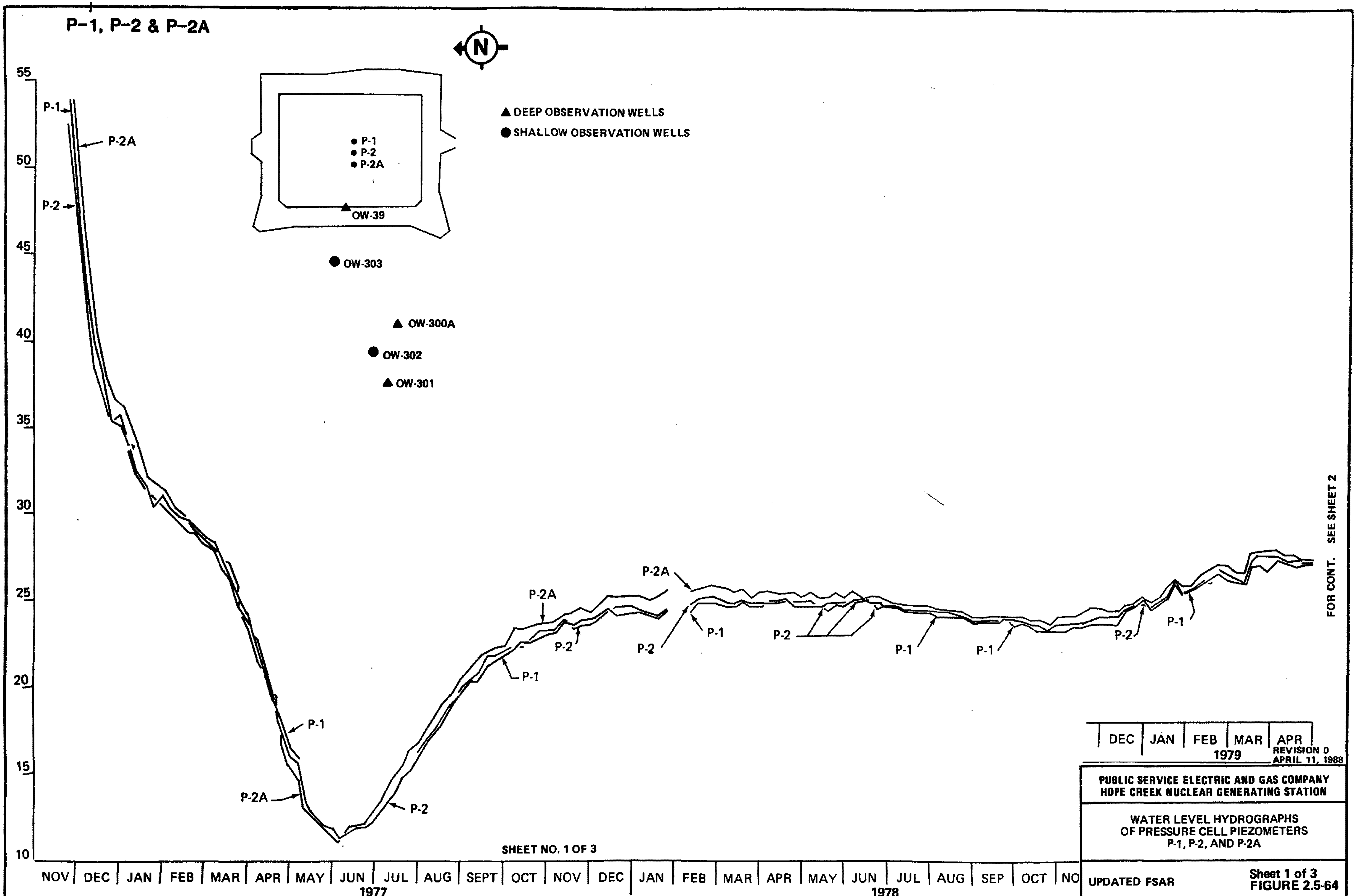
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY HOPE CREEK NUCLEAR GENERATING STATION	
ISOPACH FORM LINES	
UPDATED FSAR	FIGURE 2.5-63



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APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

WATER LEVEL HYDROGRAPHS
OF PRESSURE CELL PIEZOMETERS
P-1, P-2, AND P-2A

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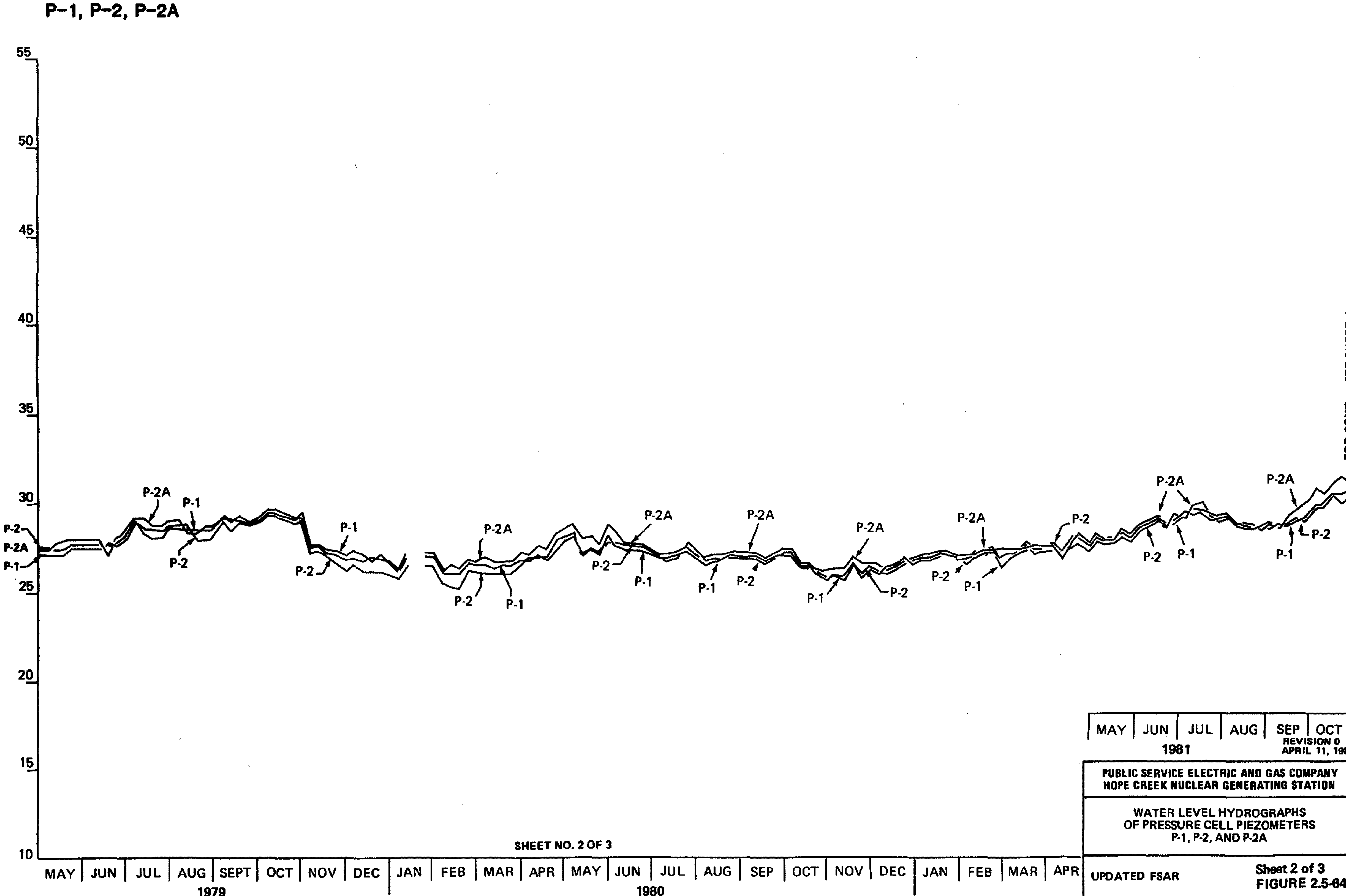
Sheet 1 of 3
FIGURE 2.5-64



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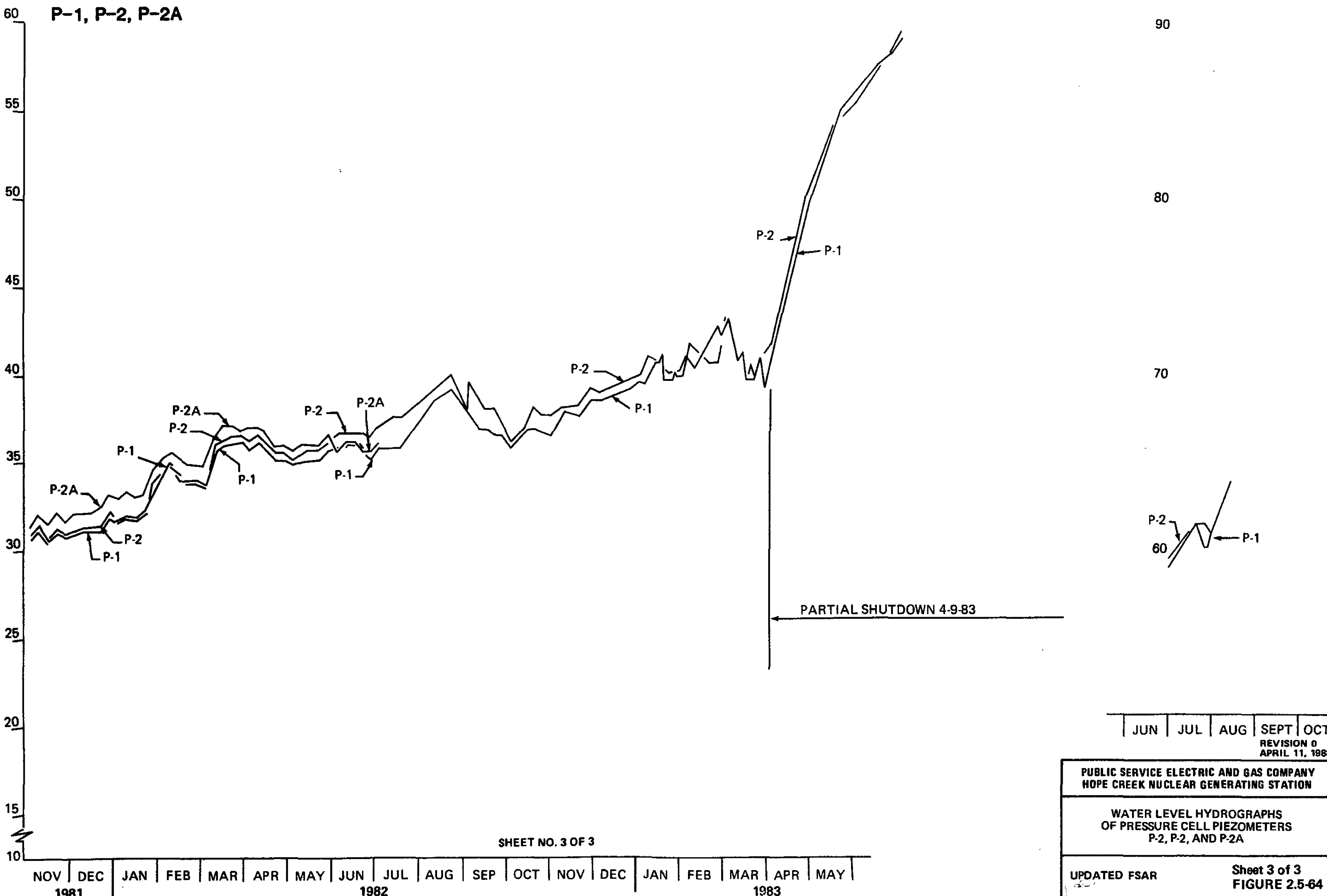
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

WATER LEVEL HYDROGRAPHS
OF PRESSURE CELL PIEZOMETERS
P-1, P-2, AND P-2A

UPDATED FSAR

Sheet 2 of 3
FIGURE 2.5-64



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APRIL 11, 1988

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HOPE CREEK NUCLEAR GENERATING STATION

WATER LEVEL HYDROGRAPHS
OF PRESSURE CELL PIEZOMETERS
P-2, P-2, AND P-2A

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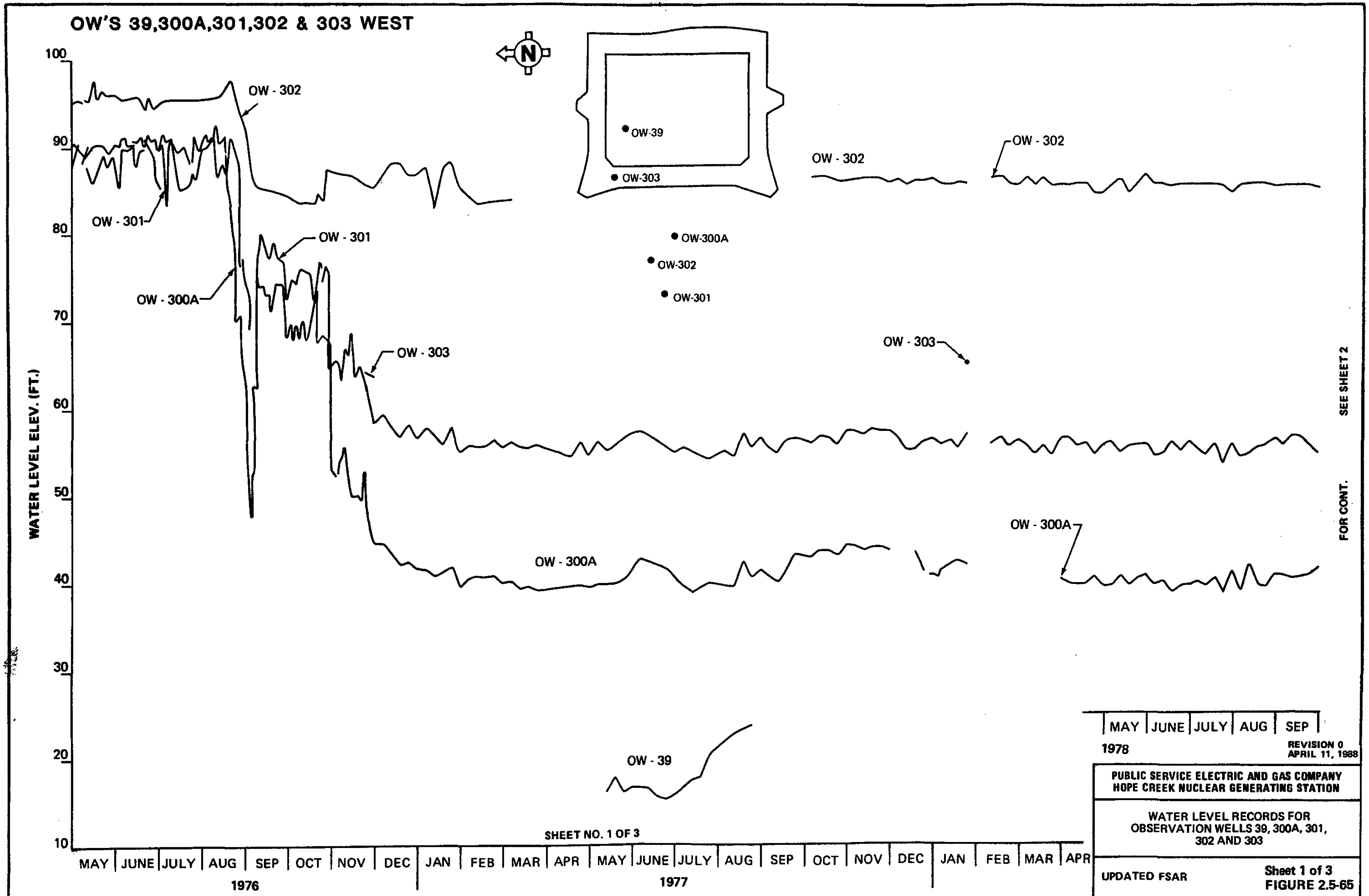
Sheet 3 of 3
FIGURE 2.5-64

OW'S 39,300A,301,302 & 303 WEST



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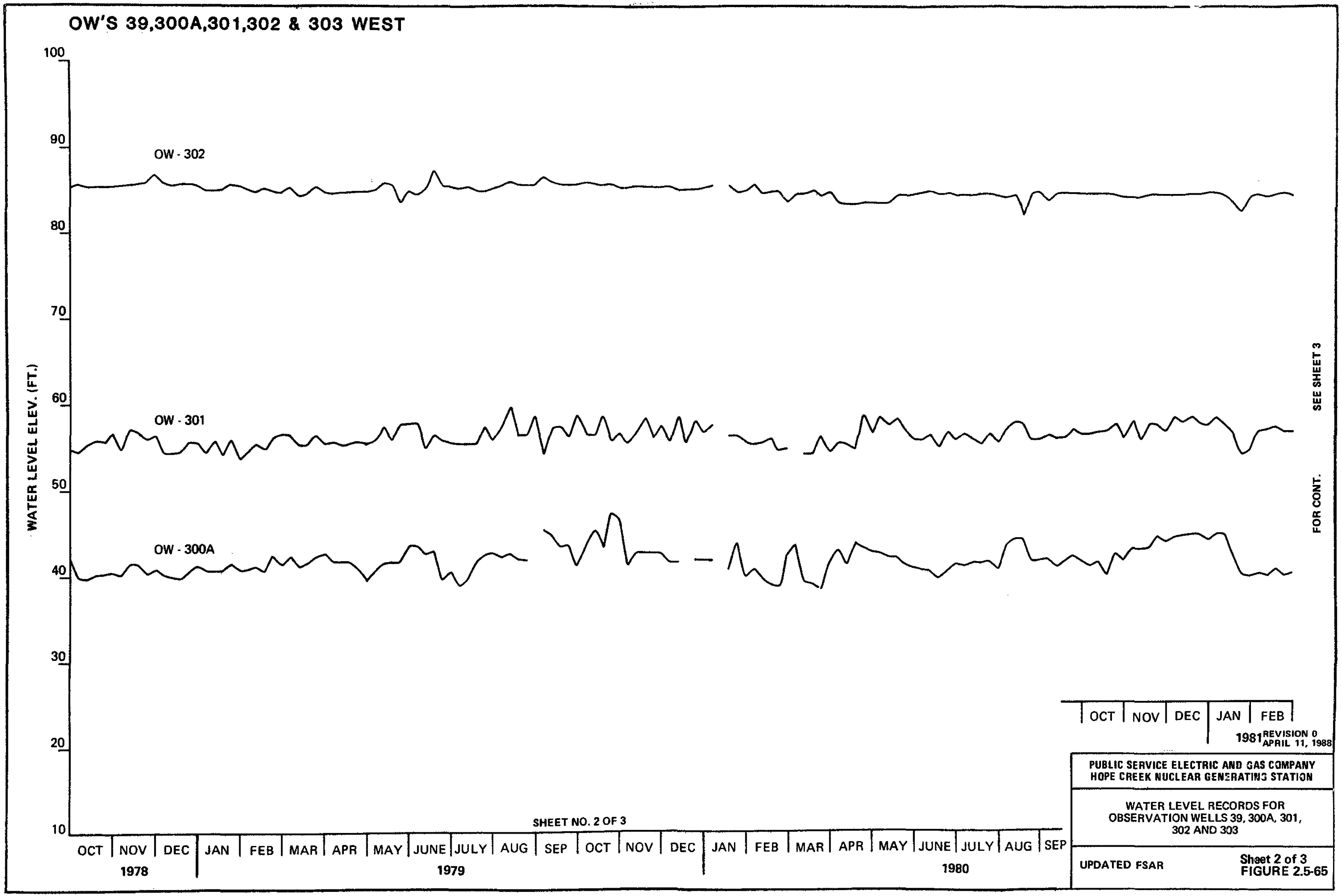
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

WATER LEVEL RECORDS FOR OBSERVATION WELLS 39, 300A, 301, 302 AND 303

UPDATED FSAR

Sheet 1 of 3
FIGURE 2.5-65



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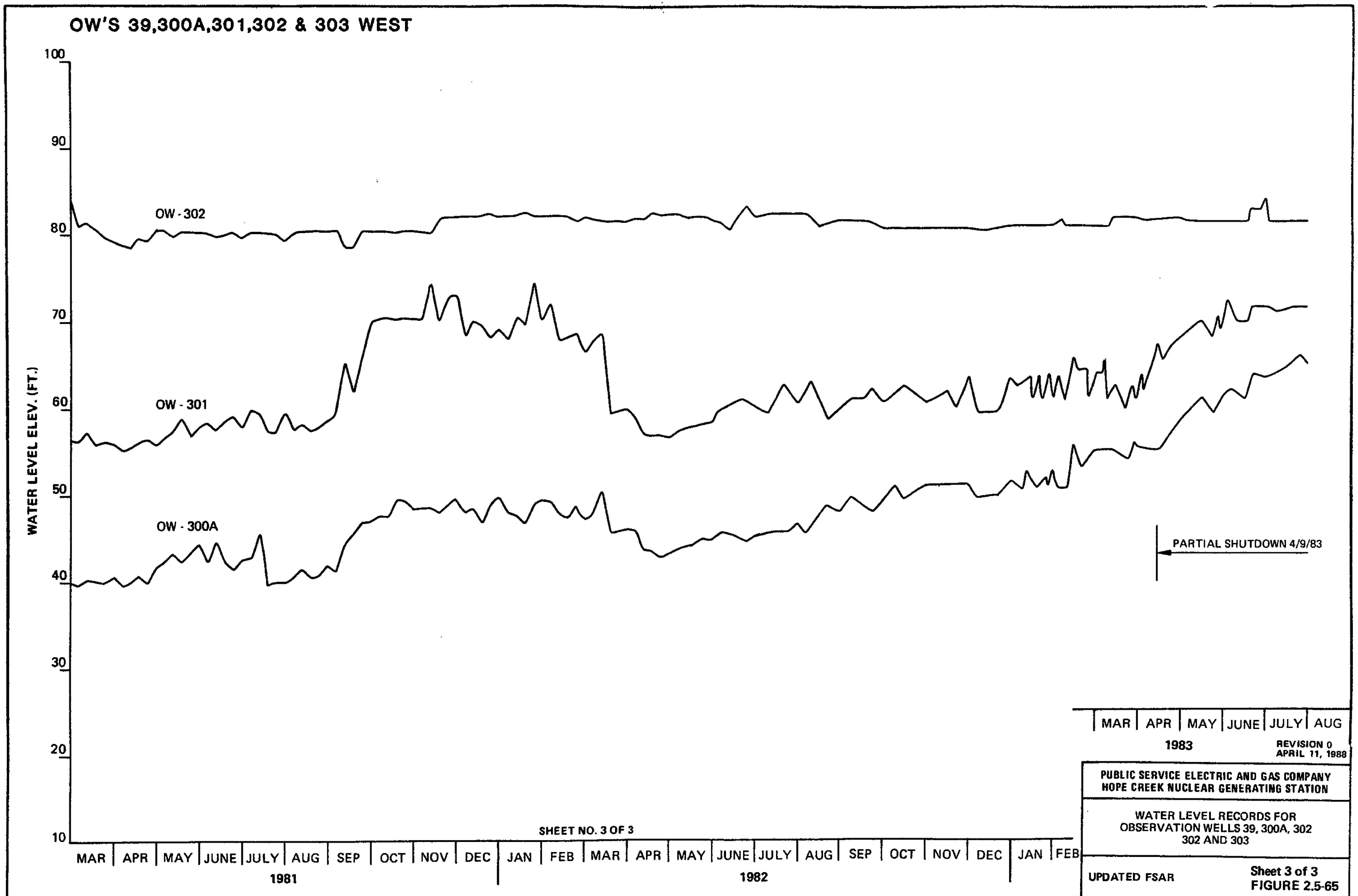
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

WATER LEVEL RECORDS FOR OBSERVATION WELLS 39, 300A, 301, 302 AND 303

UPDATED FSAR

Sheet 2 of 3
FIGURE 2.5-65



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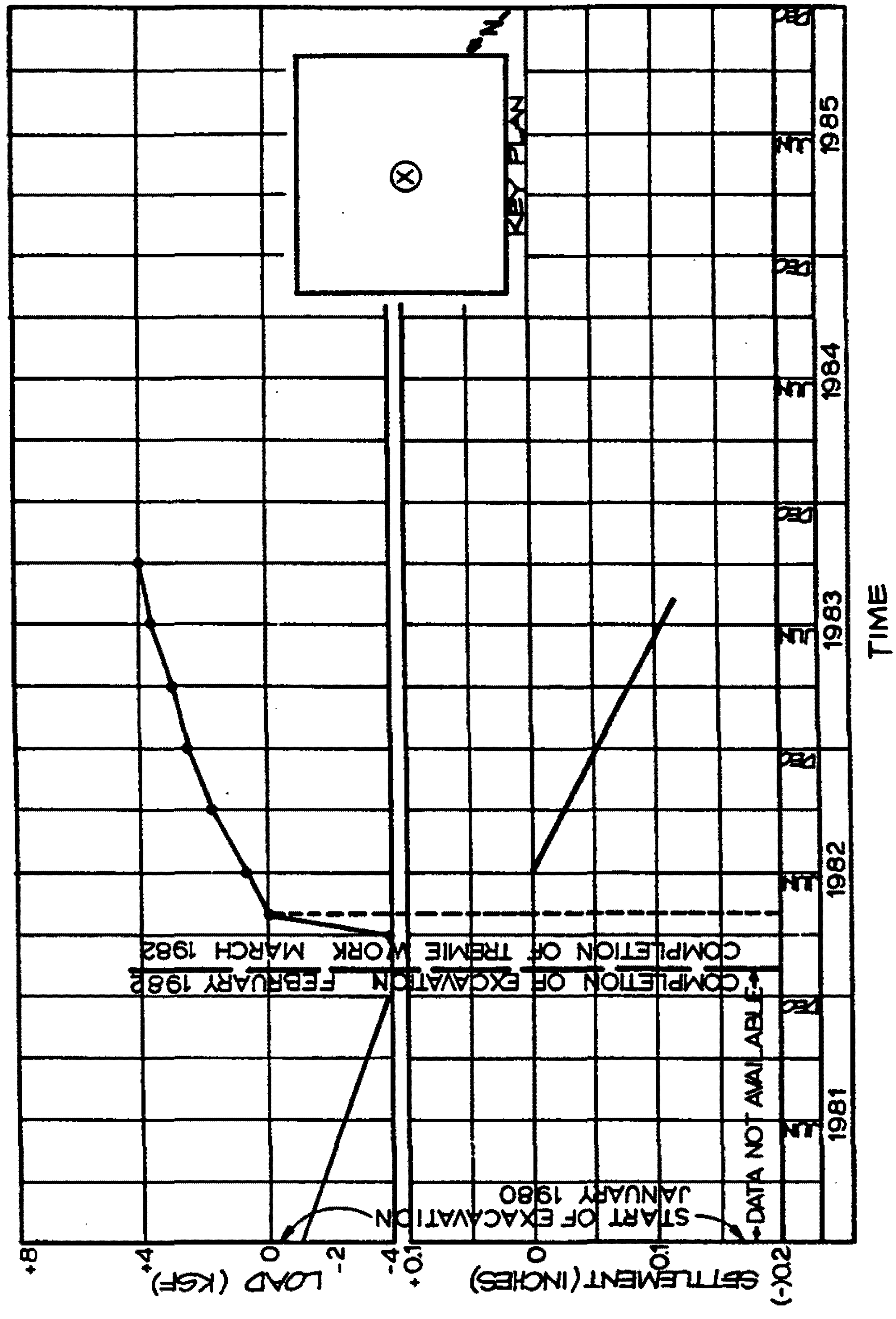
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

WATER LEVEL RECORDS FOR OBSERVATION WELLS 39, 300A, 302 302 AND 303

UPDATED FSAR

Sheet 3 of 3
FIGURE 2.5-65



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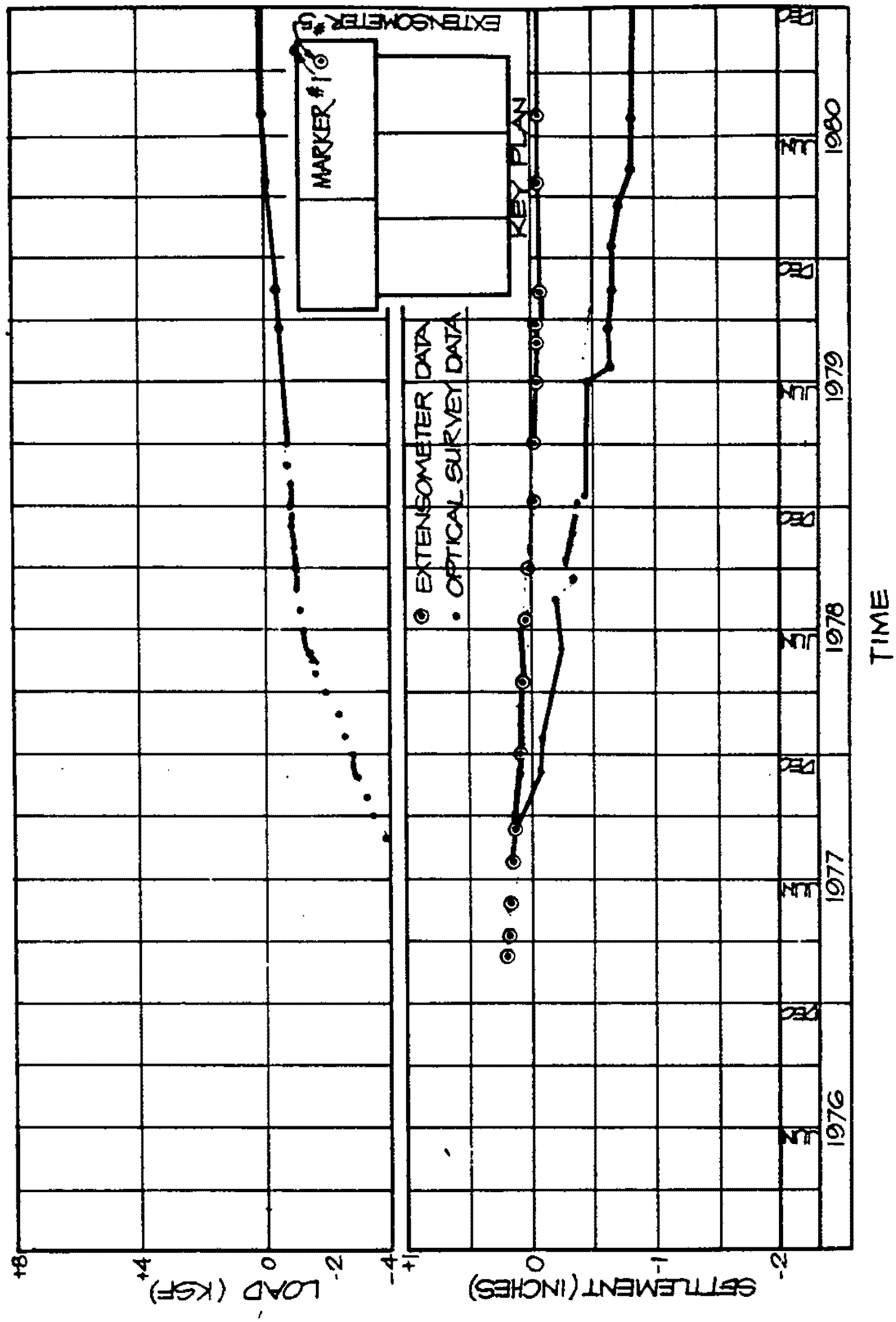
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR INTAKE STRUCTURE

UPDATED FSAR

FIGURE 2.5-66



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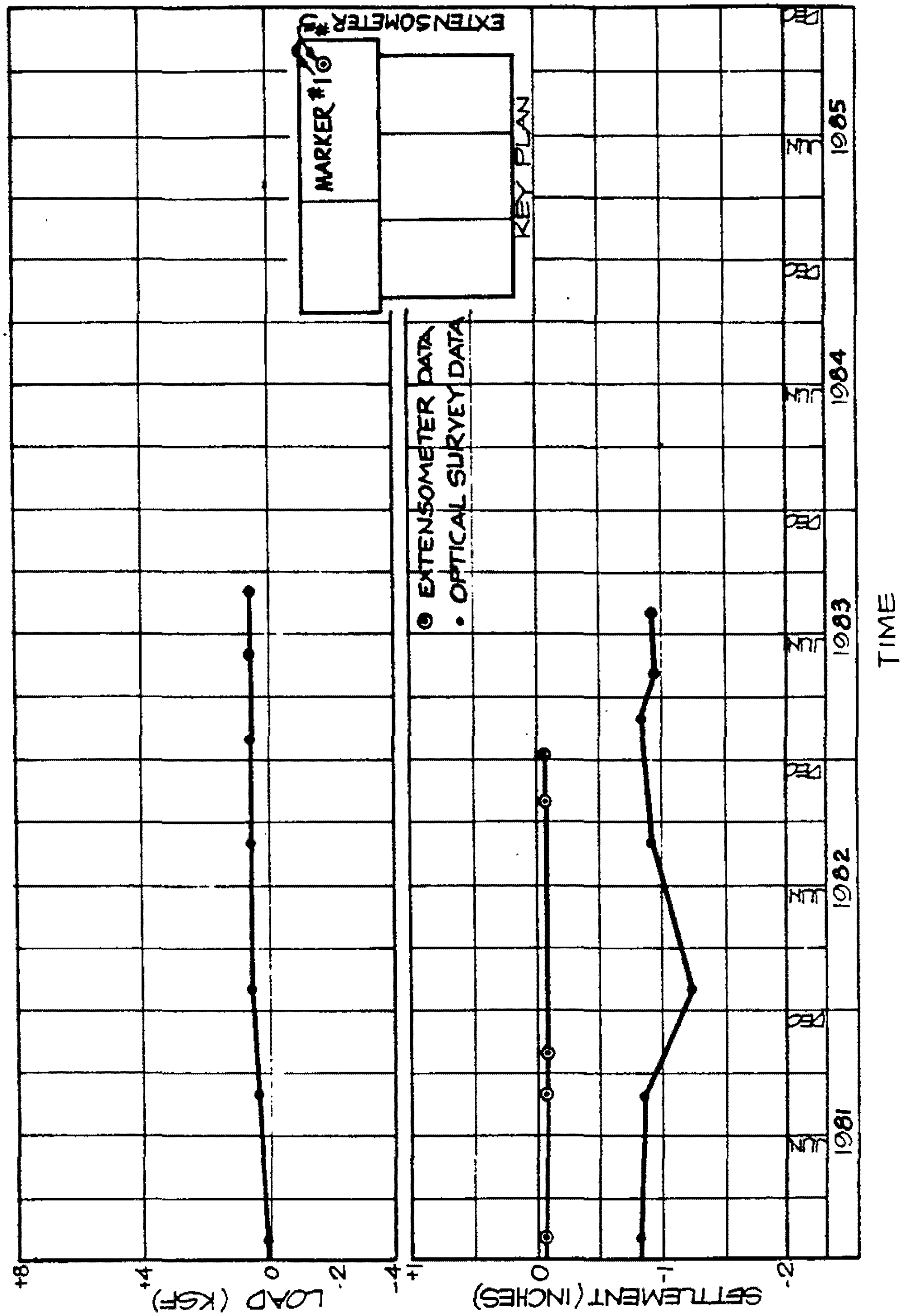
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 1

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-67



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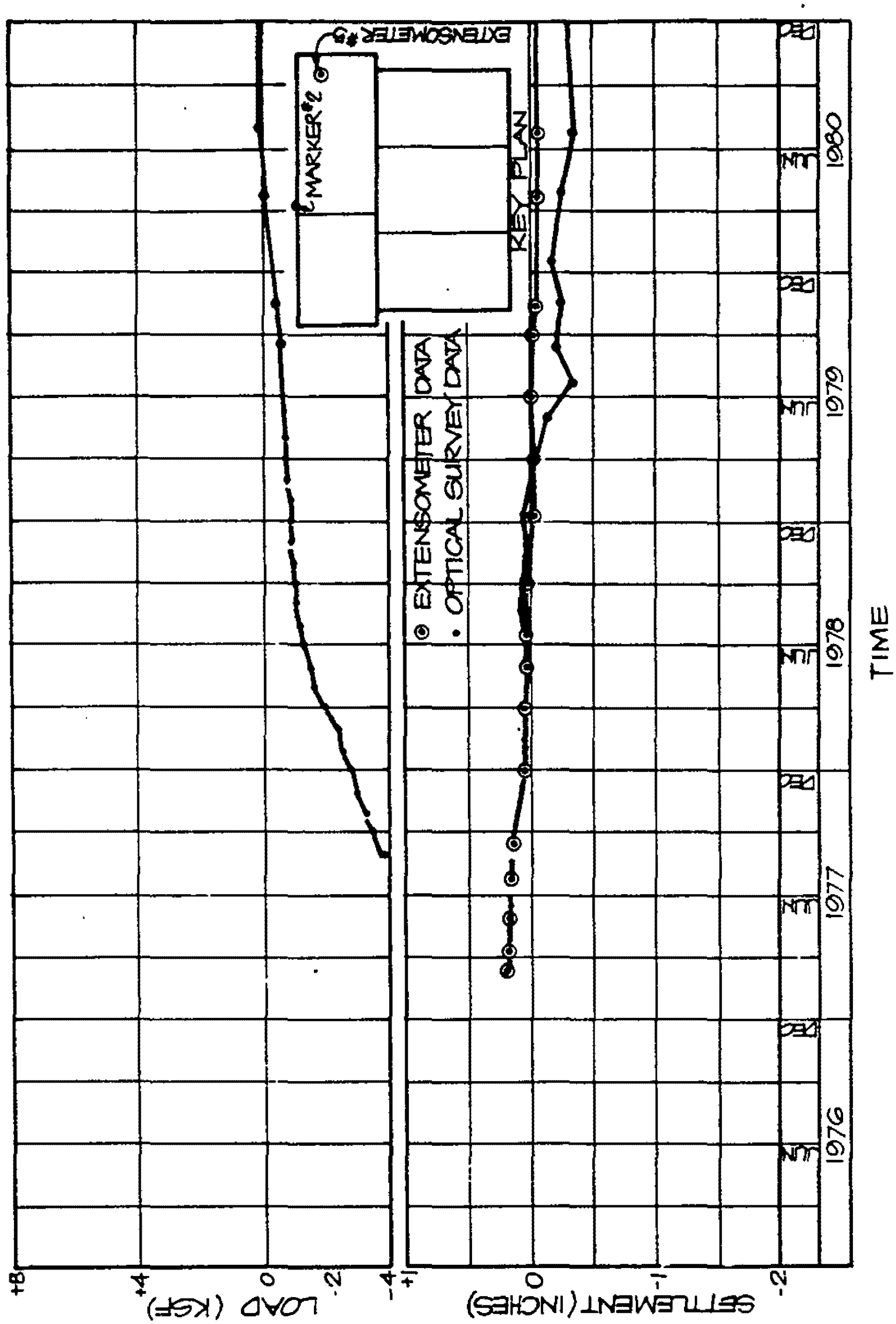
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 1

UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-67



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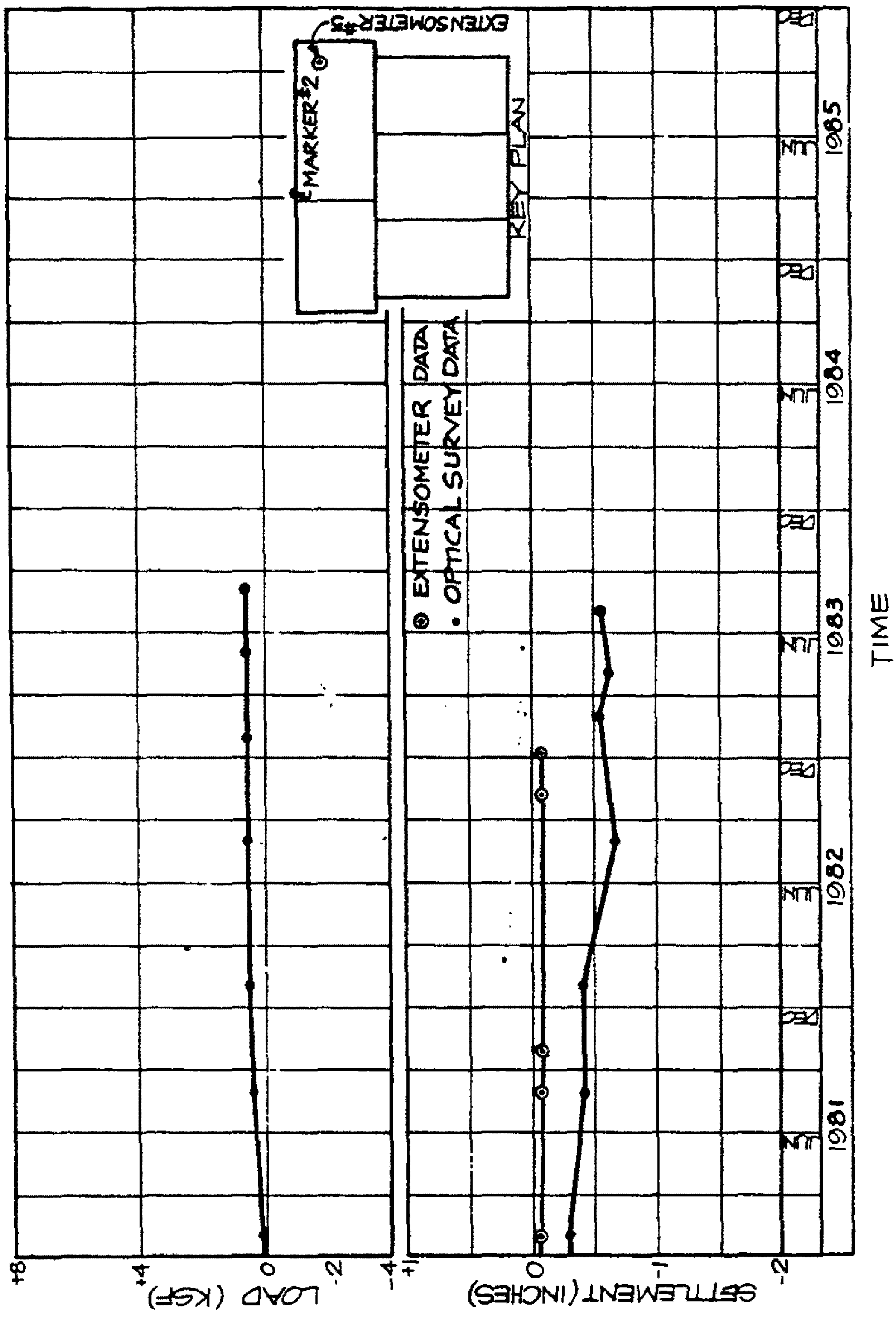
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY HOPE CREEK NUCLEAR GENERATING STATION	
LOAD/SETTLEMENT PLOT FOR MARKER NO. 2	
UPDATED FSAR	Sheet 1 of 2 FIGURE 2.5-68



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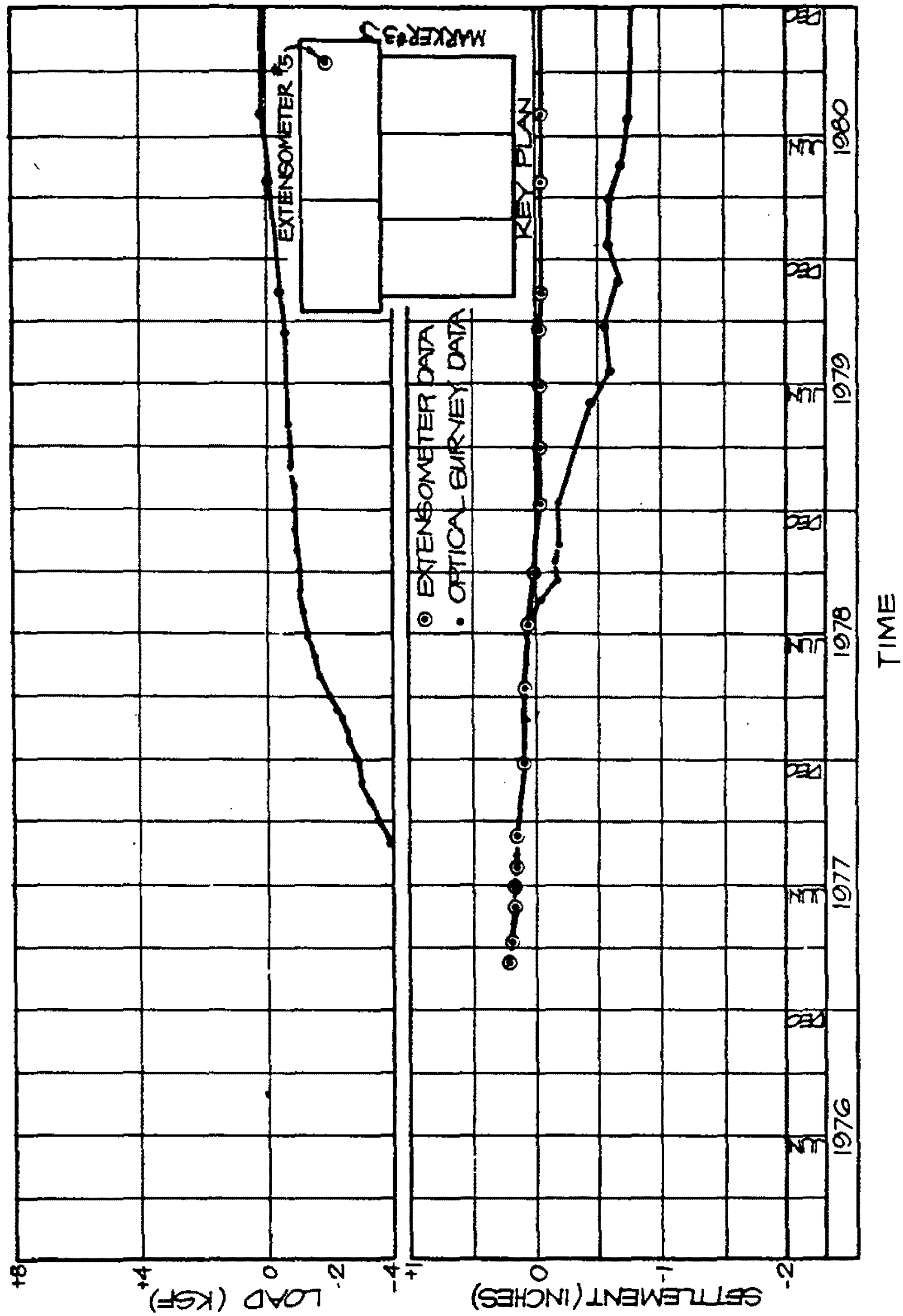
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 2

UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-68



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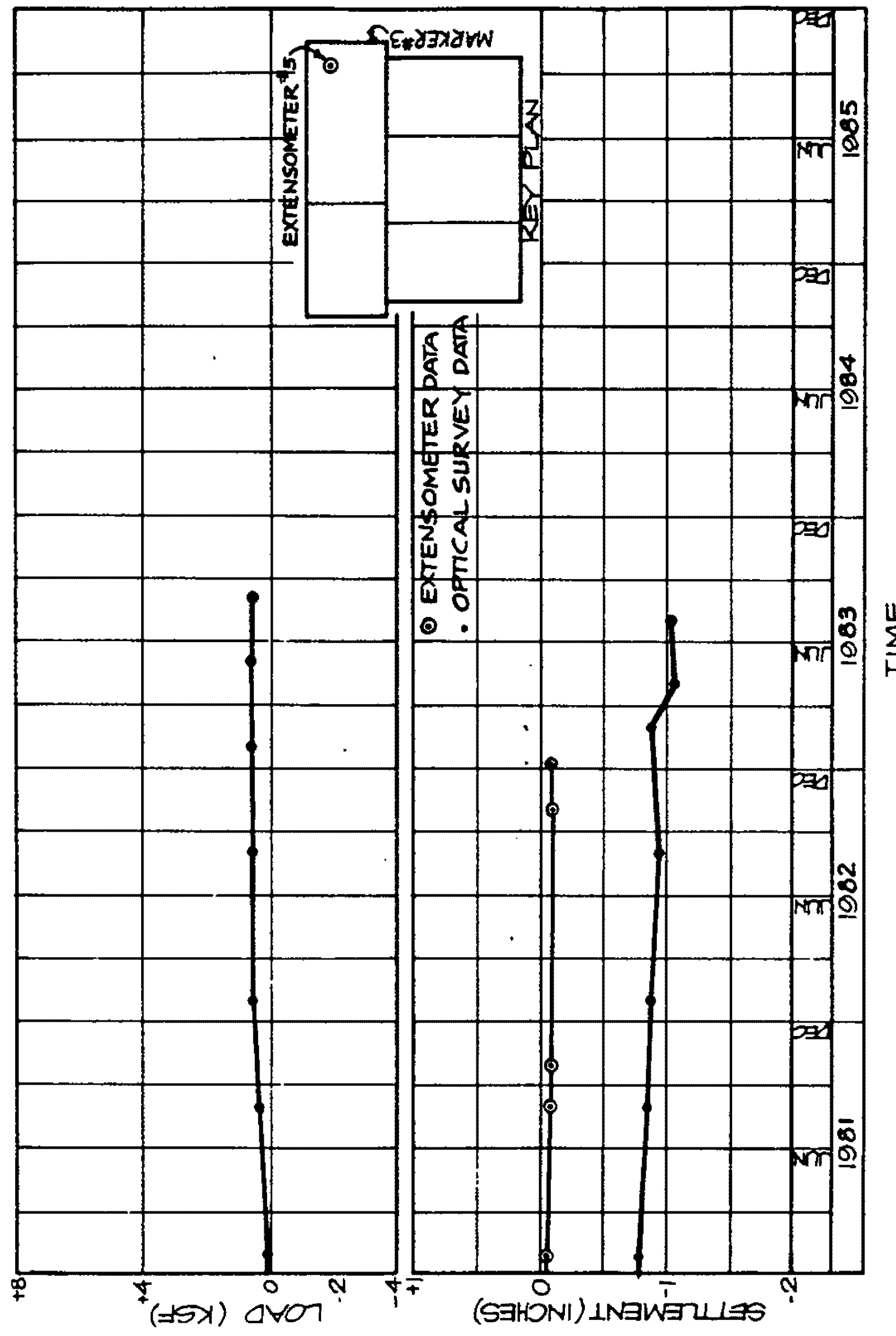
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 3

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-69



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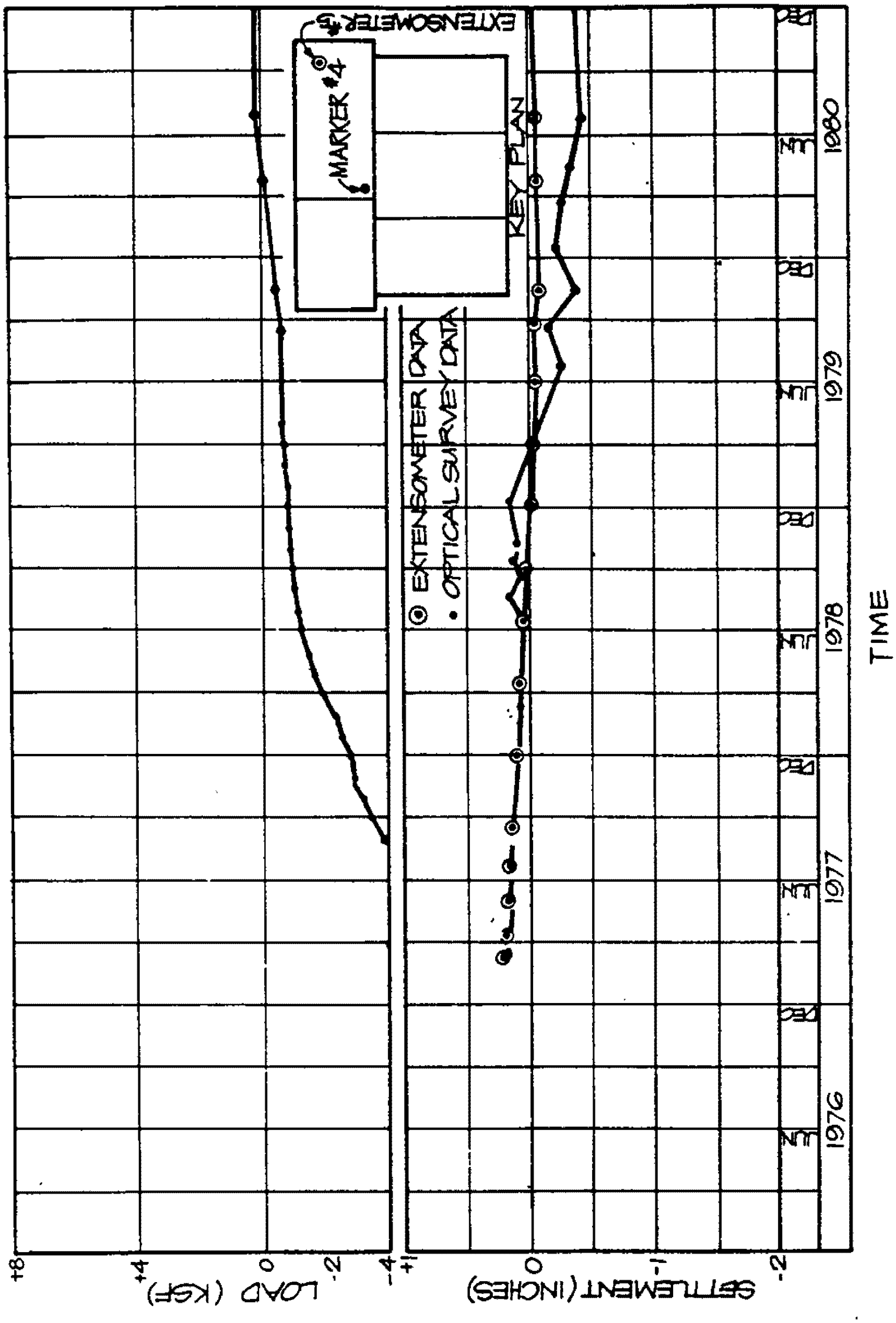
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 3

UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-69



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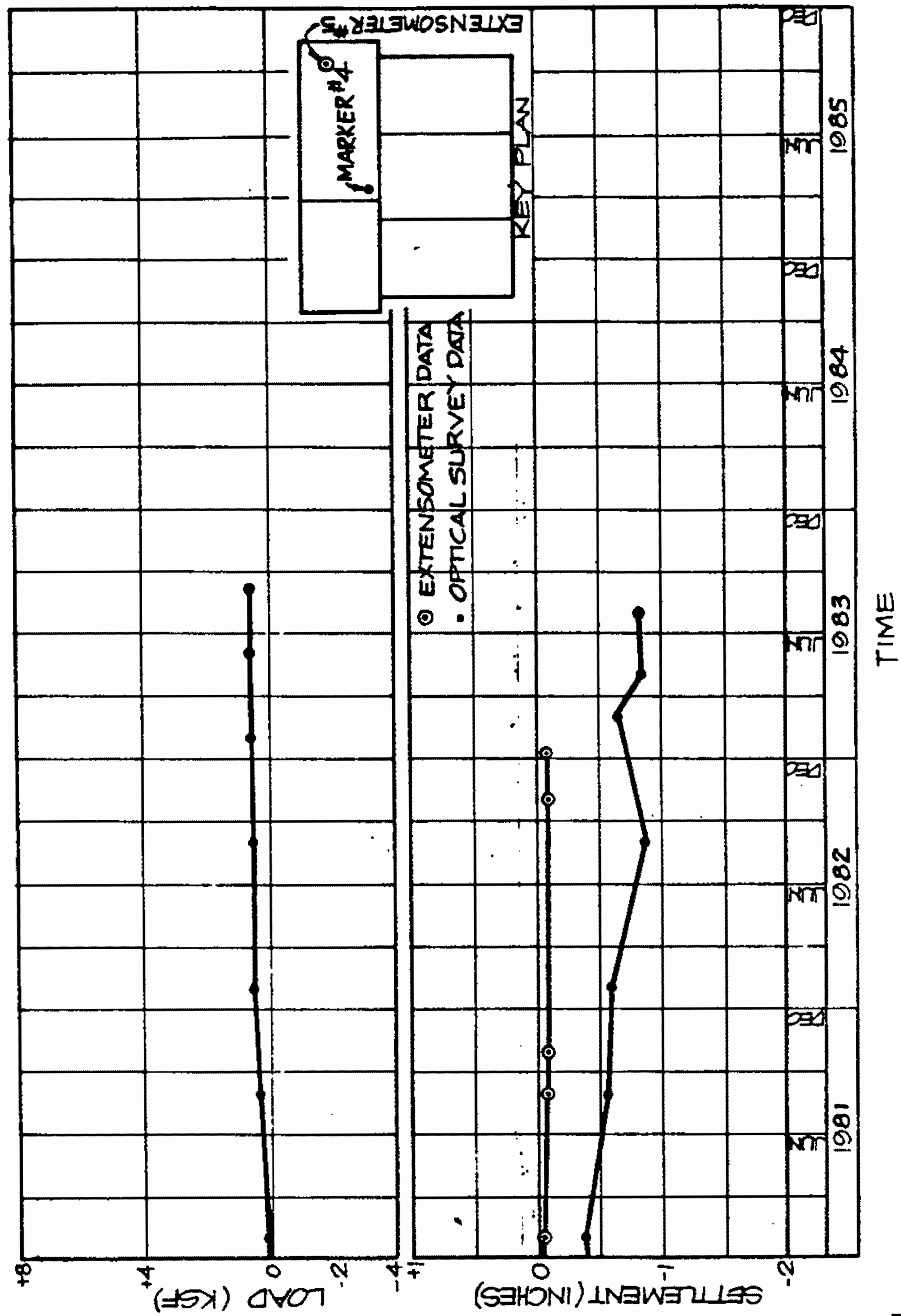
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 4

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-70



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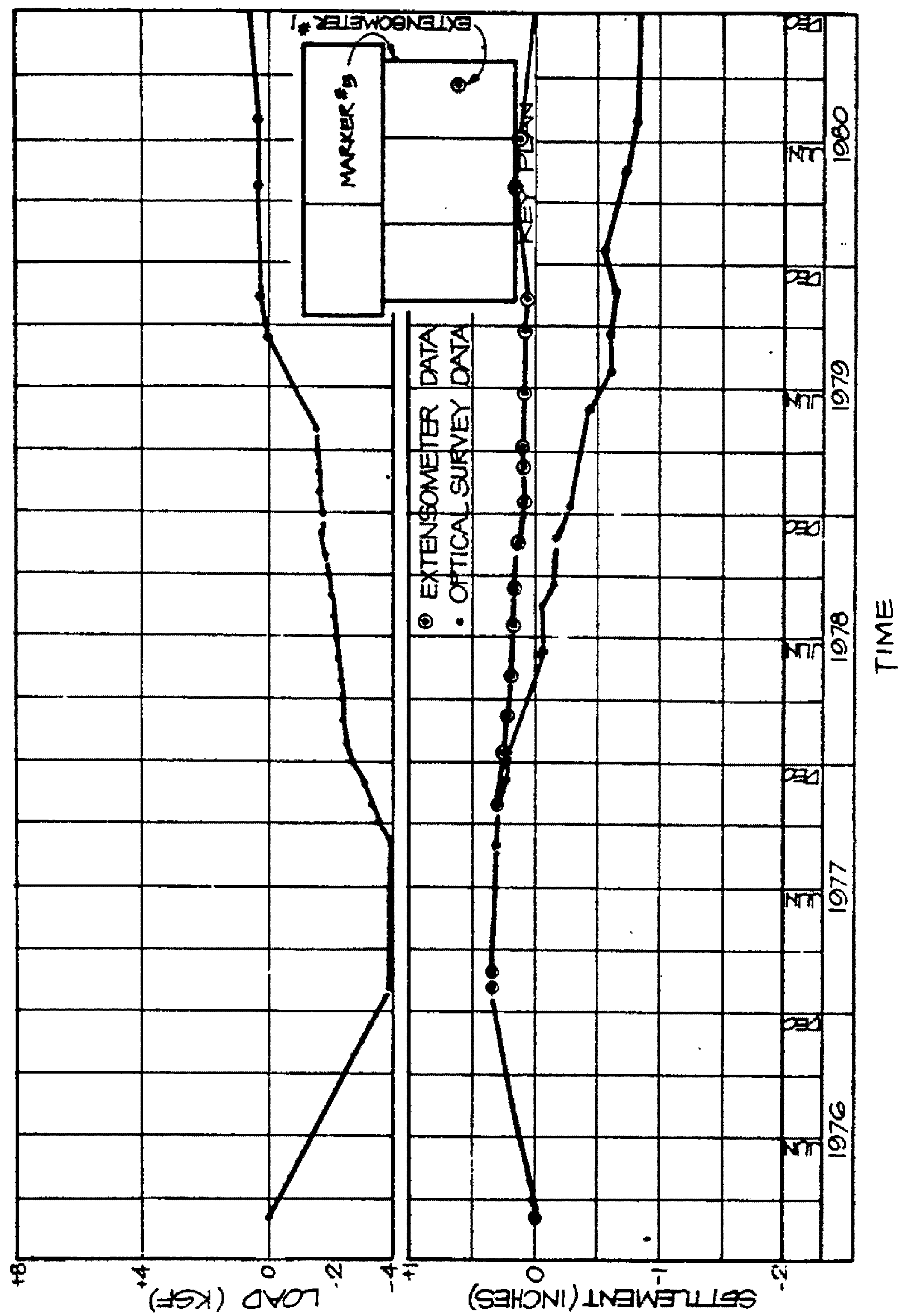
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY HOPE CREEK NUCLEAR GENERATING STATION	
LOAD/SETTLEMENT PLOT FOR MARKER NO. 4	
UPDATED FSAR	Sheet 2 of 2 FIGURE 2.5-70



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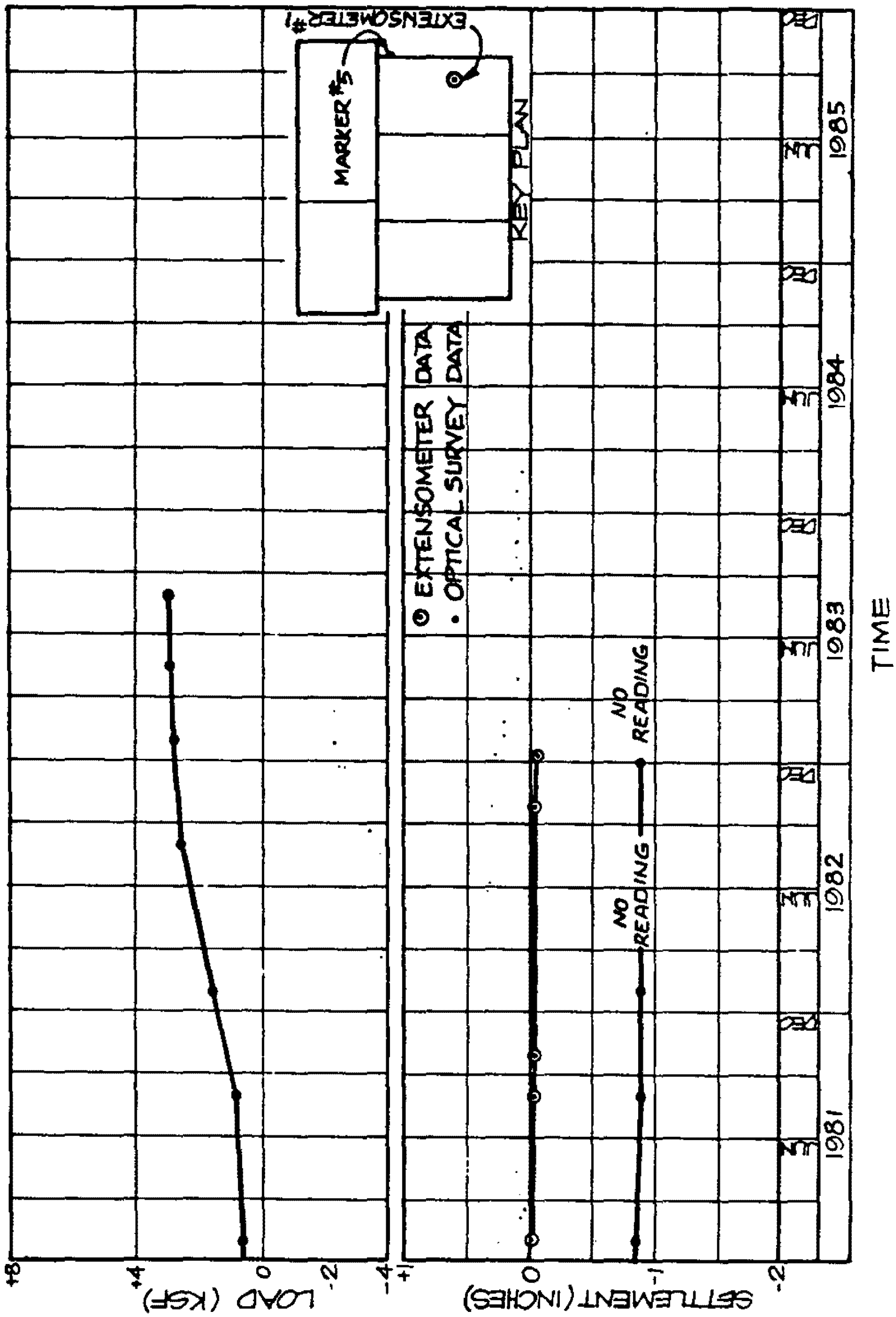
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 5

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-71



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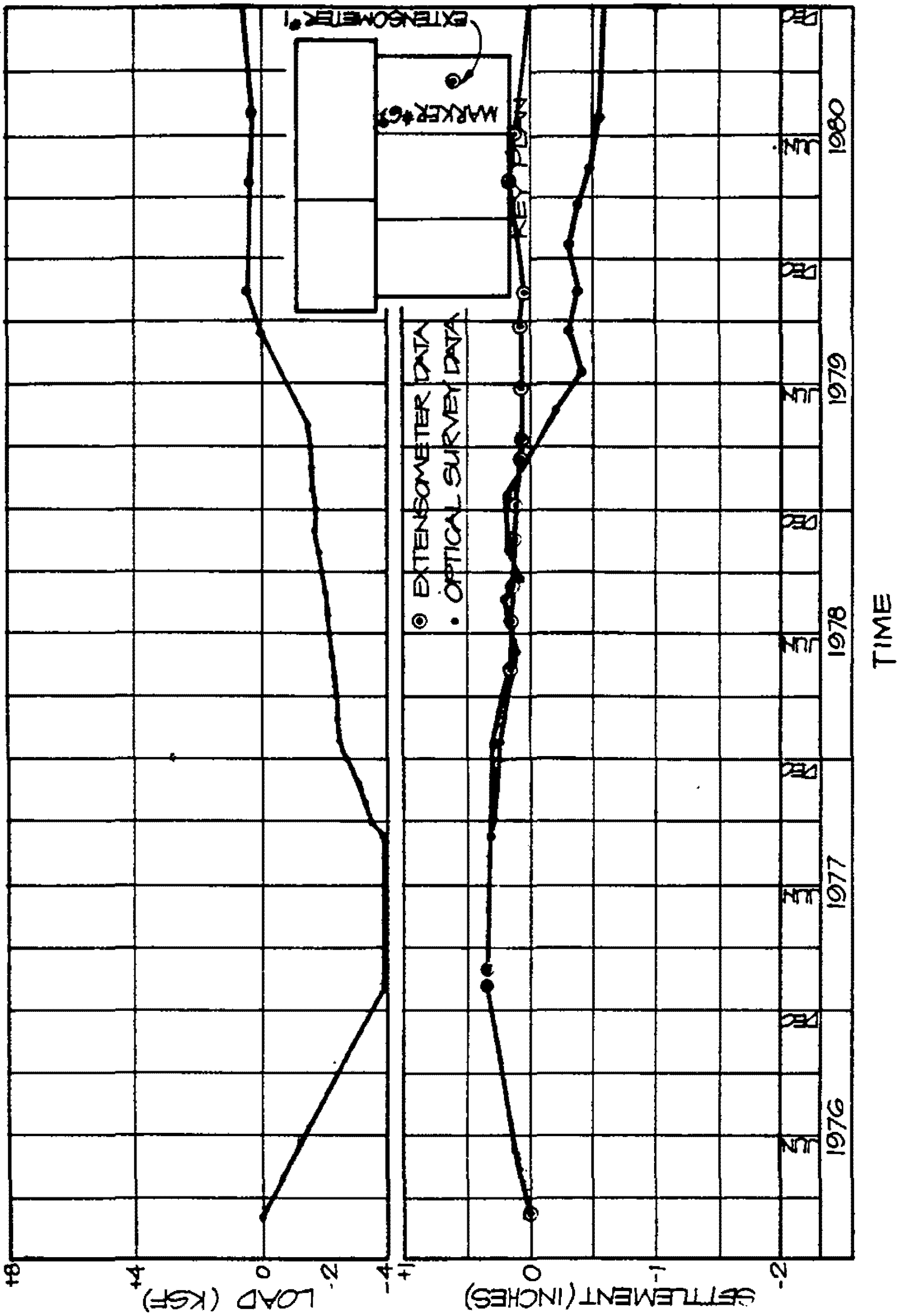
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY HOPE CREEK NUCLEAR GENERATING STATION	
LOAD/SETTLEMENT PLOT FOR MARKER NO. 5	
UPDATED FSAR	Sheet 2 of 2 FIGURE 2.5-71



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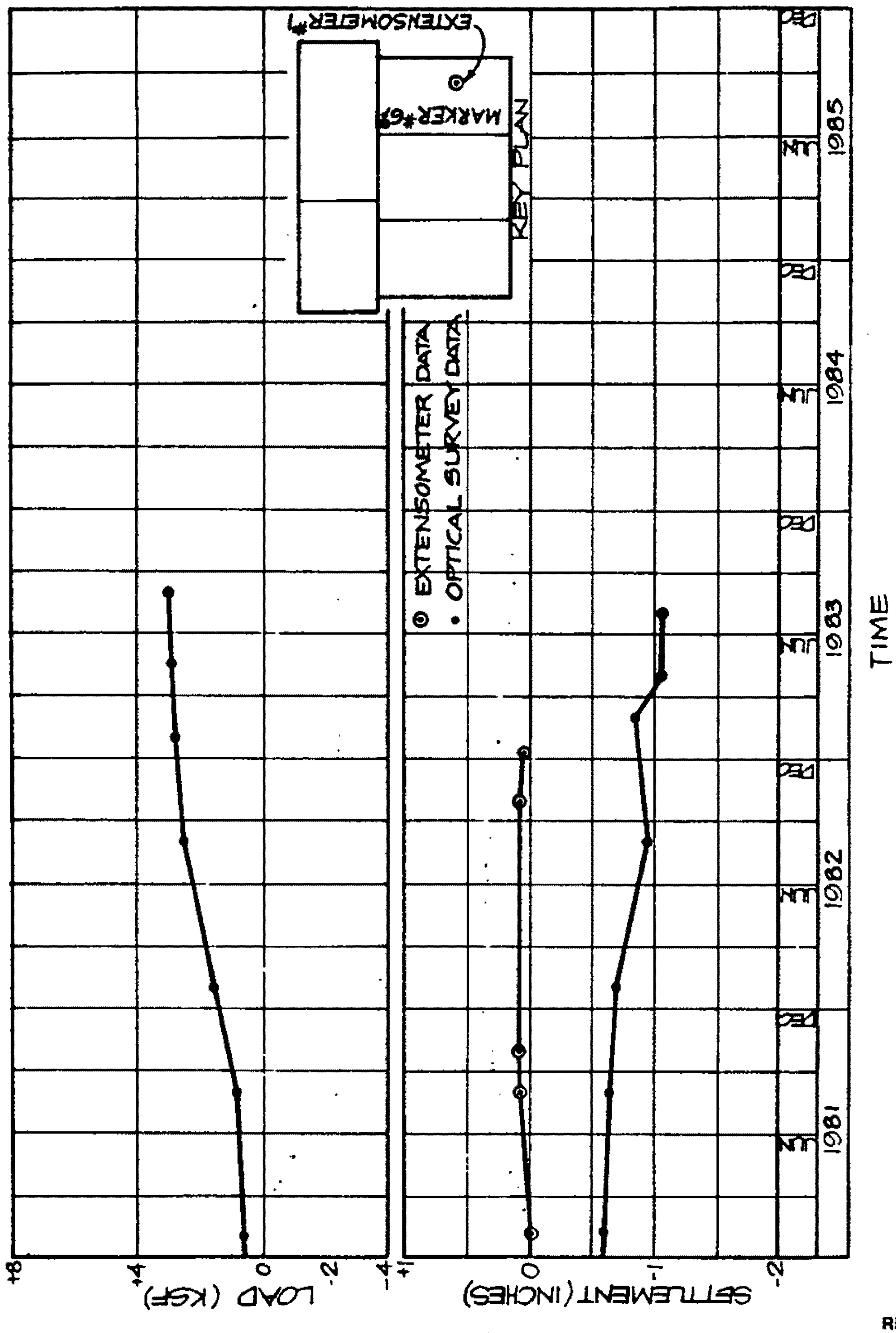
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 6

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-72



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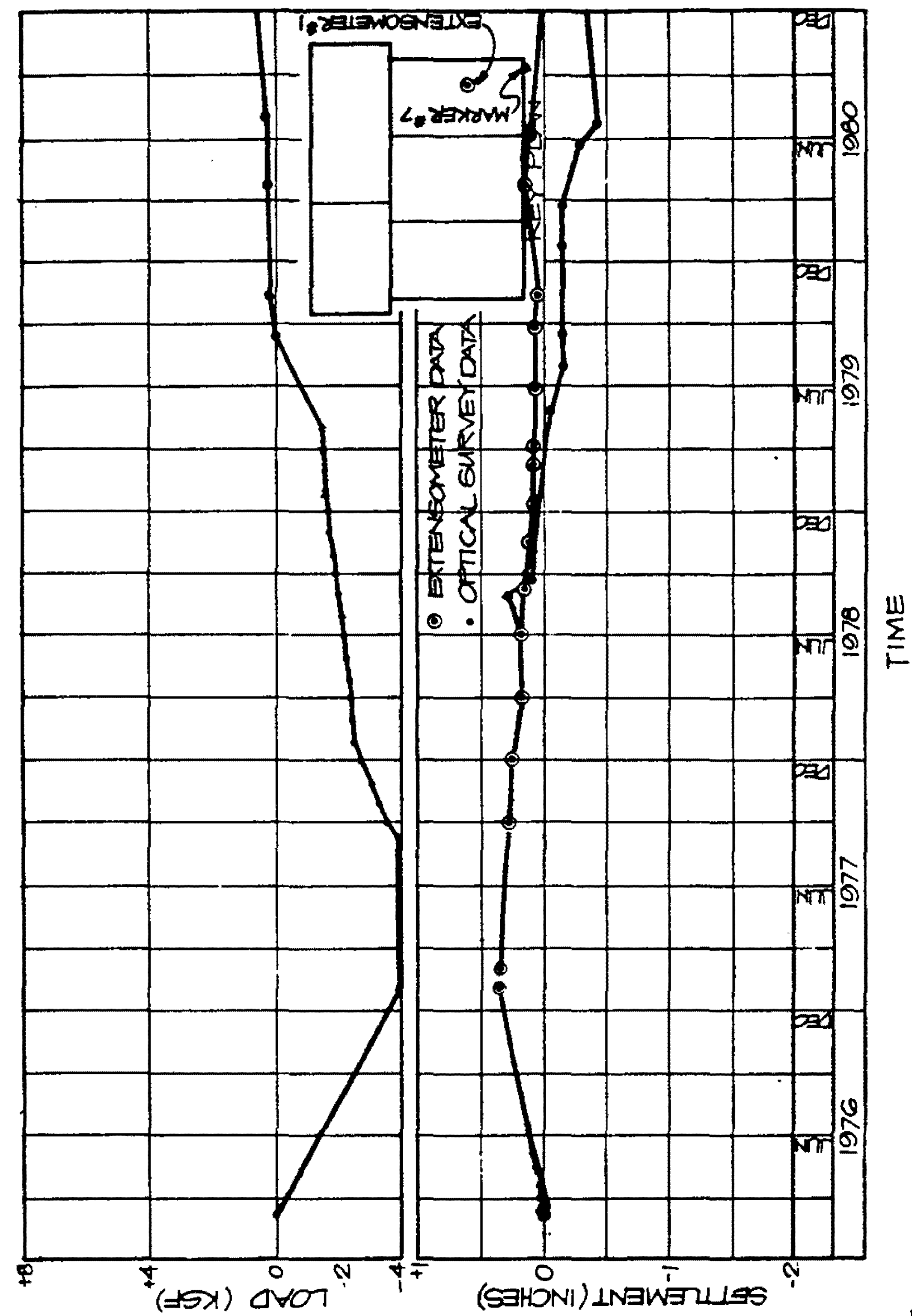
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 6

UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-72



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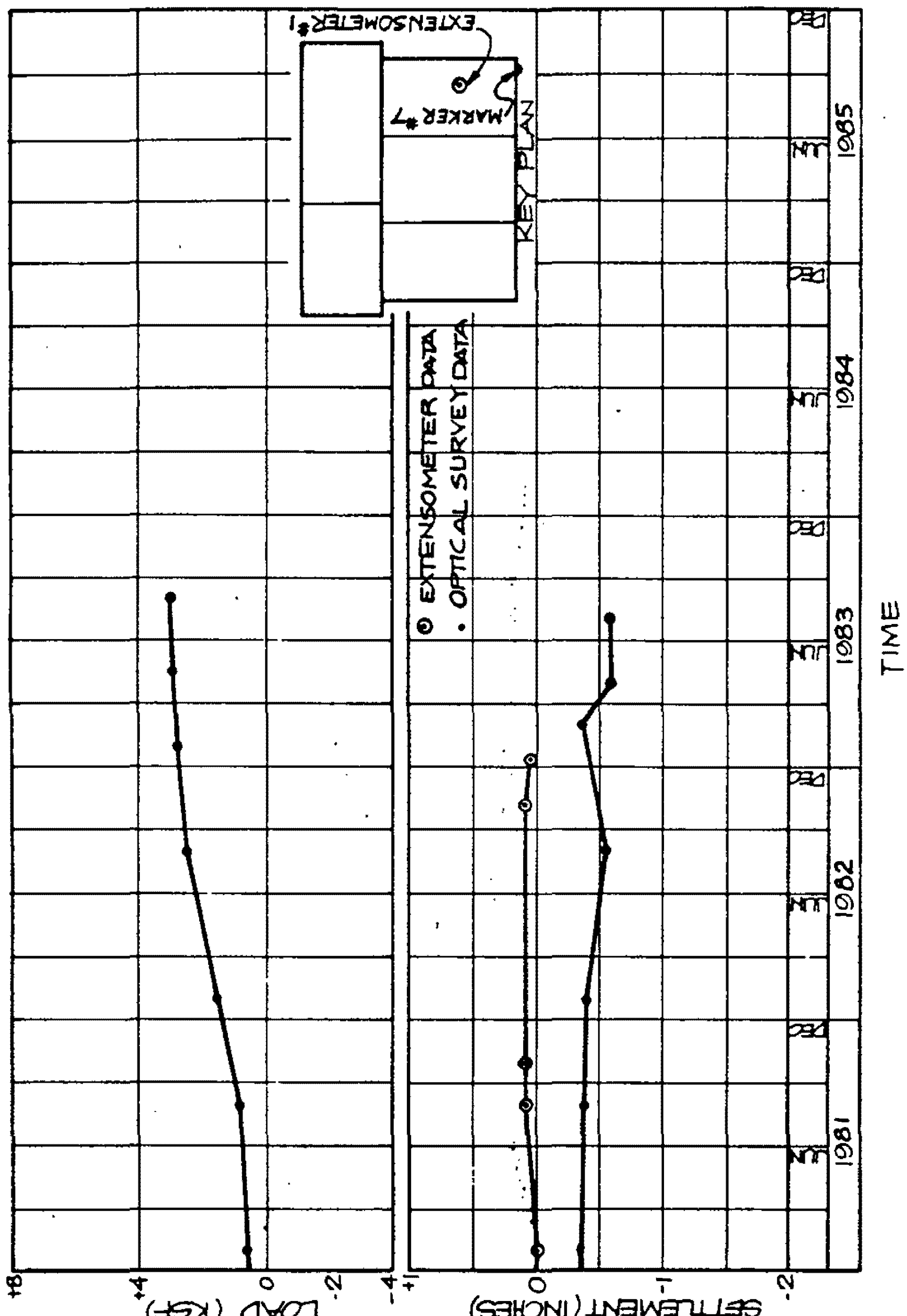
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 7

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-73



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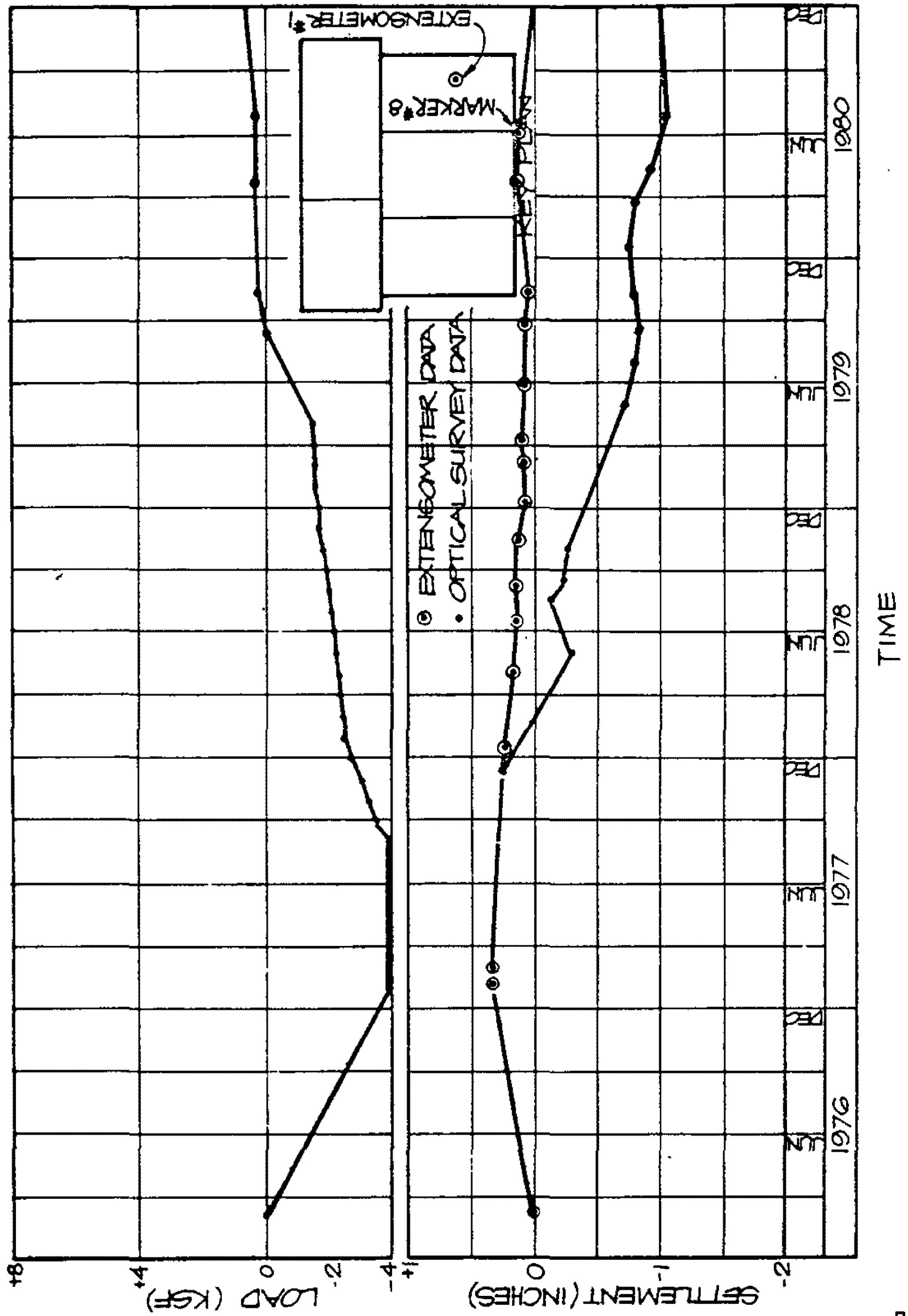
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 7

UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-73



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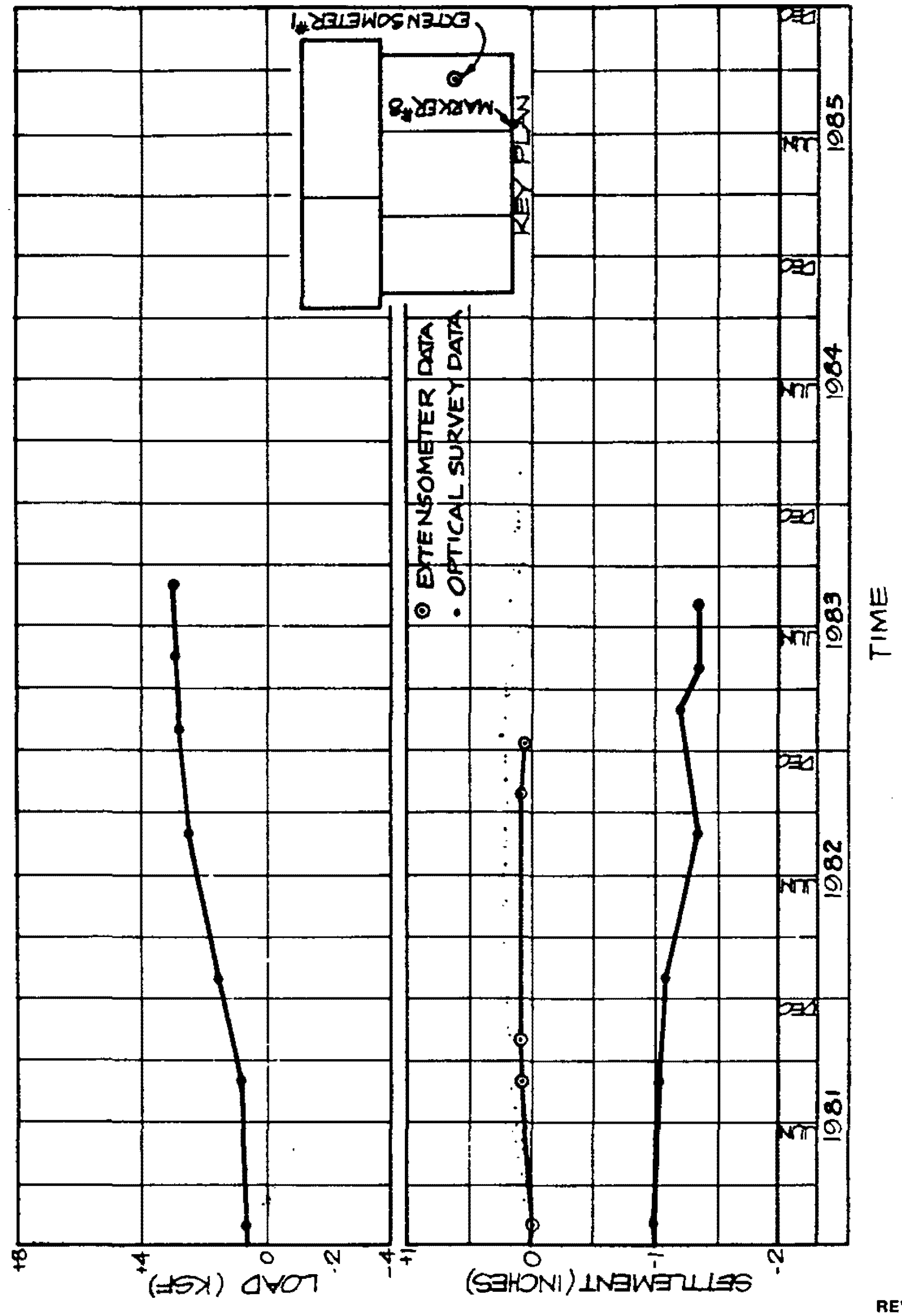
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 8

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-74



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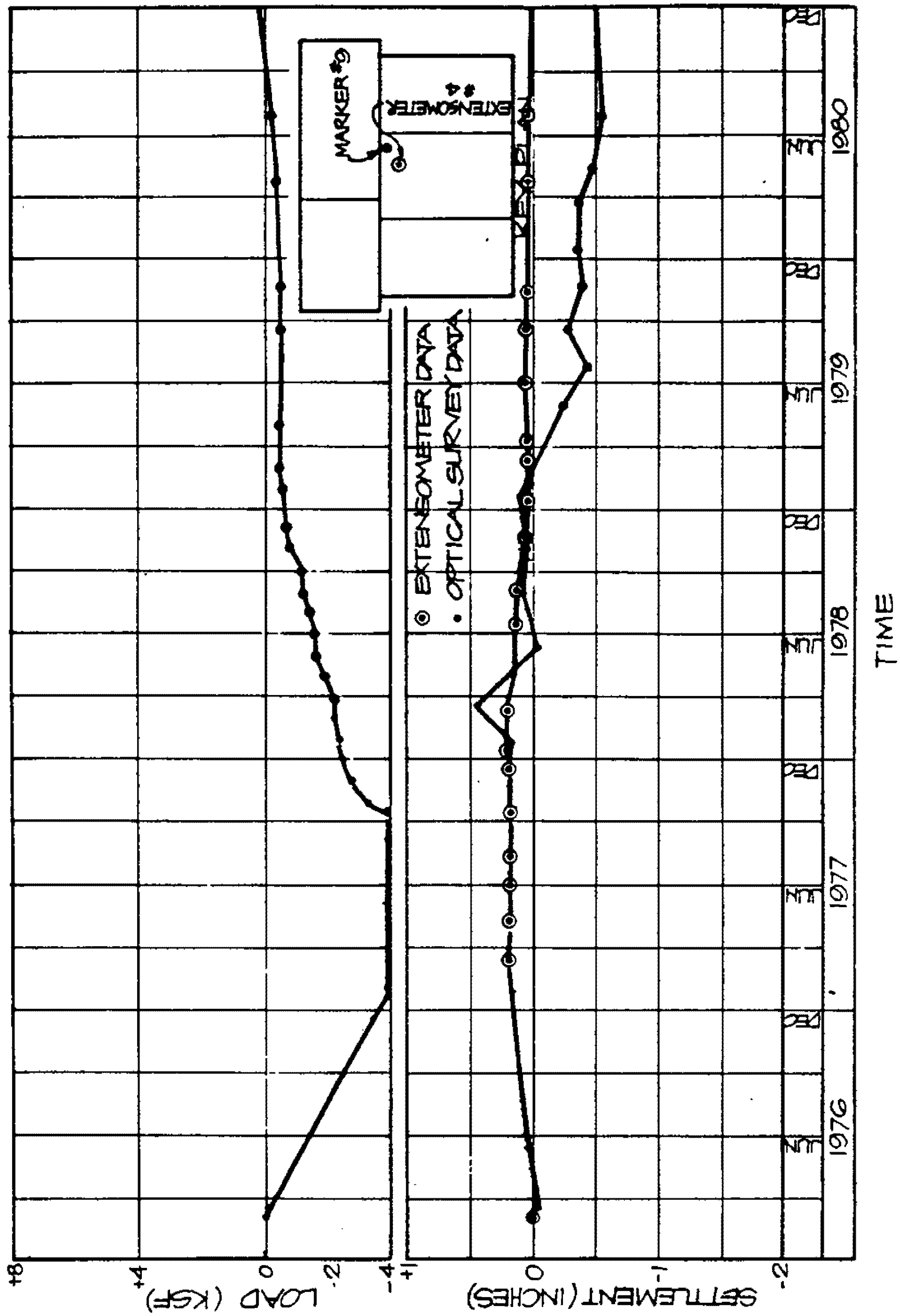
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY HOPE CREEK NUCLEAR GENERATING STATION	
LOAD/SETTLEMENT PLOT FOR MARKER NO. 8	
UPDATED FSAR	Sheet 2 of 2 FIGURE 2.5-74



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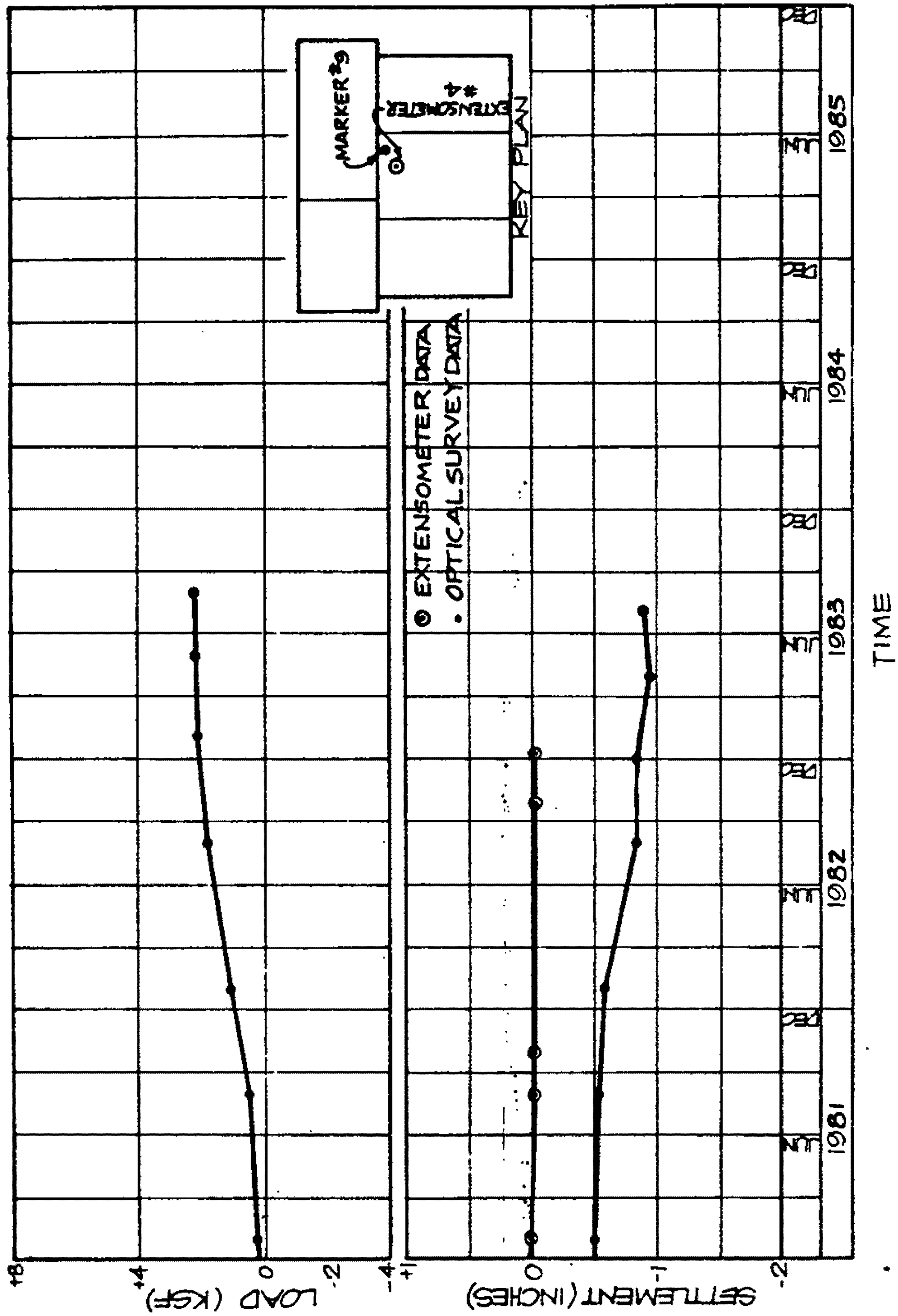
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 9

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-75



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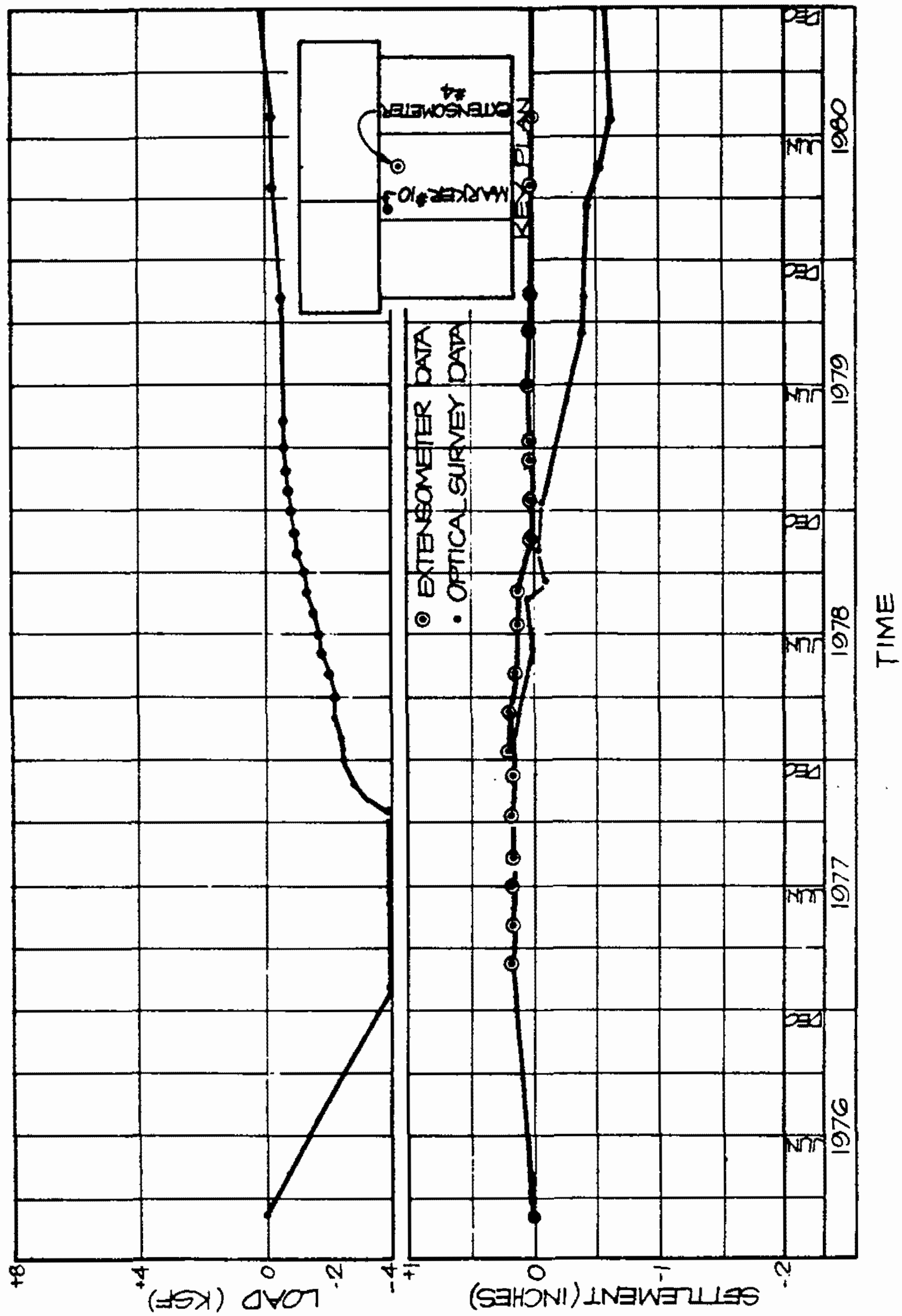
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 9

UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-75



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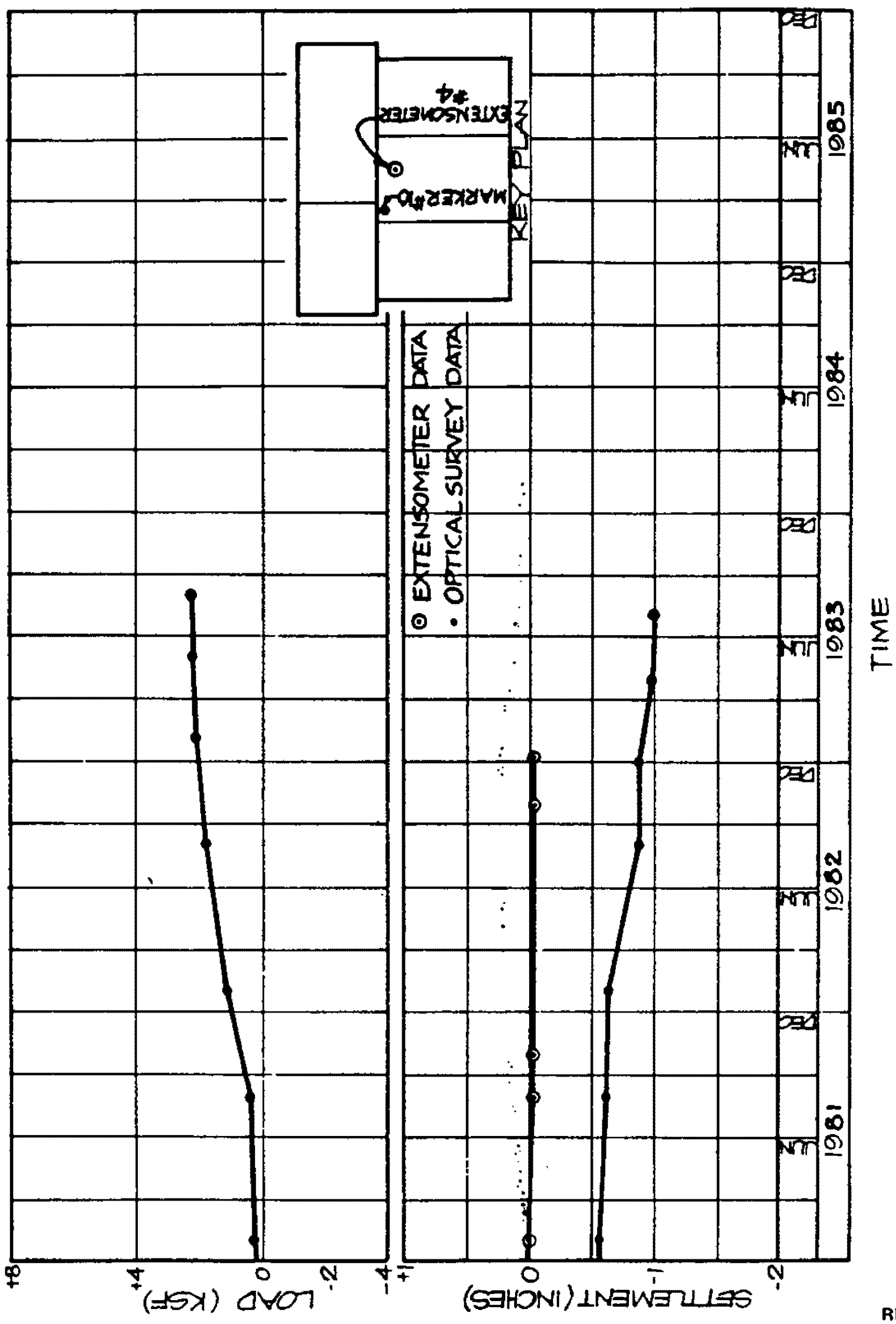
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 10

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-76



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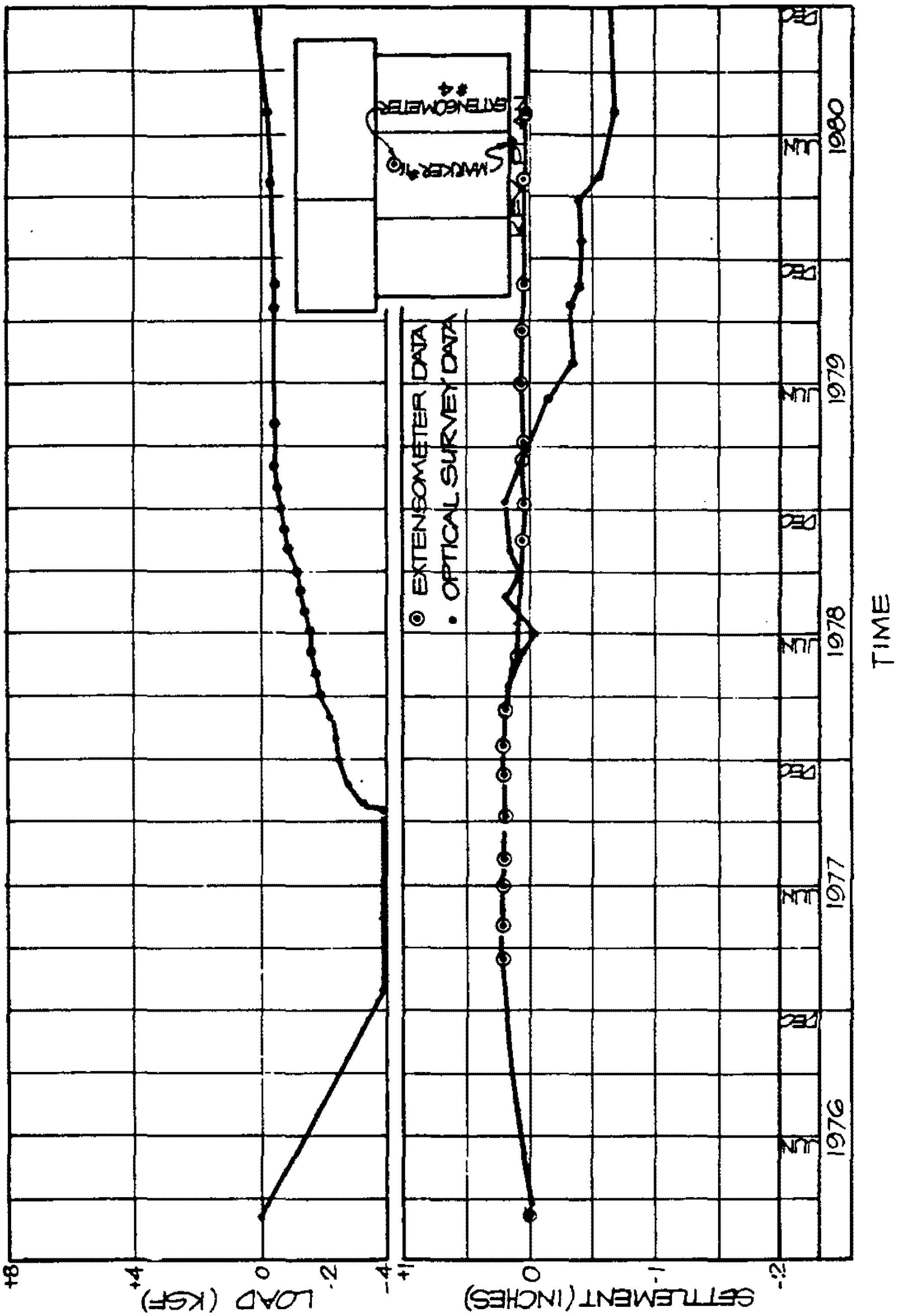
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 10

UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-76



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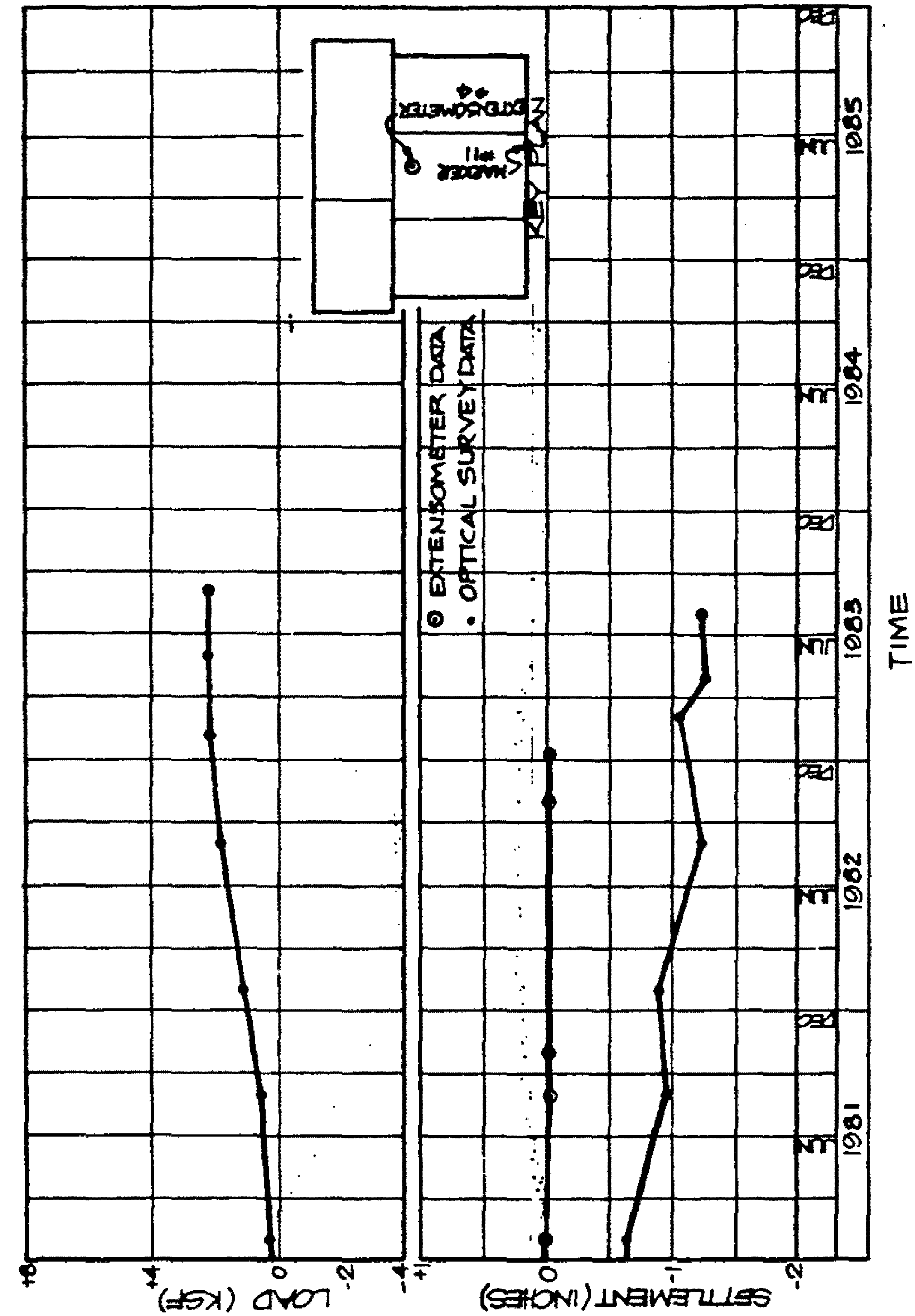
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY HOPE CREEK NUCLEAR GENERATING STATION	
LOAD/SETTLEMENT PLOT FOR MARKER NO. 11	
UPDATED FSAR	Sheet 1 of 2 FIGURE 2.5-77



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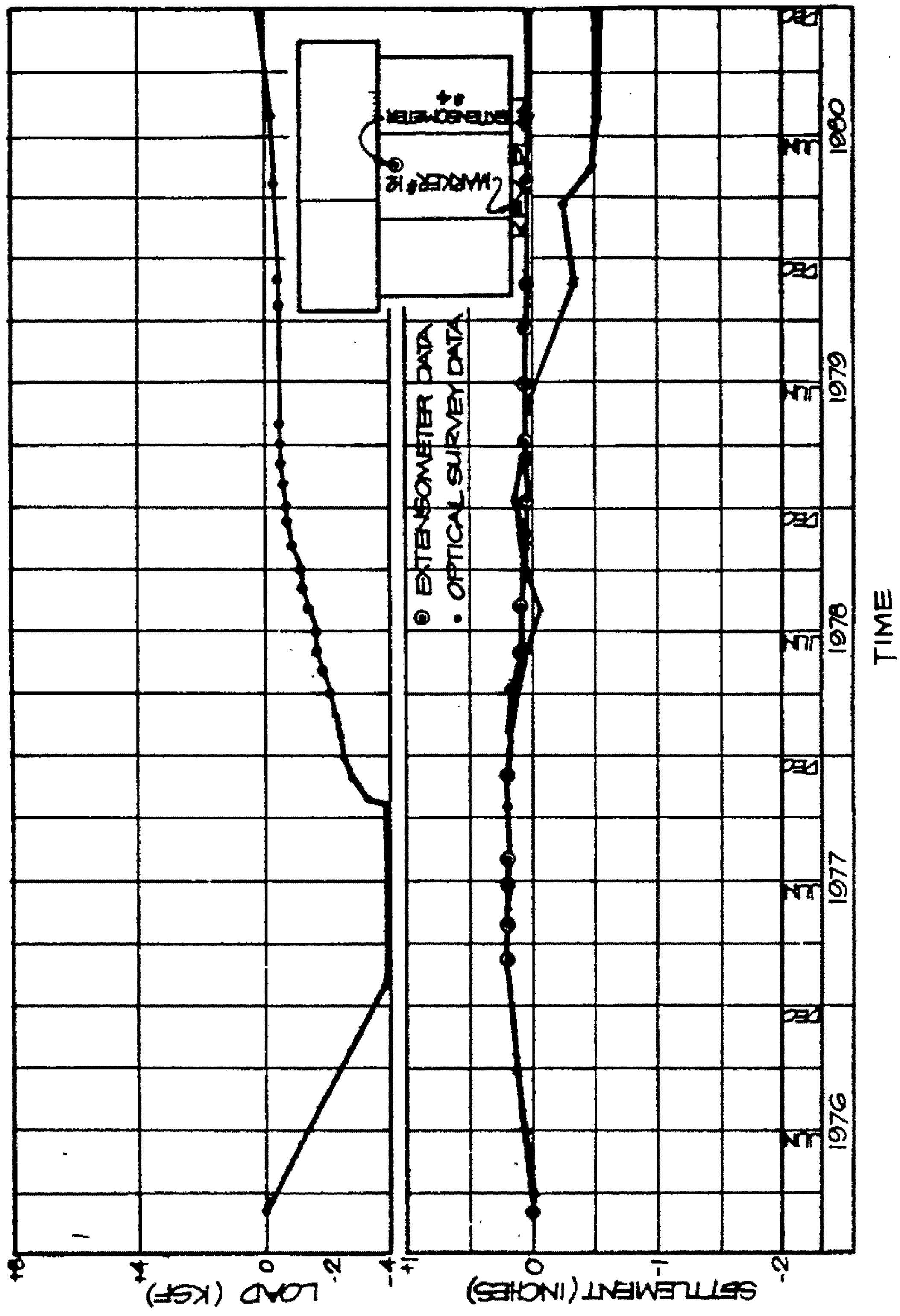
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 11

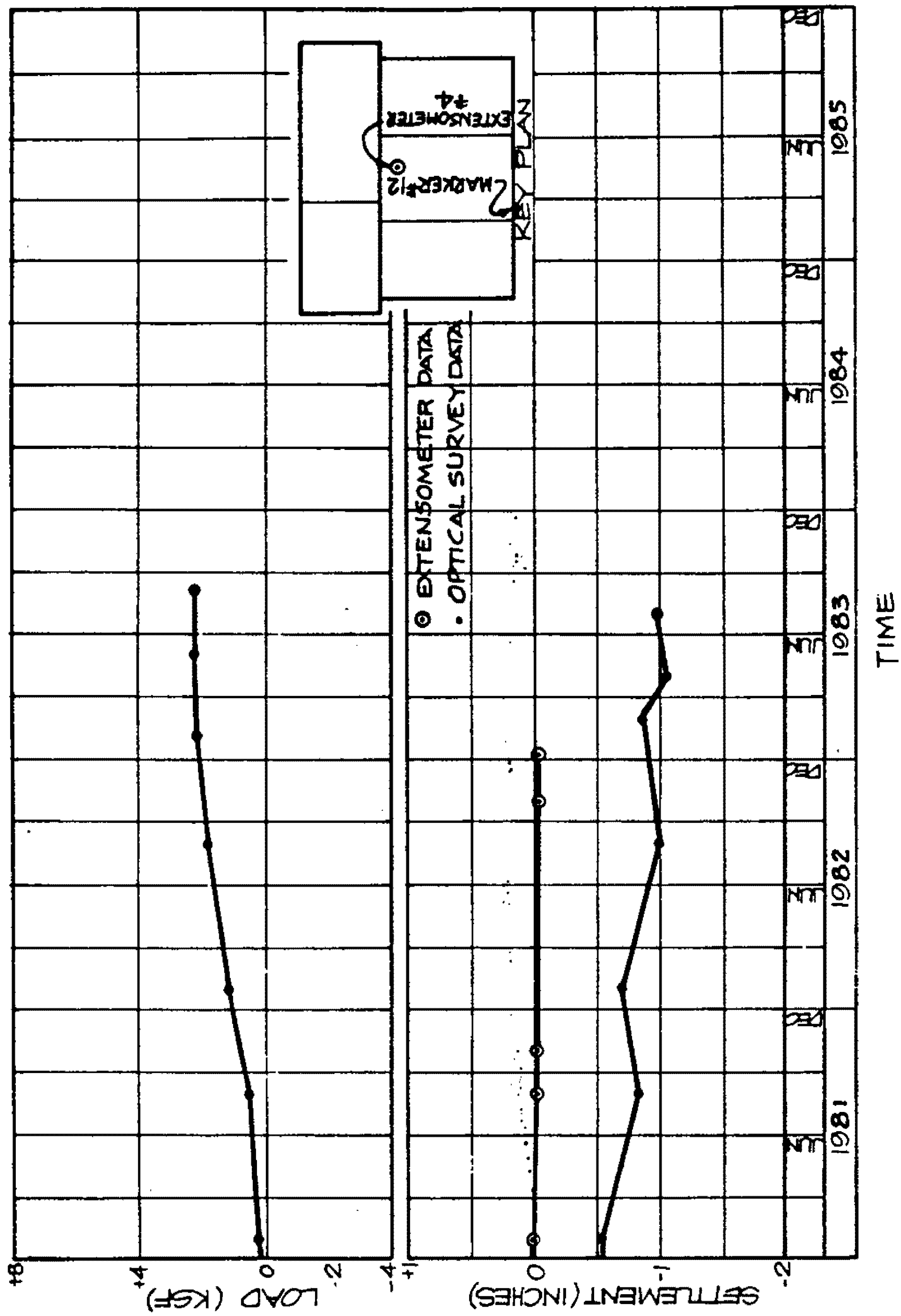
UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-77



REVISION 0
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY HOPE CREEK NUCLEAR GENERATING STATION	
LOAD/SETTLEMENT PLOT FOR MARKER NO. 12	
UPDATED FSAR	Sheet 1 of 2 **FIGURE 2.5-78**



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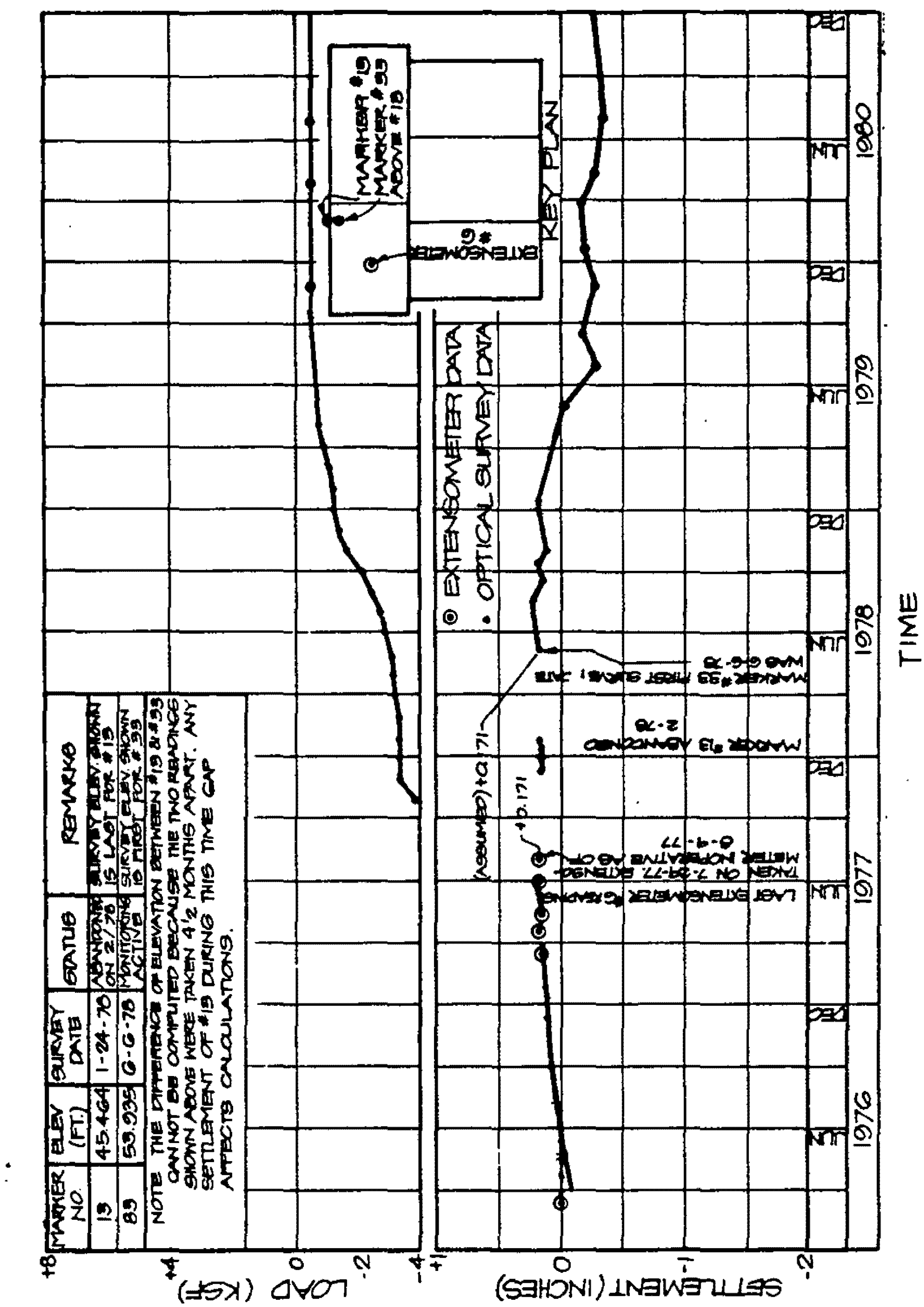
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 12

UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-78



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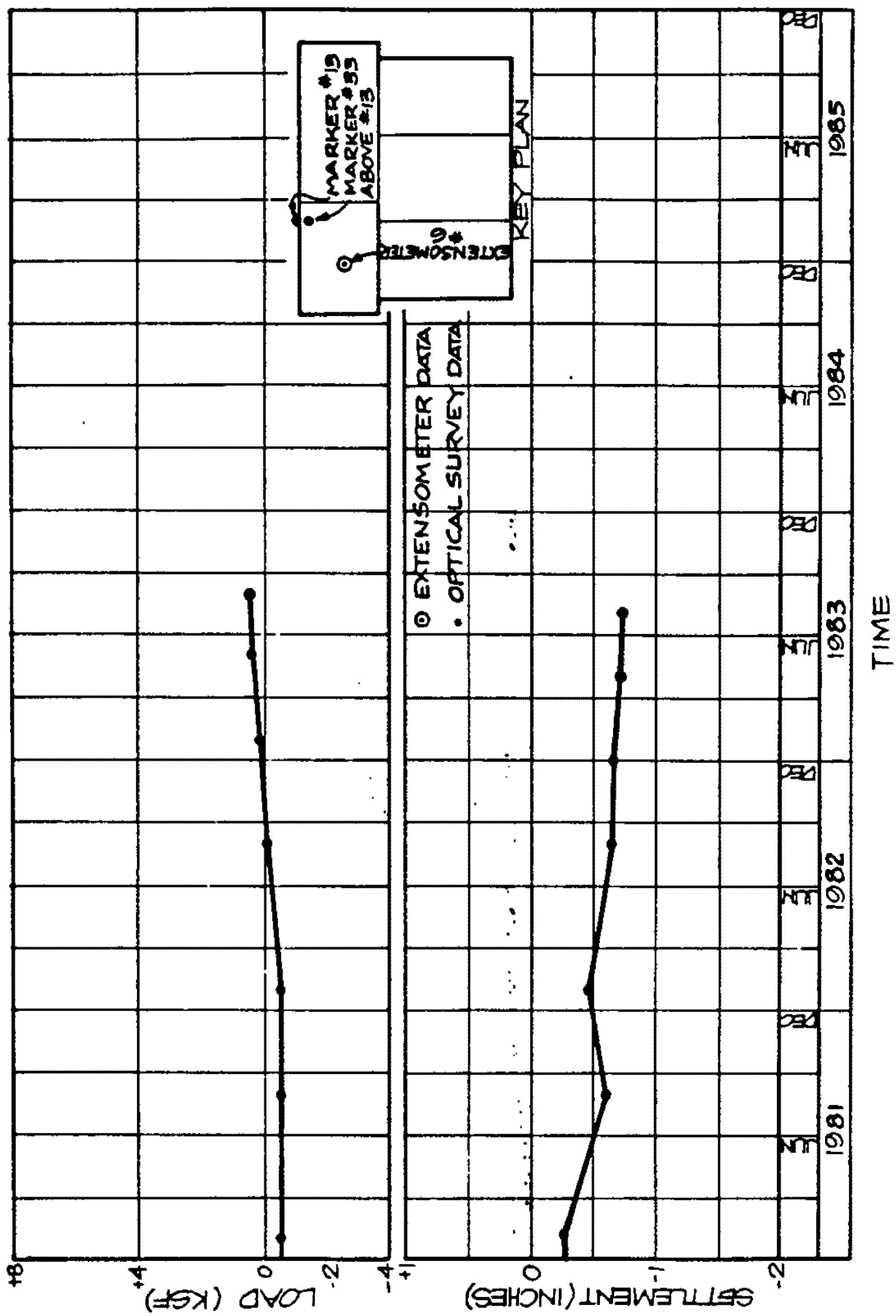
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 13

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-79



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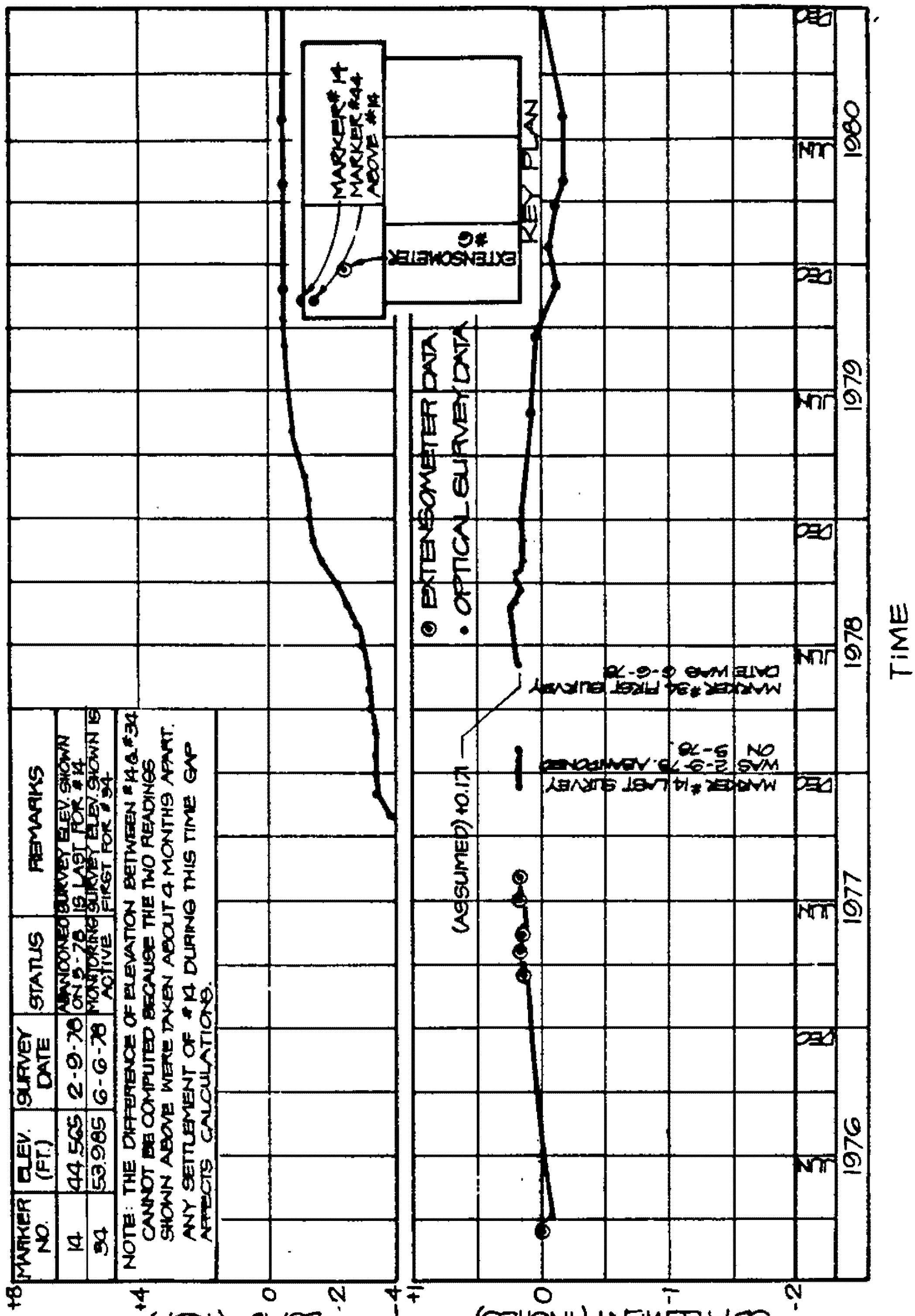
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY HOPE CREEK NUCLEAR GENERATING STATION	
LOAD/SETTLEMENT PLOT FOR MARKER NO. 13	
UPDATED FSAR	Sheet 2 of 2 FIGURE 2.5-79



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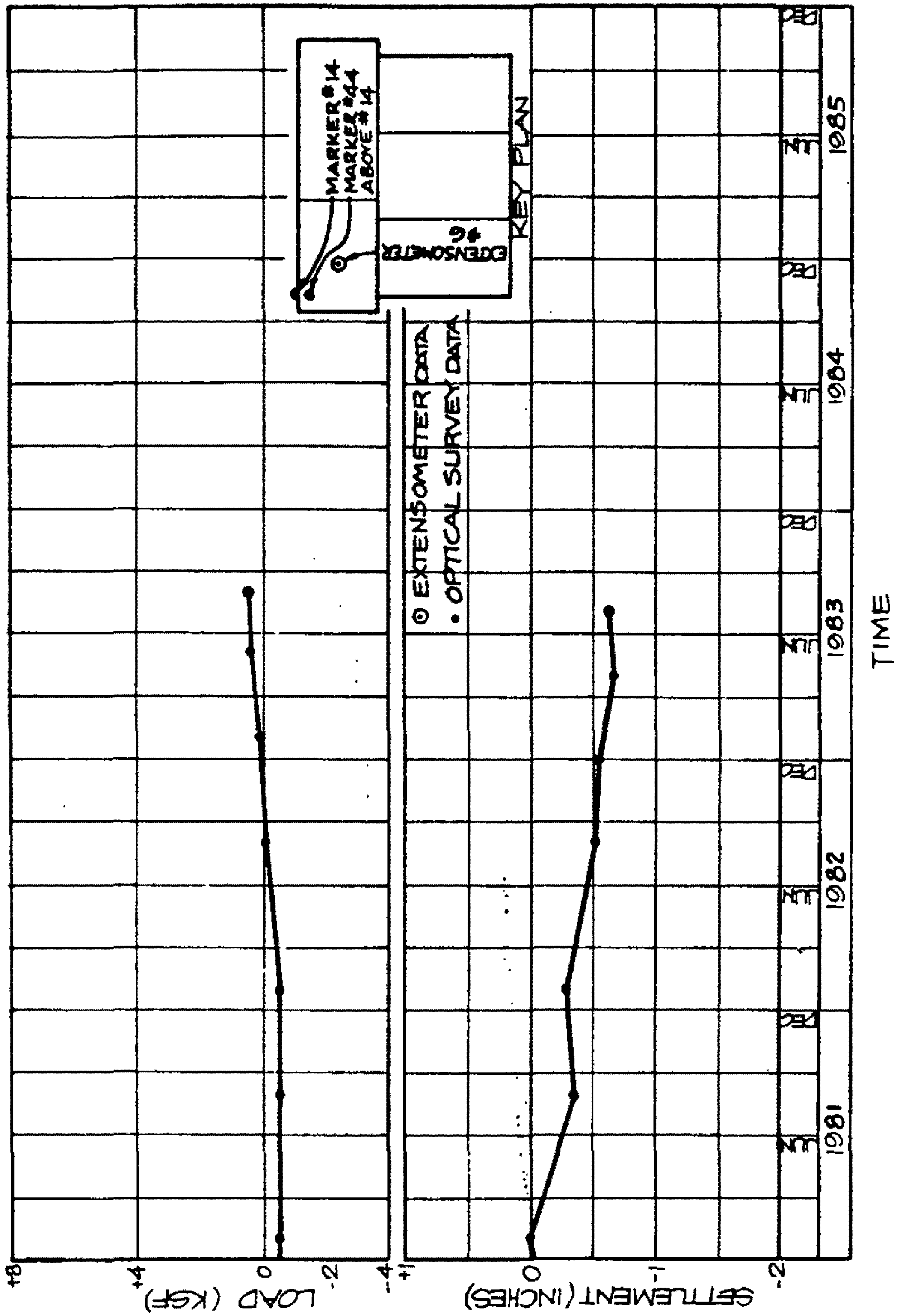
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 14

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-80



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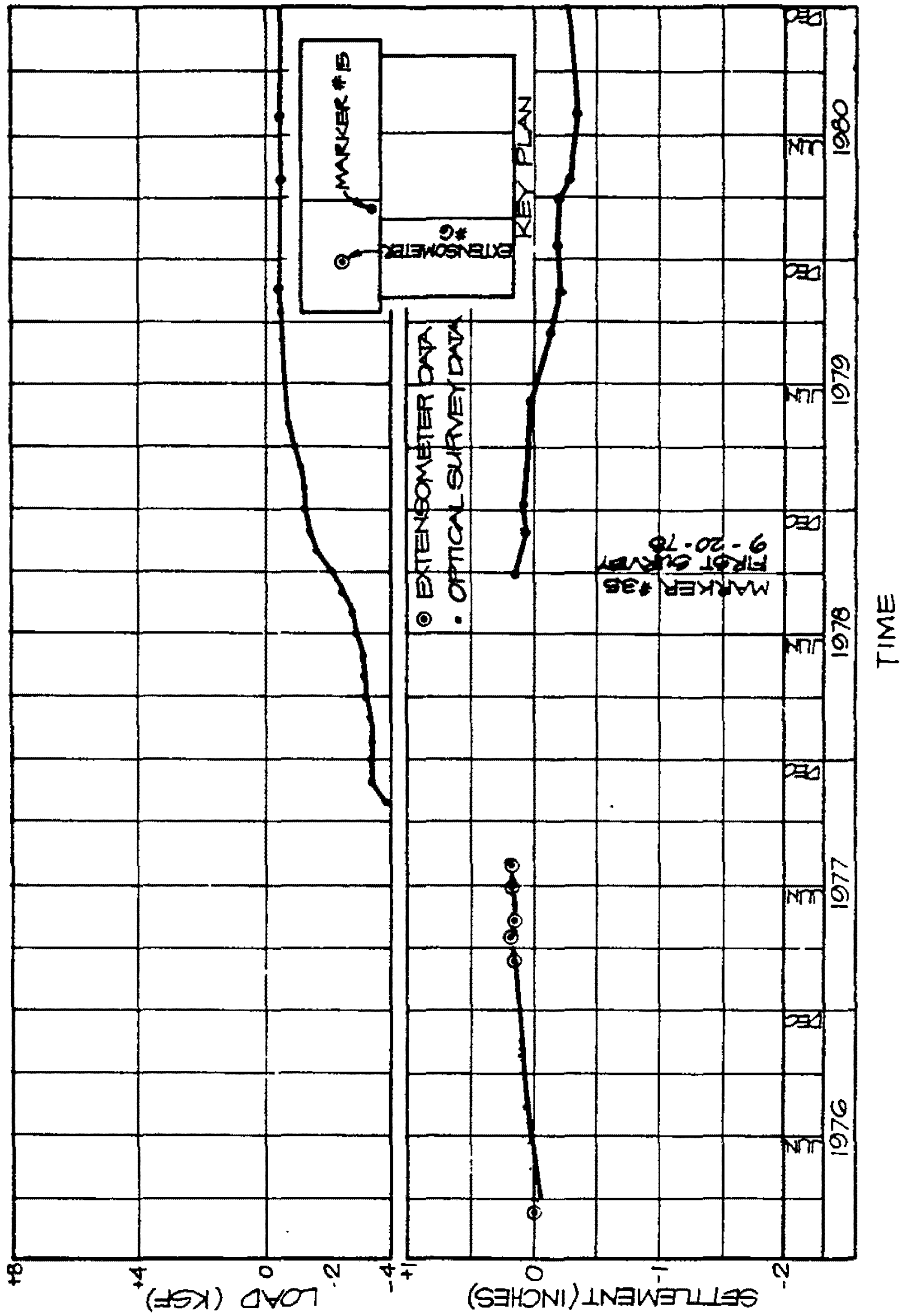
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 14

UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-80



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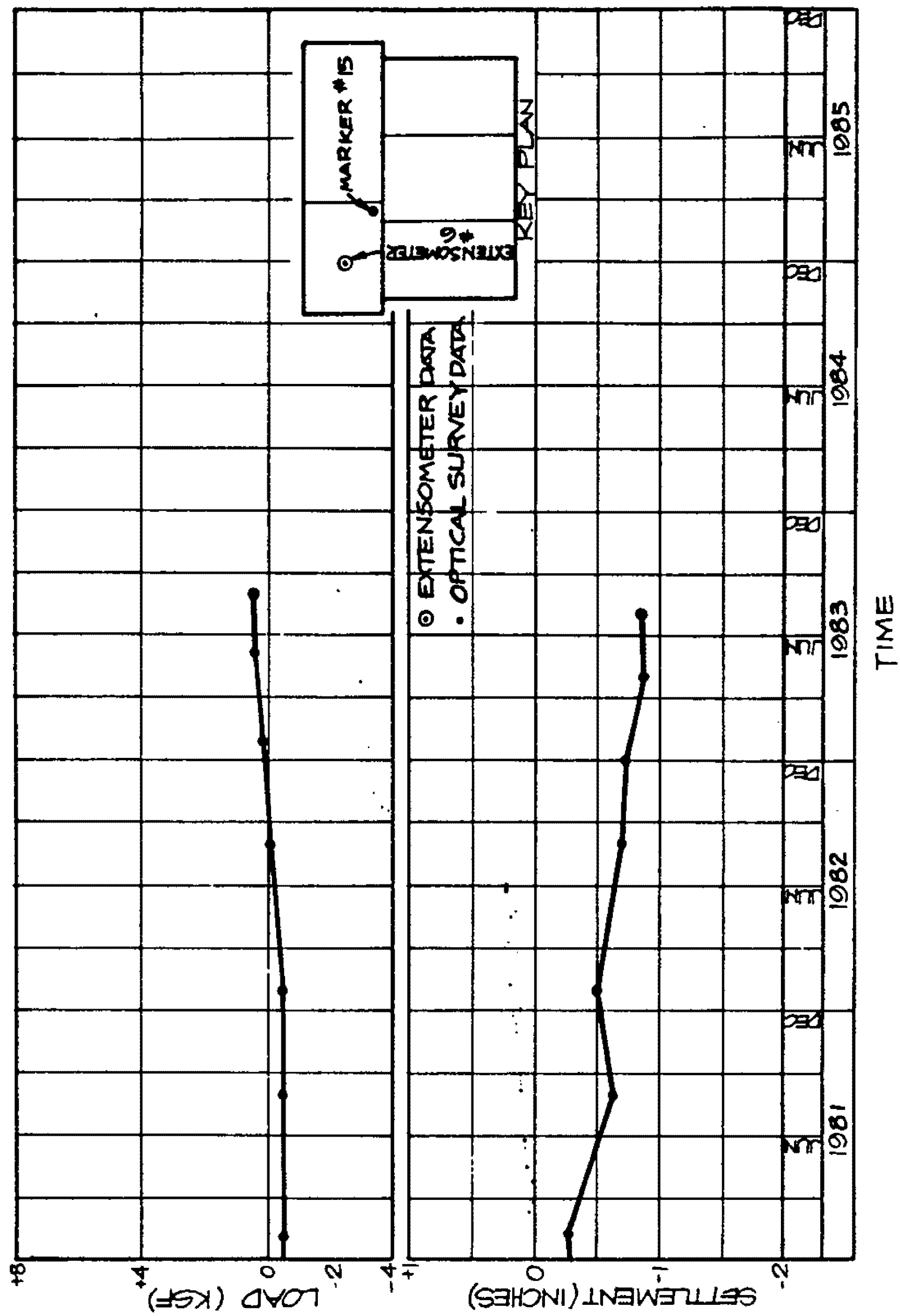
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 15

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-81



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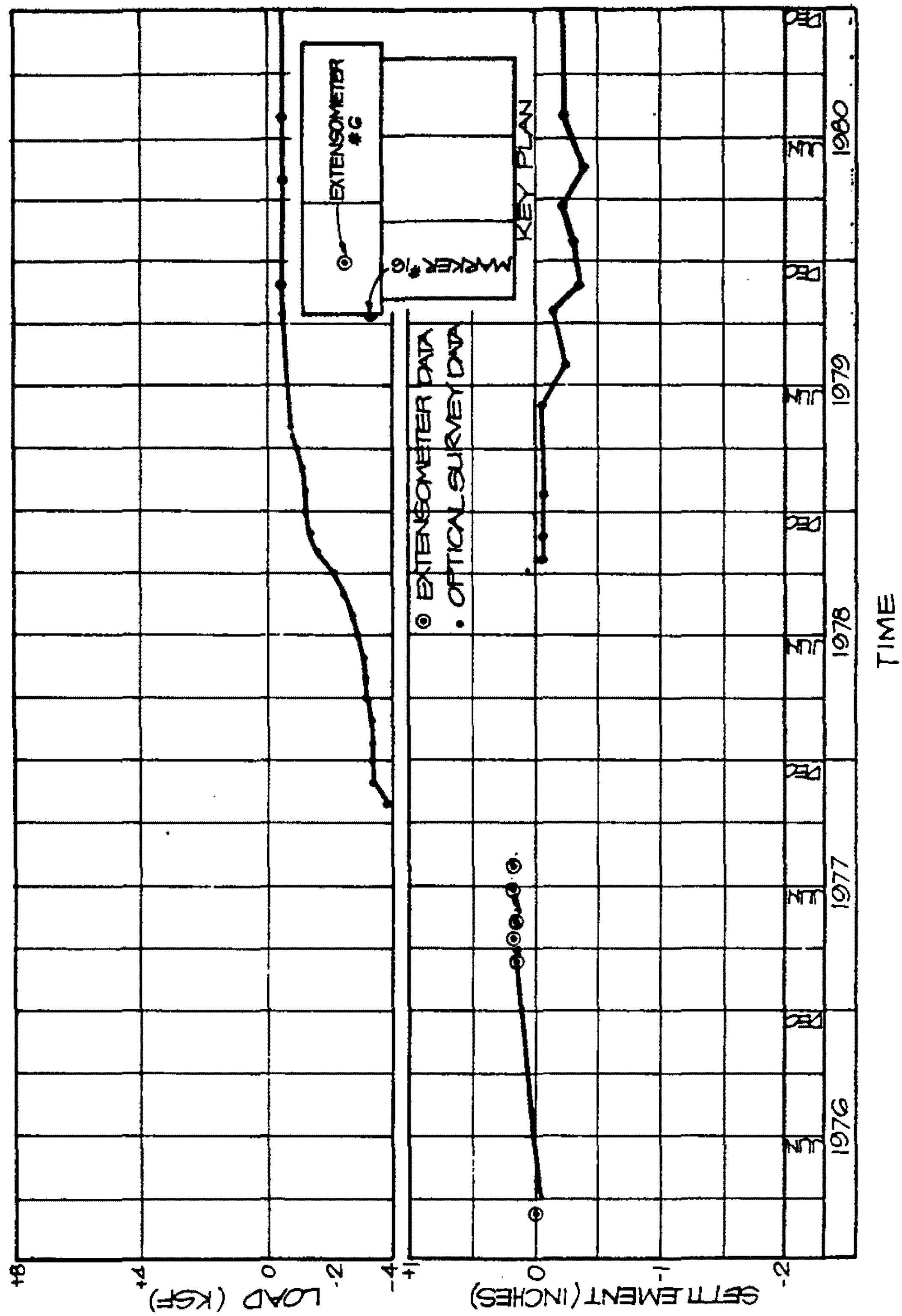
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 15

UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-81



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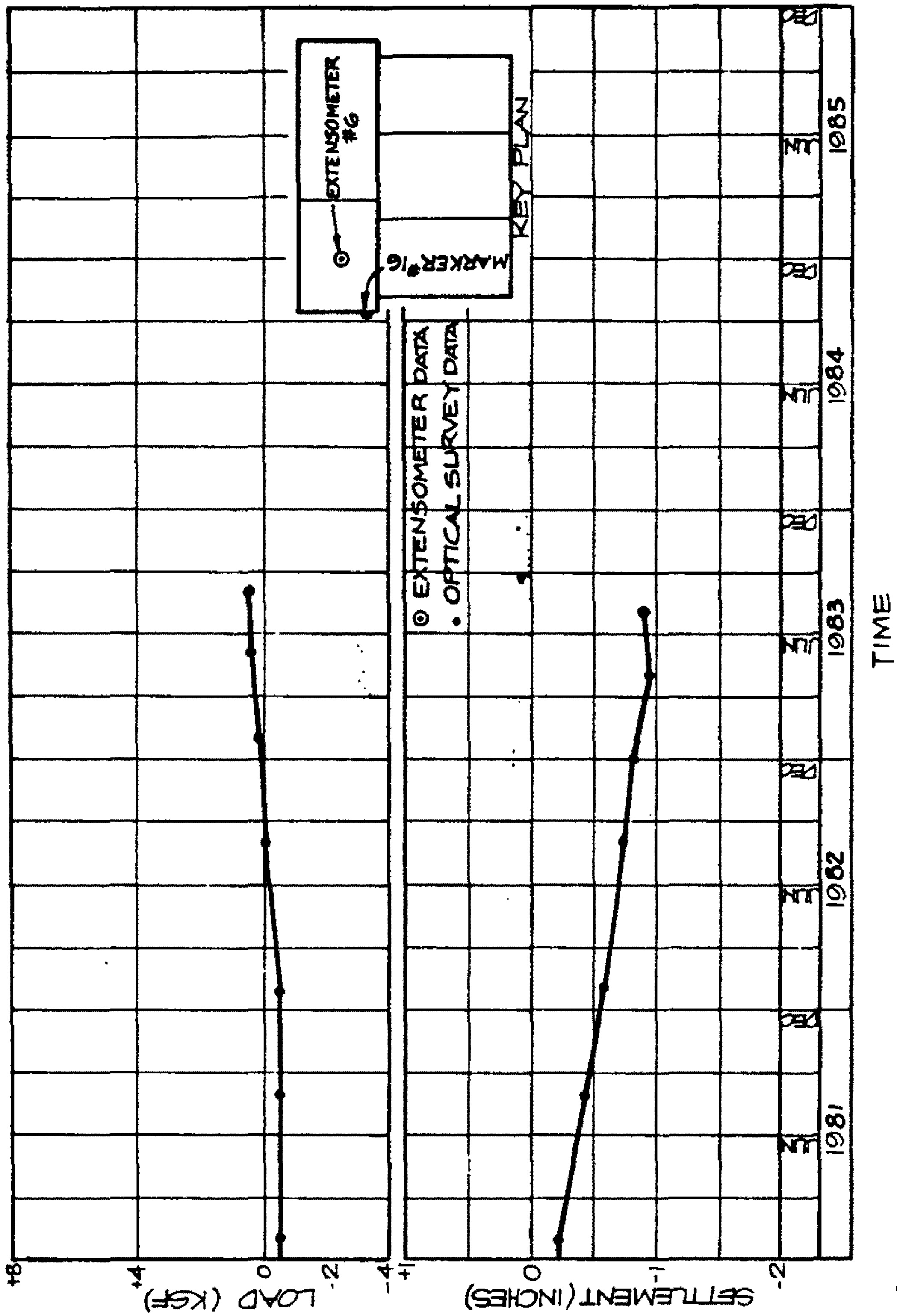
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 16

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-82



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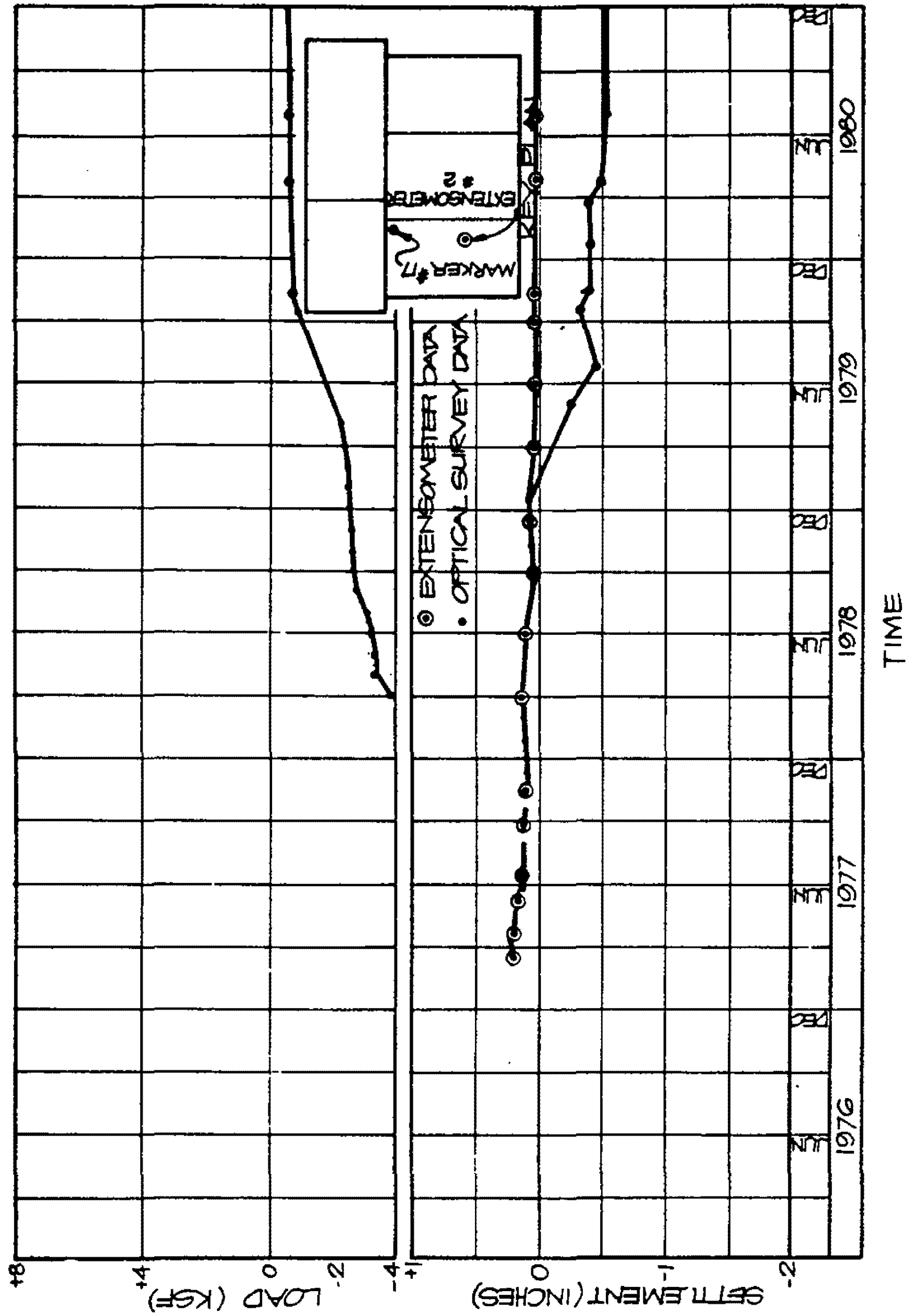
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 16

UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-82



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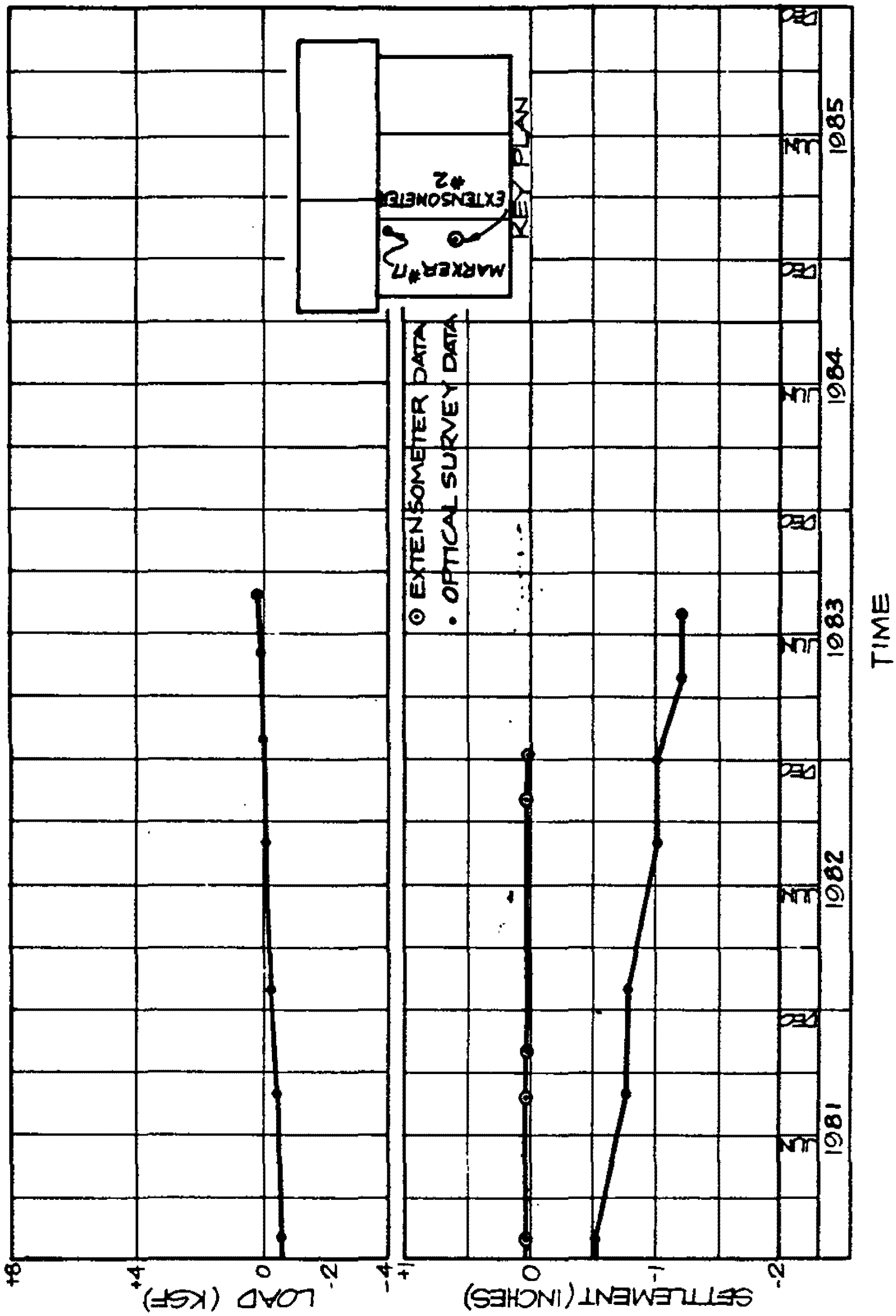
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 17

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-83



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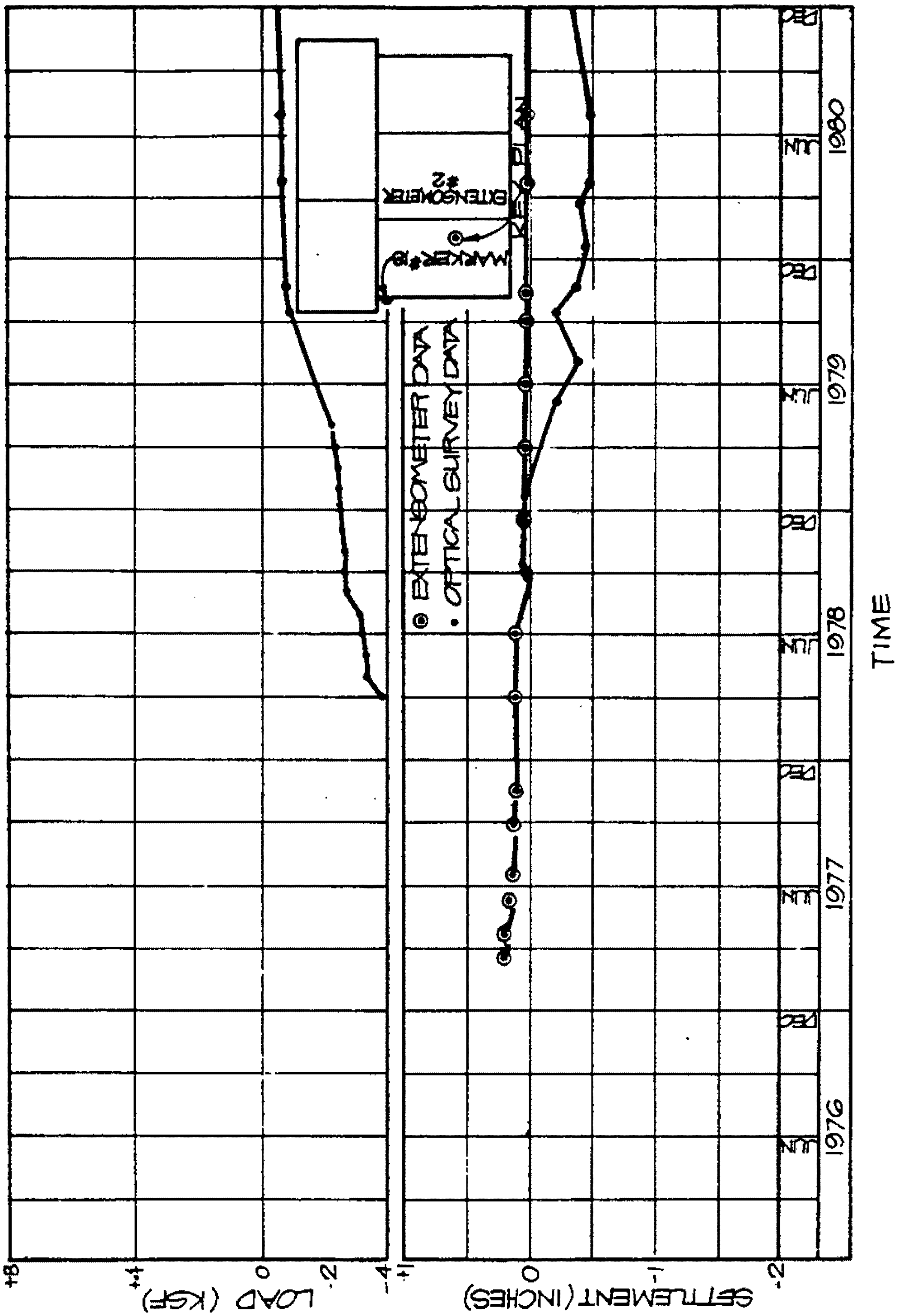
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 17

UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-83



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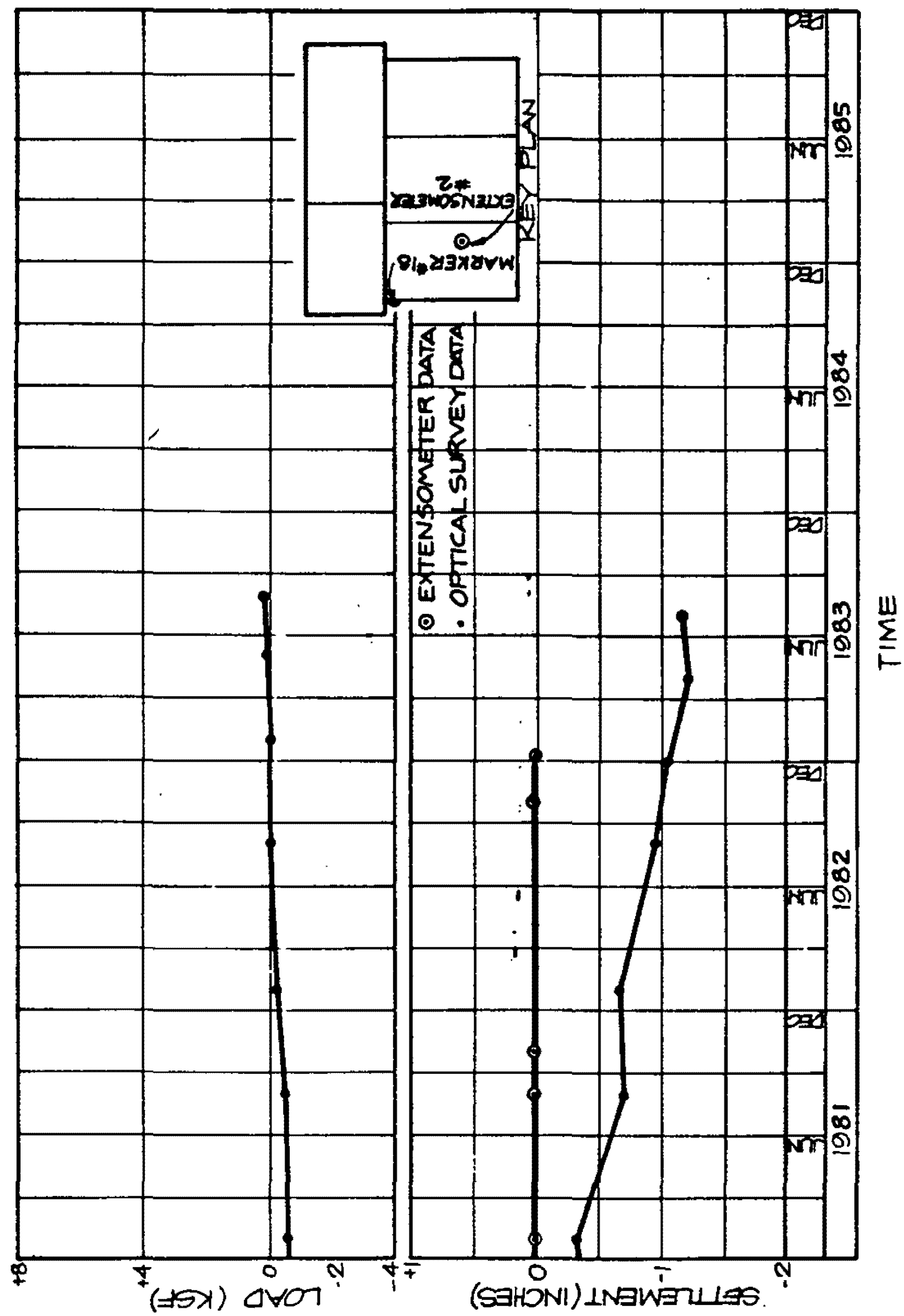
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 18

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-84



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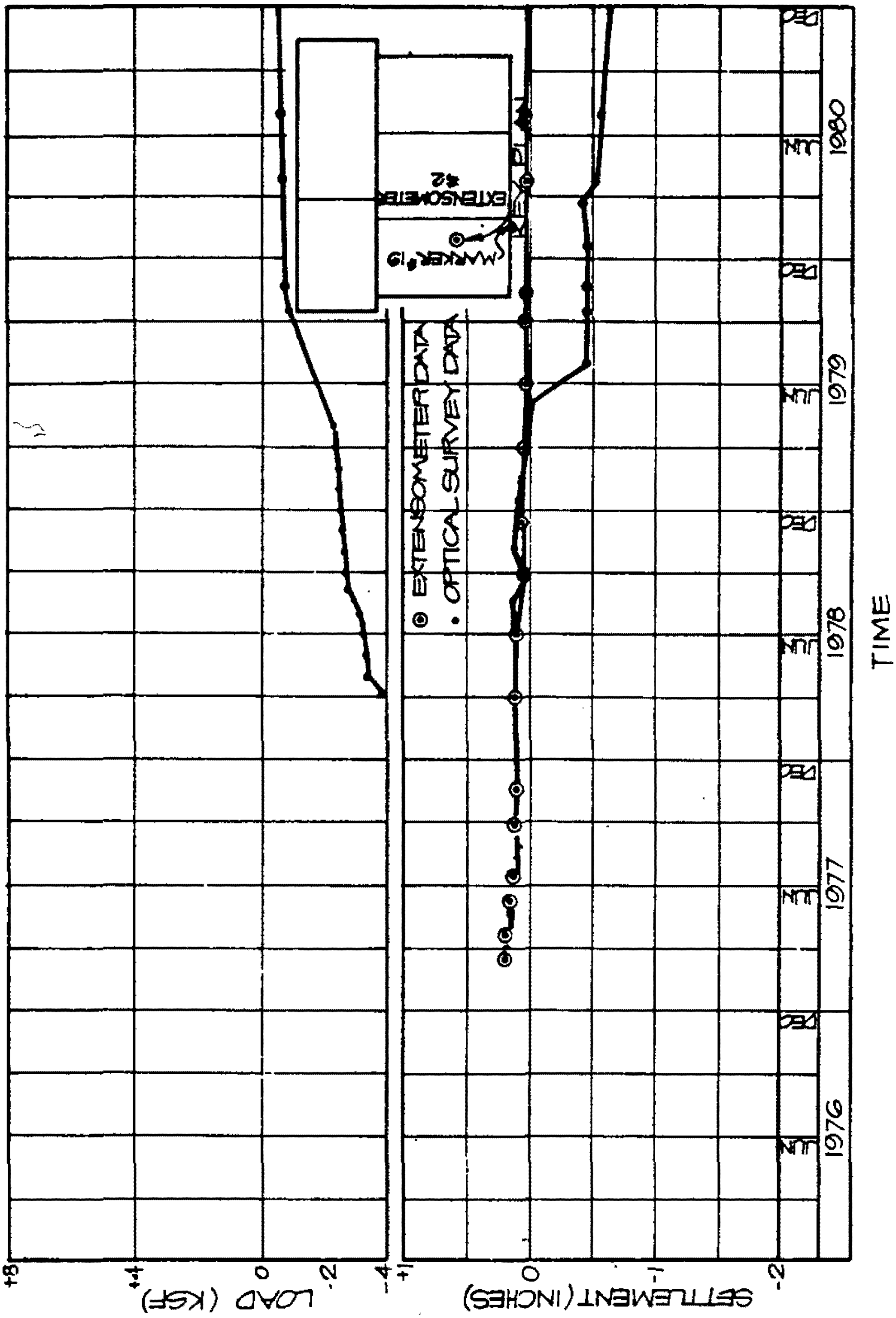
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 18

UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-84



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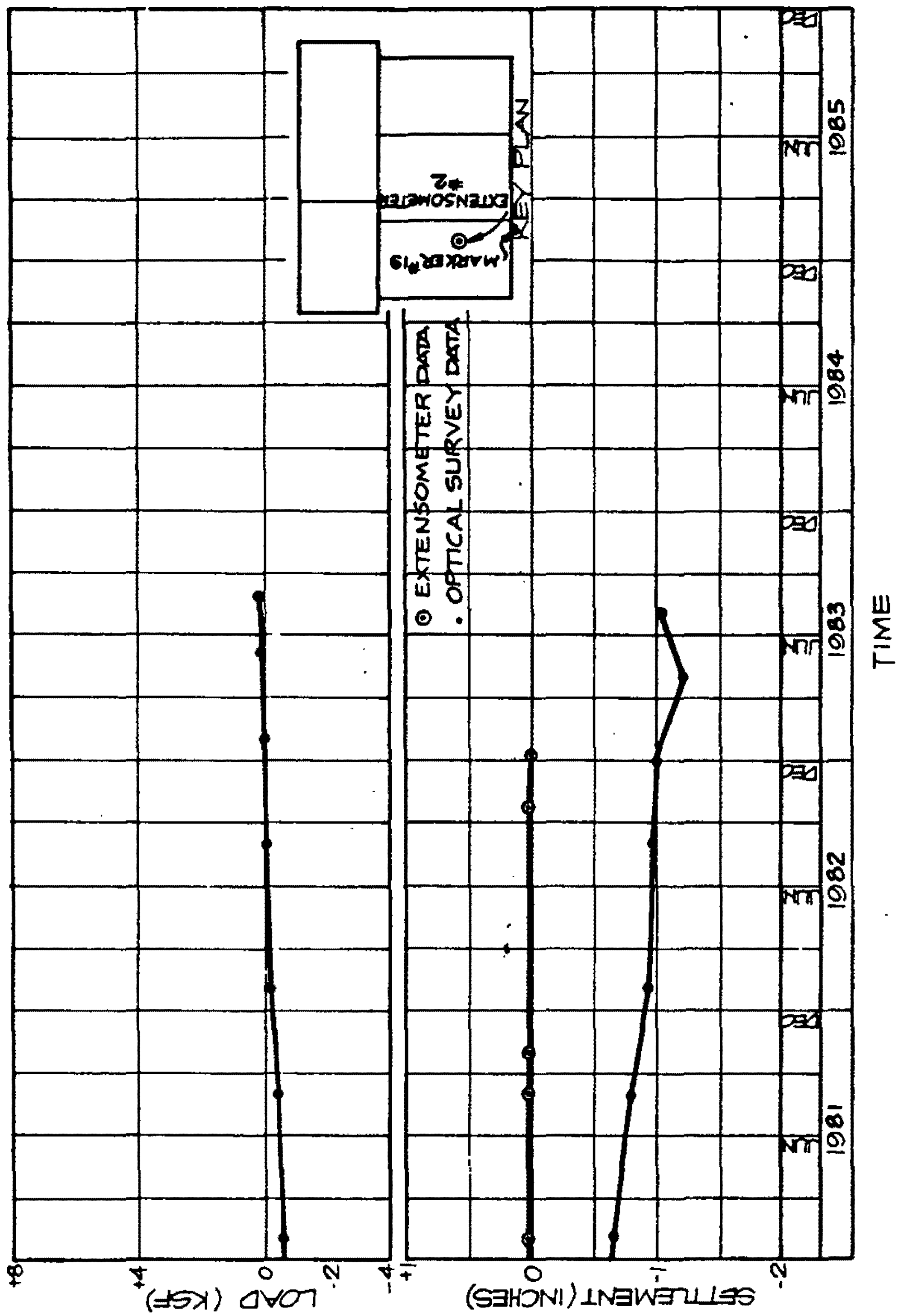
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 19

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-85



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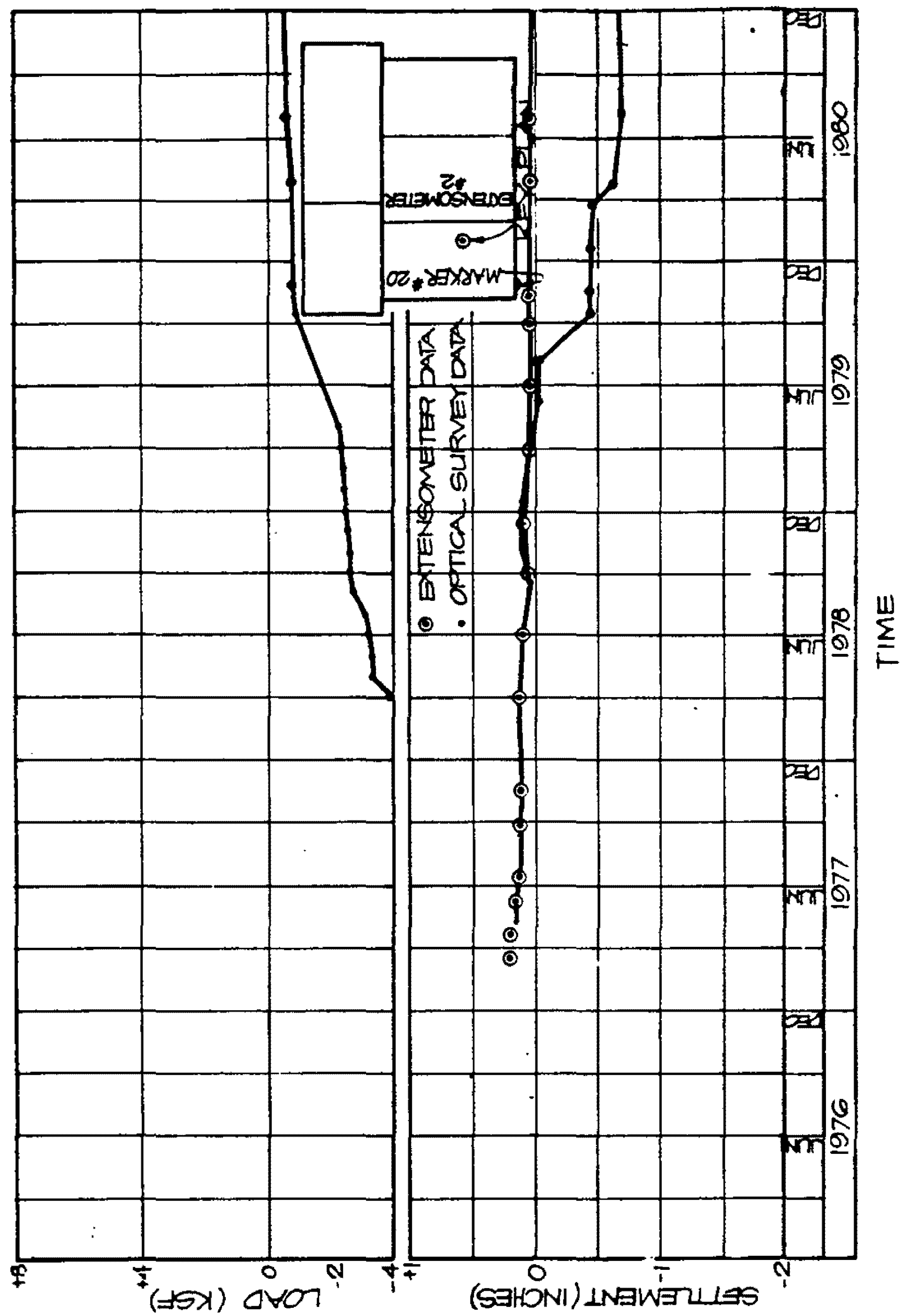
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 19

UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-85



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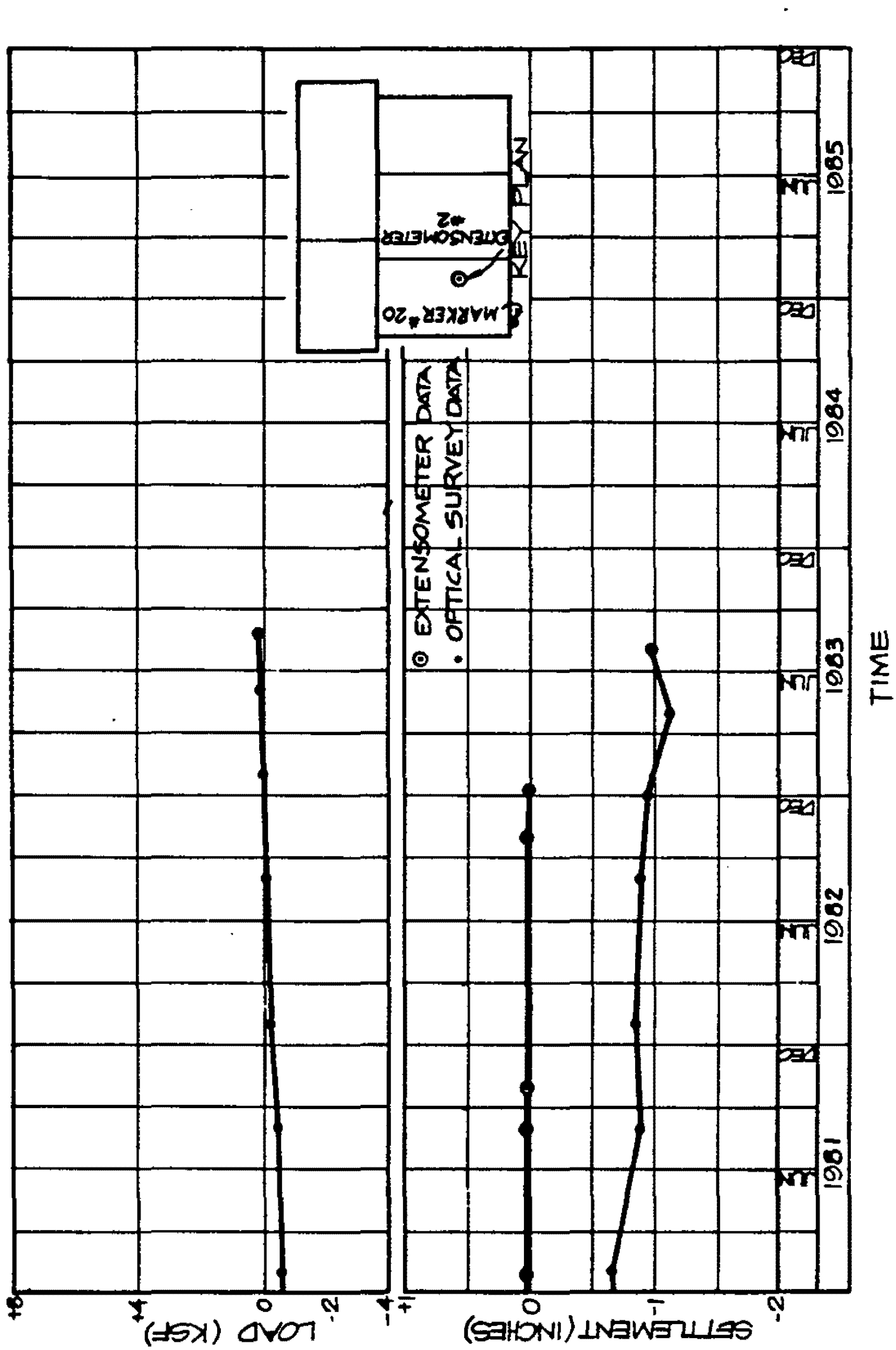
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 20

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-86



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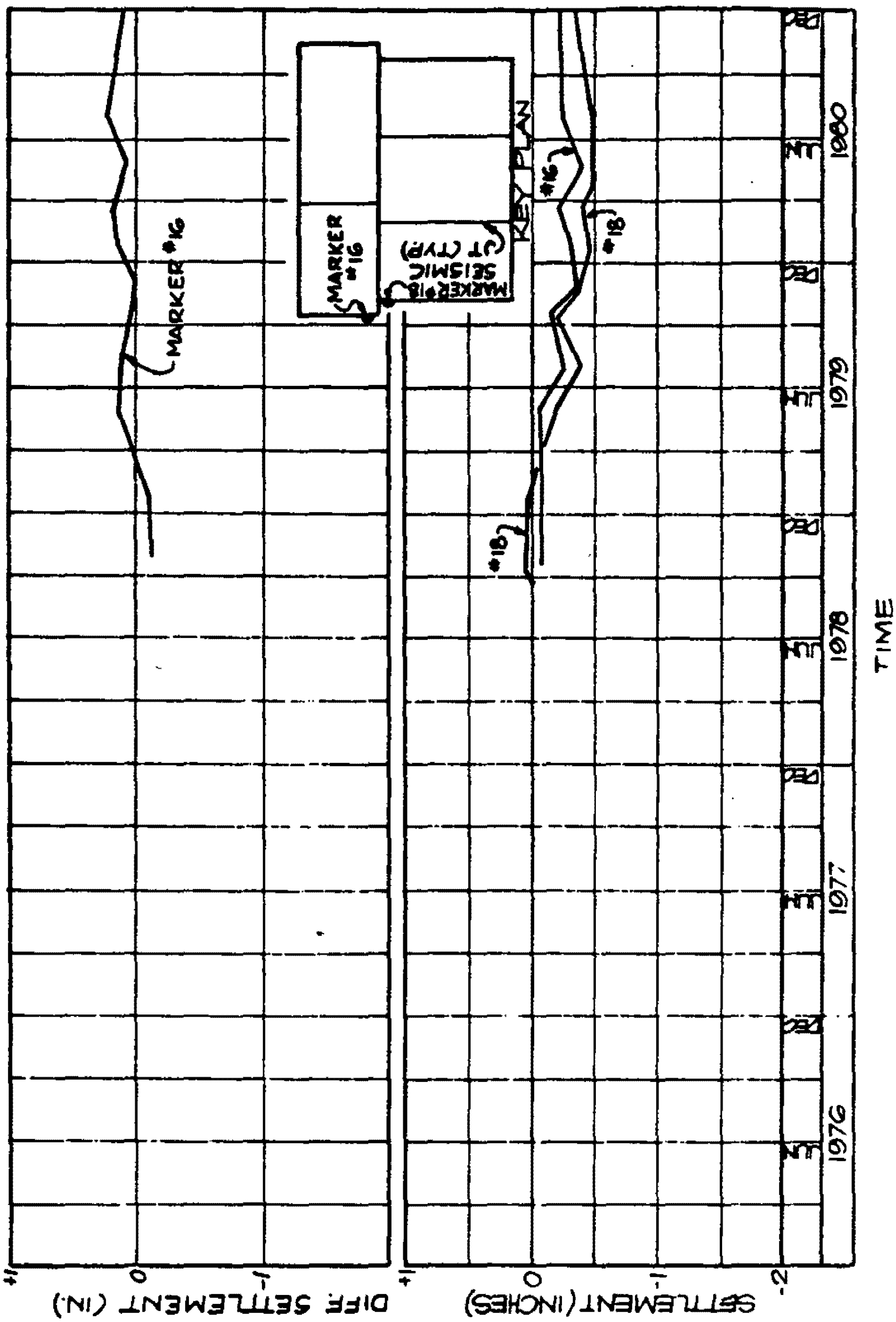
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

LOAD/SETTLEMENT PLOT
FOR MARKER NO. 20

UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-86



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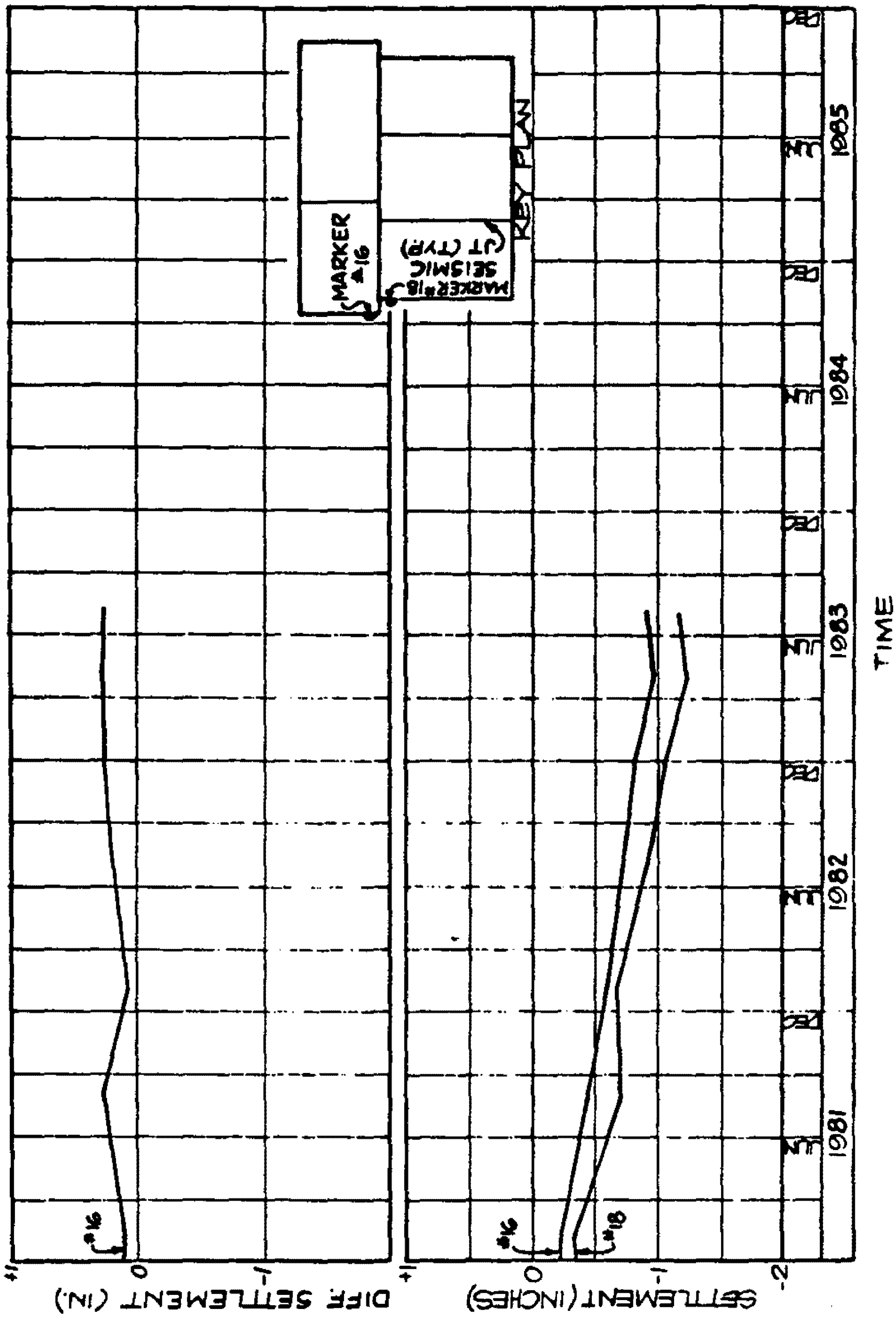
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

DIFFERENTIAL/LOAD SETTLEMENT
PLOT FOR MARKER
NO. 16 vs 18

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-87



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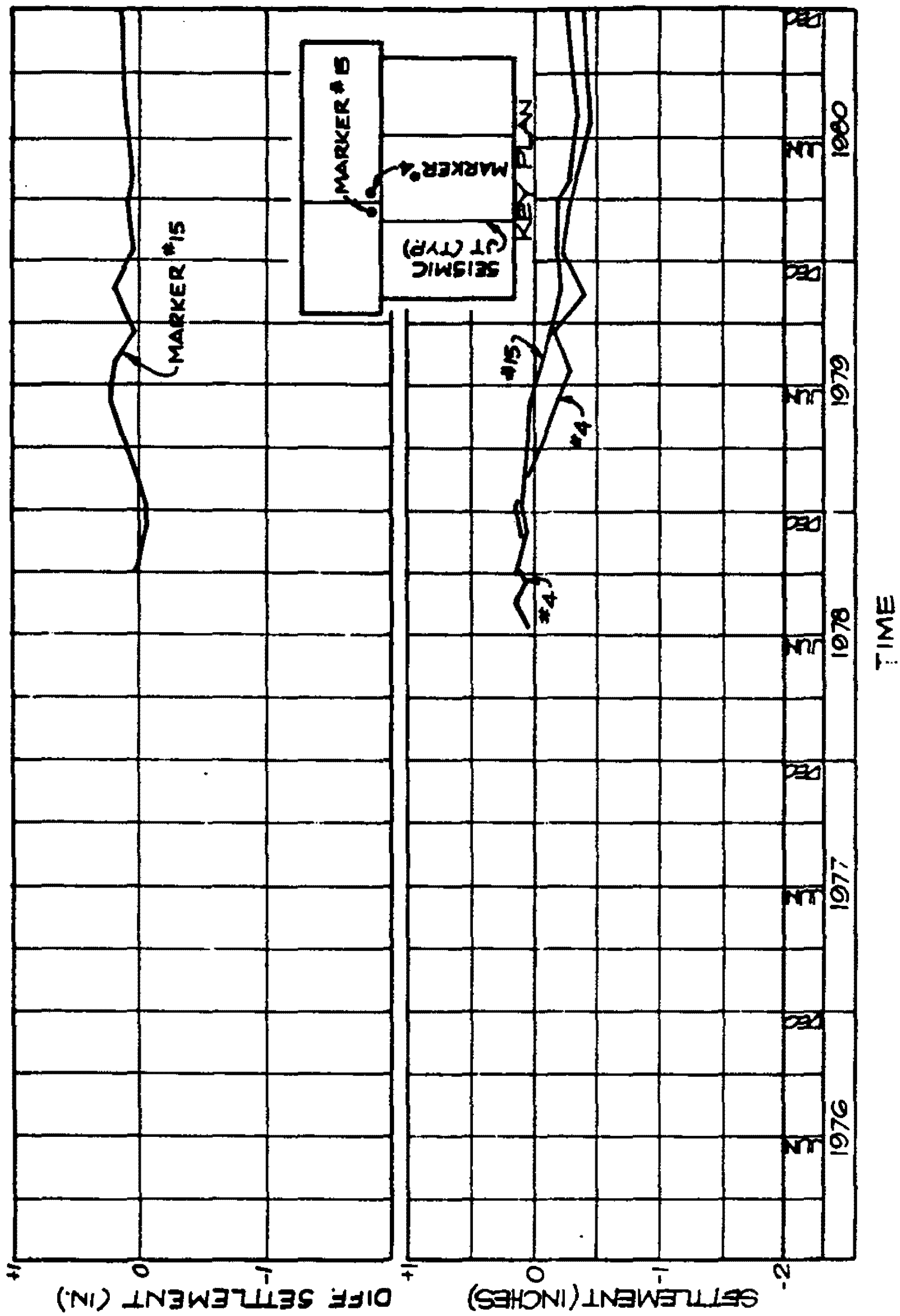
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

DIFFERENTIAL/LOAD SETTLEMENT
PLOT FOR MARKER
NO. 16 vs 18

UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-87



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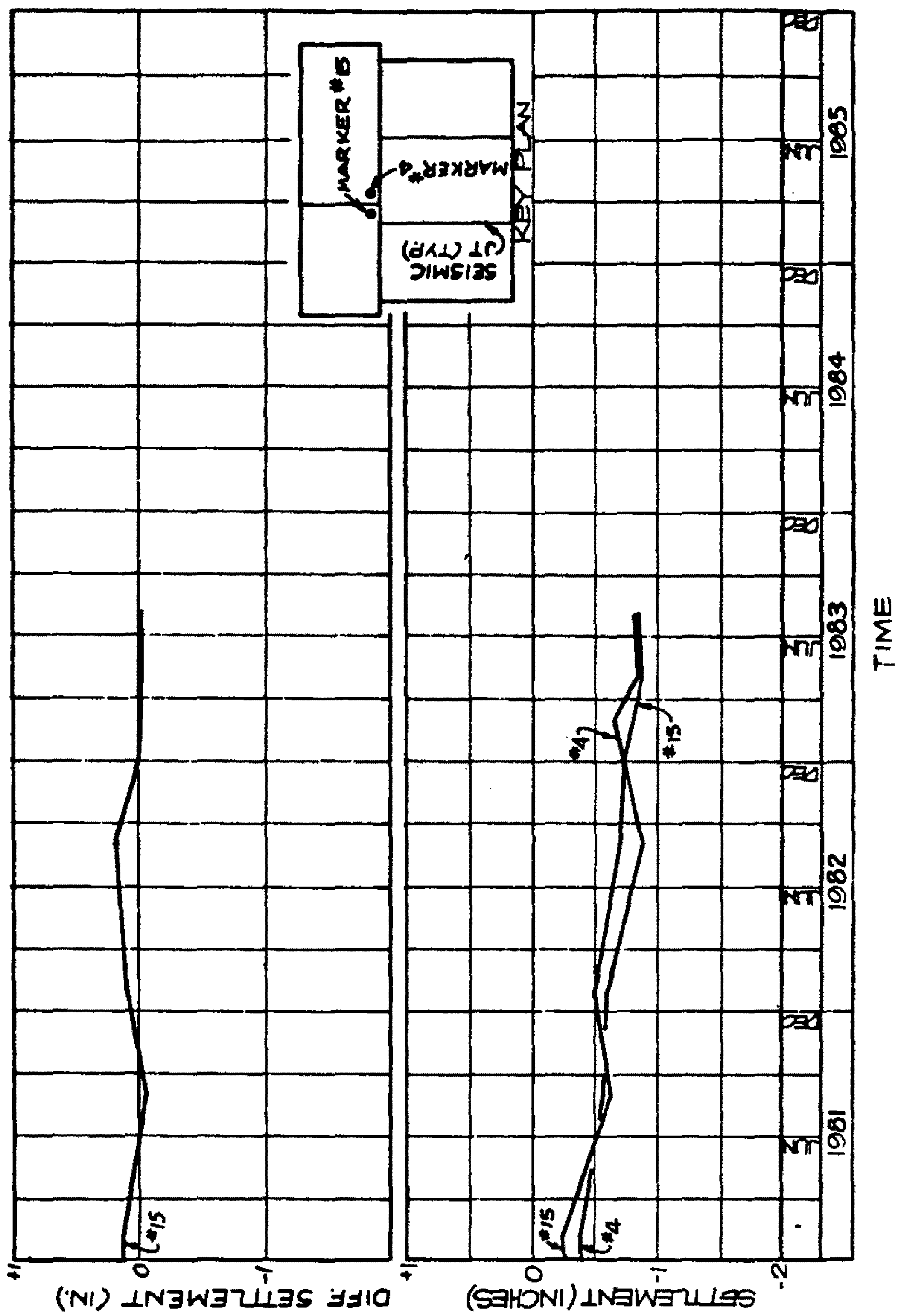
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

DIFFERENTIAL/LOAD SETTLEMENT
PLOT FOR MARKER
NO. 15 vs 4

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-88



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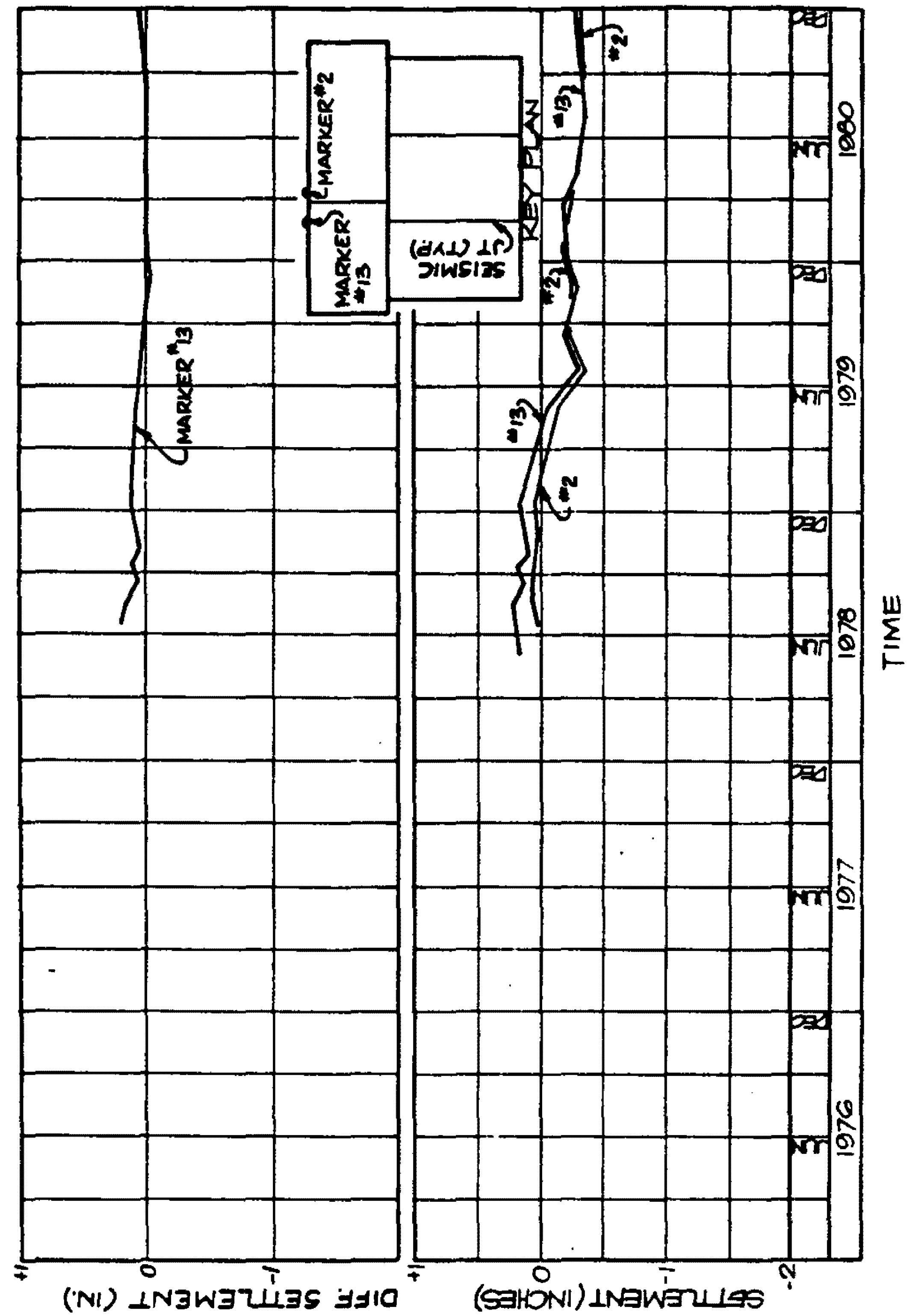
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY HOPE CREEK NUCLEAR GENERATING STATION
DIFFERENTIAL/LOAD SETTLEMENT PLOT FOR MARKER NO. 15 vs 4
UPDATED FSAR — Sheet 2 of 2 — FIGURE 2.5-88



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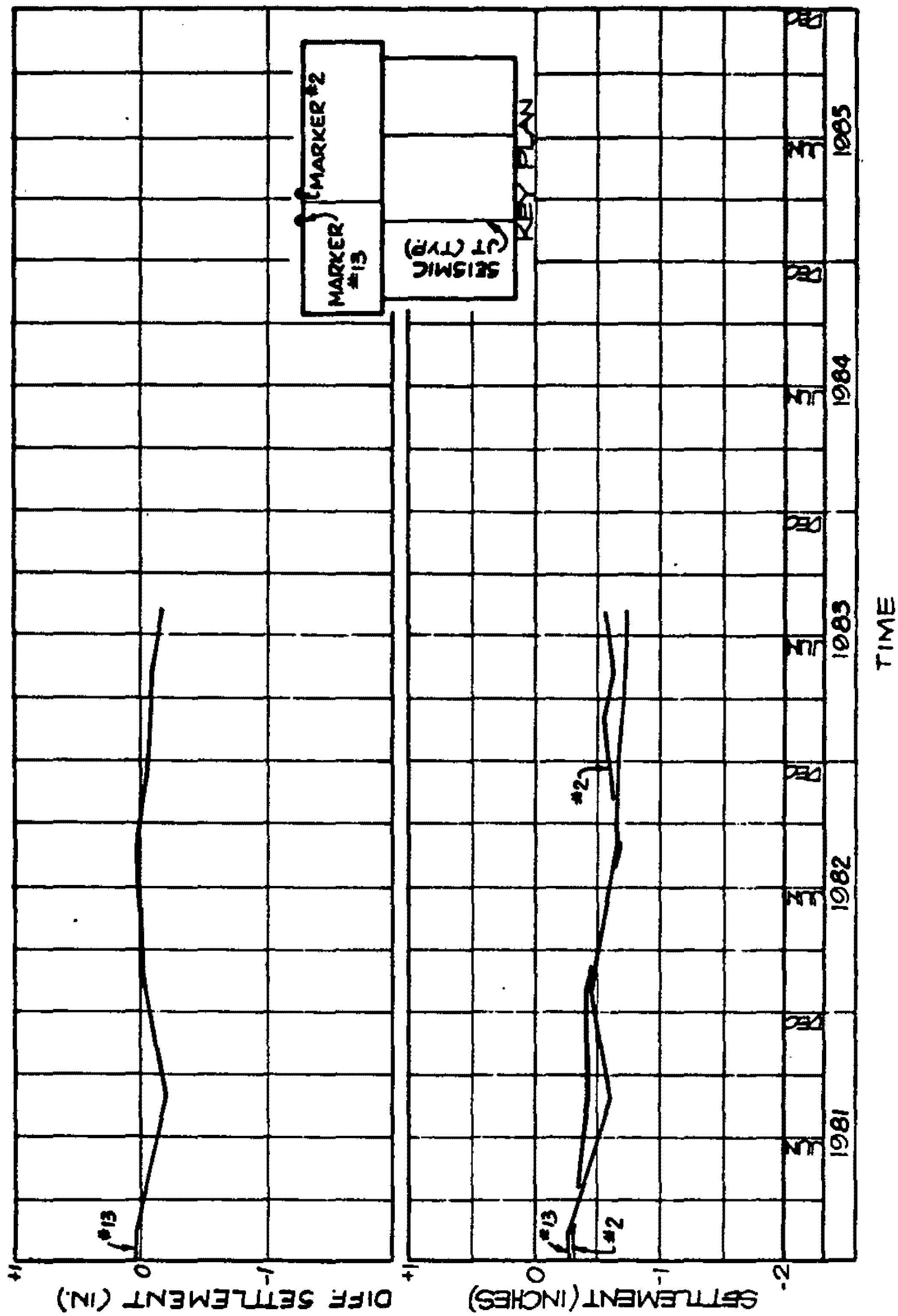
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

DIFFERENTIAL/LOAD SETTLEMENT
PLOT FOR MARKER
NO. 13 vs 2

UPDATED FSAR	Sheet 1 of 2 **FIGURE 2.5-89**



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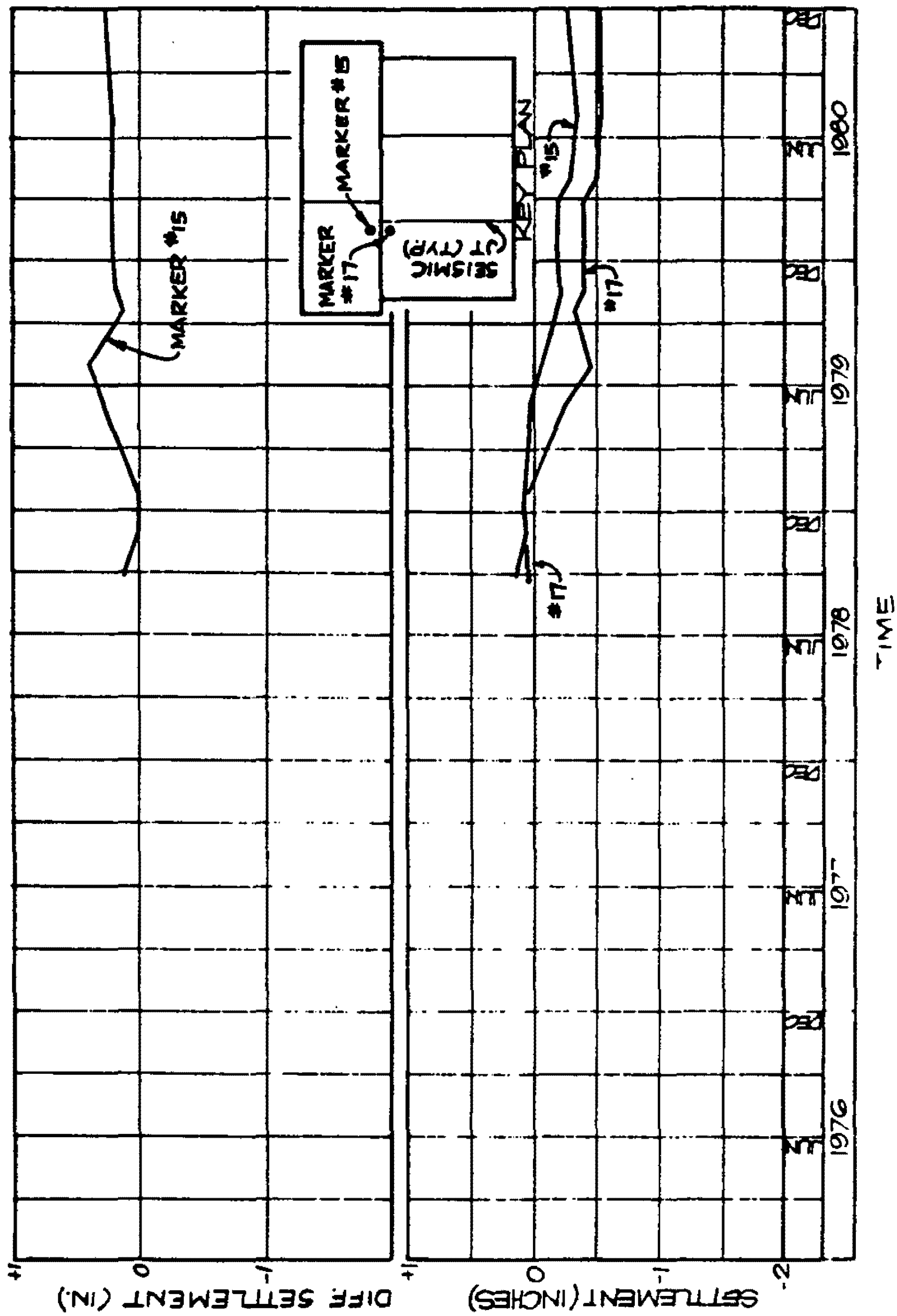
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

DIFFERENTIAL/LOAD SETTLEMENT
PLOT FOR MARKER
NO. 13 vs 2

UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-89



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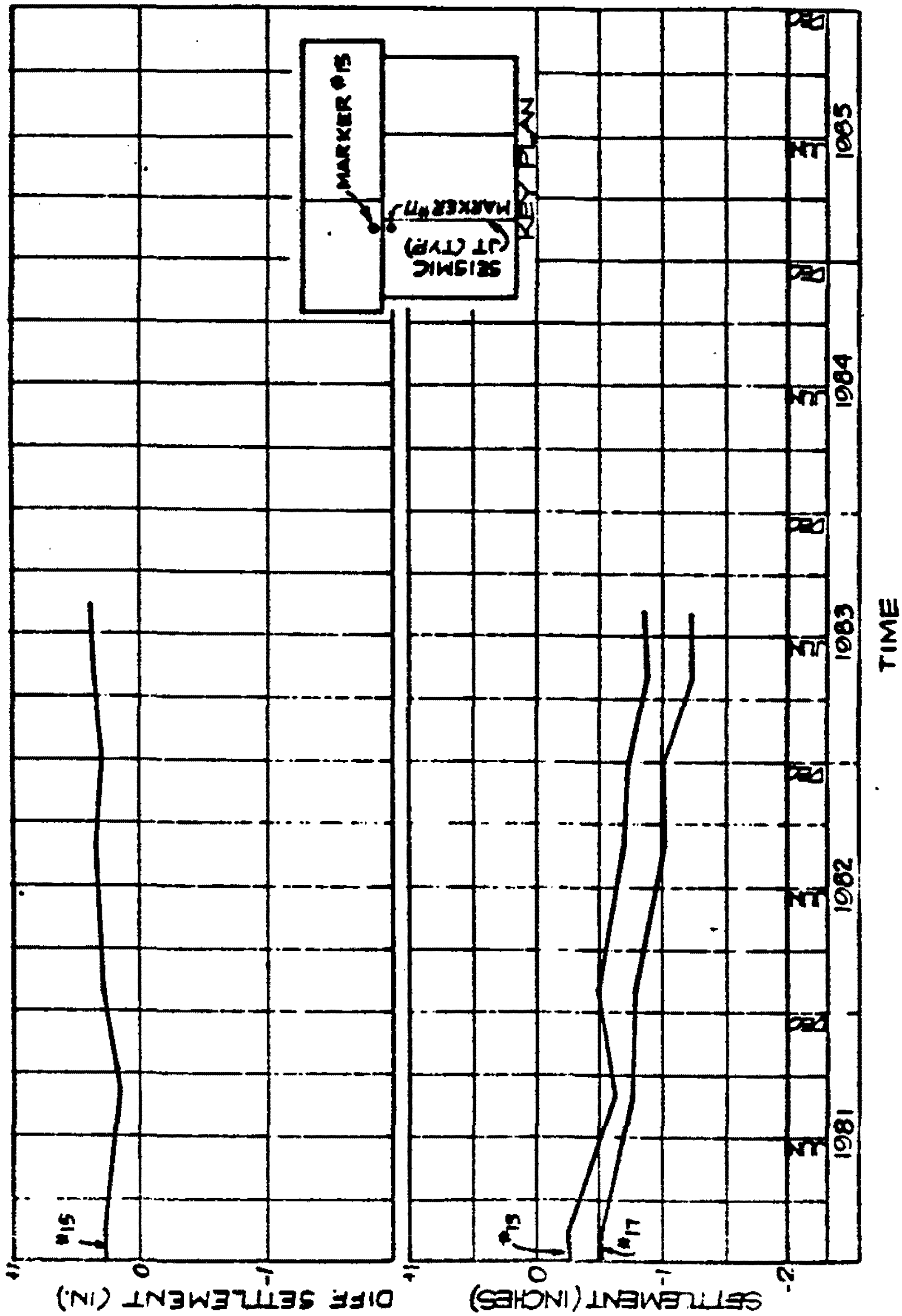
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

DIFFERENTIAL/LOAD SETTLEMENT
PLOT FOR MARKER
NO. 15 vs 17

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-90



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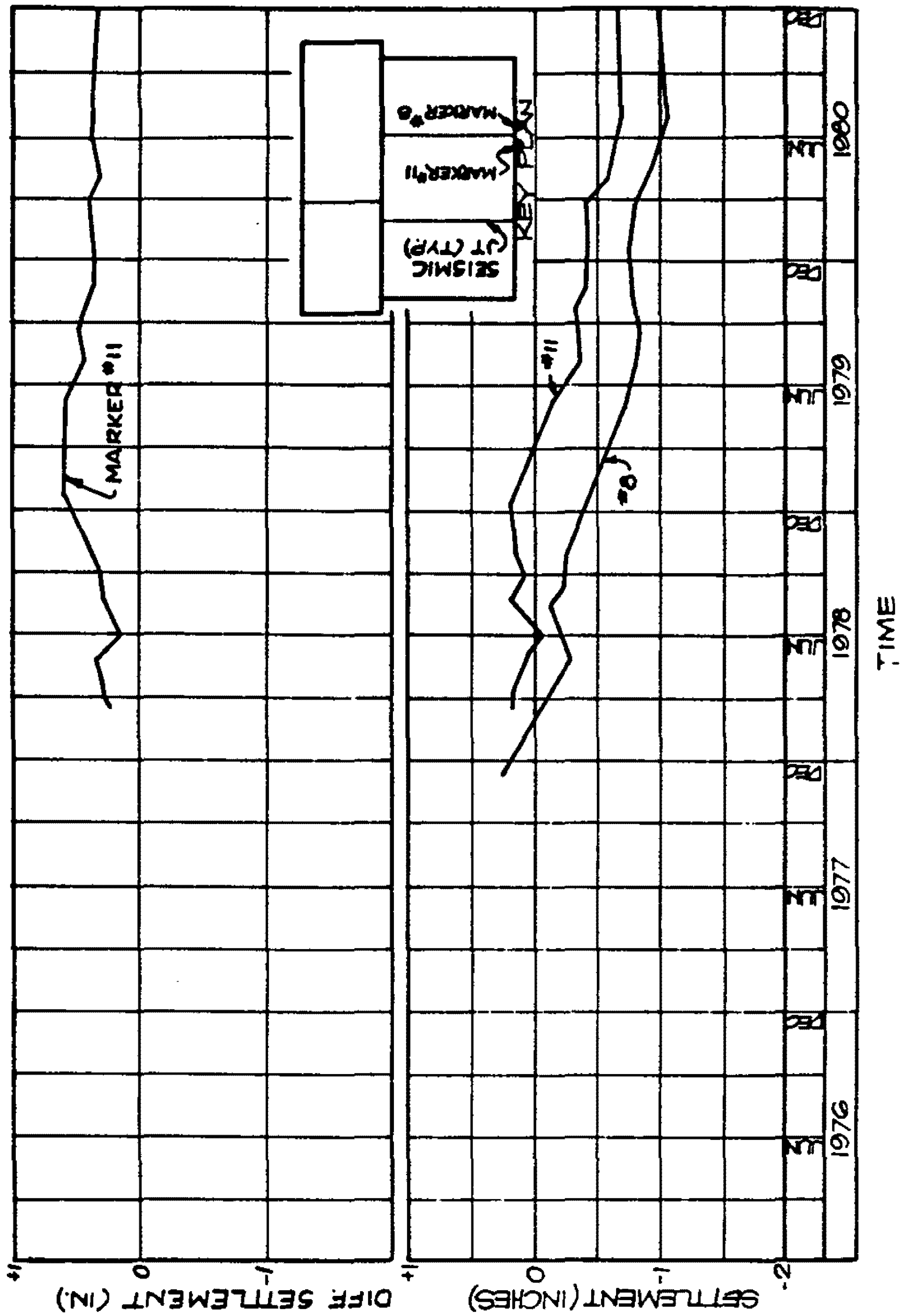
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

DIFFERENTIAL/LOAD SETTLEMENT
PLOT FOR MARKER NO. 15 vs 17

UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-90



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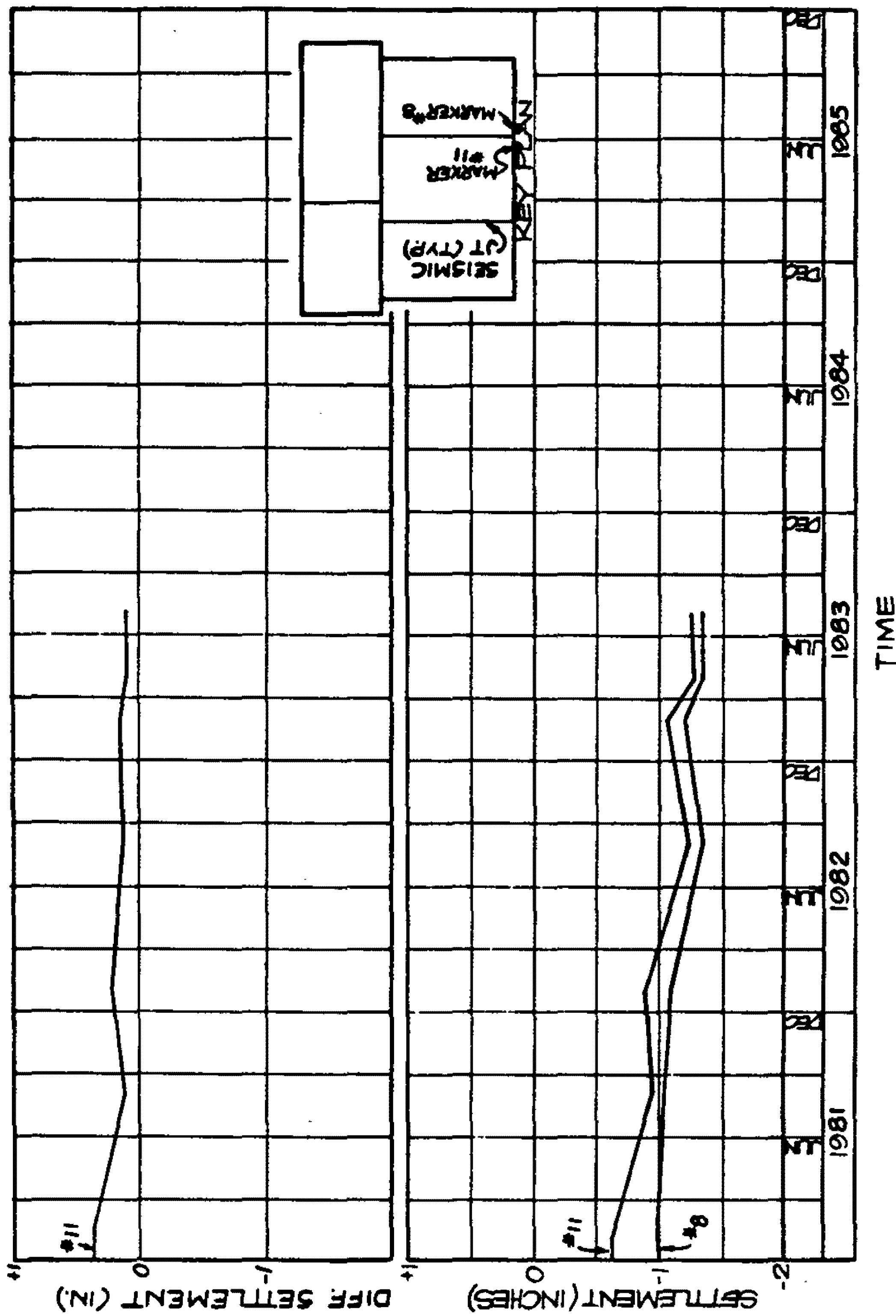
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

DIFFERENTIAL/LOAD SETTLEMENT
PLOT FOR MARKER
NO. 11 vs 8

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-91



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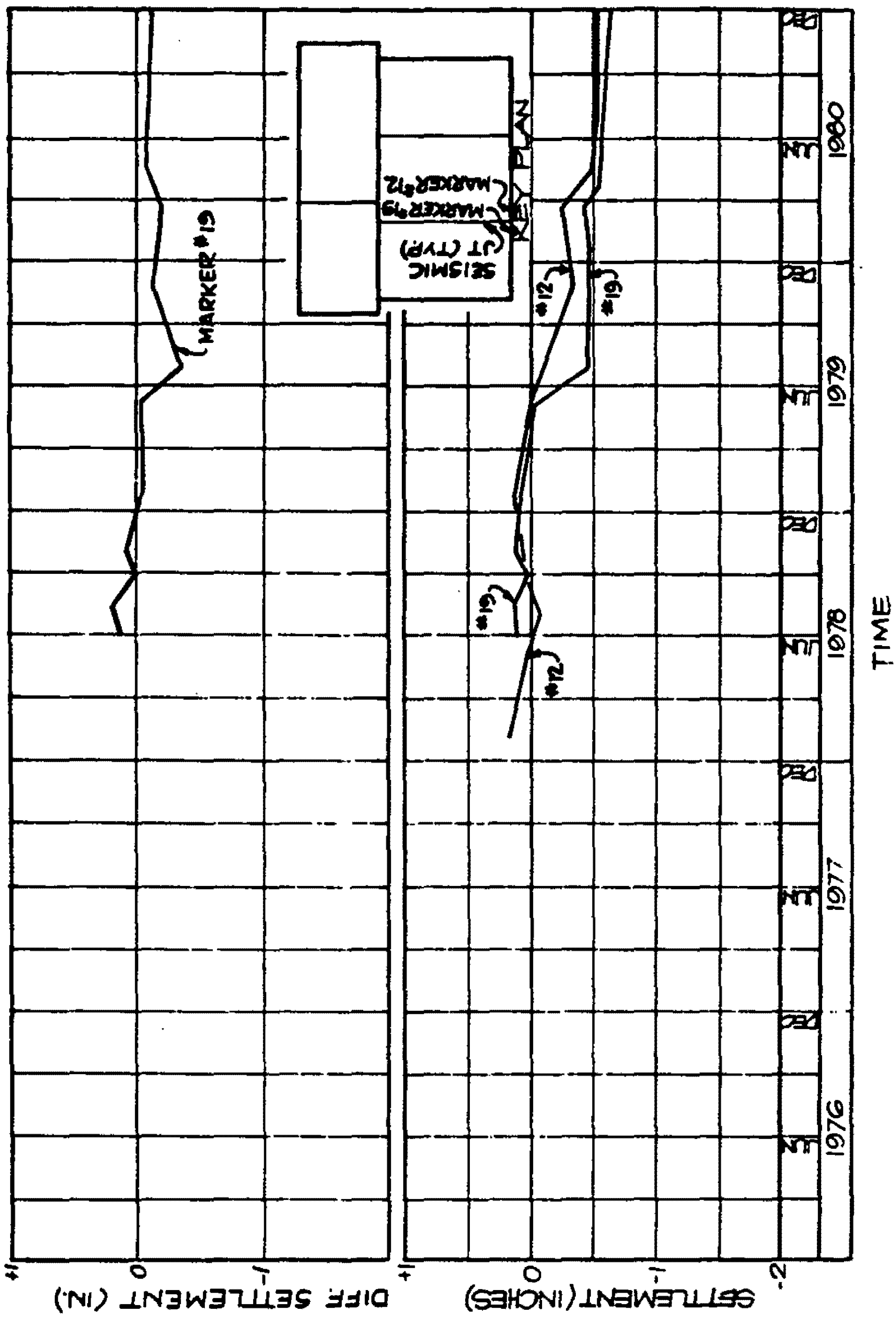
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

DIFFERENTIAL/LOAD SETTLEMENT
PLOT FOR MARKER
NO. 11 vs 8

UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-91



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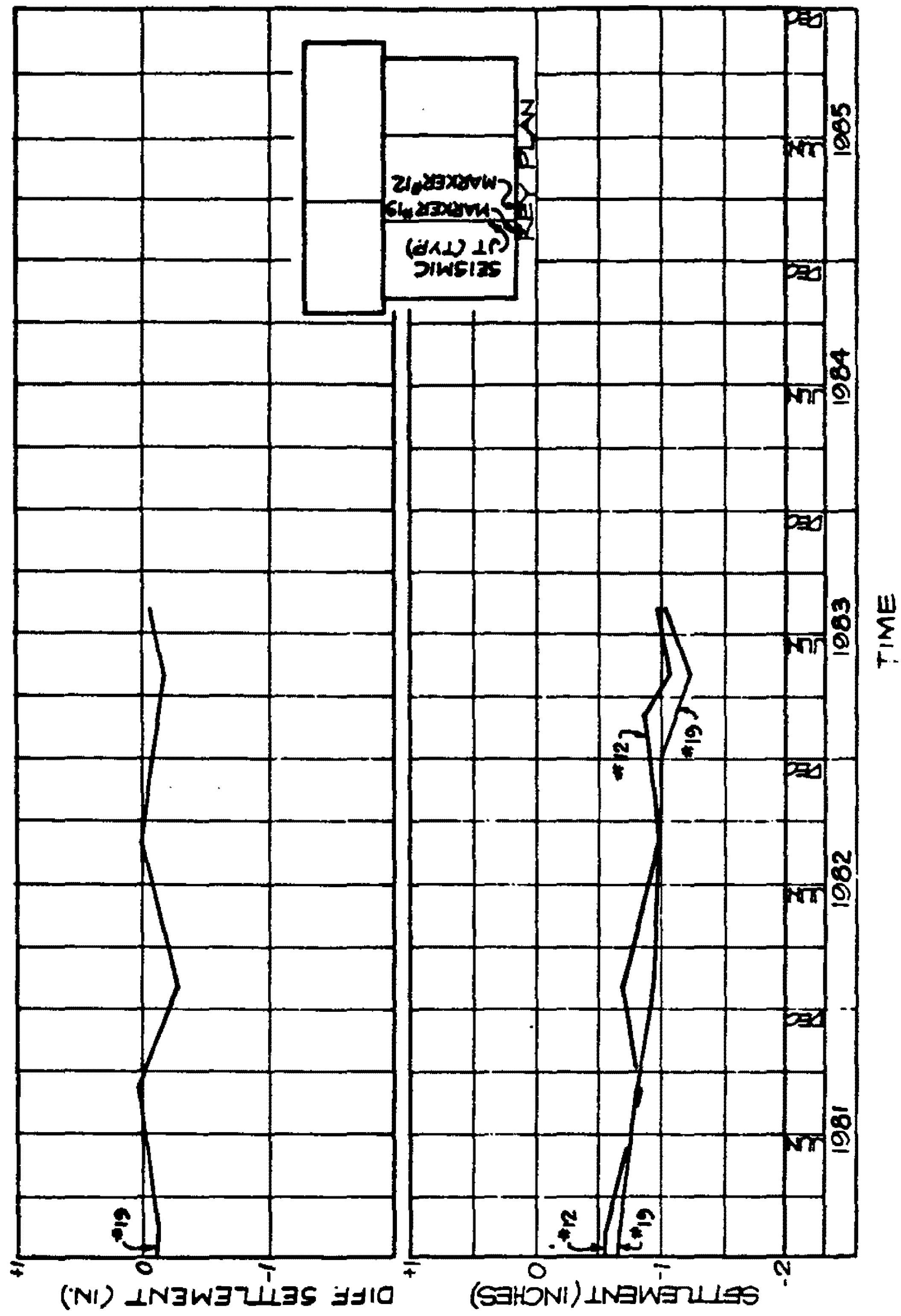
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

DIFFERENTIAL/LOAD SETTLEMENT
PLOT FOR MARKER
NO. 19 vs 12

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-92



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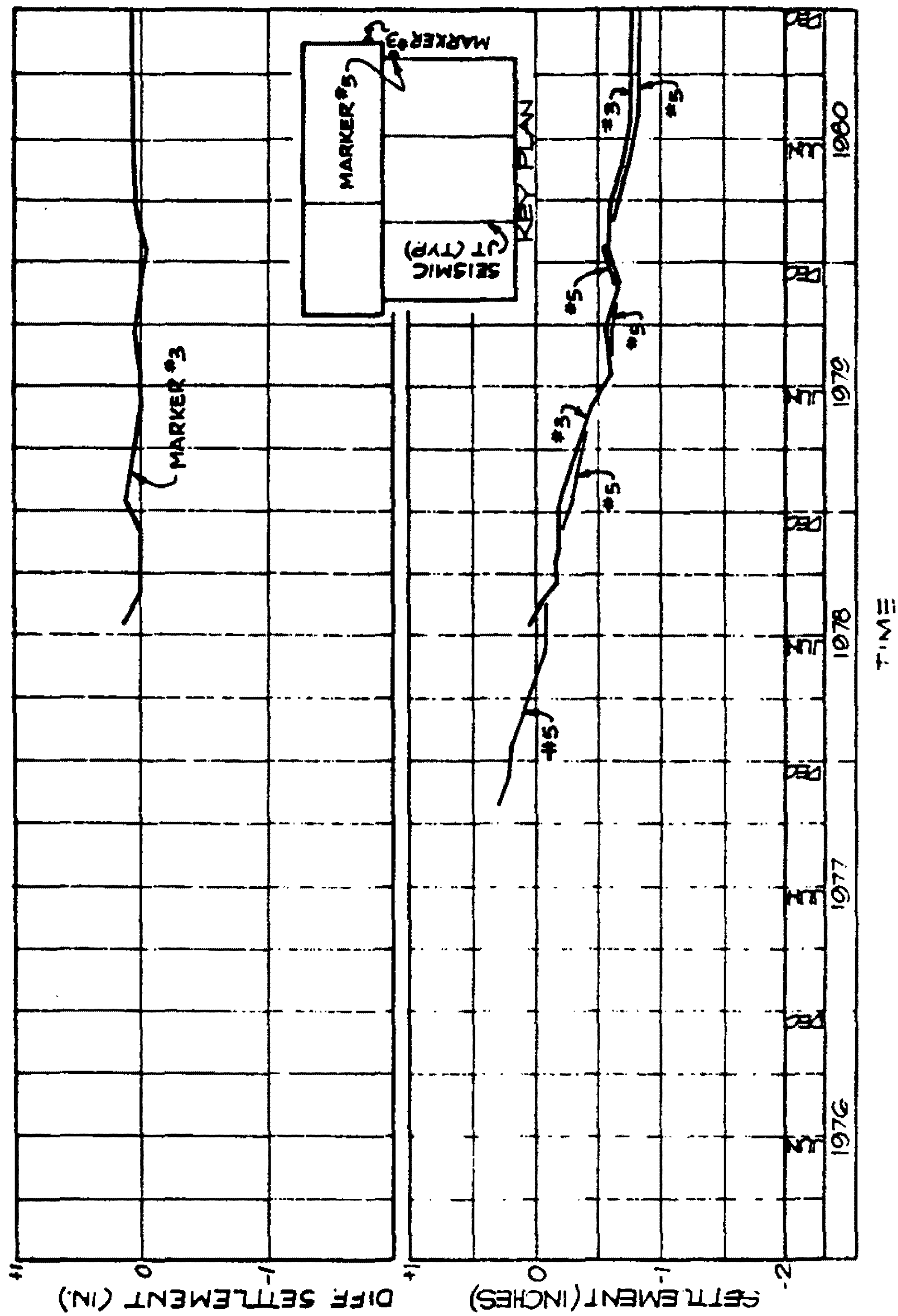
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

DIFFERENTIAL/LOAD SETTLEMENT
PLOT FOR MARKER
NO. 19 vs 12

UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-92



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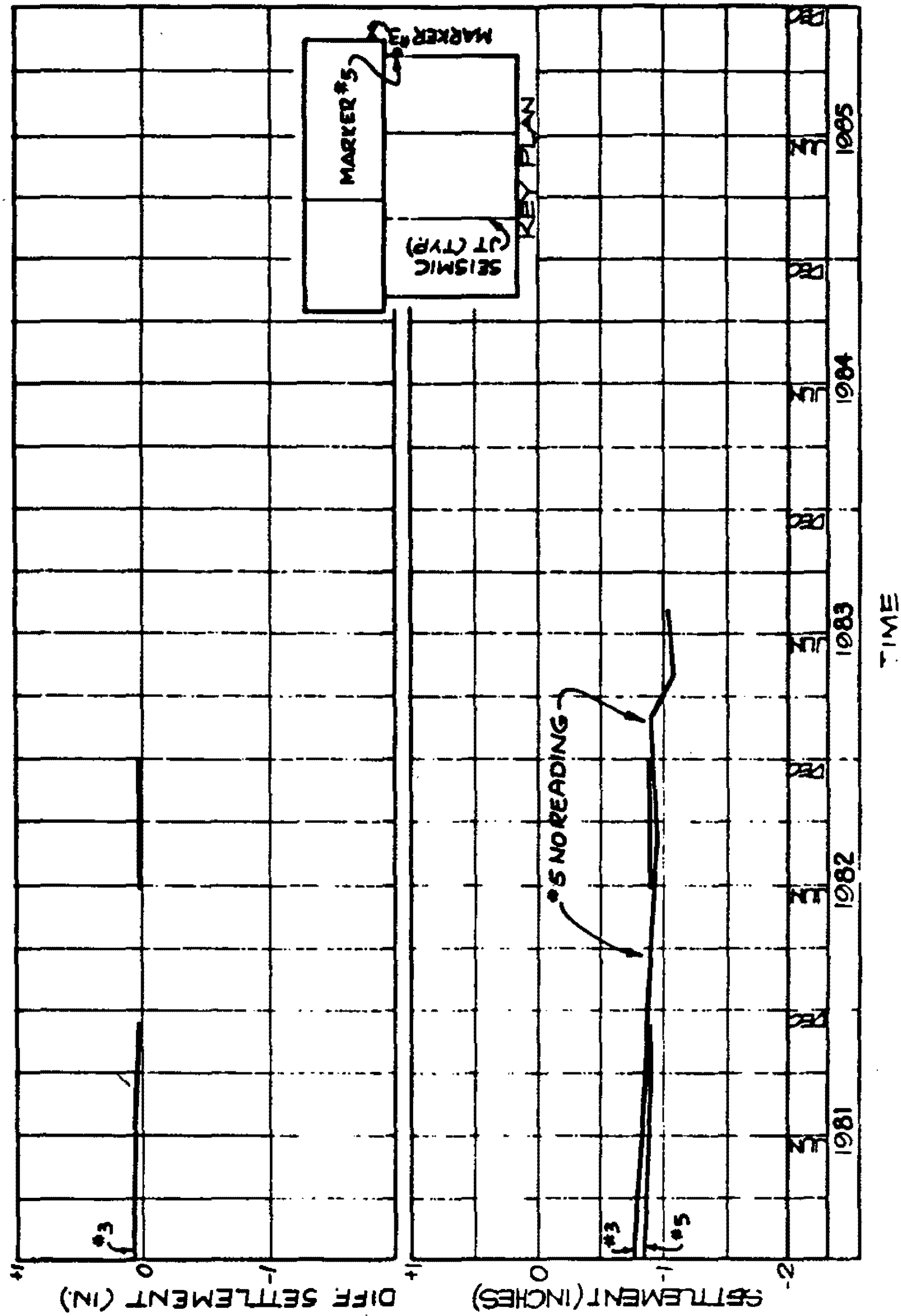
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

DIFFERENTIAL/LOAD SETTLEMENT
PLOT FOR MARKER
NO. 3 vs 5

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-93



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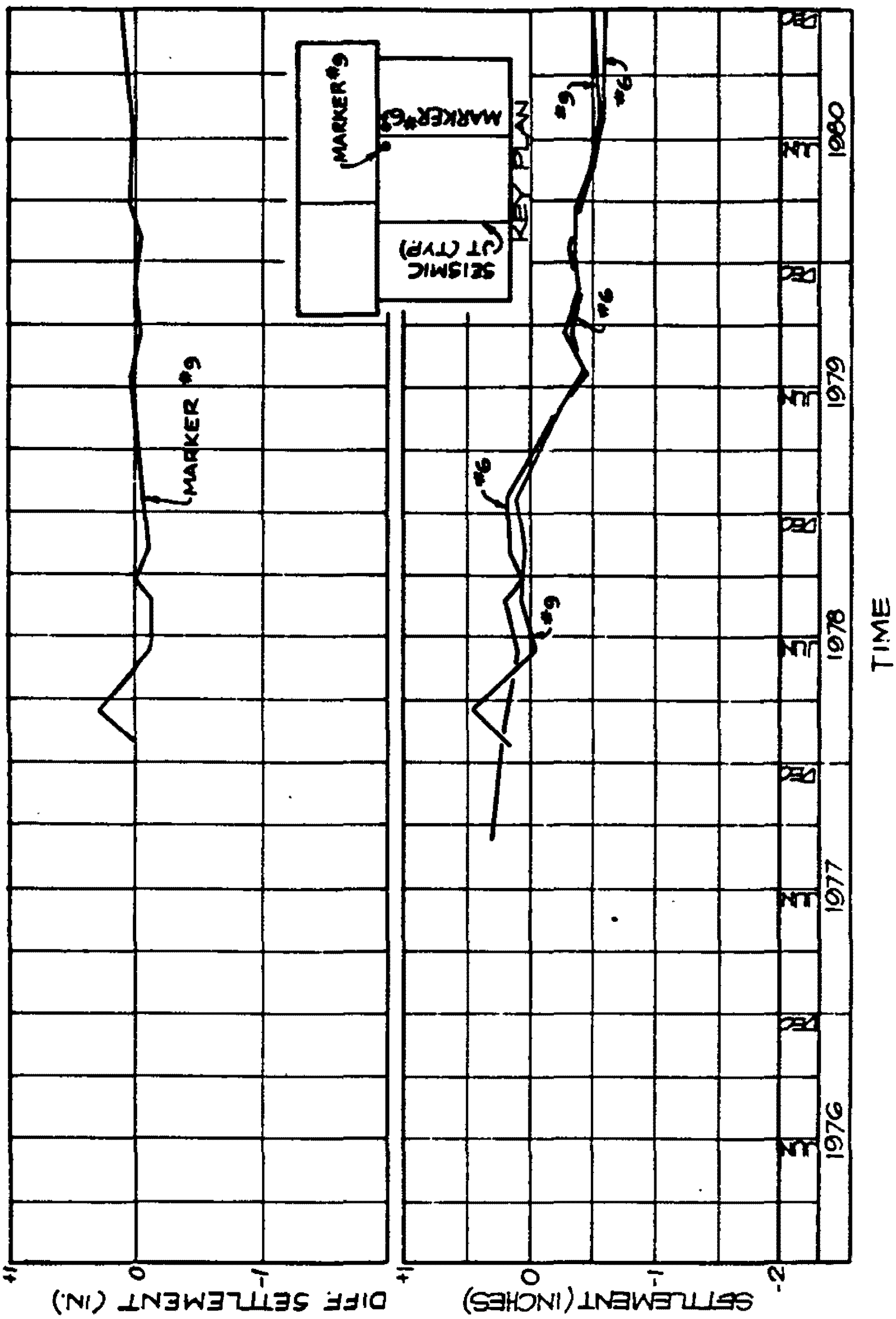
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

DIFFERENTIAL/LOAD SETTLEMENT
PLOT FOR MARKER
NO. 3 vs 5

UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-93



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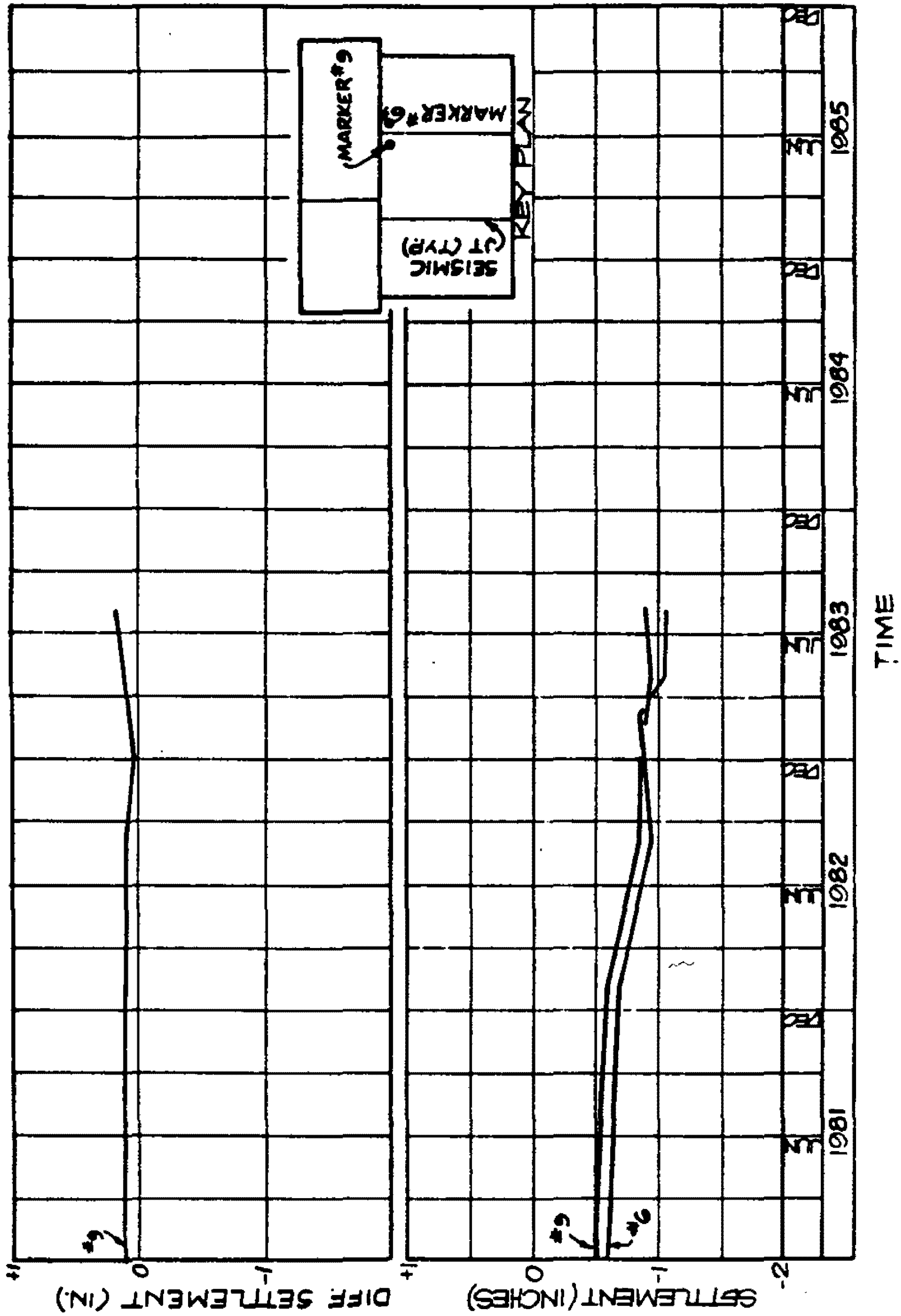
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY HOPE CREEK NUCLEAR GENERATING STATION	
DIFFERENTIAL/LOAD SETTLEMENT PLOT FOR MARKER NO. 9 vs 6	
UPDATED FSAR	Sheet 1 of 2 FIGURE 2.5-94



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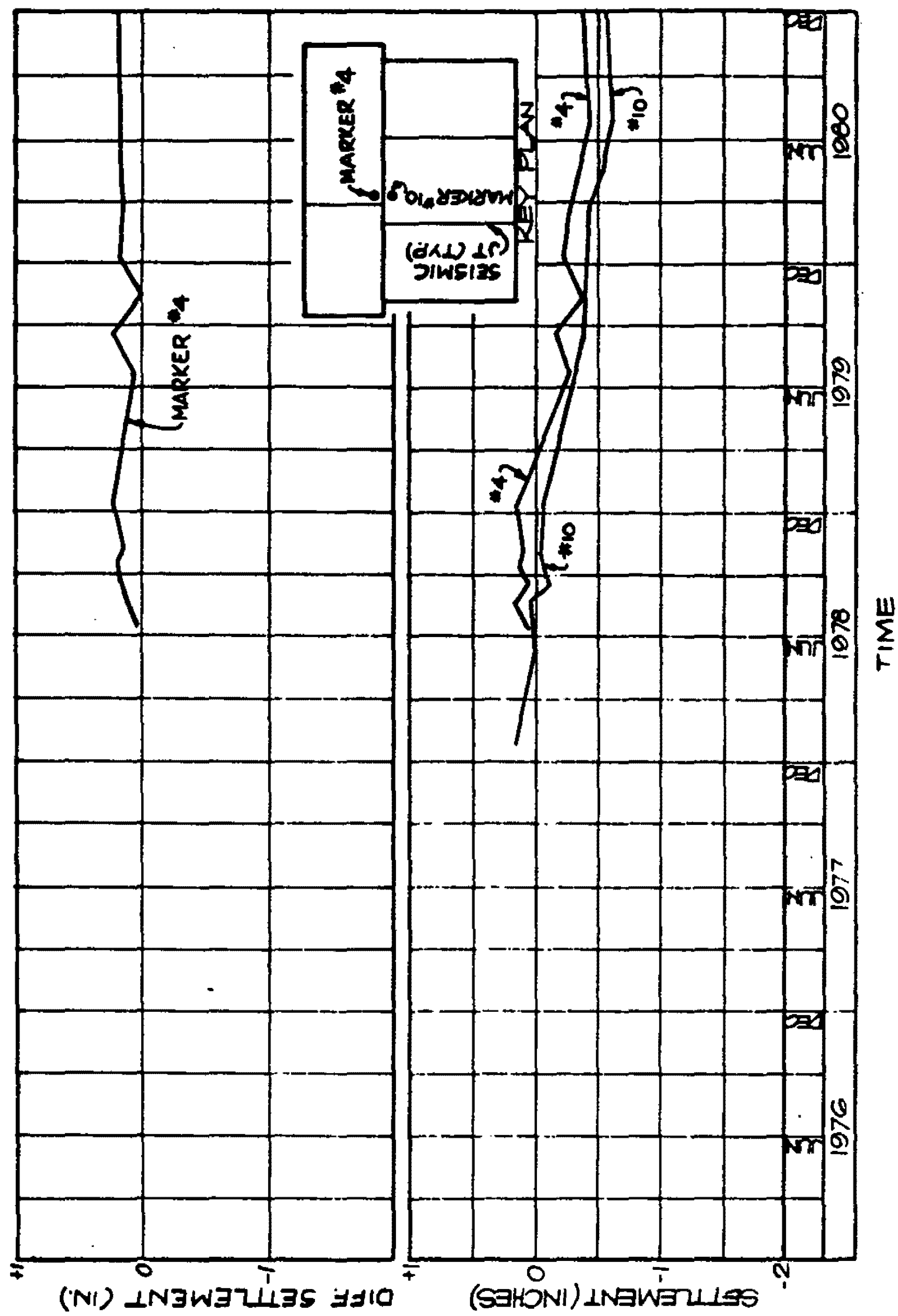
APRIL 11, 1988

PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

DIFFERENTIAL/LOAD SETTLEMENT
PLOT FOR MARKER
NO. 9 vs 6

UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-94



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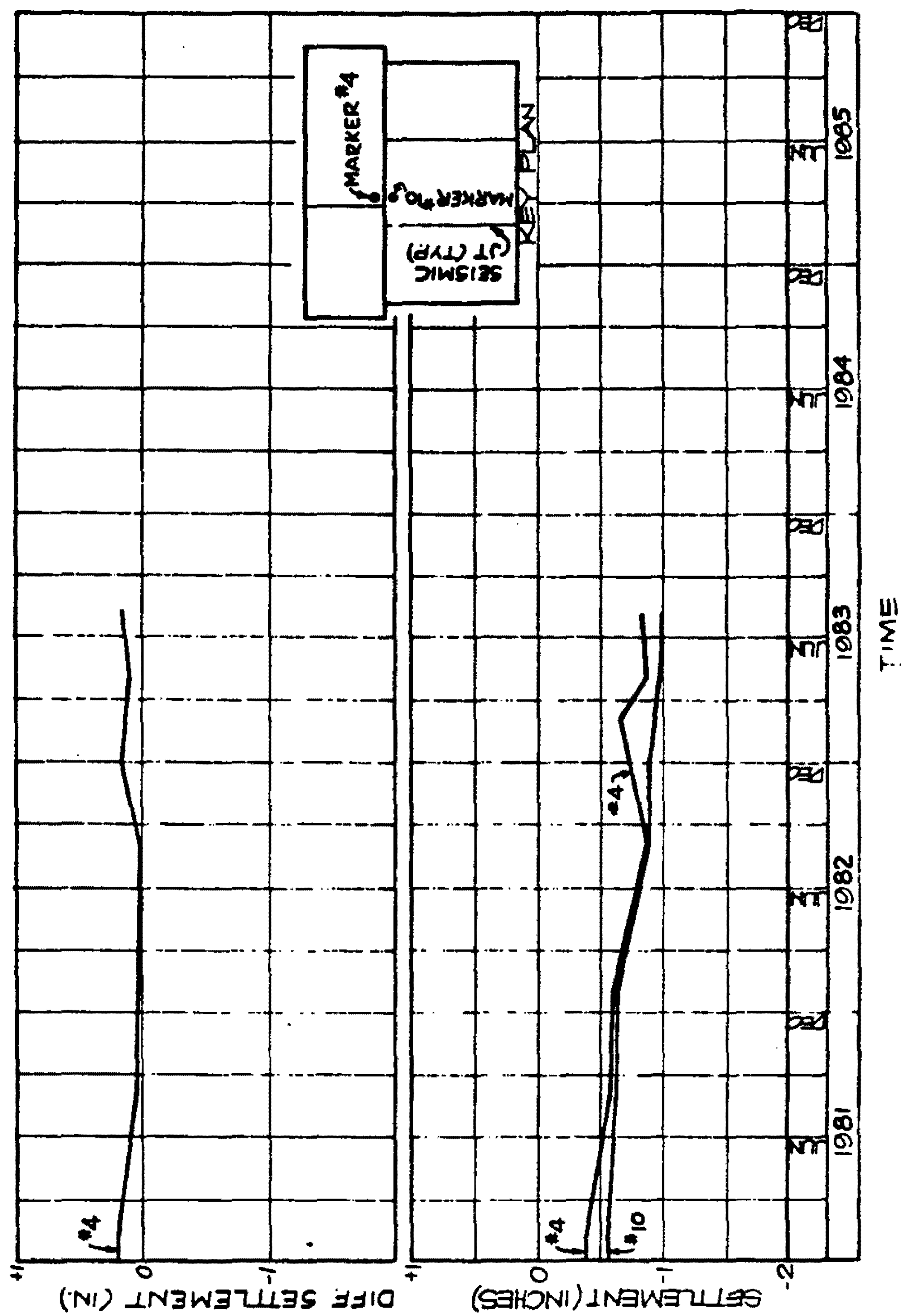
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

DIFFERENTIAL/LOAD SETTLEMENT
PLOT FOR MARKER
NO. 4 vs 10

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-95



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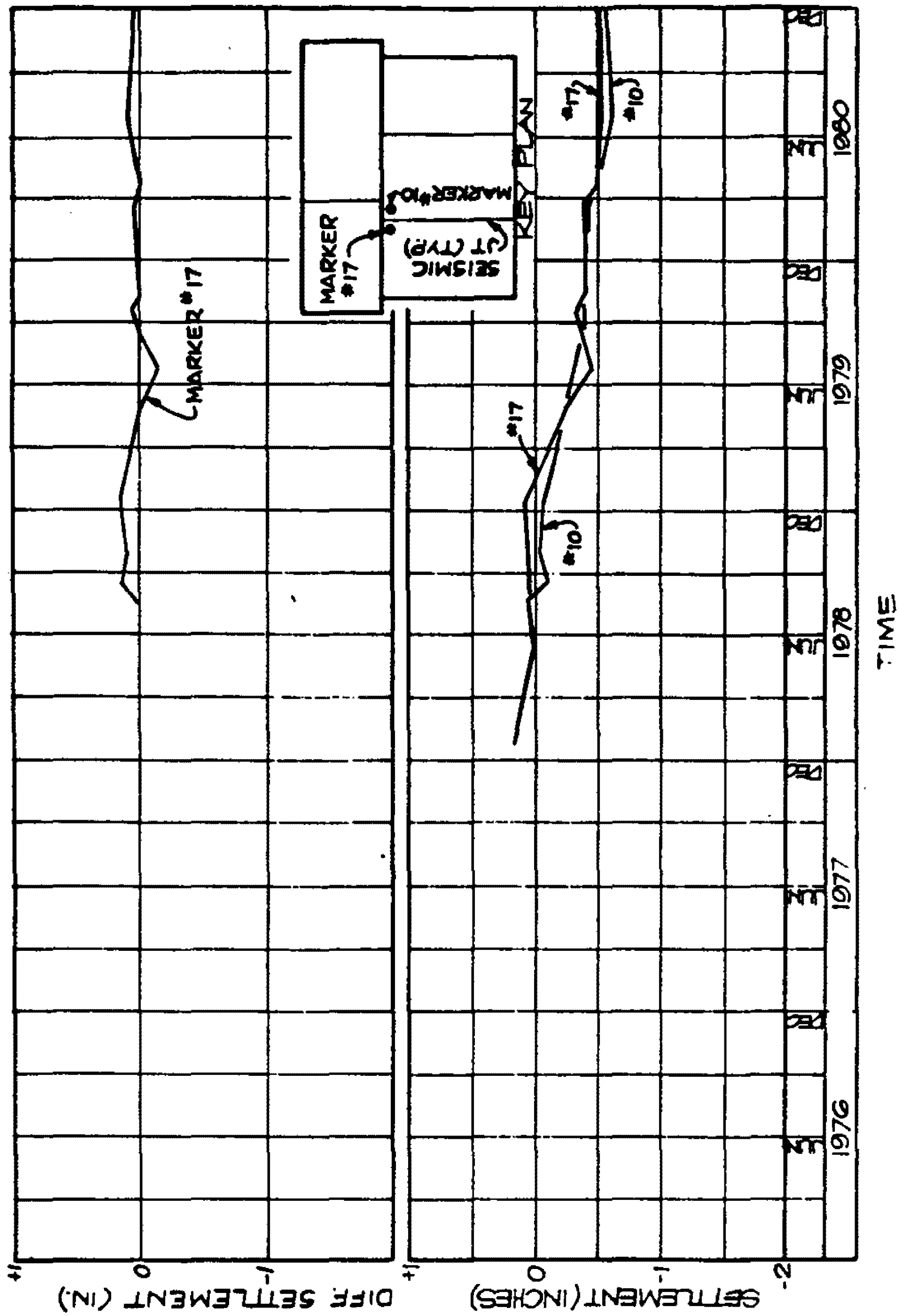
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

DIFFERENTIAL/LOAD SETTLEMENT
PLOT FOR MARKER
NO. 4 vs 10

UPDATED FSAR

Sheet 2 of 2
FIGURE 2.5-95



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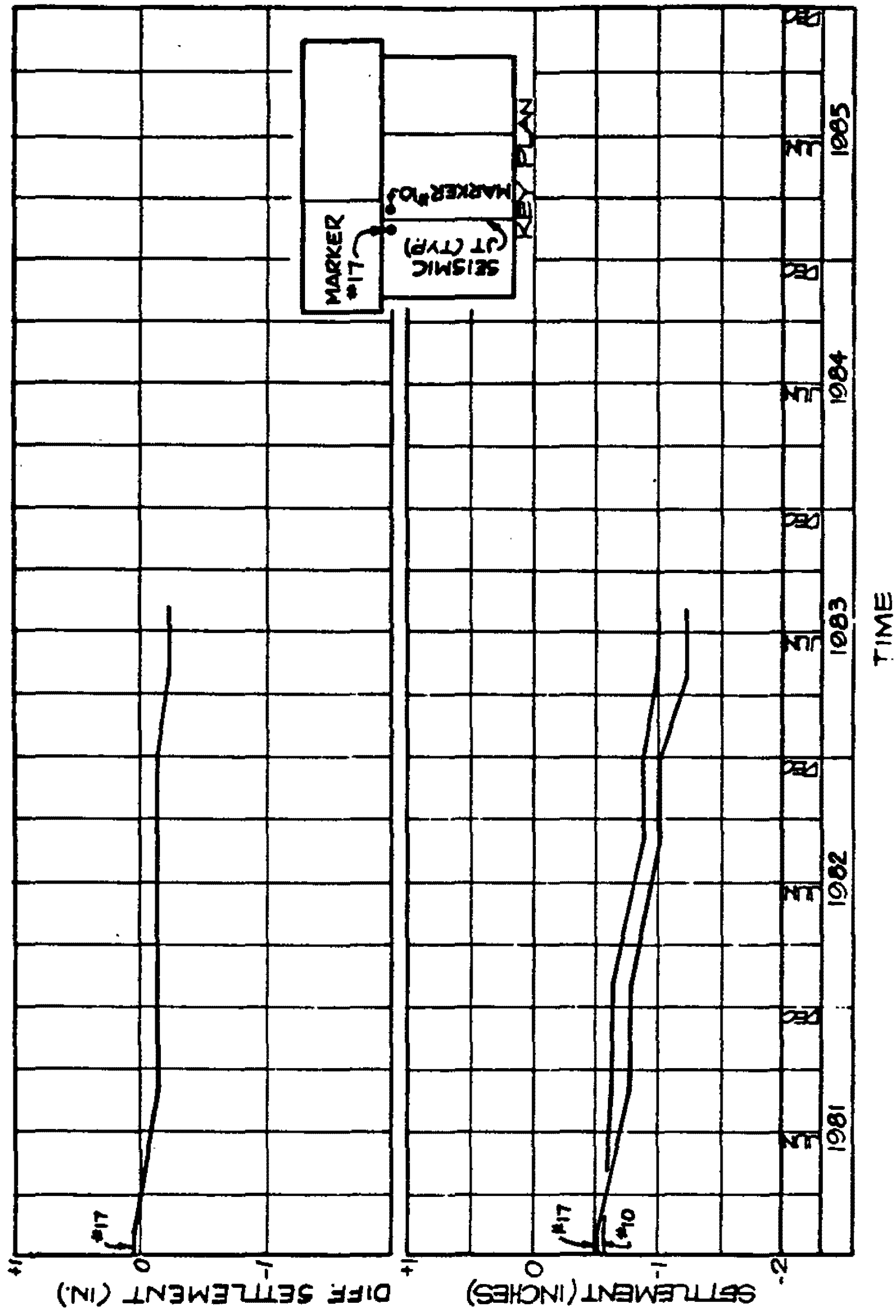
PUBLIC SERVICE ELECTRIC AND GAS COMPANY
HOPE CREEK NUCLEAR GENERATING STATION

DIFFERENTIAL/LOAD SETTLEMENT
PLOT FOR MARKER
NO. 17 vs 10

UPDATED FSAR

Sheet 1 of 2
FIGURE 2.5-96



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PUBLIC SERVICE ELECTRIC AND GAS COMPANY HOPE CREEK NUCLEAR GENERATING STATION	
DIFFERENTIAL/LOAD SETTLEMENT PLOT FOR MARKER NO. 17 vs 10	
UPDATED FSAR	Sheet 2 of 2 FIGURE 2.5-96