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2.0 SITE CHARACTERISTICS

2.1 GEOGRAPHY AND DEMOGRAPHY

2.1.1 SITE LOCATION AND DESCRIPTION

2.1.1.1 Specification of Location

Florida Power & Light Company's (FPL) St. Lucie site is located on Hutchinson Island, St. Lucie County, Florida. The coordinates for St. Lucie Unit 1 are latitude 27° 20' 58" north and longitude 80° 14' 48" west. Approximately 300 feet to the south of St. Lucie Unit 1 is FPL's St. Lucie Unit 2. St. Lucie Unit 2 is located at latitude 27° 20' 55" north and longitude 80° 14' 47" west. The Universal Transverse Mercator (UTM) coordinates for the midpoint (FPL's nos.) are 3025173 meters north and 574326 meters east.

The eastern boundary of the site is the Atlantic Ocean and the western boundary is the Indian River, a tidal lagoon. Other prominent natural features within 50 miles of the site include Lake Okeechobee, 30 miles to the west-southwest of the site and a portion of the Everglades approximately 24 miles to the south of the site. **Figure 2.1-1** shows the site in relation to the region within 50 miles. **Figure 2.1-11** also shows the site with the 50-mile grid positioned over county boundaries. **Figure 2.1-2** shows the area within 5 miles of the site. **Figures 2.1-13 and 2.1-14** show the area within 10 miles of the site.

Prominent cities within 10 miles of the site include Fort Pierce, approximately seven miles to the northwest of the site on the mainland; Port St. Lucie, 4.5 miles to the west-southwest; and Stuart, eight miles to the south.

Transportation corridors within five miles of the site include U.S. Highway 1 (US 1); State Roads (SR) A1A, 712, and 707: the Florida East Coast Railroad; the Atlantic Ocean and the Intracoastal Waterway which is located in the Indian River. SR A1A, the major north-south route on Hutchinson Island, traverses FPL's property to the east of St. Lucie Units 1 and 2. **Figure 2.1-2** shows the location and Subsection 2.2.2 further describes these transportation corridors.

2.1.1.2 Site Area Map

A map of FPL's St. Lucie site is shown on **Figure 2.1-3**. This map includes plant property lines, the site perimeter, principal plant structures, and boundary lines of the exclusion area and low population zone. FPL owns approximately 1,132 acres of land on Hutchinson Island. The site is generally flat, and has dense vegetation characteristic of Florida coastal mangrove swamps. At the ocean shore, the land rises slightly to a dune or ridge approximately 19 feet above mean sea level. The area pre-empted by the plant is about 300 acres, or approximately 27 percent of the total land owned by FPL. There are no industrial, commercial, institutional, or residential structures within the plant area.

The exclusion area and low population zones are shown on **Figures 2.1-2 and 2.1-3**. The radius of the exclusion area is 0.97 miles from the center of the St. Lucie Plant. The low population zone includes that area within one mile of the center of the St. Lucie Plant. The land within this area is owned by FPL. State Road (SR) A1A traverses FPL property in a north-south direction, approximately 1,000 feet east of the St. Lucie Plant. There are no residents within the LPZ. However, the Walton Rocks public beach access lies within the LPZ. Recreational facilities for limited use by FPL employees and their families are also located within the LPZ.

2.1.1.3 Boundaries for Establishing Effluent Release Limits

The minimum boundary distance for establishing gaseous effluent release limits is that noted on Figure 2.1-4 directly north of the St. Lucie Plant. Also indicated on Figure 2.1-4 are other boundary line distances from plant liquid and gaseous release points. The restricted area as defined in 10 CFR 20 includes the fenced-in area shown in Figure 1.2-2.

2.1.2 EXCLUSION AREA AUTHORITY AND CONTROL

2.1.2.1 Authority

As indicated and authorized within the Appendices to the St. Lucie Plant Radiological Emergency Plan, FPL controls the use of all land and water areas inside the site boundary (property) lines.

2.1.2.2 Control of Activities Unrelated to Plant Operation

All activities conducted within the plant (restricted) areas during plant operation are related to the facility operation. The plant area is the fenced-off area surrounding St. Lucie Units 1 and 2. As indicated in and authorized by the St. Lucie Plant Radiological Emergency Plan and the State of Florida Radiological Emergency Management Plan for Nuclear Power Plants, formal arrangements are made to control the traffic and activities of the public on SR A1A which traverses FPL's property east of the plant area, and on the State and Federal waters and beach adjacent to the FPL property, if necessary, in the event of an emergency to assure the radiological health and safety of the public. Specific details are enumerated in the St. Lucie Plant Radiological Emergency Plan (see Section 13.3).

St. Lucie County has constructed a wastewater treatment facility on Hutchinson Island, approximately 2 miles south of St. Lucie Units 1 and 2. Reclaimed water from this facility will be used for irrigation of properties on Hutchinson Island. During periods of high flow and/or rainy weather, excess reclaimed water will be discharged through an outfall to the St. Lucie Plant discharge canal. In their Letter of Agreement with FPL (Reference 219), St. Lucie County committed not to discharge into the discharge canal any raw sewage or sewage which does not meet the effluent discharge criteria established in the FDEP permit for the wastewater treatment facility. The increased flow in the discharge canal from the excess reclaimed water outfall is insignificant compared to the circulating water flow in the canal from St. Lucie Units 1 and 2.

2.1.2.3 Arrangements for Traffic Control

Formal arrangements are made for traffic control in the event of an emergency as described in the St. Lucie Plant Radiological Emergency Plan and in the State of Florida Radiological Emergency Management Plan for Nuclear Power Plants.

2.1.2.4 Abandonment or Relocation of Roads

There were no public roads subject to abandonment or relocation as a result of construction of the St. Lucie Plant.

2.1.3 POPULATION DISTRIBUTION

In accordance with Section 2.1.3.9, a population estimate update was submitted to the NRC in 2003 as part of an emergency planning study. Excerpts from that report were provided as Unit 2 UFSAR Appendix 2.1A via Amendment 15 to the Unit 2 UFSAR. Rather than repeat this shared plant information herein, Appendix 2.1a of the Unit 2 UFSAR is incorporated by reference into the Unit 1 UFSAR. As a result of this update, the information in Sections 2.1.3.1 through 2.1.3.8 has been superceded and these sections have been deleted.

Pages 2.1-4 through 2.1-28 have been deleted

2.1.3.9 Periodic Update of Population Data

FPL will periodically obtain and submit to the NRC the actual and projected population around the St. Lucie site in order to determine what additional measures, if any, should be undertaken to assure the public health and safety.

Commencing in April, 1993 and at least every 5 years thereafter, FPL will prepare and submit to the NRC an estimate of the actual population within 10 miles of the plant, including the distribution by distance and direction, and listing permanent residents, seasonal residents and transients. The basis for the population estimates will also be provided. In addition, commencing in April, 1993 and at least every 10 years thereafter, FPL will prepare and submit to the NRC an estimate of the actual population within 50 miles of the plant. Seasonal residents and transients within 10 miles will also be listed. Per the above commitment, Unit 2 UFSAR Appendix 2.1A provides the latest update of the estimate of population within 50 miles of the plant and Appendix 2.1B provides an estimate of the actual population within 10 miles of the plant.

Based on the revised population data, FPL will determine what changes, if any, should be incorporated into the Emergency Plan to reflect the most recent population data. It is understood that NRC staff will, upon consideration of the population data, plant design features and operational characteristics of the St. Lucie Plant in relation to other nuclear power plants, make a determination of what additional measures, if any, are deemed necessary to assure the public health and safety.

2.1.4 FUTURE LAND USE ON THE APPLICANT'S PROPERTY

Current recreational use of land within the LPZ has been described previously. There are no other proposed land uses within the applicant's property boundaries other than the structure and facilities related to the constructions and operation of St. Lucie Units 1 and 2, and the associated Independent Spent Fuel Storage Installation.

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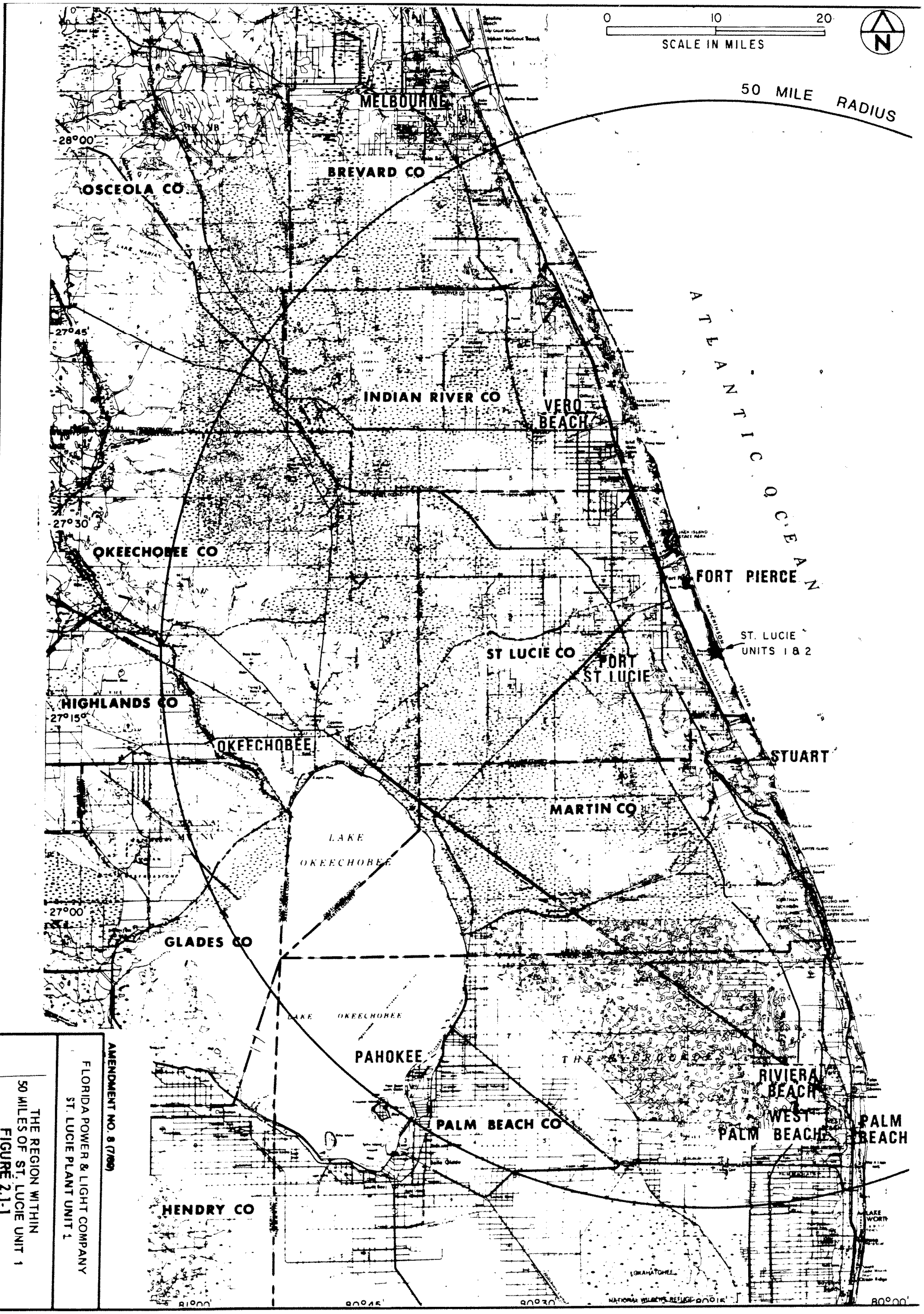
218. City of Port St. Lucie Comprehensive Plan. 1992.

219. Letter No. JNO-RC-93-142 from W. J. Barrow, dated September 15, 1993, with attachments:

    Letter of Agreement between St. Lucie County, Florida, and Florida Power & Light Company, dated November 10, 1992 (executed by St. Lucie County on August 17, 1993, and by FPL on September 14, 1993)

    Letter of Agreement between St. Lucie County, Florida, and Florida Power & Light Company, dated August 12, 1993 (executed by St. Lucie County on August 17, 1993, and by FPL on September 14, 1993)

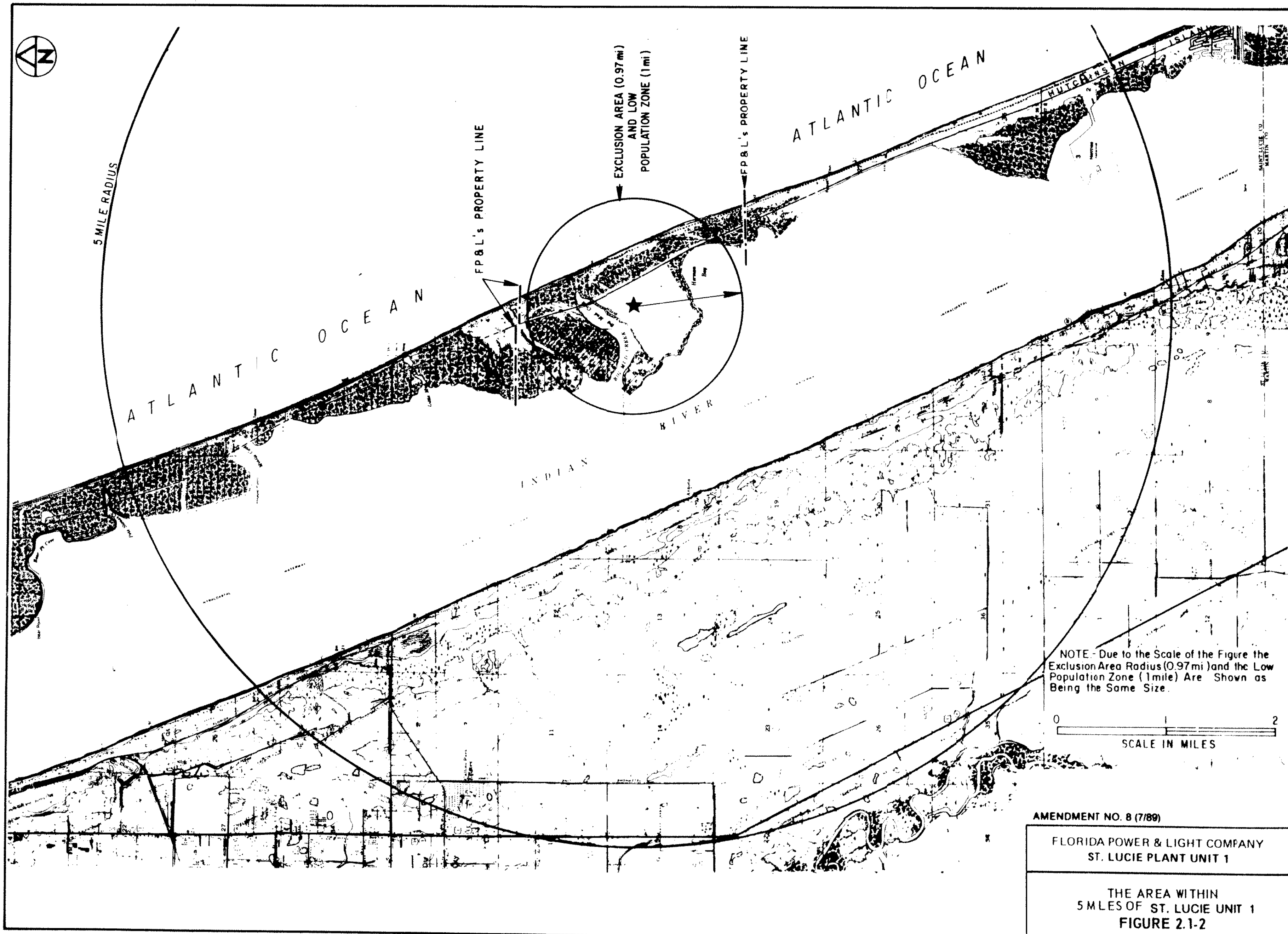
    Executed copy of the agreement between St. Lucie County, Florida, and Florida Power & Light, dated August 17, 1993



AMENDMENT NO. 8 (7/89)

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

THE REGION WITHIN 50 MILES OF ST. LUCIE UNIT 1
FIGURE 2.1-1



AMENDMENT NO. 8 (7/89)

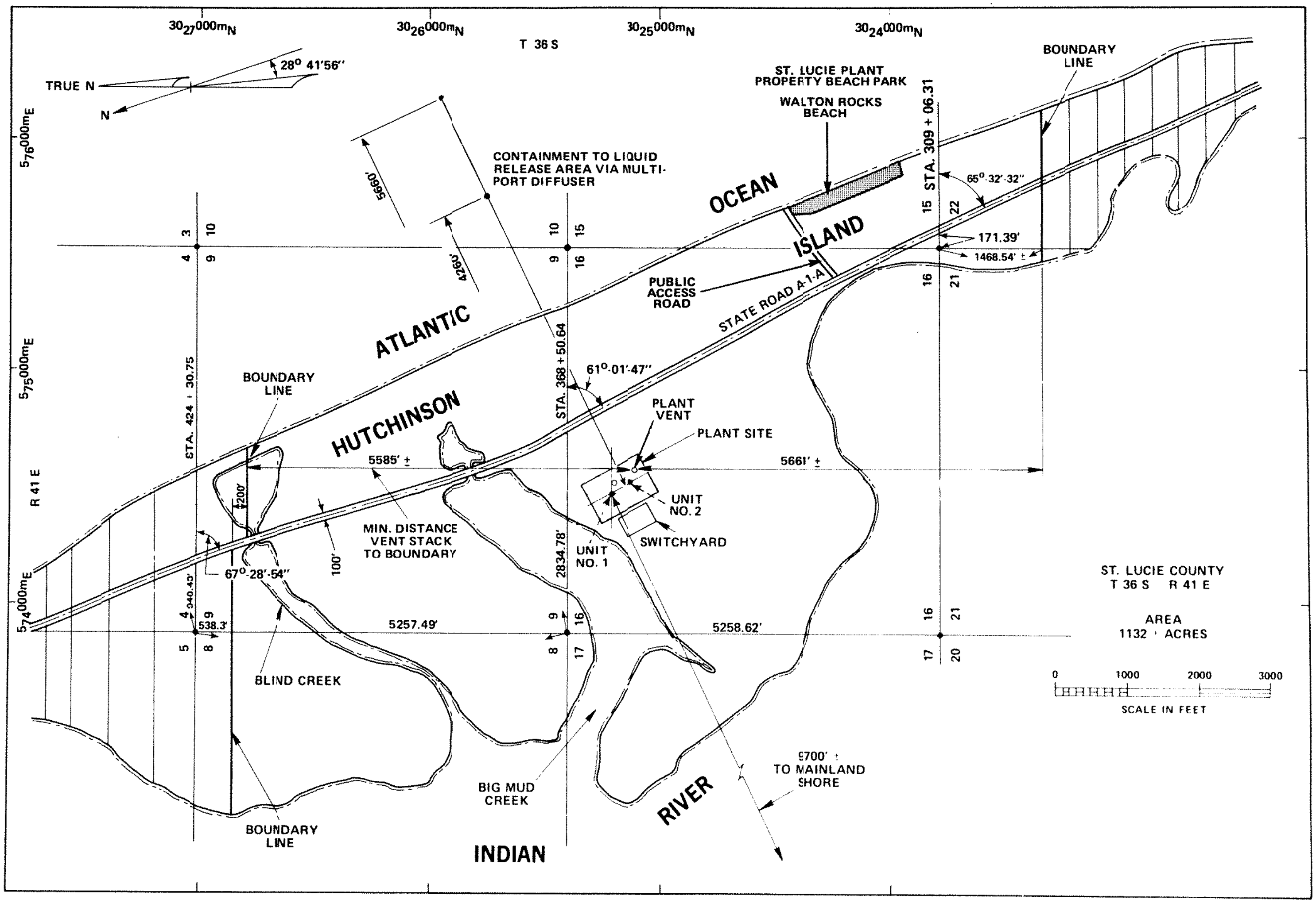
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

THE AREA WITHIN 5 MILES OF ST. LUCIE UNIT 1
FIGURE 2.1-2

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SITE AREA MAP

FIGURE 2.1-3

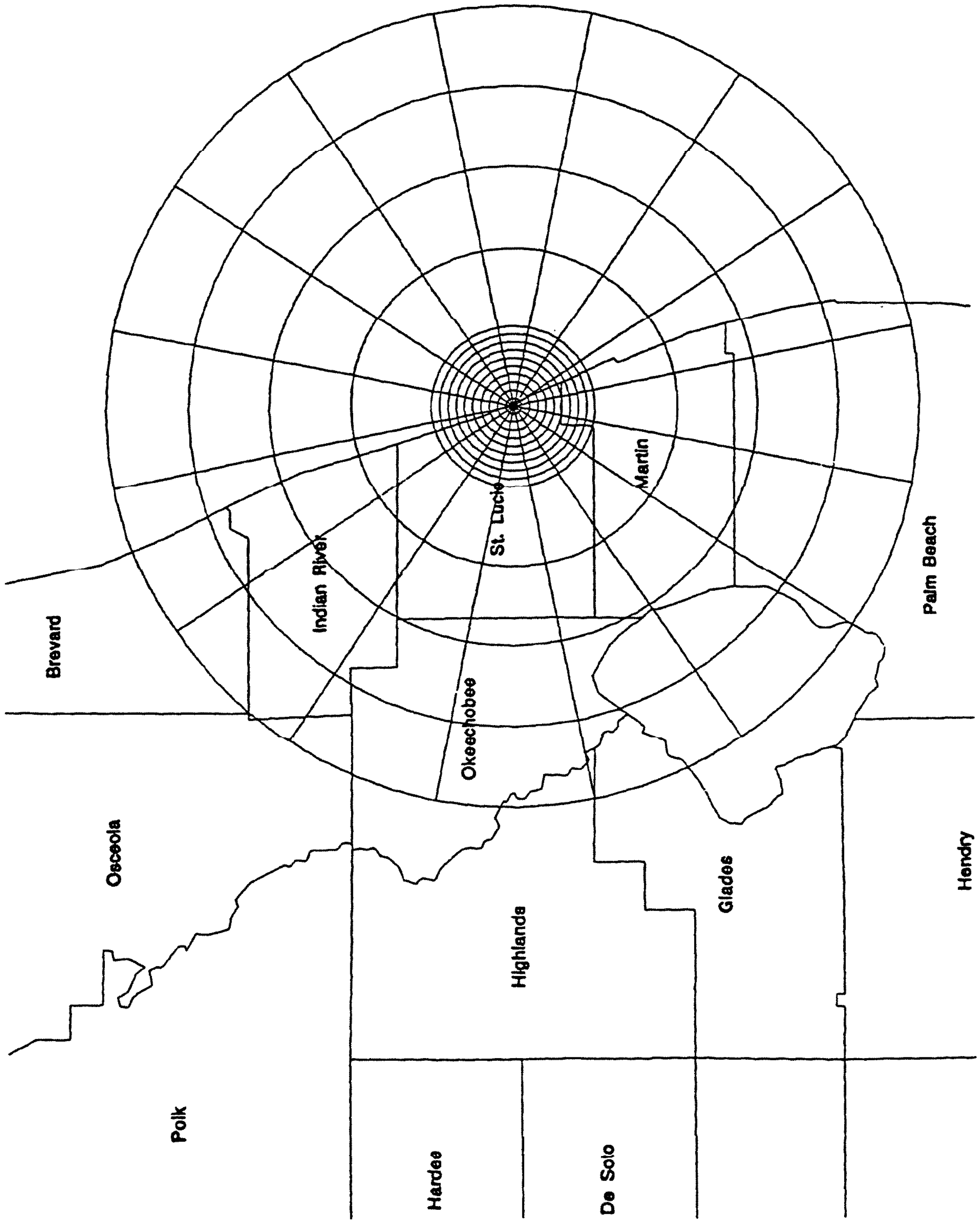


AMENDMENT NO. 9 (7/90)

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

PROPERTY PLAN

FIGURE 2.1-4

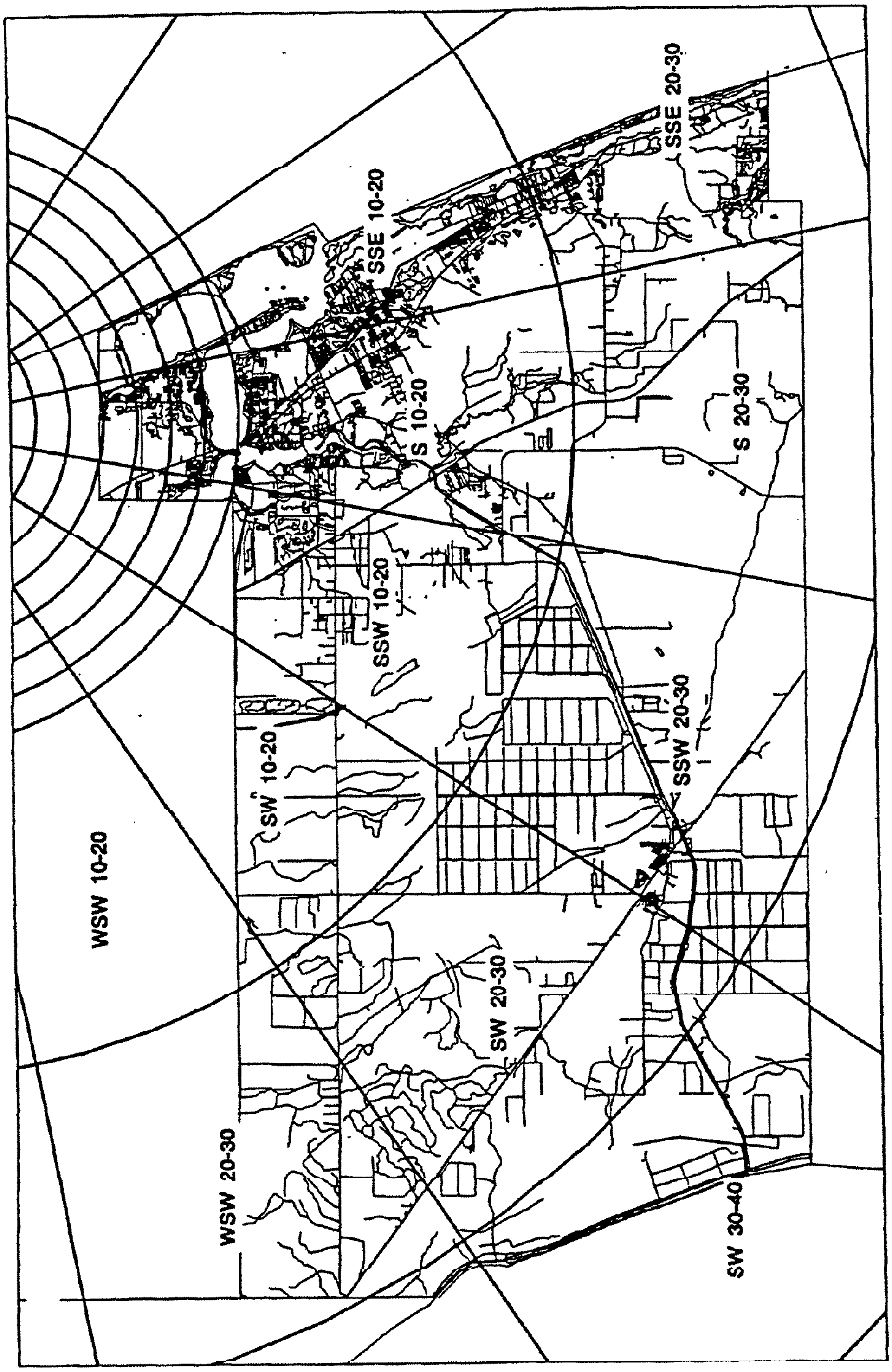


AMENDMENT NO. 12 (12/93)

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT – UNIT 1

COUNTIES WITHIN A
50 MILE RADIUS
SHEET 7

FIGURE 2.1-11

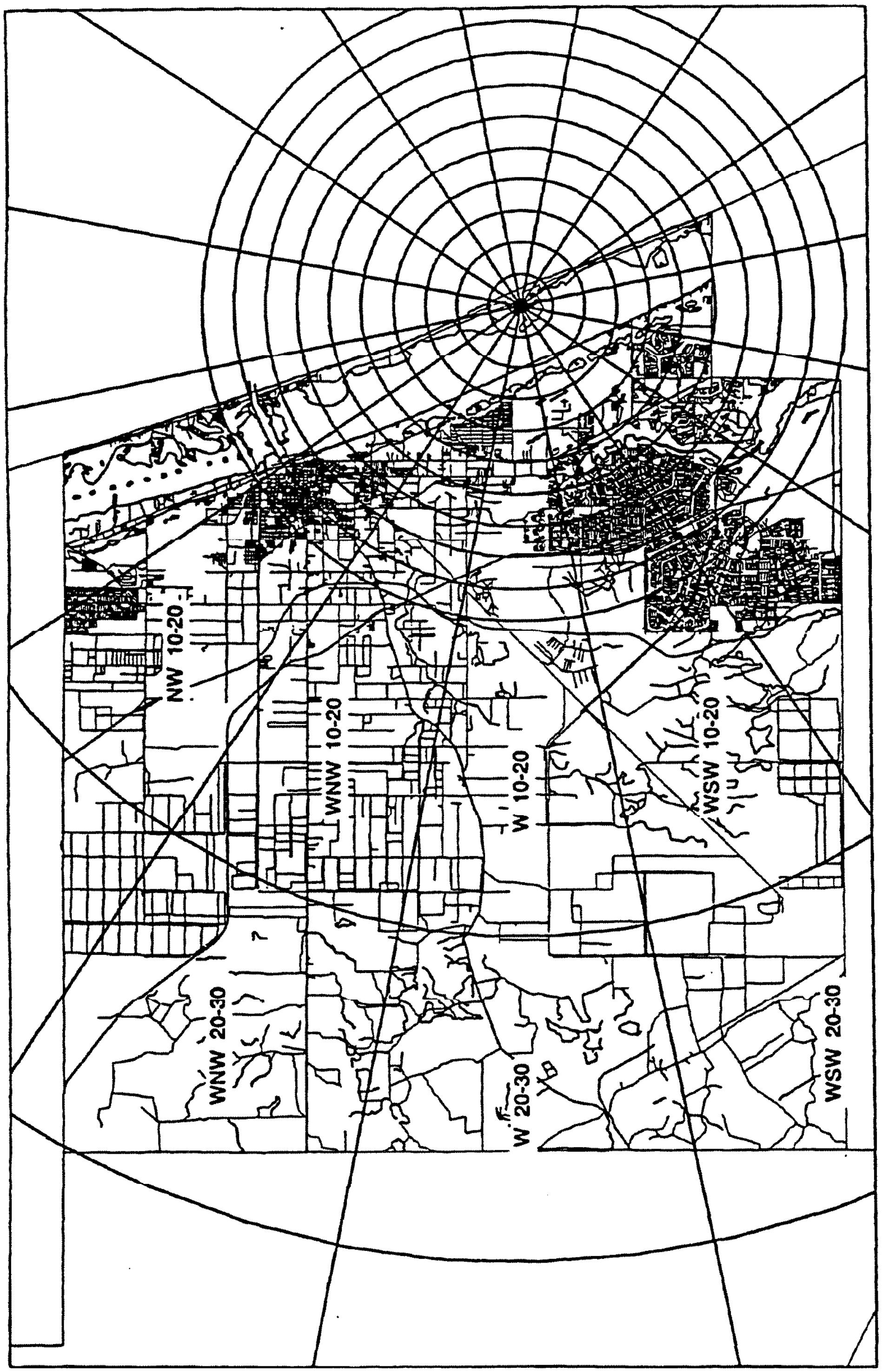


AMENDMENT NO. 12 (12/93)

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT - UNIT 1

MARTIN COUNTY
WITH SECTOR SEGMENTS
SHEET 9

FIGURE 2.1-13



AMENDMENT NO. 12 (12/93)

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT - UNIT 1

ST. LUCIE COUNTY
WITH SECTOR SEGMENTS
SHEET 10

FIGURE 2.1-14

2.2 NEARBY INDUSTRIAL, TRANSPORTATION AND MILITARY FACILITIES

2.2.1 LOCATIONS AND ROUTES

Information contained in this section is considered historical. It may be acceptable to update this section if such changes are determined by the UFSAR Update Group to be appropriate.

There are no military bases or firing ranges, missile sites, manufacturing plants, chemical plants and storage facilities, airports, nearby airplane low level flight and landing patterns, oil and gas pipe lines or tank farms within five miles of the containment building. The land and water transportation routes are indicated on Figure 2.1-5.

St. Lucie County Airport is located about two miles northwest of Fort Pierce and is owned and operated by the Fort Pierce Port and Airport Authority. It has four 5,000 foot runways 200 feet wide. The east-west runway is equipped with lighting facilities. The airport is approximately 12 miles from the plant site. Two small, private airports -- Godwin and Sunrise -- are three and a half miles west and two and a half miles southwest, respectively, of Fort Pierce. Godwin is 12 miles from the plant site and Sunrise is 10 miles from the plant site. None of the airports in St. Lucie County are served by commercial airlines.

The deepwater port of Fort Pierce is federally maintained but owned and operated by the Fort Pierce Authority. It is located at Fort Pierce Inlet and the Intracoastal Waterway, 18 miles north of the eastern terminus of the cross-state canal running from Stuart to Fort Myers. The port is served by the Florida East Coast Railway. A coast guard station is located on the causeway just inside the Fort Pierce inlet. The publicly owned utilities of the City of Fort Pierce include an electrical power generating plant and a 6 million gallon per day water treatment plant.

St. Lucie County has constructed a wastewater treatment facility on Hutchinson Island, approximately 2 miles south of St. Lucie Units 1 and 2. This facility provides treatment for the domestic wastewater generated on Hutchinson Island; reclaimed water from this facility will be used for irrigation of properties on the island. During periods of high flow and/or rainy weather, excess reclaimed water will be discharged through an outfall to the St. Lucie Plant discharge canal.

In Martin County, Stuart Airport, Witham Field, a county-owned airport having mile-long paved runways, is located just outside the city limits of Stuart and at its southeastern corner. It is 11-1/2 miles from the plant site. No commercial airlines serve this airport.

Stuart owns and operates its own water system, supplied by deep wells and subsequently treated.

The largest industrial employer is Grumman Aircraft Engineering Corporation, located at the airport and employing about 300 people. Smaller industries are: Hoosier Metal Fabricators; R & H Fittings; Outboard Marine Corporation; and Southeastern Printing Company.

In Indian River County, about 35 miles from the plant site, the Pelican Island National Wildlife Refuge is located along the Intracoastal Waterway at Sebastian.

There are two airports in the county -- Sebastian Municipal located 35 miles from the plant site at the City of Sebastian and Vero Beach Municipal -- located at Vero Beach 20 miles from the plant site. Vero Beach Municipal is served by commercial airlines. Sebastian is not.

Okeechobee County Airport is at the northwest corner of the City of Okeechobee 35 miles from the plant site. A small, private airport, known as Dixie Ranch Airport, is located about eight miles northwest of the city 40 miles from the plant site. Neither of these airports is served by commercial airlines. Lake Okeechobee, with its 700 square miles of area and 110 miles of shoreline, forms part of the southern boundary of Okeechobee County.

Among the important industrial employers in Palm Beach County are:

Pratt Whitney Aircraft - research and development on jet engines, missile propulsion components	4,500 Employees
RCA - data processing systems	1,000 Employees
ITT Semiconductors - semiconductors and integrated circuits	1,300 Employees
Solitron Devices - transistor and cryogenic thermometers	600 Employees

Palm Beach International Airport, with runways to 8,000 feet, is located 45 miles from the plant site about three and a half miles southwest of downtown West Palm Beach and is served by commercial airlines. The Flying Cow Airport is located 38 miles from the plant site. Glades Airport is located near and south of Pahokee, 48 miles southwest of the plant site. Neither of the latter is served by commercial airlines.

The U. S. Navy Base is located at Jupiter Inlet, as is Jupiter Auxiliary Air Force Base and Jupiter Light House. A Coast Guard Station is located on Peanut Island.

The J. W. Corbett Wildlife Management Area occupies about 500 square miles of the western portion of the study area in Palm Beach County and is largely in its virgin state.

Pahokee State Park is being Management developed near that city. It is located atop the Hoover Dike on Lake Okeechobee and will feature camping, swimming, picnic facilities, boat ramps and fishing.

The Port of Palm Beach serves both inland barges and deep sea vessels. Minimum depths of the channel and turning basin are 35 feet and 33 feet, respectively, below mean low water. The Port is located at Riviera Beach at the Lake Worth Inlet.

Forty miles from the site and about four miles north of the Brevard-Indian River Counties boundary, just west of Micco, is a small private airport known as Canaveral Indian River Airport. It is not served by commercial airlines.

The Air Force Test Range, with aircraft landing facilities, identified as Valkaria Missile Tracking Annex, is located near the community of Valkaria, south of the town of Malabar and just within the 50-mile radius.

The Kennedy Space Center is 75 to 80 miles north of the site.

2.2.2 DESCRIPTIONS OF MANUFACTURED, TRANSPORTED OR STORED MATERIALS

Information contained in this section is considered historical. It may be acceptable to update this section if such changes are determined by the UFSAR Update Group to be appropriate.

For a description of the manufactured, transported or stored materials located on site refer to the St. Lucie Unit 2 FSAR Subsections 2.2.2 and 2.2.3.

For a description of the materials stored at and transported to the St. Lucie County wastewater treatment facility located on Hutchinson Island, refer to the St. Lucie Unit 2 FSAR Subsections 2.2.2 and 2.2.3.

2.2.3 EVALUATIONS

Because the plant cooling water intake structure is located in a commercially non-navigable area off-shore in the Atlantic Ocean, no reasonable hazard exists from barges or ships that pass the site and no corrosive liquids or oils accidentally released could enter the intake structure.

No gas pipelines nor stone quarries are located near the site, so no explosion or concussion hazard exists from these potential sources. The potential hazard from fires off-site are negligible because no appreciable flammable mass of appreciable size exists or has purpose for existing in the area.

No effects attributed to airborne pollutants will be observable on critical components because no potential industrial source exists in the area.

The effect of aircraft impact on site structures is nil because no airports exist within five miles of the site, and those nearest to the site serve only light aircraft.

2.3 METEOROLOGY

2.3.1 REGIONAL CLIMATOLOGY

2.3.1.1 Data Sources

The general climatic data for the Florida region was developed from two sources, from on-site meteorological measurements for short term records and from the National Oceanographic and Atmospheric Administration weather stations in southern Florida for long term records.

2.3.1.2 General Climate

The prevailing climatology of the coastal site of Hutchinson Island is dominated by the presence of the Azores-Bermuda high pressure system resulting in a subtropical marine type climate for the eastern Florida coast. The westward position of this system in May to October occurs together with the highest percentage of rainfall. Sporadic penetrations of cooler continental air occur in the winter months. Hurricane activity is limited to the summer and fall months while tornadoes and waterspouts have been observed throughout the year.

The monsoonal nature of the general circulation in the area together with the proximity of the site to the ocean results in a high percentage of easterly component (on-shore) winds. Moist unstable air generates frequent rain showers and thunderstorms.

The warm waters of the adjacent Gulf Stream current, located a few miles offshore, inhibit the formation of strong persistent low level inversions while instability during the day is aided by the strong insolation.

2.3.1.3 Severe Weather

a) Precipitation

The southeastern coastal region of Florida has an annual rainfall exceeding fifty inches. Most of the heavy rainfall is associated with thunderstorms or passage of hurricanes. Measurable precipitation falls on approximately thirty-six percent of the days of the year.

b) Hail

In the period 1945-1953 the number of hailstorms recorded in all of Florida totaled only 37 cases(1). However, in the period 1955-1967, 116 cases of surface hailstorms were reported or an average of 9 storms per year for the state of Florida(2).

c) Hurricanes

For the period 1885-1958 a total of 119 tropical cyclones of all intensities significantly affected the Florida area[3]. Of this total 71 were of full hurricane intensity. The average number of hurricanes for the period was 1.6 storms per year. Individual years range from none to five.

d) Tornadoes and Waterspouts

Two independent studies have been made on Florida tornadoes. One submitted to the Commission during the Unit 1 construction permit review in October 1969 is provided as Appendix 2C, and a current study is provided as Appendix 2F. Both conclude that the severe 360 mph (Region I) tornadoes are not applicable to Florida, and that historical data does not substantiate speeds exceeding about 200 mph in Florida tornadoes. The earlier study utilized all known Florida tornado data from 1887 to 1968, whereas the current study analyzed data for the period from 1950 to 1972. The current study went beyond the earlier work in that it developed a DBT (Design Basis Tornado) for the Atlantic coast of the United States and the Atlantic coast of Florida. The results of the DBT analysis indicate that a Region III (240 mph) DBT is appropriate for the St. Lucie site.

Waterspout reporting in the United States has been coordinated in a systematic way since 1952 by the National Weather Service. A total of 190 waterspouts have been reported in Storm Data (published by the National Climatic Center), along a 200 mile zone of the Florida Atlantic Coast centered at St. Lucie. Of those, 178 were reported to have occurred within 25 miles of the shore.

Because of their very nature (over-water trajectory) only occasionally are remarks given as to their size and their direction of motion.

Eleven of the 178 waterspouts were reported to penetrate on-shore and their worst reported damage falls into the "weak tornado" category (estimated wind speeds of 72-112 mph), as defined by Fujita (1971). Golden's analysis of the Lower Matecumbe Key waterspout indicated a maximum estimated wind speed of 170 mph (Ref: Monthly Weather Review, Waterspouts and Tornadoes over South Florida, February 1971, Vol. 99, No. 2). In a personal communication, Golden indicated that errors inherent in this technique are about 15-20% and that his estimate is probably on the high side. This tangential speed when coupled with the translational speed of the waterspout, results in a maximum horizontal velocity of less than 200 mph which is well below the AEC tornado design criteria.

In order to compute the probability and recurrence interval of a waterspout at a point, estimates of waterspout diameter and path length are necessary. Since very little information was available, a conservative average waterspout width-path length of 200 feet by 4 miles was assumed.

Recurrence intervals were then computed for various offshore distances along the 200 mile coastal zone. The method used follows Thom (1963).

The probability of waterspout striking a given point is:

$$\text{Probability} = \frac{Z\ T}{A}$$

Z = Mean Area of Waterspout

T = Frequency of Waterspout (Annual)

A = Area Examined.

The recurrence interval (in years) is defined as:

$$\text{Recurrence} = \frac{1}{\text{Probability}}$$

The probability and recurrence intervals of waterspouts for the various distances offshore are given in Table 2.3-1A.

The monthly distribution of waterspouts occurring within 25 miles offshore and along a 200 mile zone centered at St. Lucie for the period of record, 1952 to 1973 is as follows:

													Total
NUMBER	6	5	8	4	14	16	51	19	30	17	7	1	178
MONTH	J	F	M	A	M	J	J	A	S	O	N	D	

Eight-two percent occur between May and October.

e) Air Pollution Potential

In the twenty-one year period from 1936-1956, there were approximately one hundred days of atmospheric stagnation in the vicinity of the plant, of which twenty-two cases lasted four or more days.[3a] Between August 1, 1960, and April 3, 1970, there were no high air pollution potential days.[3b]

2.3.2 LOCAL METEOROLOGY

2.3.2.1 Data Sources

Meteorological and climatological data from the Hutchinson Island site and from the West Palm Beach Weather Bureau-station are presented for the purpose of validating previous estimates of site climatology and to provide estimates of the local diffusion meteorology based on site data.

Site data, collected for one year period beginning March 1, 1971 indicate that the Hutchinson Island site has excellent diffusion characteristics. A subsequent year's additional meteorological site data indicates that the diffusion characteristics obtained from the original year's data are acceptably conservative. Open flat terrain and proximity to the ocean provides a well ventilated area with strong on-shore wind conditions. The coastal location tends to inhibit deep inversions from occurring and persisting and the majority of the inversion wind directions are associated with off-shore flow.

The climatological evaluation of the site is partially or totally supplemented by meteorological data from West Palm Beach Airport, Florida[4,5] because of its longer period of record (1939-1970). This station is located approximately forty miles south of Hutchinson Island and somewhat less than three miles inland from the ocean. It is the closest weather station to the site and has climatological characteristics that are very similar to Hutchinson Island. To demonstrate applicability of the West Palm Beach data to the Hutchinson Island site the one year climatological record at Hutchinson Island is compared with the same period of record from West Palm Beach and also with the long term record at West Palm Beach.

2.3.2.2 Normal and Extreme Values of Meteorological Parameters

a) Wind Distribution

Seasonal (and annual) wind frequency distribution in percent of seasonal occurrence by wind direction versus wind speed classes are shown in Table 2.3-2 through 2.3-6 for the 50-foot level of the meteorological tower at Hutchinson Island. Detailed interpretation of these tables is provided below.

A calm wind is defined as a measured wind speed less than 1.0 mph (the threshold speed of the anemometer). A variable wind is defined when the wind directional trace is in a steplike pattern and the measured wind speed is between 1.0 and 2.5 mph (the starting speed of the directional value). Table 2.3-7 provides a seasonal and annual distribution of hourly calm and variable wind conditions. The summer season dominates with the highest occurrence of calm (51 hours) and variable (122 hours) winds and also the greatest consecutive hours of non-directional winds (11 hours). The annual percent occurrence of calms at the 50-foot level is 1.05 percent and 4.10 percent of the wind conditions have been identified as variable.

Table 2.3-8 illustrates that the 9 wind sectors, NNW clockwise through SSE inclusive, that represent the on- and along-shore (over water trajectory) wind directions dominate both the percent of occurrence and the higher mean wind speeds. The higher on-shore wind speeds are attributed to the reduction of frictional forces associated with overwater flow. However the seasonal distribution of the 1-3mph wind speed class (tabulated below) into on- and off-shore wind directions indicates that the lower wind speeds are predominately off-shore land breezes.

	Winter	Spring	Summer	Fall	Annual
On Shore (9 sectors)	0.80%	1.95%	4.21%	2.48%	2.30%
Off Shore (7 sectors)	2.70%	3.10%	2.97%	3.43%	3.06%

Table 2.3-9 delineates the primary and secondary prevailing wind directions and speeds on a seasonal and annual basis. An easterly component of winddirection, associated with the ocean breeze, is identified in all the time periods except the spring season.

Summarizing the wind distribution data for Hutchinson Island, there is a high percentage occurrence of strong on-shore winds and a fairly small percentage of calms (1.05%). Off-shore winds comprise approximately 35 percent and on-shore represent 60 percent. High wind speeds are associated with the ocean breeze and low wind speeds are predominately offshore land breezes.

Quasi-steady-state wind (two minute average) and peak wind data for the Eastern Test Range at Cape Kennedy, Florida, located approximately 90 miles north of the St. Lucie site, is provided in tables 2.3-9A and 2.3-9B for the period of record January 1950 to December 1964. Values of peak wind speed were obtained by multiplying the steadystate wind speeds by a gust factor of 1.4.[5a] In the vicinity of the site, the annual extreme mile wind (30 feet above the ground)[5b] for the mean recurrence interval of 11, 25, 50, and 100 years is 49, 60, 71, and 82 mph., respectively.

b) Temperature

Table 2.3-10 illustrates the mean monthly distribution of temperature and the extremes recorded in the area. It can be seen from a comparison of the Hutchinson Island and West Palm Beach temperature data presented in Table 2.3-10 that the temperature data for the two locations are quite comparable. The moderating effects of the proximity to water at Hutchinson Island results in generally slightly lower average maximum temperatures and slightly higher average minimum temperatures at Hutchinson Island than at West Palm Beach but it is obvious that the long-term temperature statistics for West Palm Beach can be reasonably applied to Hutchinson Island. The long-term temperature statistics at West Palm Beach indicate that the maximum extreme of 101 degrees F occurred in 1942 and the minimum extreme of 29 degrees F occurred in 1970. These long-term maximum and minimum extremes would not be higher or lower at Hutchinson Island because of the moderating influence of the adjacent ocean waters.

c) Atmospheric Water Vapor

The characteristic moisture content of maritime air does not differ appreciably between the West Palm Beach and Hutchinson Island areas. However, the proximity of the site to the ocean tends to produce slightly high humidities at the coastal site. Long term (1965-1970) relative humidity data taken at 0100, 0700, 1300, and 1900 hours EST for West Palm Beach show a mean yearly pattern of 81, 82, 60 and 73 percent respectively. The 1971-72 mean yearly relative humidity values at West Palm Beach are 80, 81, 59 and 71 percent while the coincident Hutchinson Island values are 77, 82, 66 and 69 percent.

Cloud cover data is based on the period of recored 1946 to 1970 for West Palm Beach, Florida. The mean yearly cloud cover from sunrise to sunset is 61 percent usually of the cumulus type with a range of 70 to 55 percent between the rainy and dry season. On a yearly basis, average daytime conditions are as follows: clear 74 days, partly cloudy 153 days, and cloudy 138 days.

d) Precipitation and Hail

At West Palm Beach, in the period of 1939-1970, annual precipitation ranged from 37.31 to 108.64 inches, with an annual mean of 61.69 inches. Rainfall amounts for a monthly period varies from a minimum of 0.04 inches in April, 1967 to a maximum of 24.86 inches recorded in September, 1960. The maximum twenty-four hour total was 15.23 inches in April, 1942. Short period rainfall amounts of 6.0 inches in one hour have been recorded west of the West Palm Beach area. For the same period of record, March, 1971 to February, 1972 the Hutchinson Island rain gauge reported approximately 17 inches less rainfall than West Palm Beach.

A comparison of monthly distributions of precipitation are shown in Table 2.3-11 for West Palm Beach and Hutchinson Island.

Table 2.3-12 illustrates the monthly distribution of thunderstorm activity for West Palm Beach. On an annual basis, thunderstorms occur seventy-eight days a year, with a maximum frequency during the months of July and August. This summer maximum is associated with the characteristics of the unstable tropical maritime air mass.

The one degree latitude by one degree longitude square that includes Hutchinson Island had an average of 3 hailstorms per year for the 1955-1967 record period[2]. The maximum probable activity may be expected in the March to May period but its point probability in the Hutchinson Island area would be very small.

e) Fog

Heavy fog with visibilities equal to or less than 1/4 mile are uncommon in the coastal areas of Florida. Table 2.3-13 indicates that, for the period of record 1943 to 1970, West Palm Beach observed an average of 8 days per year with heavy fog.

f) Hurricanes

Since the general area of southeastern Florida is under the influence of the subtropical Azores-Bermuda high pressure area, seasonal pressure changes are relatively small. The lowest pressures are associated with hurricanes or tornadoes. To date, the lowest pressure recorded in the West Palm Beach area was 27.43 inches on September 16, 1928. Pressures lower than this are calculated to occur at the center of tornadoes. Theoretical estimates of the minimum pressure in the center of a tornado can account for a rapid change of three inches in pressure(6). The maximum pressure recorded at Hutchinson Island was 30.97 and the minimum was 28.82 inches, for the period of record March 1971 to February 1972.

In the vicinity of Palm Beach, the probability of experiencing hurricane force winds in any given year is one in ten. Further up the coast, near Vero Beach, the probability reduces to one in twenty. Since Hutchinson Island is situated north of Palm Beach and south of Vero Beach the probability of hurricane force winds affecting the site would be approximately one in fifteen. Most of the hurricanes occur in the August to October period, although occurrences range from June to December. During the 1958-1971 period, only five hurricanes and four tropical storms have affected the east coast of Florida in the vicinity of Hutchinson Island. Paths of the hurricanes are generally in a west-northwesterly direction entering the area from the ocean(5). A few move in a northerly course entering over the southern peninsula of Florida. The forward speed of the hurricane varies, averaging 12 mph in the region. Storms moving inland from the east and south of the site would place the site in the area of maximum winds, the right forward quadrant of the hurricane. Paths either over land or in a northerly direction offshore would place the site in areas of less intense wind speeds.

g) Tornadoes

Waterspouts and funnel clouds are reported throughout the year occurring mostly in the May to July period during daytime hours[(2)]. In 1964, twenty-three waterspouts were reported in the West Palm Beach area. The number reported in a specific area is heavily biased by the population density and aircraft activity in the area. Since the West Palm Beach area is served by a major airport facility, more reports are possible than for a rural site in the same locality.

The site is located in the St. Lucie county area, where a total of six tornadoes were reported in the last 13 years. Applying Thom's technique[(7)] for estimating the probability of a tornado striking a point, the probability for the Hutchinson Island area included in a one degree square, was 0.00094 per year. The recurrence interval for the site was 1062 years. This technique is based on the path width of tornadoes in the midwest area. For the Florida region, the tornado paths are narrower and the total frequency for tornadoes in the Hutchinson Island vicinity from Thom's data is fourteen for the period of 1953-1962, with a mean annual frequency of 1.4. ESSA Technical Memorandum WBTM FCST 12, "Severe Local Storm Occurrences," 1955-67 indicates mean annual tornado frequency of 2.5 in the one degree latitude-longitude square containing the plant site. Applying Thom's technique for this data yields a probability of 0.00185 with a recurrence interval of 541 years.

h) Maximum Winds

The flat open terrain exposes the site to high winds from any direction. Wind speeds of hurricane force, greater than 75 mph, can be expected to occur several times each decade. At West Palm Beach the highest one minute wind speed recorded was from the ESE at 86 mph (August 1964). Estimates of average peak winds associated with a hurricane are an extreme velocity twenty-five percent greater than one minute velocities, momentary gusts fifty percent higher than the sustained wind. In the August 1949 hurricane affecting the West Palm Beach area, the anemometer was blown away when velocity reached 110 mph and gusting to 125 mph. Speeds of 140 mph and 155 mph were estimated from privately owned anemometers[(3)]. The existing ground level anemometers are not designed to record at extremely high speeds, peak winds have to be estimated from damage assessment or theoretical considerations relating the pressure distribution to the velocity field. Potentially, the highest peak winds would be associated with a tornado passing directly over the site. Estimates of the mid-western tornadic peak winds are approximately 300 mph. For Florida tornadoes it is felt that a reasonable upper limit for tornadic velocities, including translational effects, is approximately 200 mph[(9)].

Based on the analysis and conclusions concerning maximum wind velocities experienced in the vicinity of the site, and providing a suitable margin to account for probable maximum velocities which reasonable could be expected, a design hurricane wind speed of 194 mph has been selected. Maximum wind speeds for use in the analysis of hurricane tides are discussed in Section 2.4 and are taken to be those associated with a probable maximum hurricane as set forth in ESSA memorandum HUR 7-97[10]. Wind used for this purpose is a sustained wind whose value is 140.6 mph. If such a wind is associated with gusts which are 30% higher, the maximum gust would be 192 mph. Therefore the design speed is consistent with the maximum winds of a probable maximum hurricane.

The tornado wind speeds used in the design of critical plant structures and equipment is 300 mph rotational and 60 mph translational velocities. A differential pressure drop of 3 psi is assumed concurrent with the tornado winds.

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

There is no potential modification of the normal and extreme values of meteorological parameters as a result of the presence and operation of the St. Lucie Plant.

2.3.2.4 Topographical Description

The site area and the surrounding five mile radius terrain is literally flat; natural elevations do not exceed 20 to 25 feet above MLW anywhere. Both the short and long term diffusion estimates are affected only within the first radial mile from the containment by on-site building facade area.

2.3.3 ON-SITE METEOROLOGICAL MEASUREMENTS PROGRAM

The on-site meteorological program is designed to provide a dispersion climatology for use in the planning of radioactive effluent releases and as a means of determining the appropriately conservative meteorological parameters to be used in estimating the potential consequences of hypothetical accidents. Analysis of collected meteorological data permits an assessment of the diffusion parameters characteristic of the site. The program in Regulatory Guide (RG) 1.23 is met. In response to Staff requirements as stated in its SER at Section 2.3.6, the onsite meteorological measurement program was upgraded to Regulatory Guide 1.23 standards. The effort specifically includes installation of low threshold wind speed and direction sensors at the 190 (57.9 meters) and 33 (10 meters) foot levels. Additionally, a display of the meteorological parameters is available in the Unit 1 control room in accordance with these standards. The upgraded instrumentation package is described in Sections 2.3.3.2 through 2.3.3.4.

EC246531

EC246531

An on-site climatological analysis is based on the period of record March 1, 1971 to December 30, 1973. The following meteorological data were collected at Hutchinson Island: wind direction and speed, temperature, precipitation, barometric pressure and dew point temperature.

2.3.3.1 Meteorological Tower

A meteorological tower was erected at the St. Lucie Plant site on Hutchinson Island. A 196-foot framed tower is located on site 2400 feet north of the reactor complex. It is situated in an area of relatively flat terrain characterized by mangrove trees in the range of 20 to 25 feet in height. Figure 1.2-1 shows the location of the meteorological tower relative to the rest of the plant site. EC246531 EC246531

2.3.3.2 Instrumentation

The system has been modified with sensors to meet the requirements of Regulatory Guide 1.23. The instruments are described below.

a) Wind Speed and Direction

Two combination wind speed and direction sensors at the 32.8 ft (10 meter) and 190 ft (57.9 meter) levels are solid-state ultrasonic instrumentation capable of measuring wind speed and direction in the U and V axes. Sonic pulses are generated at the transducers and are received by opposing transducers. Mathematics derived for these sonic pulses provide a wind velocity measurement in each of the corresponding axes. A microprocessor-based, electronic measurement system is used to control the sample rate and compute the wind speed and wind direction.

Wind speed and direction transducers output 0 - 5.0 volts corresponds to 0 - 111.8 mph (speed) and 0 - 360° (direction converted to 0 - 540° by the datalogger).

b) Air Temperature

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Air temperature is measured by two RTD sensors at the 32.8 ft (10 meters) and 190 ft (57.9 meters) levels. An additional sensor at the 110.3 ft (33.5 meters) level is measured for differential temperature calculations should either the 10 or 57.9 meter levels fail. Differential temperature is calculated by subtracting the 10 meter temperature from the 57.9 meter temperature multiplied by 50/47.9 to obtain an accurate 50 meter temperature delta. (approximately 929 ohms to 1190 ohms corresponds to 0 – 120°F).

c) Precipitation Gauge

The precipitation sensor is a tipping bucket rain gauge with a 7.9 inch orifice. This type of sensor funnels rain into a small receptacle which tilts when it has received 0.01 inch of rain and activates a reed switch on every tilt. Each switch closure is recorded by the datalogger pulse channel. The sensor includes a siphoning mechanism that allows the rain to flow at a steady rate regardless of rainfall intensity. The siphon reduces typical rain bucket errors and produces accurate measurements over a range of 0 to 27.6 inches per hour.

2.3.3.3 Data Reduction

The meteorological data for the diffusion evaluation is presently recorded on Dataloggers located in the meteorological tower building and Unit 1 Control Room. Meteorological data is also available through the ERDADS/SAS computer system. This present data includes: EC246531

a) wind direction for the 32.8 ft (10 meter) and 190 ft (57.9 meter) levels of the meteorological tower;

b) wind speed for the 32.8 ft (10 meter) and 190 ft (57.9 meter) levels;

c) vertical temperature lapse rates between the 32.8 ft (10 meter) and 190 ft (57.9 meter) levels;

d) ambient temperature for the 32.8 ft (10 meter) and the 190 ft (57.9 meter) levels;

e) a redundant temperature parameter is available if needed (located at the 110.3 ft (33.5 meter) level and capable of listing ambient temperature and vertical temperature lapse rates between the 110.3 (33.5 meter) and the 32.8 ft (10 meter) levels or the 190 ft (57.9 meter) level); and

f) precipitation at the surface.

NOTE: If the 110.3 ft (33.5 meter) temperature is utilized, the dispersion model algorithm must be modified to show the 110.3 ft (33.5 meter) instead of the 190 ft (57.9 meter) or the 32.8 ft (10 meter). This is to be done when meteorological data is necessary for the diffusion evaluation.

The following discussion, to the end of this sub-section, is considered historical information included for the purposes of licensing the St. Lucie site. As stated in the previous sections, the meteorological equipment has been replaced with updated, more accurate instruments and now complies with the operational requirements of RG 1.23. This discussion will be maintained as-is for historical purposes.

The data reduction program was initiated in mid-January 1971, however the complete on-line instrument system started March 1, 1971. To obtain the highest percent of valid observations, the latter date was selected as the start of the computer input data.

Some concern has existed with regard to the potential effects of the instrument shed blower discharge on the 10-foot tower level temperature sensor. In order to address this concern, a comparison of the annual frequency of Pasquill stability, based upon the thermal gradient criteria stated in USAEC Safety Guide #23, "Onsite Meteorological Programs, February 17, 1972," and derived from the temperature gradient between the 10- to 200-foot tower levels and the 110- to 200- foot tower levels, was performed. The annual frequency distribution between 10 to 200 feet and 110 to 200 feet is presented in Table 2.3-14A.

The results of this comparison indicate that the blower does not appear to have an effect on the 10-foot temperature sensor. It would be anticipated that the 10- to 200-foot data would have a greater frequency of extremely stable (Pasquill F&G) and extremely unstable (Pasquill A) conditions than the 100- to 200-foot data and that 110- to 200-foot data would have a higher frequency of near neutral conditions (Pasquill D). These are indeed the observed differences between the two observational layers. The results are completely explainable based upon the proximity to the ground of the 10-foot sensor. Note that the temperature difference based diffusion analyses presented in Section 2.3.5 is conservative, since there is a higher frequency of stable conditions (Pasquill E, F & G) from the 10- to 200-foot data than would have been the case if 110- to 200-foot data were utilized instead.

On April 6, 1973, the 10 foot temperature and dew point sensors were elevated to the 32.8 foot height of the meteorological tower. Presently, with two exceptions, the meteorological instrumentation at the St. Lucie facility meets the requirements of Regulatory Guide 1.23.

a) The lower wind speed sensor is located at the fifty foot level because at the time of installation, November 1970, there were a number of trees in the vicinity of the weather station and the fifty foot was a reasonable height for the lower wind sensor. All wind speeds at the fifty foot level are reduced to the 32.8 foot level by the wind power law. The trees have been reduced in height since the site has been in operation.

b) The starting speed of the anemometer blades is between 1.0 and 1.5 mph. However the starting speed (2 to 3 mph) of the directional vane does not comply with Regulatory Guide 1.23. The justification for maintaining the present instrumentation is that the equipment was purchased and installed prior to the issuance of Regulatory Guide 1.23 and that St. Lucie is a fairly windy location; only 1% of the wind speeds were less than 1.0 mph and 4% of the winds were 1 to 2 mph.

The frequency of measured low wind speed conditions is provided in Table 2.3-7. This tabulation indicates that the annual distribution of wind speeds less than 1.0 mph is 1.05%, 1.0 mph wind speeds occurred 1.01% and 2.0 mph wind speeds occurred 3.09%. Translating these low wind speed conditions into their respective seven Pasquill vertical temperature categories: A represents approximately 0.1%; B = 0.1%; C = 0.2%; D = 0.4%; E = 1.4%; F = 2.2% and G = 0.7% (Refer to Tables 2.3-20 to 2.3-68).

If the digital recorder values of 1.0 mph wind speeds are suspect because of the high threshold starting speed of the anemometer, and if the annual distribution of 1.0 mph wind speeds were combined with the calm wind conditions, the calculated χ/Q values would not alter because all calm wind conditions are assigned the value of 1.0 mph for calculational purposes. (Refer to Sections 2.3.4.1 and 2.3.4.2). EC246531

In general, the starting speed of the Bendix-Friez anemometer probably results in a higher occurrence of recorded calm wind conditions and provides more conservative χ/Q values because of the inverse relationship of wind speed. The higher response speed required for the wind directional vane will also result in a more conservative χ/Q value (running time average and annual average calculations) because the wind directional trace will persist or lag a longer period of time in a wind sector when the wind speeds are light and variable.

If the wind speed values were below 3.0 mph, the horizontal and vertical Pasquill categories were both determined by the vertical temperature gradient in the 0-2 hour accident model. (Refer to Section 2.3.5.1).

The annual distribution of the simultaneous occurrences of low wind speeds (calm to 2 mph) and poor atmospheric diffusion conditions, Pasquill F and G based on vertical temperature difference, are 2.2% and 0.7%, respectively. Any modifications in the assumed values of wind speeds during these low wind speed and poor diffusion conditions would not alter the 5 percentile worst χ/Q value.

The control room operator can determine the wind speed at the 10 meter altitude in the event of an accidental release by multiplying the 50 foot wind speed by 0.9 (0.8999) for atmospheric stabilities A, B, C and D, or multiplying the 50 foot wind speed by 0.8 (.8099) for stabilities E, F, and G. The atmospheric stabilities would be determined from the digital recorder. The multiplication factors were calculated from the wind power law. (Refer to Section 2.3.5.2). A training session in meteorological procedures for control room operators would include the interpretation, conversion and use of meteorological data. Meteorological procedures will be available for the control room operators. EC246531

2.3.3.4 Telemetric and Data Recording System Description

The revised meteorological data acquisition system for the St. Lucie Plant is designed in accordance with the requirements listed in Regulatory Guide 1.23. This data acquisition equipment is at the onsite meteorological tower. The data output of the sensing equipment is routed to a local recording station located at the base of the meteorological tower and to the Unit 1 control room.

Equipment in the Unit 1 control room meteorological cabinets processes, displays and records the data, including:

a) Wind direction at the 10 and 57.9 meter levels

b) Wind speed at the 10 and 57.9 meter levels

C) Air temperature at the 10 and 57.9 meter levels*

* Air differential temperature is also displayed.

Additional parameters are relayed to, and displayed/recorded on the St. Lucie Unit 1 control room meteorological cabinet from the discharge canal site via VHF radio, and from the intake canal structure via hardwiring. These parameters are discharge canal water level and temperature and intake canal water temperatures.

There is a fiber optic data input from the meteorological cabinet, in the St. Lucie Unit 1 control room, to the Safety Assessment System (SAS). The SAS console displays provide six (6) data points of meteorological information which are wind direction, wind speed and air temperature** at the 10 and 57.9 meter levels. This information as displayed on the SAS console, affords compliance with Regulatory guide 1.97, and enables the control room operators to estimate atmospheric stability for release assessment. EC246531

**An algorithm in SAS furnishes the differential temperature display between the 10 and 60 meter levels, and is average over a 15 minute period.

2.3.3.5 Calibration and Maintenance

a) Bypass Switching

Software switching is programmed into the microprocessor at the Met Tower to switch the sensor data between the "A" and "B" Trains and the various sensor levels on the tower structure. The switching is manually selected at the Met tower datalogger by maintenance personnel. Bypass Switching is used for local analog calibrations of the sensors. While train "A" is being calibrated, train "B" sensor data will be transmitted to ERDADS Unit 1, ERDADS Unit 2 and to the digital recorder in the Unit 1 Control Room. While train "B" is being calibrated, train "A" sensor data will be transmitted to ERDADS Unit 1, ERDADS Unit 2 and to the chart recorder in the Unit 1 Control Room. Operations would receive valid meteorological data at all times except during train swap over. EC246531

b) Trouble and Breakdown Switching

Trouble and Breakdown switching will allow maintenance the time necessary to troubleshoot, acquire parts and develop work orders in the event of a sensor failure. The switching is manually selected at the Met Tower datalogger by maintenance personnel. Switching enables any sensor, at any level in train "A" to be bypassed and the associated train "B" sensor would be automatically swapped into the measured data transmitted to ERDADS Unit 1, ERDADS Unit 2 and to the digital recorder in the Unit 1 Control Room (109.9 ft 33.5 meter temperature sensor is train "A" only and is bypassed and swapped independently of all other sensors). Switching also enables any sensor, at any level in train "B" to be bypassed and the associated train "A" sensor would be automatically swapped into the measured

data transmitted to ERDADS Unit 1, ERDADS Unit 2 and to the digital recorder in the Unit 1 Control Room. Operations in both units would receive valid meteorological data until the actual repair work is scheduled and the tower is powered down for repair.

c) Analog Calibrations

Instrument calibrations are to validate technical requirements of meteorological instruments wind speed, wind direction and air temperature (delta T). Calibrations are performed in accordance with surveillance maintenance procedures on a semiannual basis. Channel checks and channel calibrations are performed in accordance with UFSAR sections 13.8.1.3.2 and table 13.8.1-4.

Calibration checks:
Air temperature probes will be removed from tower carriage and installed in temperature bath. The temperature bath will be set near 32 degrees F for ice bath, ambient temperature for warm, and near 100 degrees F for hot. Loop acceptance criteria are actual temperature +/- 0.9 degrees F.
Wind speed and direction sensors are tested at wind speed of min. 0.00 mph and max. 111.8 mph, and wind direction of 10 and 160 degree angles. Loop acceptance criteria are actual speed +/- 0.36 mph and actual degrees +/- 1.6 degrees.

Calibration of loop instruments will be checked using the AutoCal function of the datalogger as stated in item d) Digital Loop Calibrations below. EC246531

d) Digital Loop Calibrations

The digital loop calibration switching is manually selected at the Met Tower datalogger by maintenance personnel using the AutoCAL function of the datalogger. When switched the dataloggers at the Met Tower and the Discharge Canal are programmed to output digital values equal to 0%, 25%, 50%, 75% and 100% of full scale for each of the variables sent to the plant. The digital values will remain at the different levels for five (5) minutes each. There are independent loop calibration switches for train "A" and train "B". Upon completion of the test, the system will automatically return to normal operations of sending measured data. Routine Bi-weekly PMs are performed by maintenance personnel to verify the digital data path between the microprocessors and the corresponding Unit 1 and 2 ERDADS systems.

Discharge Canal loop calibration testing will be the same as the tower, while train "A" is being tested, sensor data will be transmitted to ERDADS via the train "B" antenna and vice versa.

2.3.4 SHORT TERM (ACCIDENT) DIFFUSION ESTIMATES

The short-term χ/Q values for the low population zone, exclusion area boundary, and control room were revised as part of the dose reanalysis project for implementing alternative source term (AST). The revised χ/Q values are based on guidance provided in Regulatory Guide 1.145, NUREG/CR-2858, NUREG/CR-6331, and Regulatory Guide 1.194. Appendix 2I discusses the determination of χ/Qs for offsite locations and Appendix 2J discusses the determination of χ/Qs for the control room. The meteorological information retained within the following sections is for historical information only, and is not used in the current accident diffusion rate determination.

2.3.4.1 Basis

The method of analysis for determining the atmospheric dispersion potential of the St. Lucie Plant site was to compute hourly relative concentrations for the total year of data and then selecting the appropriate occurrence level for the time period of interest. Invalid data that represented 20 hours or 0.2 percent of the one-year period of record were not considered part of the sample population. Calm wind conditions and extrapolated winds less than 1.0 mph were assigned a value of 1.0 mph.

2.3.4.2 Calculations

0 - 2 Hour Meteorology

The following equation was utilized in calculating the relative concentration at the exclusion area boundary (EAB) distance of 1555 meters (5100 feet) for each hour of the one-year data sample: (16)

$$\frac{\chi}{Q} = \frac{1}{\mu(\pi\sigma_y\sigma_z + CA)}$$

where

χ/Q = relative concentrations (seconds per cubic meter)

μ = average hourly wind speed adjusted to 10-meter elevation (meters/second).

σ_y = horizontal dispersion as determined by the standard deviation of the hourly horizontal wind variability (m). See Table 2.3-14

σ_z = vertical dispersion as determined by the vertical temperature lapse rate (m). See Table 2.3-15

CA = one-half the minimum cross-sectional area of the containment structure (1363 m^2).

The hourly on-shore relative concentrations values were ranked in sequential order of their magnitude to determine cumulative percent distribution. The annual cumulative frequency distribution of the hourly on- or along-shore χ/Q values at the minimum exclusion radius of 1555 meters are plotted in Figure 2.3-3. The calm winds were distributed into on- and off-shore directions by the frequency occurrence of the 1-3 mph speed class per direction and per stability. It is USAEC policy to utilize the dilution factor which has a cumulative on-shore annual hourly occurrence of five percent for its estimate of potential doses from accidental releases for the 0-2 hour period(11.12). This is the probability of having an on-shore dilution factor which could be exceeded 5% of the time which means that better diffusion conditions exist during the year 95% of the time. Based upon this analysis the χ/Q value appropriately conservative for estimating the 0-2 hour potential accident doses, at the minimum exclusion radius of 1555 meters, is 8.55 x 10^{-5} sec/m^3 which is equivalent to the dilution factor that would be calculated using Pasquill F and 2.6 mps. The AEC Safety Guide 4(13) dilution value (Pasquill Type F and 1.0 mps) would be 2.3 x 10^{-4} sec/m^3 and the dilution value used in the PSAR 1.15 x 10^{-4} sec/m^3 (type F and 2.0 mps based on Table 14.15-5 of Hutchinson Island's PSAR).

In response to the NRC's letter of September 3, 1974, FPL committed to revise the short-term accident χ/Q values used in the analysis of the design basis functional requirements of the Engineering Safety Features (ESF) and Technical Specifications for containment leak rate. These χ/Q values are based on NRC conservative methodology and on-site meteorological data. Eleven on-shore wind directions were included in the calculation of the χ/Q value for the 0-2 hour accident period. The 5% worst atmospheric conditions are assumed to occur during the first two hours of the LOCA event. FPL's response is documented in Appendix 6B of the UFSAR and the χ/Q values to be used for the calculation of the bounding dose consequences of the design basis LOCA event are shown in Table 1 of Appendix 6B. The χ/Q value at the EAB of 1555 meters from this table is 1.80×10^{-4} sec/m^3.

Indian River Atmospheric-Dispersion

The degree of over-water atmospheric dispersion is dependent-upon airwater temperature difference, hence, dependent upon the time of day and year. Atmospheric dispersion could be restricted somewhat during extended over-water trajectories when the water is cooler than the air above it. This effect is of particular concern during night-time stable atmospheric conditions.

The question of air-water temperature difference can provide a basis for determining whether Indian River over-water atmospheric dispersion is less than over-land dispersion. Since long-term Atlantic Ocean temperature data is available, Indian River surface temperature data could be collected, correlated with Atlantic Ocean data, and compared to the 10 meter St. Lucie meteorological tower temperature data. In this manner the collection of limited time-period Indian River temperature data is expected to provide sufficient information to determine if the Indian River temperature is warmer, equivalent or colder than Atlantic Ocean temperatures. This in turn would provide the relationship required between Indian River temperature and air temperature.

Before proceeding to a discussion of a temperature measurements program, it must be noted that this concern is somewhat academic for this site because;

i) The Applicant will consider the Staff's very conservative bounding estimates of overwater effect in the development of protective action times for the west bank of the Indian River.

ii) Once the safeguards modifications required by the Staff's September 3, 1974 letter are implemented, the 30 day dose at one mile will be less than the guideline values of 10 CFR 100.

The meteorological measurements gathered at the meteorological tower at the St. Lucie plant site reflect the maritime environment of the Atlantic Coast. The meteorological measurements, wind variability and temperature difference with height, are converted to diffusion parameters based on the methodology of AEC Regulatory Guide 1.23.

The on-shore maritime air mass flow possesses tile characteristics of hundreds of miles of Atlantic Ocean over water flow. Specifically, the horizontal wind variability is small and wind speed is fairly high because of the reduced frictional drag associated with over water flow. The vertical temperature profile, as measured at the meteorological tower, approximately 2,000 feet west of the Atlantic shoreline, represents the vertical temperature profile over the Atlantic Ocean. Hutchinson Island is a flat and narrow impediment (a maximum width of one mile in the vicinity of St. Lucie) to the penetration of on-shore air masses.

Modifications of the maritime air masses induced by the narrow island would be insignificant during unstable conditions, minimal under stable conditions, and generally confined to the lowest ten meter height. The span of the Indian River varys from a minimum of 1 1/2 miles to the west of St. Lucie and 5 miles northwest toward Fort Pierce.

Prophet[18] , Craig[19] , and Bowman[20] , have concluded that thermal adjustments in the vertical direction (and the heights above water to which mixing extends) responds quite rapidly to the air-water temperature differences. Less stable lapse rates and higher values of vertical diffusivity occur when the air is over warmer water and more stable lapse rates and reduced vertical diffusivity occur when air is over colder waters. Van der Hoven recognized the greater significance of these thermal effects at a cooling pond reservoir, as compared with the influence of horizontal wind variance by that reservoir, and concluded that the presence of a warm water body would have a favorable influence on the diffusion climate of the site because of its warming-from-below influence.

Craig [19,22] and Montgomery [23] discuss the rate of modification of the boundary layer temperatures when air moves over water warmer than the surface air temperatures. In all the data presented, a super adiabatic layer forms below 20 feet with an adiabatic layer above, extending to a minimum height, in one case of 400 feet, and in other cases to above 1,000 feet. The over water trajectories are long, compared with the St. Lucie site, extending from 8 miles to 100 miles. The data do show, however, that the modification process is much more rapid when the air column is, heated from below than when it is cooled from below. Furthermore, Craig[24] also finds fairly significant, i. e. 20 to 40%, changes in air temperature, occurring in the lowest fifty feet for a two mile over water trajectory during low wind speed conditions.

Accordingly, the flux of sensible heat from the water into the air may be estimated from standard meteorological wind and temperature measurements under stable conditions. The sensible heat, $H = K (\theta_w - \theta_a) \overline{V}_a$, where θ_w is the surface potential temperature and θ_a, $\overline{V}_a$ is the air potential temperature and mean wind speed at anemometer level (10 meters). A nomogram presented by Van der Hoven[26] and Prophet[18] was used investigate stability modification associated with the on-shore sea breeze[24] flow for the Pilgrim Nuclear Power Station. The air mass modification, due to a solar heated land surface, follows the same physical laws as the thermal adjustment to cool air moving over warm water. In both cases, with sufficient time, an adiabatic lapse rate results where the temperature at the base of the modified air is equal to the temperature of the underlying surface, Knowing the meteorological conditions, one may then determine the extent of the modification based upon the expected over-water trajectory and the height to which the adiabatic

layer extends. For example, given a Pasquill G condition and $\theta_w - \theta_a = 2°F$, an adiabatic lapse rate of about 60 feet deep, results in a new lower stability of Pasquill F for over-water trajectories of twenty minutes or more.

In order to evaluate the relative magnitude of the effects of the Indian River upon St. Lucie stability and, consequently, χ/Q values, the following assumptions will be made regarding temperature differences between the Indian River and the St. Lucie site:

	Early Winter	Early Summer
Day	Case 1: Water temperature similar to air temperature	Case 2: Water temperature cooler than air temperature.
Night	Case 3: Water temperature warmer than air temperature	Case 4: Water temperature similar to air temperature

The seasons early winter and early summer were chosen because the air-water temperature differences are the greatest; other seasons are transitional. Case 2 and 3 above are the conditions when the greatest differences between air over land and air over water pertain. Therefore, during Cases 2 and 3, when the wind is blowing from the site toward the river, the effect of the Indian River upon stability is at a maximum.

Utilizing a 3°F temperature modification, a wind speed of 1 m/sec. and a distance from the source of 1000 m, the standard Gaussian ground level release diffusion equation is; $\frac{\chi}{Q} = \frac{I}{\pi \sigma_y \sigma_z \mu}$

the components σ_y and σ_z are the horizontal and vertical standard deviations from plume centerline, respectively, and μ is the wind speed. For Case 2, an initial stability of A is assumed to be modified to a B stability by passage over the river, and, similarly, for Case 3, an F stability is modified to E. Such changes are based on the work done by Van der Hoven[26]. Then,

Stability	A	B	E	F
χ/Q	3.29×10^{-6}	1.86×10^{-5}	2.96×10^{-4}	6.79×10^{-4}

The change for Case 2 is B-A = 1.53×10^{-5} and for Case 3, F-E = 3.83×10^{-4}.

The χ/Q difference caused by over-water passage is much greater for stable conditions than unstable conditions (by a factor of 25) because of the exponential variability of σ_y and σ_z with stability class. This is of particular significance at St. Lucie since it is expected that the river surface temperature will provide a destabilizing (warming from below) modification at night when stable atmospheric conditions are prevalent.

Therefore, as concluded by Slade[25] and Van der Hoven[26], "the magnitude of over-water diffusion is greatly influenced by the water-air temperature difference". Thus monitoring water-air temperature differences provides a first approximation of over-water atmospheric dispersion characteristics.

Measurement Program

The Indian River surface temperature will be monitored and recorded.
The maximum and minimum daily Indian River temperature data will be reduced to the nearest degree Fahrenheit. The Indian River temperature data will be compared with the hourly ten meter temperature data at the St. Lucie meteorological tower and correlated with Atlantic Ocean data.

Preliminary Analysis

Since proximal Indian River temperature data are not available for the coincident St. Lucie meteorological measurement program period, a preliminary comparison of Atlantic Ocean surface temperature versus St. Lucie meteorological tower 10 foot temperature data has been made. The Atlantic Ocean surface temperatures are measured several thousand feet east of St. Lucie at a depth of several feet below the surface. Five day average Atlantic Ocean temperature data are presented in the St. Lucie Unit 2 Environmental Report, Table 2.7-6. Comparisons of the Atlantic Ocean surface water temperatures to the Indian River surface water temperatures are sparse. Pending the correlations of coincident water body temperatures, it can be assumed that the Atlantic Ocean surface water temperatures are colder than the Indian River surface water temperatures. This assumption is based on the fact that a small shallow (4 to 10 feet deep) water body will respond to day-time heating faster than a larger water body.

Table 2.3-1B presents a comparison of St. Lucie 10 foot meteorological tower temperature with the Atlantic Ocean five-day average surface temperatures (°F) for a period from March 1, 1971 to February 29, 1972. This analysis for the Atlantic Ocean indicates that average monthly surface water temperature is not cooler than average monthly night time atmospheric temperatures.

Refer to Appendix 2H for additional information.

Analysis Of The Probability of Fumigation Occurence

Fumigation is a meteorological condition whereby an elevated effluent is released during stable (inversion) periods. It then diffuses in a narrow plume above the ground and slowly grows laterally and vertically as is characteristic of inversion conditions. However, as the plume moves downwind from its source, it is brought rapidly down to the ground because the ground level conditions have changed from inversion to lapse conditions while aloft inversion conditions still may exist. This condition can develop in the early morning hours (shortly after sunrise) as the earth's surface is heated by incoming solar radiation. This condition can also occur with a sea breeze situation in the late morning or early afternoon with the trapping of a shallow unstable layer close to the ground and inversion conditions aloft.

An analysis was performed to develop a fumigation probability of occurrence at the St. Lucie Site. Onsite meteorological data from March 1, 1971 to December 30, 1972 were reviewed. Temperature difference measurements were made between approximately 10 and 100 feet and between 10 and 200 feet. All digital recorders were reviewed for fumigation occurrence. In this study, a fumigation condition was defined to be a situation whereby inversion conditions exist for a least one hour followed by a crossover to either lapse or neutral conditions. For each case of fumigation that was identified, the stability, wind speed, and wind direction conditions that persisted for the previous hour were listed. These listed conditions were separated according to on-shore and off-shore winds (SSW through WNW being off-shore and NW through S in a clockwise direction being on-shore). Relative concentrations were calculated for each occurrence of fumigation associated with on-shore winds. The equation used was taken from Safety Guide 5 "Assumptions Used For Evaluating The Potential Radiological Consequences Of A Steam Line Break Accident For Boiling Water Reactors". EC246531

$$X/Q = 0.0133/\sigma_Y u$$

where:

X/Q =Relative concentration (sec/m^3)

σ_Y = the horizontal standard deviation of the plume (meters) - This value was based on the stability class which prevailed for the hour prior to fumigation and was calculated at a distance of 1555m.

u = wind speed (m/sec) - This was the average 50 foot wind speed for the hour preceding the fumigation.

For the 22 month period there were 255 occurrences of fumigation conditions associated with on-shore winds. Fumigation conditions generally persist only for periods of 30 minutes or less. The relative probability of occurrence of fumigation with on-shore winds is 255-30 minute periods (fumigation) divided by 31680-30 minute periods in 22 months, or less than 1 percent of the time. The probability of occurrence is less than the

traditional NRC value of 5 percent worst diffusion factors utilized in assessing potential accident consequences.

The 255 cases of fumigation were grouped according to the stability class which persisted for the hour prior to fumigation. The maximum, minimum, and average wind speeds were determined for each stability class. The following reflects these statistics:

FUMIGATION OCCURRENCES PER STABILITY CLASS
WITH ASSOCIATED WIND SPEEDS (m/sec)

	Stability Class		
	E	F	G
FREQUENCY	162	74	19
MIN WS	.4	.2	.2
AVE WS	2.7	1.8	1.8
MAX WS	6.7	3.1	3.1

A cumulative frequency listing of relative concentration values was prepared. Every ten percentile value was plotted. This plot is presented in Figure 2.3-10. The 50 percentile X/Q is 9.4×10^{-5} sec/m^3, which is equivalent to a stability of F and a wind speed of 2.55 m/sec.

In summary, we feel that it is not necessary to consider the consequences of fumigation since it occurs less than I percent of the time in association with on-shore winds. However, if the NRC desires to evaluate these consequences, the X/Q value to be evaluated is 9.4×10^{-5} sec/m^3, which is equivalent to F stability and a wind speed of 2.55 m/sec.

30 Day Meteorology

The 30-day potential accident has been divided into four time periods as described in AEC Safety Guide 4[13], i.e., 0-8 hours, 8-24 hours, 1-4 days, and 4-30 days. For each of these time periods of 8 hours, 16 hours, 3 days and 26 days, the total sample moving average relative concentrations were computed at the low population zone distance of 5 miles or 8047 meters. The method of computation was to determine the moving average relative concentration, spread over a 22-1/2 degree sector, for one year of consecutive hourly observations for each of the 16 cardinal directions.

The following equation was used:[13]

$$\frac{\chi}{Q} = \frac{1}{n} \sum_{i=1}^{n} \frac{2.032}{u_I \ \sigma_{zi} \ D}$$

where

$\frac{\chi}{Q}$ = average relative concentration in seconds per cubic meter for time period of interest for each sector.

u = average hourly wind speed in meters per second with a minimum assignment of 0.447 mps (1.0 mph).

σ_z = vertical dispersion as determined by the vertical temperature lapse rate in meters. See Table 2.3-15.

D = the distance in meters to the low population zone distance of 8047 meters (Revised to 1609 meters for the analysis of the dose consequences of the LOCA event. See Appendix 6B.)

n,i = the average time periods of 8 hours, 16 hours, 3 days, and 26 days.

2.032 = constant that incorporates a sector spread of 22-1/2 degrees or π/8 radians.

The resulting running averages of the atmospheric dilution factors from the one year of data, for the individual four time periods, were tabulated in descending order of magnitude by class interval with the most conservative dilution factor first. The percent distribution of the computed dilution factors were grouped such that a cumulative percent probability of having a value more or less conservative than a given value can be obtained. The cumulative probability distribution of the atmospheric dilution factors is presented on Figures 2.3-4 through 2.3-7 for the four time periods of interest. The off-shore wind directions were considered part of the sample population and the basis of selecting the worst case. Tables 2.3-16 through 19 lists the highest relative concentration by sector for the four time periods. The time periods are not mutually exclusive, i.e., the worst 0-8 hour period is also an integral part of the worst 0-16 hours and 0-3 day episode.

In response to the NRC's letter of September 3, 1974, FPL committed to revise the short term accident χ/Q values used in the analysis of the design basis functional requirements of the Engineered Safety Features (ESF) and Technical Specifications for containment leak rate. In this response FPL committed to modify the ESF design to provide sufficient dose reduction in the LOCA event to meet the dose requirements of 10 CFR 100 at a low population zone distance (LPZ) of one mile. A very conservative approach has been used in the determination of these LPZ χ/Q values. The worst annual average χ/Q sector and 0-2 hour accident χ/Q were plotted on a log-time scale and a straight line interpolation provided the χ/Q values at 0-8 hour, 0-16 hour, 1-4 days and 4-30 days. FPL's response is documented in Appendix 6B of the UFSAR and the χ/Q values to be used for the calculation of the bounding dose consequences of the design basis LOCA event are shown in Table 1 of Appendix 6B.

2.3.4.3 Conclusions

The summary of the recommended conservative χ/Q values to be used in the accident analysis section of the UFSAR are tabulated. The χ/Q values given below are not applicable to any analysis performed using AST methodology.

Time Period	Location / Distance (meters)	Safety Basis for Selection	Recommended χ/Q (sec/m^3)	Guide 4[13] χ/Q (sec/m^3)
0-2 hours	Exclusion Zone (1555)	5%	8.55×10^{-5}	2.30×10^{-4}
0-8 hours	Low Population Zone (1609)	*Worst Case	7.97×10^{-6}	3.05×10^{-5}
8-24 hours	Low Population Zone (1609)	Worst Case	4.26×10^{-6}	6.00×10^{-6}
1-4 days	Low Population Zone (1609)	Worst Case	9.63×10^{-7}	2.00×10^{-6}
4-30 days	Low Population Zone (1609)	Worst Case	3.68×10^{-7}	4.15×10^{-7}

*Worst running average dilution factor for any of the 16 cardinal point directions.

Figures 2.3-8 and 2.3-9 present graphically the accident analysis dilution factors versus distance for each of the time periods of interest.

In response to the NRC's letter of September 3, 1974 FPL committed to revise the short term accident χ/Q values used in the analysis of the design basis functional requirements of the Engineered Safety Features (ESF) and Technical Specifications for containment leak rate. In this response FPL committed to modify the ESF design to provide sufficient dose reduction in the LOCA event to meet the dose requirements of 10 CFR 100 at a low population zone (LPZ) distance of one mile. FPL's response is documented in Appendix 6B of the UFSAR and the χ/Q values to be used for the calculation of the bounding dose consequences of the design basis LOCA event are shown in Table 1 of Appendix 6B and summarized below. Subsequently, the radiological dose consequences of the non-LOCA events have also been revised to incorporate these more conservative χ/Q values.

Time Period	**Location / Distance (meters)**	**Basis for Selection**	**Recommended χ/Q (sec/m^3) Appendix 6B Table 1**	**9/3/74 NRC staff position χ/Q (sec/m^3)**
0-2 hours	EAB (1555)	NRC Methodology	1.80×10^{-4}	2.2×10^{-4}
0-8 hours	LPZ (1609)	NRC Methodology	8.50×10^{-5}	1.1×10^{-4}
8-24 hours	LPZ (1609)	NRC Methodology	5.90×10^{-5}	7.2×10^{-5}
1-4 days	LPZ (1609)	NRC Methodology	2.70×10^{-5}	3.1×10^{-5}
4-30 days	LPZ (1609)	NRC Methodology	8.50×10^{-6}	9.2×10^{-6}

The selection of the accident analysis meteorological assumptions are extremely conservative and have been developed specifically for Hutchinson Island using one year of on-site measured data.

2.3.5 LONG TERM (ROUTINE) DIFFUSION ESTIMATES

2.3.5.1 Basis

Based upon on-site meteorological measurements a frequency distribution of the Pasquill diffusion categories has been developed using two independent methods. The meteorological tower vertical temperature between the 10 and 57.9 m levels was used to determine the vertical temperature gradient. The categorization of vertical temperature gradient are listed in Table 2.3-15[14]. The horizontal Pasquill stability categorization is based upon the horizontal wind variability (sigma-theta).

Sigma-theta was calculated by dividing the wind direction range by 6[15]. If the wind speed was below 3 mph, the minimum threshold speed of the wind vane, the horizontal and vertical Pasquill categorization were both determined by the vertical temperature gradient method using the relationships listed in Table 2.3-15. The breakdown of horizontal Pasquill types versus sigma-theta are listed in Table 2.3-14[16]. This split technique of determining the vertical and horizontal Pasquill categories provides a better means of reflecting local micro-climatological effects than either technique by itself could, i.e., site surface roughness may result in increased wind variability with a resulting enhanced lateral plume growth but yet not effect the vertical turbulence. Using the sigma-theta technique alone could over-predict potential diffusion, while using vertical temperature gradient techniques alone could under-predict potential diffusion in this case. There are other situations which could have the reverse effect, i.e., over-predict diffusion using vertical temperature gradient and under-predict diffusion using the sigma-theta technique. This split technique uses the best of both techniques to provide a more realistic categorization of the vertical and horizontal Pasquill types.

2.3.5.2 Calculations

The 50-foot tower level wind data were selected for the wind distribution stability study because they were determined to be appropriately conservative due to the near-ground level potential releases from the St. Lucie Plant. However, if the 50-foot wind data were missing or invalid (7.7 percent of annual record period), the 190-foot wind data were substituted. All meteorological tower wind speeds were reduced to a 10-meter level by the Pasquill definition of the vertical temperature lapse rate (Table 2.3-15) and the following wind speed power law:[17]

$$^{\mu}10\text{ meter} = {}^{\mu}h\left(\frac{10\text{ meter}}{h\text{ meter}}\right)^{n}$$

where μ = wind speed (mph)
h = 50-foot height (or the 190-foot height when the 50-foot wind speed was missing or invalid)
n = .25 for Pasquill A, B, C, D, or .50 Pasquill E, F, and G

The above technique is desired by the USAEC for standardization purposes so that different sites can be compared on equal terms.

The one year of hourly data was distributed into a 7 by 7 matrix of vertical and horizontal classifications by wind direction versus computed 10-meter wind speeds. Tables 2.3-20 through 2.3-68 are the annual hourly percent frequency distributions of wind direction by wind speed classes for 49 discreet stability categories using the 50-foot level wind data reduced to the 10-meter elevation, the 10-to-200-foot vertical temperature gradient for the vertical Pasquill categorization and wind direction variability at the 50-foot level for the horizontal

Pasquill categorization. The inclusive range in mph of the wind speed classes is assigned in the following manner: 1.0 to 3.4 for 1-3 class; 3.5 to 7.4 for the 4-7 class, etc. Extrapolated wind speed below 1.0 mph were assigned to the calm and variable class.

Average annual values of relative dilution were computed by summing each of the hourly dilution factors during the year for the 16 sectors out to a distance of 50 miles by the following equation:

$$\frac{\overline{\chi}}{Q(s,x)} = \frac{1}{n}\sum_{i=1}^{n\ \text{hrs}} \frac{2.032}{\sigma_{z(x,p)}\,\mu_s\, x}$$

where $\dfrac{\overline{\chi}}{Q(s,x)}$ = average annual relative dilution for each sector s and distance x (meters).

$\sigma_z(x,p)$ = vertical dispersion coefficient at distance x as defined by the vertical temperature gradient Pasquill category p (m).

μ_s = average hourly wind speed in the specific sector, s (mps) with a minimum assignment of 0.447 mps (1.0 mph)

n,i = the valid hourly data sample for the year.

An additional 22 months of data collection has been analyzed by use of Code LSD-5, which incorporates the diluting equation recommended by RG 1.42, and is summarized in Tables 2.3-70 through 2.3-93 showing the annual average χ/Q values by distance and direction for 1972 and 1973 (including a redundancy in the use of the January and February 1972 data) and the joint wind speed and direction frequency distribution by stability classes for the years 1971-1973. The 7 joint wind frequency tables representing the 7 vertical stability classes for each of the years were totaled from 49 frequency tables that were broken down into both horizontal and vertical stability categories.

2.3.5.3 Conclusions

The wind distribution of the dual stability categories indicates that the majority of the stable, poor atmospheric diffusion conditions (Pasquill E F and G) are associated with the seven off-shore wind directions (S clockwise through NW). The unstable and neutral atmospheric conditions (Pasquill A B C and D) are predominantly associated with the nine off-shore wind directions (NNW clockwise through SSE).

Tables 2.3-69 through - 71 summarize the average annual relative dilution as a function of direction and distance. The highest annual average relative dilution factor of 1.73 x 10^{-6} was in the NE direction at the edge of the restricted distance of 1555 meters.

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TABLE 2.3-1

TORNADOES IN SOUTHEASTERN FLORIDA
1958-1970

	County				
	Palm Beach	Martin	St. Lucie	Indian River	Okeechobee
1958	13	0	1	1	0
1959	1	1	0	0	1
1960	2	0	1	1	0
1961	1	0	1	1	1
1962	1	0	1	0	0
1963	2	1	0	1	0
1964	4	1	0	0	0
1965	1	0	0	0	0
1966	0	0	0	1	0
1967	2	0	1	0	0
1968	5	0	0	1	1
1969	4	0	0	0	2
1970	4	1	1	1	1

TABLE 2.3-1A

CUMULATIVE FREQUENCY OF WATERSPOUTS
Occurring within 100 miles
From St. Lucie for Various Distances Offshore and the
Probability and Recurrence Internals Based Upon
Storm Data records from 1952 to 1973

	Distance From Land (miles) 0 - 2	0 - 4	0 - 8	0 - 25
Frequency	8	17	41	52
Adjusted Frequencies By Apportioning 126 of the Unknown Observations	27	58	140	178
Area Examined (square miles)	400	800	1600	5000
Probability (10^{-4})	4.60	4.94	5.97	2.40
Recurrence Interval (years)	2174	2024	1675	4167

Table 2.3-1B

Average Monthly Temperature and Temperature Difference
Comparisons (°F) Between The Ten-Foot Level
of the St. Lucie Meteorological Tower
and the Atlantic Ocean

Period of Record: March 1, 1971 to Feb. 29, 1972

Note: A negative temperature difference indicates the ocean is warmer than the 10-foot level.

	Average Temperatures			Temperature Difference			
	Atlantic	St.Lucie		Daytime Minus	Nighttime Minus	St.Lucie One Hour	
Month	Ocean	Day time	Night Time	Ocean	Ocean	Maximum	Minimum
March	69.5*	69.3	63.4	-0.2	-6.1	85	44
April	71.3*	74.4	68.9	+3.1	-2.4	94	49
May	74.7	77.3	73.1	+2.6	-1.6	89	59
June	79.3	80.7	75.2	+1.4	-4.1	90	68
July	79.8	82.7	77.8	+2.9	-2.0	88	71
August	78.0	83.9	78.0	+5.9	0.0	93	72
September	82.2	81.4	77.8	-0.8	-4.4	86	66
October	81.3	79.6	76.1	-1.7	-5.2	87	65
November	75.8	74.2	70.3	-1.6	-5.5	82	54
December	73.5	74.4	71.2	+0.9	-2.3	81	57
January	74.0	73.8	69.6	-0.2	-4.4	83	51
February	70.5	67.9	63.2	-2.6	-7.3	82	41

*Several missing surface Atlantic Ocean temperatures replaced by ocean bottom temperatures

TABLE 2.3-2

FLORIDA POWER & LIGHT COMPANY
HUTCINSON ISLAND METEOROLOGICAL TOWER 50 FOOT HEIGHT

WIND FREQUENCY DISTRIBUTION IN PERCENT

SEASON; WINTER
DEC. 1, 1971 TO FEB. 29, 1972

WIND SECTOR	WIND SPEED CLASSES IN M.P.H. 1-3	4-7	8-12	13-18	19-24	25-31	31+	TOTAL PERCENT	AVERAGE SPEED M.P.H.
NNE	0.14	1.19	1.24	0.37	0.46	0.00	0.00	3.39	10.19
NE	0.05	1.10	1.97	1.51	0.73	0.14	0.00	5.50	12.43
ENE	0.05	1.51	2.38	1.37	0.78	0.14	0.00	6.23	11.60
E	0.05	1.92	2.79	1.37	0.55	0.00	0.00	6.69	10.68
ESE	0.05	5.18	5.41	1.37	0.09	0.00	0.00	12.09	8.56
SE	0.05	2.47	5.36	1.47	0.09	0.00	0.00	9.44	9.67
SSE	0.18	2.02	3.85	1.15	0.05	0.00	0.00	7.24	9.31
S	0.37	5.22	4.81	0.60	0.00	0.00	0.00	10.99	7.86
SSW	0.32	2.06	1.74	0.60	0.18	0.00	0.00	4.90	8.42
SW	0.55	2.43	1.69	0.60	0.18	0.00	0.00	5.45	8.07
WSW	0.46	1.28	0.78	0.05	0.00	0.00	0.00	2.57	6.14
W	0.41	0.55	0.41	0.18	0.14	0.00	0.00	1.69	8.16
WNW	0.41	0.96	0.96	0.50	0.18	0.00	0.00	3.02	8.74
NW	0.18	2.66	2.02	0.60	0.00	0.00	0.00	5.45	7.68
NNW	0.18	1.83	2.34	0.82	0.00	0.00	0.00	5.18	8.81
N	0.05	1.37	2.52	1.47	0.69	0.00	0.00	6.09	11.47
CALM AND VARIABLE		0.00	0.00	0.00	0.00	0.00	0.00	4.08	1.66
	-----	----	----	----	----	----	----	----	----
TOTALS:	3.52%	33.76	40.27%	14.02%	4.12%	0.27%	0.00%	100.00%	9.00

NUMBER OF VALID SEASONAL OBSERVATIONS = 2183

NUMBER OF SEASONAL INVALID OBSERVATIONS = 1

NOTE: ALL PERCENTS ARE BASED ON SEASONAL VALID OBSERVATIONS FOR WIND SPEEDS AT THE 50 FT ELEVATION

TABLE 2.3-3

FLORIDA POWER & LIGHT COMPANY
HUTCINSON ISLAND METEOROLOGICAL TOWER 50 FOOT HEIGHT

WIND FREQUENCY DISTRIBUTION IN PERCENT

SEASON; SPRING
MAR. 1, 1971 TO MAY 31, 1971

WIND SECTOR	1-3	4-7	8-12	13-18	19-24	25-31	31+	TOTAL PERCENT	AVERAGE SPEED M.P.H.
	WIND SPEED CLASSES IN M.P.H.								
NNE	0.16	0.84	3.42	0.95	0.00	0.00	0.00	5.37	9.65
NE	0.21	2.74	2.95	0.21	0.00	0.00	0.00	6.11	7.93
ENE	0.05	2.42	2.37	0.00	0.00	0.00	0.00	4.85	7.27
E	0.37	3.27	4.21	0.05	0.00	0.00	0.00	7.90	7.59
ESE	0.26	2.58	3.37	0.11	0.00	0.00	0.00	6.32	7.47
SE	0.42	1.53	4.32	1.32	0.00	0.00	0.00	7.59	9.61
SSE	0.32	1.58	2.74	1.58	0.00	0.00	0.00	6.22	9.60
S	0.11	1.21	2.85	0.79	0.11	0.00	0.00	5.06	9.53
SSW	0.47	1.37	2.11	1.48	0.47	0.00	0.00	5.90	10.30
SW	0.47	3.32	2.85	1.69	0.90	0.11	0.00	9.33	10.01
WSW	0.58	2.32	2.69	0.37	0.42	0.00	0.00	6.38	8.84
W	0.42	2.48	1.26	0.26	0.00	0.00	0.00	4.43	7.10
WNW	0.58	2.00	2.90	1.00	0.00	0.00	0.00	6.48	8.41
NW	0.47	1.74	4.58	1.63	0.00	0.00	0.00	8.43	9.46
NNW	0.11	1.05	1.05	0.68	0.00	0.00	0.00	2.90	9.27
N	0.05	0.79	2.11	2.21	0.00	0.00	0.00	5.16	11.52
CALM AND VARIABLE		0.00	0.00	0.00	0.00	0.00	0.00	1.58	1.37
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TOTALS	5.05%	31.24%	45.79%	14.33%	1.90%	0.11%	0.00%	100.00%	8.88

NUMBER OF VALID SEASONAL OBSERVATIONS = 1898

NUMBER OF SEASONAL INVALID OBSERVATIONS = 310

NOTE: ALL PERCENTS ARE BASED ON SEASONAL VALID OBSERVATIONS FOR WIND SPEEDS AT THE 50 FT ELEVATION

TABLE 2.3-4

FLORIDA POWER & LIGHT COMPANY
HUTCHINSON ISLAND METEOROLOGICAL TOWER 50 FOOT HEIGHT

WIND FREQUENCY DISTRIBUTION IN PERCENT
SEASON; SUMMER
JUNE 1, 1971 TO AUG. 31, 1971

WIND SECTOR	WIND SPEED CLASSES IN M.P.H. 1-3	4-7	8-12	13-18	19-24	25-31	31+	TOTAL PERCENT	AVERAGE SPEED M.P.H.
NNE	0.11	1.03	0.65	0.11	0.00	0.00	0.00	1.89	7.23
NE	0.70	2.06	0.76	0.11	0.00	0.00	0.00	3.63	6.01
ENE	0.70	3.47	2.27	0.05	0.00	0.00	0.00	6.50	6.70
E	0.65	5.85	5.31	0.22	0.00	0.00	0.00	12.02	7.26
ESE	0.54	4.87	4.44	0.11	0.00	0.00	0.00	9.96	7.06
SE	0.70	6.17	7.47	0.32	0.00	0.00	.0.00	14.67	7.64
SSE	0.43	3.63	3.47	0.60	0.00	0.00	0.00	8.12	7.61
S	0.54	2.44	3.74	1.08	0.00	0.00	0.00	7.80	8.78
SSW	0.22	2.76	2.17	0.43	0.05	0.00	0.00	5.63	7.64
SW	0.60	4.93	2.00	0.11	0.00	0.00	0.00	7.63	6.20
WSW	0.43	1,89	0.32	0.38	0.05	0.00	0.00	3.09	6.56
W	0.81	1.84	0.81	0.05	0.00	0.00	0.00	3.52	5.74
WNW	0.32	1.68	0.16	0.05	0.00	0.00	0.00	2.22	5.39
NW	0.05	0.87	0.60	0.00	0.00	0.00	0.00	1.52	6.82
NNW	0.11	0.22	0.11	0.05	0.00	0.00	0.00	0.49	6.89
N	0.27	1.03	0.60	0.05	0.00	0.00	0.00	1.95	6.36
CALM AND VARIABLE		0.00	0.00	0.00	0.00	0.00	0.00	9.37	1.18
	----	----	----	----	----	----	----	----	----
TOTALS:	7.19%	44.72%	34.87%	3.74%	0.11%	0.00%	0.00%	100.00%	6.59

NUMBER OF VALID SEASONAL OBSERVATIONS = 1847

NUMBER OF SEASONAL INVALID OBSERVATIONS = 361

NOTE: ALL PERCENTS ARE BASED ON SEASONAL VALID OBSERVATIONS FOR WIND SPEEDS AT THE 50 FT ELEVATION

TABLE 2.3-5

FLORIDA POWER & LIGHT COMPANY
HUTCHINSON ISLAND METEOROLOGICAL TOWER 50 FOOT HEIGHT

WIND FREQUENCY DISTRIBUTION IN PERCENT

SEASON: FALL
SEP. 1, 1971 TO NOV. 30, 1971

WIND SECTOR	WIND SPEED CLASSES IN M.P.H. 1-3	4-7	8-12	13-18	19-24	25-31	31+	TOTAL PERCENT	AVERAGE SPEED M.P.H
NNE	0.05	0.69	2.43	1.47	0.23	0.00	0.00	4.85	11.34
NE	0.14	1.88	2.61	0.92	0.73	0.00	0.00	6.27	10.67
ENE	0.78	2.75	5.27	2.61	0.50	0.00	0.00	11.90	9.97
E	0.18	3.89	6.00	2.98	0.23	0.00	0.00	13.28	9.89
ESE	0.37	2.88	3.43	1.28	0.05	0.00	0.00	8.01	8.90
SE	0.37	3.11	3.02	0.55	0.00	0.00	0.00	7.05	7.74
SSE	0.27	2.70	1.05	0.18	0.00	0.00	0.00	4.21	6.82
S	0.23	1.51	1.28	0.14	0.00	0.00	0.00	3.16	7.36
SSW	0.23	2.01	0.96	0.41	0.14	0.00	0.00	3.75	8.11
SW	0.96	3.48	2.20	0.46	0.00	0.00	0.00	7.10	6.97
WSW	0.46	2.15	1.28	0.23	0.00	0.00	0.00	4.12	6.99
W	0.27	1.01	0.27	0.00	0.00	0.00	0.00	1.56	5.47
WNW	0.41	1.60	0.46	0.05	0.00	0.00	0.00	2.52	5.73
NW	0.87	3.89	1.28	0.27	0.00	0.00	0.00	6.32	6.45
NNW	0.23	1.51	1.65	0.78	0.00	0.00	0.00	4.17	8.58
N	0.09	0.73	2.43	2.66	0.00	0.00	0.00	5.91	11.70
CALM AND VARIABLE		0.00	0.00	0.00	0.00	0.00	0.00	5.82	1.57
	----	----	----	----	----	----	----	----	----
TOTALS:	5.91%	35.81%	35.62%	14.97%	1.88%	0.00%	0.00%	100.00%	8.36

NUMBER OF VALID SEASONAL OBSERVATIONS = 2184

NUMBER OF SEASONAL INVALID OBSERVATIONS = 0

NOTE: ALL PERCENTS ARE BASED ON SEASONAL VALID OBSERVATIONS FOR WIND SPEEDS AT THE 50 FT ELEVATION

TABLE 2.3-6

FLORIDA POWER & LIGHT COMPANY
HUTCHINSON ISLAND METEOROLOGICAL TOWER 50 FOOT HEIGHT

WIND FREQUENCY DISTRIBUTION IN PERCENT

ANNUAL WIND REPORT
MAR. 1, 1971 TO FEB. 29, 1972

WIND SECTOR	WIND SPEED CLASSES IN M.P.H. 1-3	4-7	8-12	13-18	19-24	25-31	31+	TOTAL PERCENT	AVERAGE SPEED M.P.H
NNE	0.11	0.94	1.94	0.74	0.18	0.00	0.00	3.91	10.07
NE	0.26	1.91	2.10	0.73	0.39	0.04	0.00	5.42	9.72
ENE	0.39	2.50	3.13	1.08	0.35	0.04	0.00	7.50	9.28
E	0.30	3.66	4.56	1.23	0.21	0.00	0.00	9.96	8.88
ESE	0.30	3.88	4.18	0.76	0.04	0.00	0.00	9.16	8.09
SE	0.37	3.27	4.97	0.92	0.02	0.00	0.00	9.55	8.57
SSE	0.30	2.47	2.75	0.86	0.01	0.00	0.00	6.39	8.44
S	0.31	2.65	3.16	0.63	0.02	0.00	0.00	6.77	8.33
SSW	0.31	2.05	1.71	0.71	0.21	0.00	0.00	4.99	8.68
SW	0.65	3.49	2.17	0.70	0.26	0.02	0.00	7.30	7.92
WSW	0.48	1.90	1.26	0.25	0.11	0.00	0.00	3.99	7.46
W	0.47	1.42	0.67	0.12	0.04	0.00	0.00	2.71	6.62
WNW	0.43	1.54	1.10	0.39	0.05	0.00	0.00	3.51	7.53
NW	0.41	2.37	2.10	0.62	0.00	0.00	0.00	5.49	7.88
NNW	0.16	1.20	1.34	0.60	0.00	0.00	0.00	3.30	8.76
N	0.11	0.99	1.96	1.64	0.18	0.00	0.00	4.88	11.09
CALM AND VARIABLE		0.00	0.00	0.00	0.00	0.00	0.00	5.15	1.42
	----	----	----	----	----	----	----	----	----
TOTALS	5.36%	36.22%	39.08%	12.01%	2.08%	0.10%	0.00%	100.00%	8.25

NUMBER OF VALID ANNUAL OBSERVATIONS = 8112

NUMBER OF ANNUAL INVALID OBSERVATIONS = 672

NOTE: ALL PERCENTS ARE BASED ON ANNUAL VALID OBSERVATIONS FOR WIND SPEEDS AT THE 50 FT ELEVATION

TABLE 2.3-7

SEASONAL HOURLY DISTRIBUTION OF CALM AND VARIABLE WINDS

Hutchinson Island 50-foot level

Period of Record: March 1, 1971 - February 29, 1972

Season[1]	Calm[2] <1.0 mph	Variable[3] 1.0 mph	2.0 mph	Total Hours	Annual Percent	Greatest Consecutive Hours Non-directional Winds
Winter	9	14	66	89	1.10	7
Spring	6	7	17	30	0.37	6
Summer	51	40	82	173	2.13	11
Fall	19	21	86	126	1.55	6
Annual Hours	85	82	251	418		
Annual Percent	1.05	1.01	3.09	5.15		

1. Months included in each season are:
   Winter December, January, February
   Spring: March , April May
   Summer: June, July, August
   Fall: September, October, November

2. Calm wind is defined as a wind speed less than 1.0 mph.

3. A variable wind is defined when the wind directional trace is in a steplike pattern and the wind speed is between 1.0 and 2.5 mph.

TABLE 2.3-8

DISTRIBUTION OF ON-SHORE (NNW-SSE) AND OFF-SHORE (S-NW) WINDS

Hutchinson Island, Florida 50-foot Level

Period of Record: March 1, 1971 - February 29, 1972

(All speed classes included except calms.)

	Off Shore 7 Sectors		On Shore 9 Sectors	
	Seasonal Percent	Mean Speed mph	Seasonal Percent	Mean Speed mph
Winter	34.07	7.91	61.85	10.09
Spring	46.01	9.23	52.42	8.81
Summer	31.41	7.06	59.23	7.20
Fall	28.53	6.86	65.65	9.62
Annual	34.76	7.91	60.07	9.04

TABLE 2.3-9

MEAN SEASONAL VALUES OF PREVAILING HOURLY WIND DIRECTION AND SPEED

Hutchinson Island, Florida 50-foot level

Period of Record: March 1, 1971 - February 29, 1972

Season	Wind Direction (Sector)	Percent	Speed (mph)
Winter	ESE	12.09	8.56
	S	10.99	7.86
Spring	SW	9.33	10.01
	NW	8.43	9.46
Summer	SE	14.67	7.64
	E	12.02	7.26
Fall	E	13.28	9.89
	ENE	11.90	9.97
Annual	E	9.96	8.88
	SE	9.55	8.57

TABLE 2.3-9A
(Based on 11 yr. record for Patrick AFB, Fla.)

SURFACE WIND SPEED ENVELOPES 95 PERCENTILE*
FOR EASTERN TEST RANGE

Height Above Natural Grade		Quasi-Steady-State Wind		Peak Wind	
(m)	(ft)	(ms^{-1})	(knots)	(ms^{-1})	(knots)
3.0	10	7.2	14.0	10.1	19.6
9.1	30	9.0	17.4	12.6	24.4
18.3	60	10.3	20.0	14.4	28.0
30.5	100	11.4	22.2	16.0	31.1
61.0	200	13.1	25.5	18.4	35.7
91.4	300	14.2	27.6	19.9	38.6
121.9	400	15.1	29.3	21.1	41.0
152.4	500	15.7	30.6	22.0	42.8

*The 95 percentile winds are, in general, exceeded during heavy rain showers, thunderstorms in the area or over the site, squall lines, some frontal passages, strong pressure gradients, and hurricanes.

TABLE 2.3-9B
(Based on 11 yr. record for Patrick AFB, Fla.)
SURFACE WIND SPEED ENVELOPES 99 PERCENTILE*
FOR EASTERN TEST RANGE

Height Above Natural Grade		Quasi-Steady-State Wind		Peak Wind	
(m)	(ft)	(ms^{-1})	(knots)	(ms^{-1})	(knots)
3.0	10	9.5	18.4	13.3	25.8
9.1	30	11.8	22.9	16.5	32.1
18.3	60	13.5	26.3	18.9	36.8
30.5	100	15.0	29.2	21.0	40.9
61.0	200	17.2	33.5	24.1	46.9
91.4	300	18.7	36.3	26.1	50.8
121.9	400	19.8	38.5	27.7	53.9
152.4	500	20.7	40.2	29.0	56.3

*The 99 percentile winds are, in general, exceeded during thunderstorms over the site, squall lines, occasional frontal passages and hurricanes.

TABLE 2.3-10

MONTHLY DISTRIBUTION OF TEMPERATURE (F)

West Palm Beach Long Term, Florida 1939-1970 Abbreviated as WPBLT

West Palm Beach Short Term, Florida March 1971 - February 1972: Abbreviated as WPBST

Hutchinson Island March 1971 - February 1972: Abbreviated as H1

	Average Maximum			Average Minimum			Monthly Mean		
	HI	WPBST	WPBLT	HI	WPBST	WPBLT	HI	WPBST	WPBLT
January	77	80	75	66	64	57	71	72	66
February	72	77	76	58	57	57	65	67	67
March	73	78	79	58	54	61	66	66	70
April	78	83	83	65	61	66	71	72	74
May	81	88	86	70	70	69	75	79	78
June	85	90	89	73	72	72	79	81	81
July	86	90	91	75	74	74	80	82	83
August	87	91	91	75	76	75	81	83	83
September	84	88	89	75	74	74	80	81	82
October	82	87	85	72	71	70	77	79	78
November	76	81	80	67	65	63	72	73	72
December	76	80	76	69	67	59	73	73	68
Annual	80	84	83	69	66	66	74	75	75

Extremes	WPBLT	WPBST	H1
Maximum	101 July 1942	99 April	94 April
Minimum	29 Jan 1970	39 March	41 February

TABLE 2.3-11

MONTHLY DISTRIBUTIONS OF PRECIPITATION (inches)

West Palm Beach, Florida 1939 - 1970 (WPBLT)

West Palm Beach, Florida March 1, 1971 - February 29, 1972 (WPBST)

Hutchinson Island, Florida March 1, 1971 - February 29, 1972 (H1)

	WPBLT	WPBST	H1
January	2.54	2.42	3.59
February	2.53	2.55	1.45
March	3.26	0.55	0.79
April	3.51	1.27	1.29
May	5.14	5.04	2.88
June	8.15	9.03	4.54
July	6.42	5.13	5.64
August	7.04	4.38	1.92
September	9.75	6.24	3.97
October	8.60	5.13	6.93
November	2.41	10.77	1.37
December	2.34	1.87	3.18
Annual	61.69	54.38	37.55

Extremes	WPBLT	WPBST	HI
Maximum monthly	24.86-Sept 1960	10.77-Nov	6.93-Oct
Maximum 24-hour	15.23-April 1942	5.44-Nov	3.33-Oct
Maximum hourly	6.00-July 1964	2.25-Nov	2.84-Oct

TABLE 2.3-12

MONTHLY THUNDERSTORM ACTIVITY

West Palm Beach, Florida 1943-1971

	Average Number of Thunderstorm Days
January	1
February	1
March	2
April	4
May	7
June	13
July	16
August	16
September	11
October	5
November	1
December	1
Annual	78

TABLE 2.3-13

NUMBER OF DAYS OF HEAVY FOG (VISIBILITY 1/4 MILE)

Period of Record: 1943-1970

January	2
February	1
March	1
April	1
May	*
June	*
July	0
August	*
September	*
October	*
November	1
December	1
Annual	8

* Less than one-half day.

TABLE 2.3-14

HORIZONTAL STABILITY CLASS BY WIND VARIABILITY

Pasquill Class	Description	Range of Standard Deviation (σ_θ),Degrees		
A	Extremely Unstable	σ_θ	$\geq$	22.5°
B	Unstable	22.4	to	17.5
C	Slightly Unstable	17.4	to	12.5
D	Neutral	12.4	to	7.5
E	Slightly Stable	7.4	to	3.8
F	Stable	3.7	to	1.3
G	Extremely Stable	σ_θ	<	1.3

TABLE 2.3-14A

COMPARISON OF THE ST. LUCIE VERTICAL PASQUILL STABILITY
AS DERIVED BETWEEN THE 200 MINUS 110-FOOT AND THE 200 MINUS 10-FOOT LEVELS
(Period of March 1, 1971 to February 29,1972)

Pasquill Stability	200 Minus 110-foot Temperature Difference Method		200 Minus l0-foot Temperature Difference Method	
	Occurrences	Percent	Occurrences	Percent
A	1122	13.2	2024	23.1
B	279	3.3	253	2.8
C	467	5.5	518	5.9
D	4144	48.9	2130	24.3
E	2035	24.0	2924	33.3
F	339	4.0	731	8.3
G	90	1.1	184	2.0
	----	------	----	----
TOTAL	8476	100	8764	100
Invalid	308		18	
% Invalid	3.5%		0.2%	

TABLE, 2.3-15

VERTICAL STABILITY CLASS BY TEMPERATURE DIFFERENCE

Pasquill Class	Description	ΔT(°F)/190 feet
A	Extremely Unstable	≤ -2.0
B	Unstable	-1.9 to -1.8
C	Slightly Unstable	-1.7 to -1.4
D	Neutral	-1.3 to -0.5
E	Slightly Stable	-0.4 to 1.5
F	Stable	1.6 to 4.2
G	Extremely Stable	> 4.2

TABLE 2.3-16

8 HOUR χ/Q AT 5 miles

FLORIDA POWER & LIGHT COMPANY

CODE*LSD-4 St. Lucie Plant Site, Florida

HIGHEST CALCULATED AVERAGE RELATIVE CONCENTRATION
BY WIND DIRECTION AND CODED DATE FOR THE TIME PERIOD OF 8 HOURS
AT THE LOW POPULATION DISTANCE OF 5 MILES

PERIOD OF RECORD: 03/01/71 TO 02/29/72

WIND DIRECTION	LAST HOUR CODED DATE MONTH	DAY	HOUR	MAXIMUM X/Q (SEC/M**3)
NNE	2	29	02	2.021D-06
NE	3	22	13	2.292D-06
ENE	6	26	08	5.793D-06
E	5	18	09	3.518D-06
ESE	5	05	09	7.969D-06
SE	6	29	04	3.001D-06
SSE	8	18	05	3.241D-06
S	12	31	08	3.010D-06
SSW	3	10	06	4.105D-06
SW	11	27	24	4.280D-06
WSW	3	12	04	2.766D-06
W	11	20	09	2.284D-06
WNW	2	26	09	3.965D-06
NW	2	01	08	3.344D-06
NNW	3	09	03	3.947D-06
N	2	29	09	2.874D-06

TABLE 2.3-17
16 HOUR χ/Q AT 5 miles
FLORIDA POWER & LIGHT COMPANY
CODE*LSD-4 St. Lucie Plant Site, Florida

HIGHEST CALCULATED AVERAGE RELATIVE CONCENTRATION
BY WIND DIRECTION AND CODED DATE FOR THE TIME PERIOD OF 16 HOURS
AT THE LOW POPULATION DISTANCE OF 5 MILES

PERIOD OF RECORD: 03/01/71 TO 02/29/72

WIND DIRECTION	LAST HOUR CODED DATE MONTH	DAY	HOUR	MAXIMUM X/Q (SEC/M**3)
NNE	2	29	10	1.074D-06
NE	3	22	13	1.146D-06
ENE	6	26	11	3.059D-06
E	5	18	09	2.132D-06
ESE	5	05	15	4.262D-06
SE	6	29	18	1.519D-06
SSE	8	18	06	1.713D-06
S	12	31	10	1.561D-06
SSW	3	10	14	2.056D-06
SW	11	28	09	2.275D-06
WSW	6	11	09	1.413D-06
W	11	20	12	1.146D-06
WNW	2	26	09	1.982D-06
NW	10	12	07	2.460D-06
NNW	3	09	10	2.484D-06
N	2	29	09	1.437D-06

TABLE 2.3-18
72 HOUR χ/Q AT 5 miles
FLORIDA POWER & LIGHT COMPANY
CODE*LSD-4 St. Lucie Plant Site, Florida

HIGHEST CALCULATED AVERAGE RELATIVE CONCENTRATION
BY WIND DIRECTION AND CODED DATE FOR THE TIME PERIOD OF 72 HOURS
AT THE LOW POPULATION DISTANCE OF 5 MILES

PERIOD OF RECORD: 03/01/71 TO 02/29/72

WIND DIRECTION	LAST HOUR CODED DATE MONTH	DAY	HOUR	MAXIMUM X/Q (SEC/M**3)
NNE	9	30	03	3.030D-07
NE	4	13	05	4.529D-07
ENE	6	27	09	6.837D-07
E	5	19	24	6.781D-07
ESE	5	05	22	9.630D-07
SE	7	01	21	6.787D-07
SSE	7	26	01	6.219D-07
S	1	12	11	7.025D-07
SSW	3	11	04	5.963D-07
SW	6	25	14	7.514D-07
WSW	1	31	02	5.992D-07
W	6	12	04	4.679D-07
WNW	10	23	07	6.140D-07
NW	2	23	07	6.840D-07
NNW	3	09	22	6.751D-07
N	2	29	09	4.348D-07

TABLE 2.3-19
26 DAY χ/Q AT 5 miles

FLORIDA POWER & LIGHT COMPANY
CODE*LSD-4 St. Lucie Plant Site, Florida

HIGHEST CALCULATED AVERAGE RELATIVE CONCENTRATION
BY WIND DIRECTION AND CODED DATE FOR THE TIME PERIOD OF 624 HOURS
AT THE LOW POPULATION DISTANCE OF 5 MILES

PERIOD OF RECORD: 03/01/71 TO 02/29/72

WIND DIRECTION	LAST HOUR CODED DATE MONTH - DAY - HOUR			MAXIMUM X/Q (SEC/M**3)
NNE	7	29	02	1.045D-07
NE	8	03	08	1.278D-07
ENE	6	26	11	1.753D-07
E	6	09	08	2.259D-07
ESE	5	28	08	2.461D-07
SE	7	24	23	2.356D-07
SSE	8	06	05	2.522D-07
S	1	25	11	3.682D-07
SSW	3	26	24	1.853D-07
SW	7	18	08	2.434D-07
WSW	6	30	03	1.849D-07
W	6	22	17	1.636D-07
WNW	2	26	09	1.931D-07
NW	2	26	09	2.548D-07
NNW	2	28	01	2.022D-07
N	2	29	09	1.443D-07

JOINT FREQUENCY DISTRIBUTIONS OF STABILITY
CLASSIFICATIONS, WIND SPEED AND WIND DIRECTIONS

TABLE 2.3-20

CODE#LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 1
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL
STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = A
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = A
PASQUILL A-A

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.01	10.05
ENE	3	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
E	4	0.02	0.02	0.00	0.00	0.00	0.00	0.00	0.05	3.90
ESE	5	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
SE	6	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.02	5.55
SSE	7	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.02	5.55
S	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSW	9	0.01	0.01	0.00	0.00	0.00	0.00	0.00	0.02	4.70
SW	10	0.01	0.02	0.01	0.00	0.00	0.00	0.00	0.05	5.85
WSW	11	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.01	10.05
W	12	0.00	0.01	0.02	0.00	0.00	0.00	0.00	0.03	8.55
WNW	13	0.03	0.02	0.01	0.00	0.00	0.00	0.00	0.07	4.65
NW	14	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CALM AND VARIABLE	0								0.07	0.99
TOTAL		0.09	0.14	0.07	0.00	0.00	0.00	0.00	0.39	4.66

NUMBER OF VALID CATEGORY OBSERVATIONS = 34

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-21

CODE#LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 2
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL
STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = A
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = B
PASQUILL A-B

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.01	10.05
NE	2	0.00	0.02	0.05	0.01	0.00	0.00	0.00	0.08	9.53
ENE	3	0.00	0.01	0.06	0.00	0.00	0.00	0.00	0.07	9.30
E	4	0.00	0.03	0.00	0.00	0.00	0.00	0.00	0.03	5.55
ESE	5	0.01	0.06	0.02	0.00	0.00	0.00	0.00	0.09	6.26
SE	6	0.00	0.03	0.05	0.00	0.00	0.00	0.00	0.07	8.12
SSE	7	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.01	10.05
S	8	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.02	5.55
SSW	9	0.00	0.03	0.00	0.00	0.00	0.00	0.00	0.03	5.55
SW	10	0.00	0.01	0.01	0.00	0.00	0.00	0.00	0.02	7.80
WSW	11	0.00	0.03	0.00	0.00	0.00	0.00	0.00	0.03	5.55
W	12	0.00	0.08	0.05	0.02	0.01	0.00	0.00	0.16	9.41
WNW	13	0.00	0.04	0.06	0.03	0.02	0.00	0.00	0.17	10.52
NW	14	0.00	0.05	0.13	0.05	0.00	0.00	0.00	0.22	10.26
NNW	15	0.00	0.02	0.02	0.02	0.00	0.00	0.00	0.07	10.36
N	16	0.00	0.02	0.02	0.00	0.00	0.00	0.00	0.14	12.11
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.01	0.31	0.48	0.14	0.03	0.00	0.00	1.23	9.29

NUMBER OF VALID CATEGORY OBSERVATIONS = 108

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-22

CODE#LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 3
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL
STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = A
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = C
PASQUILL A-C

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.16	0.46	0.06	0.00	0.00	0.00	0.67	9.45
NE	2	0.00	0.57	0.24	0.00	0.00	0.00	0.00	0.81	6.88
ENE	3	0.01	0.55	0.43	0.06	0.00	0.00	0.00	1.05	7.92
E	4	0.01	0.40	0.23	0.03	0.01	0.00	0.00	0.68	7.76
ESE	5	0.00	0.68	0.40	0.05	0.00	0.00	0.00	1.13	7.54
SE	6	0.00	0.36	0.57	0.09	0.00	0.00	0.00	1.04	8.90
SSE	7	0.00	0.05	0.09	0.06	0.00	0.00	0.00	0.19	10.01
S	8	0.00	0.00	0.06	0.01	0.00	0.00	0.00	0.07	10.97
SSW	9	0.00	0.08	0.03	0.01	0.00	0.00	0.00	0.13	7.69
SW	10	0.00	0.03	0.15	0.05	0.03	0.00	0.00	0.30	12.34
WSW	11	0.00	0.10	0.24	0.08	0.02	0.00	0.00	0.45	10.59
W	12	0.00	0.09	0.07	0.03	0.01	0.00	0.00	0.21	9.61
WNW	13	0.01	0.08	0.15	0.02	0.01	0.00	0.00	0.27	9.35
NW	14	0.00	0.07	0.17	0.07	0.03	0.00	0.00	0.34	10.29
NNW	15	0.01	0.10	0.21	0.05	0.00	0.00	0.00	0.37	9.23
N	16	0.00	0.09	0.36	0.37	0.00	0.00	0.00	0.85	11.97
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.05	3.46	3.89	1.08	0.09	0.00	0.00	8.57	9.01

NUMBER OF VALID CATEGORY OBSERVATIONS = 751

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-23

CODE#LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 4
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL
STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = A
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = D
PASQUILL A-D

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.17	0.27	0.03	0.00	0.00	0.00	0.48	8.34
NE	2	0.02	0.42	0.15	0.00	0.00	0.00	0.00	0.59	6.75
ENE	3	0.06	0.83	0.16	0.00	0.00	0.00	0.00	1.05	6.00
E	4	0.01	1.97	0.86	0.11	0.00	0.00	0.00	2.94	7.23
ESE	5	0.00	0.90	0.60	0.01	0.00	0.00	0.00	1.52	7.42
SE	6	0.01	0.79	1.77	0.13	0.00	0.00	0.00	2.69	8.96
SSE	7	0.01	0.11	0.37	0.22	0.00	0.00	0.00	0.71	10.36
S	8	0.02	0.05	0.13	0.06	0.00	0.00	0.00	0.25	9.77
SSW	9	0.00	0.07	0.11	0.01	0.00	0.00	0.00	0.17	8.29
SW	10	0.02	0.10	0.17	0.07	0.01	0.00	0.00	0.35	9.70
WSW	11	0.00	0.05	0.03	0.00	0.00	0.00	0.00	0.09	7.48
W	12	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.02	5.55
WNW	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NW	14	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.02	5.55
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.00	0.05	0.34	0.11	0.00	0.00	0.00	0.50	10.89
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.16	3.56	4.96	0.75	0.01	0.00	0.00	11.44	9.13

NUMBER OF VALID CATEGORY OBSERVATIONS = 1003

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-24

CODE#LSO-2 FLORIDA POWER & LIGHT COMPANY PAGE 5
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = A
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = E
PASQUILL A-E

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.02	5.55
ENE	3	0.00	0.05	0.00	0.00	0.00	0.00	0.00	0.05	5.55
E	4	0.00	0.10	0.01	0.00	0.00	0.00	0.00	0.19	5.01
ESE	5	0.00	0.05	0.07	0.00	0.00	0.00	0.00	0.11	9.25
SE	6	0.00	0.06	0.33	0.06	0.00	0.00	0.00	0.45	10.18
SSE	7	0.00	0.01	0.03	0.03	0.00	0.00	0.00	0.09	11.76
S	8	0.00	0.06	0.06	0.06	0.00	0.00	0.00	0.17	10.38
SSW	9	0.01	0.02	0.07	0.03	0.00	0.00	0.00	0.14	10.03
SW	10	0.00	0.10	0.08	0.01	0.00	0.00	0.00	0.19	7.99
WSW	11	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
W	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WNW	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NW	14	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.02	0.50	0.65	0.19	0.00	0.00	0.00	1.43	8.91

NUMBER OF VALID CATEGORY OBSERVATIONS = 125

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-25

CODE#LSO-2 FLORIDA POWER & LIGHT COMPANY PAGE 6
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = A
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = F
PASQUILL A-F

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENE	3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
E	4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ESE	5	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.01	10.05
SE	6	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.01	[illegible]
SSE	7	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
S	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSW	9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SW	10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WSW	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
W	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WNW	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NW	14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.00	0.01	0.01	0.01	0.00	0.00	0.00	0.03	10.34

NUMBER OF VALID CATEGORY OBSERVATIONS = 3

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-26

CODE#LSO-2 FLORIDA POWER & LIGHT COMPANY PAGE 7
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = A
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = G
PASQUILL A-G

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENE	3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
E	4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ESE	5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SE	6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSE	7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
S	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSW	9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SW	10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WSW	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
W	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WNW	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NW	14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

NUMBER OF VALID CATEGORY OBSERVATIONS = 0

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-27

CODE#LSO-2 FLORIDA POWER & LIGHT COMPANY PAGE 8
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = B
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = A
PASQUILL B-A

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
NE	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENE	3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
E	4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ESE	5	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
SE	6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSE	7	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
S	8	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.02	5.55
SSW	9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SW	10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WSW	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
W	12	0.01	0.00	0.01	0.00	0.00	0.00	0.00	0.02	6.15
WNW	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NW	14	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.01	10.05
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.01	0.06	0.02	0.00	0.00	0.00	0.00	0.09	6.26

NUMBER OF VALID CATEGORY OBSERVATIONS = 8

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-28

CODE=LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 9
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = B
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = B
PASQUILL B-B

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.02	0.01	0.00	0.00	0.00	0.00	0.03	7.05
ENE	3	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.01	15.55
E	4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ESE	5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SE	6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSE	7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
S	8	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
SSW	9	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
SW	10	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
WSW	11	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
W	12	0.00	0.02	0.01	0.00	0.00	0.00	0.00	0.03	7.05
WNW	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NW	14	0.00	0.03	0.00	0.00	0.00	0.00	0.00	0.03	5.55
NNW	15	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.01	10.05
N	16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CALM AND VARIABLE	0								0.05	0.11
TOTAL		0.02	0.10	0.03	0.01	0.00	0.00	0.00	0.22	5.38

NUMBER OF VALID CATEGORY OBSERVATIONS = 19

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-29

CODE=LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 10
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = B
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = C
PASQUILL B-C

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.07	0.02	0.02	0.00	0.00	0.00	0.11	9.43
NE	2	0.00	0.03	0.05	0.00	0.00	0.00	0.00	0.08	8.12
ENE	3	0.00	0.07	0.02	0.02	0.00	0.00	0.00	0.14	7.97
E	4	0.01	0.06	0.03	0.02	0.00	0.00	0.00	0.13	8.30
ESE	5	0.01	0.05	0.01	0.01	0.00	0.00	0.00	0.08	7.15
SE	6	0.00	0.00	0.01	0.02	0.00	0.00	0.00	0.03	13.72
SSE	7	0.00	0.00	0.02	0.00	0.00	0.00	0.00	0.02	10.05
S	8	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.02	5.55
SSW	9	0.01	0.00	0.00	0.01	0.00	0.00	0.00	0.02	8.90
SW	10	0.00	0.06	0.02	0.02	0.00	0.00	0.00	0.10	8.77
WSW	11	0.00	0.06	0.02	0.02	0.01	0.00	0.00	0.11	10.05
W	12	0.00	0.01	0.02	0.00	0.00	0.00	0.00	0.03	8.55
WNW	13	0.00	0.02	0.00	0.01	0.00	0.00	0.00	0.03	8.88
NW	14	0.00	0.02	0.01	0.01	0.00	0.00	0.00	0.05	9.17
NNW	15	0.00	0.02	0.03	0.05	0.00	0.00	0.00	0.10	11.49
N	16	0.00	0.02	0.08	0.10	0.03	0.00	0.00	0.24	13.62
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.03	0.54	0.37	0.33	0.05	0.00	0.00	1.31	9.79

NUMBER OF VALID CATEGORY OBSERVATIONS = 115

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-30

CODE=LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 11
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = B
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = D
PASQUILL B-D

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.02	0.03	0.01	0.00	0.00	0.00	0.07	9.47
NE	2	0.00	0.09	0.01	0.00	0.00	0.00	0.00	0.10	6.05
ENE	3	0.03	0.07	0.02	0.00	0.00	0.00	0.00	0.13	5.47
E	4	0.00	0.09	0.02	0.00	0.00	0.00	0.00	0.11	6.45
ESE	5	0.00	0.06	0.02	0.00	0.00	0.00	0.00	0.08	6.84
SE	6	0.00	0.08	0.05	0.01	0.03	0.00	0.00	0.14	7.88
SSE	7	0.00	0.03	0.07	0.05	0.00	0.00	0.00	0.15	10.70
S	8	0.01	0.03	0.03	0.00	0.00	0.00	0.00	0.08	7.01
SSW	9	0.01	0.03	0.01	0.00	0.00	0.00	0.00	0.06	5.79
SW	10	0.00	0.01	0.06	0.02	0.03	0.00	0.00	0.13	13.70
WSW	11	0.00	0.01	0.01	0.00	0.00	0.00	0.00	0.02	7.80
W	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WNW	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NW	14	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.00	0.00	0.02	0.00	0.00	0.00	0.00	0.02	10.05
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.06	0.53	0.37	0.09	0.03	0.00	0.00	1.10	8.21

NUMBER OF VALID CATEGORY OBSERVATIONS = 96

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-31

CODE=LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 12
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = B
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = E
PASQUILL B-E

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENE	3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
E	4	0.00	0.06	0.00	0.00	0.00	0.00	0.00	0.06	5.55
ESE	5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SE	6	0.01	0.02	0.00	0.00	0.00	0.00	0.00	0.03	4.72
SSE	7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
S	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSW	9	0.00	0.01	0.03	0.00	0.00	0.00	0.00	0.04	8.92
SW	10	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
WSW	11	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
W	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WNW	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NW	14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.01	0.13	0.03	0.00	0.00	0.00	0.00	0.17	6.23

NUMBER OF VALID CATEGORY OBSERVATIONS = 15

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-32

CODE*LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 13
HUTCHINSON ISLAND SITE
PERIOD OF RECORD 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = B
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = F
PASQUILL B-F

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENE	3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
E	4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ESE	5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SE	6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSE	7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
S	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSW	9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SW	10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WSW	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
W	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WNW	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NW	14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

NUMBER OF VALID CATEGORY OBSERVATIONS = 0

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-33

CODE*LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 14
HUTCHINSON ISLAND SITE
PERIOD OF RECORD 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = B
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = G
PASQUILL B-G

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENE	3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
E	4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ESE	5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SE	6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSE	7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
S	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSW	9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SW	10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WSW	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
W	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WNW	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NW	14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

NUMBER OF VALID CATEGORY OBSERVATIONS = 0

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-34

CODE*LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 15
HUTCHINSON ISLAND SITE
PERIOD OF RECORD 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = C
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = A
PASQUILL C-A

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENE	3	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
E	4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ESE	5	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.01	10.05
SE	6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSE	7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
S	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSW	9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SW	10	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.01	10.05
WSW	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
W	12	0.01	0.01	0.01	0.00	0.00	0.00	0.00	0.03	5.75
WNW	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NW	14	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.00	0.01	0.01	0.00	0.00	0.00	0.00	0.02	7.80
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.02	0.03	0.05	0.00	0.00	0.00	0.00	0.10	6.82

NUMBER OF VALID CATEGORY OBSERVATIONS = 9

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-35

CODE*LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 16
HUTCHINSON ISLAND SITE
PERIOD OF RECORD 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = C
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = B
PASQUILL C-B

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
NE	2	0.00	0.01	0.01	0.00	0.00	0.00	0.00	0.02	7.80
ENE	3	0.00	0.01	0.00	0.00	0.02	0.00	0.00	0.03	10.22
E	4	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
ESE	5	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
SE	6	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.01	15.55
SSE	7	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.01	10.05
S	8	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.01	10.05
SSW	9	0.00	0.02	0.01	0.00	0.00	0.00	0.00	0.03	7.05
SW	10	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.02	5.55
WSW	11	0.01	0.00	0.01	0.00	0.00	0.00	0.00	0.02	6.15
W	12	0.00	0.05	0.01	0.00	0.00	0.00	0.00	0.06	5.45
WNW	13	0.00	0.05	0.01	0.01	0.00	0.00	0.00	0.07	7.97
NW	14	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
NNW	15	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.02	5.55
N	16	0.00	0.02	0.01	0.00	0.00	0.00	0.00	0.03	7.05
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.02	0.24	0.09	0.02	0.02	0.00	0.00	0.40	7.88

NUMBER OF VALID CATEGORY OBSERVATIONS = 35

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-36

CODE#LSD-2 — FLORIDA POWER & LIGHT COMPANY — PAGE 17
HUTCHINSON ISLAND SITE
PERIOD OF RECORD 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = C
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = C
PASQUILL C-C

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.07	0.07	0.03	0.02	0.00	0.00	0.17	10.79
NE	2	0.00	0.13	0.11	0.03	0.00	0.00	0.00	0.27	8.67
ENE	3	0.05	0.05	0.08	0.01	0.00	0.00	0.00	0.18	7.32
E	4	0.00	0.03	0.02	0.07	0.00	0.00	0.00	0.13	11.82
ESE	5	0.00	0.07	0.07	0.00	0.00	0.00	0.00	0.14	7.48
SE	6	0.00	0.06	0.02	0.01	0.00	0.00	0.00	0.09	7.93
SSE	7	0.01	0.05	0.00	0.02	0.00	0.00	0.00	0.08	7.94
S	8	0.01	0.02	0.02	0.00	0.00	0.00	0.00	0.05	6.97
SSW	9	0.00	0.01	0.01	0.01	0.00	0.00	0.00	0.03	10.30
SW	10	0.01	0.06	0.08	0.03	0.00	0.00	0.00	0.18	9.19
WSW	11	0.00	0.06	0.02	0.01	0.01	0.00	0.00	0.10	7.44
W	12	0.00	0.05	0.00	0.01	0.00	0.00	0.00	0.06	7.55
WNW	13	0.01	0.10	0.03	0.00	0.00	0.00	0.00	0.15	6.33
NW	14	0.02	0.05	0.03	0.01	0.00	0.00	0.00	0.11	7.24
NNW	15	0.02	0.03	0.10	0.00	0.00	0.00	0.00	0.14	7.97
N	16	0.00	0.03	0.05	0.06	0.02	0.00	0.00	0.16	12.69
CALM AND VARIABLE	0								0.21	0.04
TOTAL		0.14	0.87	0.73	0.32	0.05	0.00	0.00	2.32	8.10

NUMBER OF VALID CATEGORY OBSERVATIONS = 203

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-37

CODE#LSD-2 — FLORIDA POWER & LIGHT COMPANY — PAGE 18
HUTCHINSON ISLAND SITE
PERIOD OF RECORD 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = C
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = D
PASQUILL C-D

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.05	0.05	0.01	0.00	0.00	0.00	0.10	[illegible]
NE	2	0.01	0.03	0.01	0.00	0.00	0.00	0.00	0.05	5.72
ENE	3	0.03	0.06	0.02	0.00	0.01	0.00	0.00	0.13	6.92
E	4	0.01	0.14	0.07	0.01	0.00	0.00	0.00	0.23	7.23
ESE	5	0.00	0.18	0.06	0.00	0.00	0.00	0.00	0.24	6.62
SE	6	0.00	0.14	0.09	0.01	0.00	0.00	0.00	0.24	7.74
SSE	7	0.00	0.09	0.16	0.09	0.00	0.00	0.00	0.34	10.32
S	8	0.02	0.02	0.15	0.01	0.00	0.00	0.00	0.22	9.04
SSW	9	0.00	0.06	0.07	0.10	0.01	0.00	0.00	0.24	11.53
SW	10	0.00	0.17	0.13	0.02	0.01	0.00	0.00	0.33	8.53
WSW	11	0.01	0.05	0.02	0.00	0.00	0.00	0.00	0.08	[illegible]
W	12	0.01	0.02	0.00	0.00	0.00	0.00	0.00	0.03	4.45
WNW	13	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.75
NW	14	0.00	0.03	0.01	0.00	0.00	0.00	0.00	0.05	6.45
NNW	15	0.00	0.01	0.01	0.00	0.00	0.00	0.00	0.02	7.80
N	16	0.00	0.03	0.07	0.04	0.00	0.00	0.00	0.13	9.32
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.11	1.12	0.92	0.29	0.03	0.00	0.00	2.4[illegible]	8.45

NUMBER OF VALID CATEGORY OBSERVATIONS = 217

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-38

CODE#LSD-2 — FLORIDA POWER & LIGHT COMPANY — PAGE 19
HUTCHINSON ISLAND SITE
PERIOD OF RECORD 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = C
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = E
PASQUILL C-E

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
ENE	3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
E	4	0.00	0.06	0.00	0.00	0.00	0.00	0.00	0.06	5.55
ESE	5	0.00	0.01	0.02	0.00	0.00	0.00	0.00	0.03	8.55
SE	6	0.00	0.02	0.01	0.00	0.00	0.00	0.00	0.03	7.05
SSE	7	0.00	0.02	0.01	0.00	0.00	0.00	0.00	0.03	7.05
S	8	0.00	0.02	0.08	0.01	0.00	0.00	0.00	0.11	9.70
SSW	9	0.01	0.08	0.06	0.02	0.00	0.00	0.00	0.17	8.16
SW	10	0.01	0.07	0.05	0.00	0.00	0.00	0.00	0.13	6.89
WSW	11	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
W	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WNW	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NW	14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.02	0.31	0.23	0.03	0.00	0.00	0.00	0.59	7.73

NUMBER OF VALID CATEGORY OBSERVATIONS = 52

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-39

CODE#LSD-2 — FLORIDA POWER & LIGHT COMPANY — PAGE 20
HUTCHINSON ISLAND SITE
PERIOD OF RECORD 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = C
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = F
PASQUILL C-F

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENE	3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
E	4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ESE	5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SE	6	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
SSE	7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
S	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSW	9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SW	10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WSW	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
W	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WNW	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NW	14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55

NUMBER OF VALID CATEGORY OBSERVATIONS = 1

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-40

CODE#LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 21
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = C
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = G
PASQUILL C-G

SECTOR	SECTOR NUMBER	1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
		SPEED(MPH) ADJUSTED TO 10-METER ELEVATION								
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENE	3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
E	4	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
ESE	5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SE	6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSE	7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
S	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSW	9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SW	10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WSW	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
W	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WNW	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NW	14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55

NUMBER OF VALID CATEGORY OBSERVATIONS = 1

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-41

CODE#LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 22
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = D
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = A
PASQUILL D-A

SECTOR	SECTOR NUMBER	1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
		SPEED(MPH) ADJUSTED TO 10-METER ELEVATION								
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.02	5.55
ENE	3	0.00	0.01	0.00	0.00	0.01	0.00	0.00	0.02	13.55
E	4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ESE	5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SE	6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSE	7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
S	8	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.02	5.55
SSW	9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SW	10	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.02	5.55
WSW	11	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
W	12	0.00	0.01	0.01	0.00	0.00	0.00	0.00	0.02	7.50
WNW	13	0.00	0.02	0.01	0.00	0.00	0.00	0.00	0.03	7.05
NW	14	0.01	0.02	0.00	0.00	0.00	0.00	0.00	0.03	4.45
NNW	15	0.00	0.01	0.00	0.01	0.00	0.00	0.00	0.02	10.55
N	16	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.01	0.17	0.02	0.01	0.01	0.00	0.00	0.23	7.13

NUMBER OF VALID CATEGORY OBSERVATIONS = 20

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-42

CODE#LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 23
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = D
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = B
PASQUILL D-B

SECTOR	SECTOR NUMBER	1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
		SPEED(MPH) ADJUSTED TO 10-METER ELEVATION								
NNE	1	0.00	0.02	0.05	0.01	0.01	0.00	0.00	0.09	11.05
NE	2	0.00	0.00	0.05	0.05	0.07	0.01	0.00	0.18	17.21
ENE	3	0.01	0.05	0.02	0.06	0.11	0.00	0.00	0.25	15.35
E	4	0.00	0.02	0.00	0.05	0.03	0.00	0.00	0.10	15.33
ESE	5	0.00	0.03	0.00	0.01	0.00	0.00	0.00	0.05	8.05
SE	6	0.00	0.01	0.02	0.00	0.00	0.00	0.00	0.03	8.55
SSE	7	0.00	0.01	0.02	0.00	0.00	0.00	0.00	0.03	8.55
S	8	0.00	0.06	0.00	0.00	0.00	0.00	0.00	0.06	5.55
SSW	9	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
SW	10	0.00	0.06	0.00	0.00	0.00	0.00	0.00	0.02	5.55
WSW	11	0.01	0.01	0.01	0.00	0.00	0.00	0.00	0.03	5.95
W	12	0.00	0.01	0.03	0.01	0.00	0.00	0.00	0.06	10.25
WNW	13	0.00	0.05	0.00	0.02	0.00	0.00	0.00	0.07	8.88
NW	14	0.00	0.03	0.02	0.01	0.00	0.00	0.00	0.07	8.72
NNW	15	0.00	0.02	0.00	0.01	0.00	0.00	0.00	0.03	8.88
N	16	0.00	0.01	0.03	0.02	0.00	0.00	0.00	0.07	11.13
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.02	0.30	0.26	0.26	0.23	0.01	0.00	1.16	12.11

NUMBER OF VALID CATEGORY OBSERVATIONS = 102

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-43

CODE#LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 24
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = D
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = C
PASQUILL D-C

SECTOR	SECTOR NUMBER	1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
		SPEED(MPH) ADJUSTED TO 10-METER ELEVATION								
NNE	1	0.00	0.03	0.16	0.21	0.07	0.00	0.00	0.47	13.92
NE	2	0.01	0.13	0.33	0.41	0.11	0.00	0.00	0.99	12.99
ENE	3	0.01	0.14	0.48	0.39	0.05	0.00	0.00	1.06	11.59
E	4	0.00	0.22	0.39	0.30	0.03	0.00	0.00	0.94	11.17
ESE	5	0.00	0.16	0.26	0.11	0.01	0.00	0.00	0.55	10.12
SE	6	0.00	0.06	0.08	0.10	0.00	0.00	0.00	0.26	10.53
SSE	7	0.00	0.10	0.09	0.03	0.00	0.00	0.00	0.23	8.35
S	8	0.00	0.07	0.05	0.00	0.00	0.00	0.00	0.11	7.25
SSW	9	0.00	0.05	0.01	0.01	0.00	0.00	0.00	0.07	7.37
SW	10	0.02	0.11	0.03	0.01	0.00	0.00	0.00	0.18	6.61
WSW	11	0.00	0.13	0.10	0.03	0.00	0.00	0.00	0.26	8.62
W	12	0.00	0.10	0.07	0.02	0.00	0.00	0.00	0.19	9.31
WNW	13	0.00	0.19	0.10	0.16	0.00	0.00	0.00	0.49	10.32
NW	14	0.01	0.27	0.21	0.13	0.00	0.00	0.00	0.62	9.53
NNW	15	0.00	0.10	0.21	0.21	0.00	0.00	0.00	0.51	11.35
N	16	0.00	0.13	0.26	0.41	0.03	0.00	0.00	0.83	12.58
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.06	2.01	2.83	2.56	0.31	0.00	0.00	7.76	11.10

NUMBER OF VALID CATEGORY OBSERVATIONS = 680

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-44

CODE=LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 25
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = D
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = D
PASQUILL D-D

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.03	0.17	0.24	0.03	0.00	0.00	0.00	0.48	8.28
NE	2	0.01	0.11	0.10	0.00	0.00	0.00	0.00	0.23	7.41
ENE	3	0.05	0.24	0.26	0.00	0.00	0.00	0.00	0.55	7.43
E	4	0.03	0.57	0.55	0.13	0.00	0.00	0.00	1.28	8.37
ESE	5	0.03	0.56	0.25	0.02	0.00	0.00	0.00	0.87	6.99
SE	6	0.06	0.43	0.23	0.01	0.00	0.00	0.00	0.73	6.85
SSE	7	0.00	0.31	0.46	0.15	0.03	0.00	0.00	0.91	9.42
S	8	0.01	0.32	0.35	0.14	0.00	0.00	0.00	0.82	9.11
SSW	9	0.01	0.30	0.23	0.14	0.01	0.00	0.00	0.68	9.26
SW	10	0.06	0.60	0.38	0.13	0.03	0.00	0.00	1.21	8.44
WSW	11	0.02	0.29	0.23	0.00	0.00	0.00	0.00	0.54	7.32
W	12	0.07	0.13	0.03	0.00	0.00	0.00	0.00	0.23	5.30
WNW	13	0.07	0.14	0.01	0.00	0.00	0.00	0.00	0.22	4.74
NW	14	0.05	0.24	0.11	0.00	0.00	0.00	0.00	0.40	6.46
NNW	15	0.05	0.14	0.03	0.00	0.00	0.00	0.00	0.23	5.79
N	16	0.02	0.11	0.26	0.03	0.00	0.00	0.00	0.43	8.89
CALM AND VARIABLE	0								0.41	0.02
TOTAL		0.57	4.71	3.74	0.78	0.06	0.00	0.00	10.27	7.65

NUMBER OF VALID CATEGORY OBSERVATIONS = 900

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-45

CODE=LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 26
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = D
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = E
PASQUILL D-E

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.01	10.05
ENE	3	0.00	0.02	0.01	0.00	0.00	0.00	0.00	0.03	7.05
E	4	0.03	0.08	0.07	0.00	0.00	0.00	0.00	0.18	6.62
ESE	5	0.01	0.02	0.02	0.00	0.00	0.00	0.00	0.05	6.69
SE	6	0.01	0.05	0.06	0.01	0.00	0.00	0.00	0.13	8.20
SSE	7	0.01	0.13	0.24	0.05	0.00	0.00	0.00	0.43	9.27
S	8	0.00	0.56	0.97	0.11	0.00	0.00	0.00	1.64	8.90
SSW	9	0.03	0.72	0.59	0.22	0.00	0.00	0.00	1.56	8.57
SW	10	0.01	0.33	0.14	0.03	0.00	0.00	0.00	0.51	7.34
WSW	11	0.00	0.03	0.00	0.00	0.00	0.00	0.00	0.03	5.55
W	12	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
WNW	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NW	14	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
NNW	15	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
N	16	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.16	1.95	2.11	0.43	0.00	0.00	0.00	4.66	8.41

NUMBER OF VALID CATEGORY OBSERVATIONS = 408

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-46

CODE=LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 27
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = D
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = F
PASQUILL D-F

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENE	3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
E	4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ESE	5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SE	6	0.00	0.03	0.00	0.00	0.00	0.00	0.00	0.03	5.55
SSE	7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
S	8	0.00	0.03	0.01	0.00	0.00	0.00	0.00	0.05	6.67
SSW	9	0.00	0.08	0.01	0.00	0.00	0.00	0.00	0.09	6.11
SW	10	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.02	5.55
WSW	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
W	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WNW	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NW	14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.00	0.18	0.02	0.00	0.00	0.00	0.00	0.21	6.05

NUMBER OF VALID CATEGORY OBSERVATIONS = 18

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-47

CODE=LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 28
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = D
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = G
PASQUILL D-G

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENE	3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
E	4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ESE	5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SE	6	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
SSE	7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
S	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSW	9	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
SW	10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WSW	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
W	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WNW	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NW	14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.02	5.55

NUMBER OF VALID CATEGORY OBSERVATIONS = 2

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-48

CDOE#LSD-2 FLORIDA POWER & LIGHT COMPANY
HUTCHINSON ISLAND SITE
PERIOD OF RECORD 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = F
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = A
PASQUILL E-A

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.01	0.02	0.00	0.00	0.00	0.00	0.00	0.03	4.45
ENE	3	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
E	4	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
ESE	5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SE	6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSE	7	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.02	5.55
S	8	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.02	5.55
SSW	9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SW	10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WSW	11	0.00	0.01	0.00	0.01	0.00	0.00	0.00	0.02	10.55
W	12	0.01	0.01	0.01	0.00	0.00	0.00	0.00	0.03	5.95
WNW	13	0.00	0.02	0.01	0.00	0.00	0.00	0.00	0.03	7.05
NW	14	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.01	10.05
NNW	15	0.02	0.01	0.01	0.00	0.00	0.00	0.00	0.05	5.02
N	16	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.08	0.14	0.05	0.01	0.00	0.00	0.00	0.27	5.75

NUMBER OF VALID CATEGORY OBSERVATIONS = 24

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-49

CDOE#LSD-2 FLORIDA POWER & LIGHT COMPANY
HUTCHINSON ISLAND SITE
PERIOD OF RECORD 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = E
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = B
PASQUILL E-B

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.02	0.01	0.06	0.01	0.00	0.00	0.00	0.10	7.43
NE	2	0.01	0.03	0.08	0.00	0.05	0.00	0.00	0.17	11.70
ENE	3	0.00	0.06	0.06	0.01	0.00	0.00	0.00	0.11	3.59
E	4	0.01	0.01	0.05	0.02	0.00	0.00	0.00	0.09	9.63
ESE	5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SE	6	0.01	0.03	0.00	0.01	0.00	0.00	0.00	0.04	6.89
SSE	7	0.00	0.05	0.00	0.00	0.00	0.00	0.00	0.05	5.55
S	8	0.01	0.03	0.00	0.00	0.00	0.00	0.00	0.05	4.77
SSW	9	0.01	0.01	0.01	0.00	0.00	0.00	0.00	0.03	5.95
SW	10	0.02	0.01	0.00	0.00	0.00	0.00	0.00	0.03	3.35
WSW	11	0.01	0.02	0.01	0.00	0.00	0.00	0.00	0.04	5.05
W	12	0.01	0.03	0.02	0.00	0.00	0.00	0.00	0.03	5.36
WNW	13	0.01	0.01	0.02	0.00	0.00	0.00	0.00	0.05	6.97
NW	14	0.00	0.06	0.01	0.00	0.00	0.00	0.00	0.07	4.30
NNW	15	0.01	0.01	0.03	0.00	0.00	0.00	0.00	0.04	7.59
N	16	0.00	0.02	0.01	0.00	0.00	0.00	0.00	0.03	7.05
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.15	0.42	0.37	0.06	0.05	0.00	0.00	1.04	7.91

NUMBER OF VALID CATEGORY OBSERVATIONS = 91

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-50

CDOE#LSD-2 FLORIDA POWER & LIGHT COMPANY
HUTCHINSON ISLAND SITE
PERIOD OF RECORD 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = E
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = C
PASQUILL E-C

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.02	0.06	0.19	0.01	0.00	0.00	0.00	0.29	8.75
NE	2	0.03	0.24	0.29	0.03	0.01	0.00	0.00	0.60	8.35
ENE	3	0.01	0.29	0.48	0.13	0.00	0.00	0.00	0.90	9.29
E	4	0.02	0.19	0.33	0.11	0.00	0.00	0.00	0.66	9.41
ESE	5	0.01	0.51	0.58	0.09	0.00	0.00	0.00	1.20	8.47
SE	6	0.02	0.15	0.18	0.11	0.00	0.00	0.00	0.47	9.58
SSE	7	0.00	0.07	0.06	0.00	0.00	0.00	0.00	0.13	7.60
S	8	0.02	0.06	0.01	0.00	0.00	0.00	0.00	0.09	5.79
SSW	9	0.02	0.05	0.01	0.00	0.00	0.00	0.00	0.08	5.25
SW	10	0.03	0.07	0.06	0.01	0.00	0.00	0.00	0.17	7.06
WSW	11	0.05	0.18	0.14	0.01	0.00	0.00	0.00	0.39	7.09
W	12	0.05	0.22	0.07	0.00	0.00	0.00	0.00	0.35	6.29
WNW	13	0.03	0.19	0.21	0.00	0.00	0.00	0.00	0.43	7.42
NW	14	0.01	0.18	0.42	0.00	0.00	0.00	0.00	0.62	8.57
NNW	15	0.01	0.14	0.27	0.00	0.00	0.00	0.00	0.42	8.38
N	16	0.02	0.07	0.17	0.06	0.00	0.00	0.00	0.32	9.51
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.38	2.66	3.49	0.57	0.01	0.00	0.00	7.11	8.41

NUMBER OF VALID CATEGORY OBSERVATIONS = 623

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-51

CDOE#LSD-2 FLORIDA POWER & LIGHT COMPANY
HUTCHINSON ISLAND SITE
PERIOD OF RECORD 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = E
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = D
PASQUILL E-D

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.03	0.23	0.13	0.00	0.00	0.00	0.00	0.37	6.71
NE	2	0.07	0.30	0.13	0.00	0.00	0.00	0.00	0.49	6.24
ENE	3	0.18	0.71	0.18	0.00	0.00	0.00	0.00	1.07	5.75
E	4	0.09	0.86	0.34	0.06	0.00	0.00	0.00	1.35	6.09
ESE	5	0.23	1.37	0.32	0.02	0.00	0.00	0.00	1.96	6.02
SE	6	0.19	1.48	0.38	0.00	0.00	0.00	0.00	2.05	6.06
SSE	7	0.17	0.67	0.31	0.04	0.00	0.00	0.00	1.19	6.53
S	8	0.07	0.32	0.21	0.01	0.00	0.00	0.00	0.60	[illegible]
SSW	9	0.11	0.13	0.06	0.01	0.00	0.00	0.00	0.31	5.53
SW	10	0.26	0.82	0.19	0.07	0.00	0.00	0.00	1.35	6.04
WSW	11	0.22	0.67	0.09	0.00	0.00	0.00	0.00	0.98	5.24
W	12	0.25	0.34	0.02	0.00	0.00	0.00	0.00	0.62	4.37
WNW	13	0.11	0.46	0.06	0.00	0.00	0.00	0.00	0.63	5.74
NW	14	0.30	0.81	0.23	0.00	0.00	0.00	0.00	1.14	6.15
NNW	15	0.06	0.33	0.10	0.00	0.00	0.00	0.00	0.49	6.11
N	16	0.02	0.21	0.14	0.00	0.00	0.00	0.00	0.37	7.03
CALM AND VARIABLE	0								0.00	0.00
TOTAL		2.18	9.70	2.86	0.21	0.00	0.00	0.00	14.96	6.07

NUMBER OF VALID CATEGORY OBSERVATIONS = 1311

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-52

CODE#LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 33
HUTCHINSON ISLAND SITE
PERIOD OF RECORD 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL
STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = F
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = E
PASQUILL F-E

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.02	0.03	0.00	0.00	0.00	0.00	0.00	0.06	4.23
NE	2	0.06	0.03	0.00	0.00	0.00	0.00	0.00	0.09	3.49
ENE	3	0.02	0.06	0.00	0.00	0.00	0.00	0.00	0.08	4.61
E	4	0.07	0.08	0.00	0.00	0.00	0.00	0.00	0.17	3.79
ESE	5	0.19	0.14	0.00	0.00	0.00	0.00	0.00	0.33	3.62
SE	6	0.29	0.30	0.00	0.02	0.00	0.00	0.00	0.65	4.51
SSE	7	0.19	1.02	0.10	0.00	0.00	0.00	0.00	1.31	5.41
S	8	0.19	1.12	0.25	0.02	0.00	0.00	0.00	1.59	6.00
SSW	9	0.18	0.50	0.27	0.03	0.00	0.00	0.00	0.99	6.53
SW	10	0.39	0.88	0.05	0.00	0.00	0.00	0.00	1.31	4.73
WSW	11	0.23	0.07	0.00	0.00	0.00	0.00	0.00	0.32	3.19
W	12	0.08	0.01	0.00	0.00	0.00	0.00	0.00	0.09	2.66
WNW	13	0.16	0.01	0.00	0.00	0.00	0.00	0.00	0.17	2.47
NW	14	0.16	0.10	0.00	0.00	0.00	0.00	0.00	0.26	3.34
NNW	15	0.03	0.01	0.00	0.00	0.00	0.00	0.00	0.05	3.07
N	16	0.03	0.01	0.00	0.00	0.00	0.00	0.00	0.05	3.07
CALM AND VARIABLE	0								1.60	0.00
TOTAL		2.33	4.47	0.67	0.08	0.00	0.00	0.00	8.96	4.33

NUMBER OF VALID CATEGORY OBSERVATIONS = 785

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-53

CODE#LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 34
HUTCHINSON ISLAND SITE
PERIOD OF RECORD 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL
STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = F
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = F
PASQUILL F-F

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENE	3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
E	4	0.05	0.03	0.00	0.00	0.00	0.00	0.00	0.08	3.66
ESE	5	0.01	0.02	0.00	0.00	0.00	0.00	0.00	0.03	4.43
SE	6	0.06	0.13	0.01	0.00	0.00	0.00	0.00	0.19	4.64
SSE	7	0.06	0.09	0.00	0.00	0.00	0.00	0.00	0.15	4.74
S	8	0.06	0.11	0.00	0.00	0.00	0.00	0.00	0.17	4.45
SSW	9	0.03	0.11	0.01	0.00	0.00	0.00	0.00	0.15	5.16
SW	10	0.03	0.03	0.00	0.00	0.00	0.00	0.00	0.07	3.70
WSW	11	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
W	12	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
WNW	13	0.05	0.01	0.00	0.00	0.00	0.00	0.00	0.05	2.91
NW	14	0.01	0.02	0.00	0.00	0.00	0.00	0.00	0.03	4.45
NNW	15	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
N	16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.39	0.58	0.02	0.00	0.00	0.00	0.00	0.97	4.36

NUMBER OF VALID CATEGORY OBSERVATIONS = 87

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-54

CODE#LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 35
HUTCHINSON ISLAND SITE
PERIOD OF RECORD 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL
STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = E
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = G
PASQUILL E-G

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENE	3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
E	4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ESE	5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SE	6	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
SSE	7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
S	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSW	9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SW	10	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
WSW	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
W	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WNW	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NW	14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.02	0.01	0.00	0.00	0.00	0.00	0.00	0.03	3.35

NUMBER OF VALID CATEGORY OBSERVATIONS = 3

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-55

CODE#LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 36
HUTCHINSON ISLAND SITE
PERIOD OF RECORD 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL
STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = F
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = A
PASQUILL F-A

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
ENE	3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
E	4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ESE	5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SE	6	0.01	0.00	0.01	0.00	0.00	0.00	0.00	0.02	4.15
SSE	7	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
S	8	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
SSW	9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SW	10	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
WSW	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
W	12	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
WNW	13	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
NW	14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.03	0.06	0.01	0.00	0.00	0.00	0.00	0.10	4.95

NUMBER OF VALID CATEGORY OBSERVATIONS = 9

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-56

CODE#LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 37
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL
STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = F
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = B
PASQUILL F-B

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENE	3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
E	4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ESE	5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SE	6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSE	7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
S	8	0.01	0.01	0.00	0.00	0.00	0.00	0.00	0.02	3.90
SSW	9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SW	10	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
WSW	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
W	12	0.02	0.01	0.00	0.00	0.00	0.00	0.00	0.03	3.35
WNW	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NW	14	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.06	0.02	0.00	0.00	0.00	0.00	0.00	0.08	3.19

NUMBER OF VALID CATEGORY OBSERVATIONS = 7
NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-57

CODE#LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 38
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL
STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = F
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = C
PASQUILL F-C

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
NE	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENE	3	0.02	0.01	0.00	0.00	0.00	0.00	0.00	0.03	3.35
E	4	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
ESE	5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SE	6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSE	7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
S	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSW	9	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
SW	10	0.02	0.01	0.01	0.00	0.00	0.00	0.00	0.05	3.02
WSW	11	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
W	12	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
WNW	13	0.01	0.02	0.00	0.00	0.00	0.00	0.00	0.03	4.43
NW	14	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
NNW	15	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.53
N	16	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.14	0.08	0.01	0.00	0.00	0.00	0.00	0.23	3.79

NUMBER OF VALID CATEGORY OBSERVATIONS = 20
NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-58

CODE#LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 39
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL
STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = F
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = D
PASQUILL F-D

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.02	0.03	0.00	0.00	0.00	0.00	0.00	0.06	4.23
NE	2	0.05	0.05	0.00	0.00	0.00	0.00	0.00	0.09	3.90
ENE	3	0.09	0.02	0.00	0.00	0.00	0.00	0.00	0.11	2.91
E	4	0.05	0.02	0.01	0.00	0.00	0.00	0.00	0.08	4.31
ESE	5	0.09	0.02	0.01	0.00	0.00	0.00	0.00	0.13	3.56
SE	6	0.07	0.09	0.02	0.00	0.00	0.00	0.00	0.18	4.38
SSE	7	0.07	0.03	0.01	0.00	0.00	0.00	0.00	0.11	4.02
S	8	0.05	0.03	0.00	0.00	0.00	0.00	0.00	0.08	3.66
SSW	9	0.07	0.03	0.00	0.00	0.00	0.00	0.00	0.10	3.35
SW	10	0.10	0.05	0.00	0.00	0.00	0.00	0.00	0.15	3.27
WSW	11	0.11	0.06	0.00	0.00	0.00	0.00	0.00	0.17	3.35
W	12	0.06	0.05	0.00	0.00	0.00	0.00	0.00	0.10	3.72
WNW	13	0.05	0.03	0.00	0.00	0.00	0.00	0.00	0.08	3.66
NW	14	0.06	0.11	0.00	0.00	0.00	0.00	0.00	0.17	4.45
NNW	15	0.02	0.19	0.00	0.00	0.00	0.00	0.00	0.22	5.20
N	16	0.02	0.03	0.00	0.01	0.00	0.00	0.00	0.07	6.12
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.97	0.87	0.06	0.01	0.00	0.00	0.00	1.91	4.06

NUMBER OF VALID CATEGORY OBSERVATIONS = 167
NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-59

CODE#LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 40
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL
STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = F
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = E
PASQUILL F-E

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.02	0.02	0.00	0.00	0.00	0.00	0.00	0.05	3.90
NE	2	0.08	0.00	0.00	0.00	0.00	0.00	0.00	0.09	2.25
ENE	3	0.02	0.01	0.00	0.00	0.00	0.00	0.00	0.03	3.35
E	4	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
ESE	5	0.10	0.02	0.01	0.00	0.00	0.00	0.00	0.14	3.43
SE	6	0.15	0.01	0.00	0.00	0.00	0.00	0.00	0.16	2.49
SSE	7	0.16	0.18	0.00	0.00	0.00	0.00	0.00	0.34	4.01
S	8	0.10	0.19	0.01	0.00	0.00	0.00	0.00	0.31	4.62
SSW	9	0.05	0.07	0.00	0.00	0.00	0.00	0.00	0.11	4.23
SW	10	0.11	0.26	0.00	0.00	0.00	0.00	0.00	0.38	4.55
WSW	11	0.06	0.07	0.00	0.00	0.00	0.00	0.00	0.13	4.35
W	12	0.05	0.02	0.00	0.00	0.00	0.00	0.00	0.07	3.35
WNW	13	0.11	0.03	0.00	0.00	0.00	0.00	0.00	0.15	3.01
NW	14	0.11	0.21	0.00	0.00	0.00	0.00	0.00	0.32	4.37
NNW	15	0.03	0.09	0.00	0.00	0.00	0.00	0.00	0.11	4.65
N	16	0.14	0.05	0.00	0.00	0.00	0.00	0.00	0.18	3.07
CALM AND VARIABLE	0								0.00	0.00
TOTAL		1.32	1.24	0.02	0.00	0.00	0.00	0.00	2.59	3.90

NUMBER OF VALID CATEGORY OBSERVATIONS = 227
NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-60

CODE#LSO-2 — FLORIDA POWER & LIGHT COMPANY — PAGE 41
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = F
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = F
PASQUILL F-F

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
NE	2	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
ENE	3	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
E	4	0.01	0.01	0.00	0.00	0.00	0.00	0.00	0.02	3.90
ESE	5	0.03	0.01	0.00	0.00	0.00	0.00	0.00	0.05	3.07
SE	6	0.07	0.03	0.00	0.00	0.00	0.00	0.00	0.10	3.35
SSE	7	0.10	0.06	0.00	0.00	0.00	0.00	0.00	0.16	3.43
S	8	0.10	0.02	0.00	0.00	0.00	0.00	0.00	0.13	2.85
SSW	9	0.06	0.01	0.00	0.00	0.00	0.00	0.00	0.07	2.60
SW	10	0.07	0.02	0.00	0.00	0.00	0.00	0.00	0.09	3.07
WSW	11	0.10	0.01	0.00	0.00	0.00	0.00	0.00	0.11	2.58
W	12	0.06	0.02	0.00	0.00	0.00	0.00	0.00	0.08	3.19
WNW	13	0.07	0.00	0.00	0.00	0.00	0.00	0.00	0.07	2.25
NW	14	0.05	0.05	0.00	0.00	0.00	0.00	0.00	0.10	4.08
NNW	15	0.02	0.05	0.00	0.00	0.00	0.00	0.00	0.07	4.45
N	16	0.02	0.01	0.00	0.00	0.00	0.00	0.00	0.03	3.35
CALM AND VARIABLE	0								2.29	0.33
TOTAL		0.82	0.32	0.00	0.00	0.00	0.00	0.00	3.37	1.41

NUMBER OF VALID CATEGORY OBSERVATIONS = 295

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-61

CODE#LSO-2 — FLORIDA POWER & LIGHT COMPANY — PAGE 42
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = F
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = G
PASQUILL F-G

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENE	3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
E	4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ESE	5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SE	6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSE	7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
S	8	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
SSW	9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SW	10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WSW	11	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
W	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WNW	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NW	14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
CALM AND VARIABLE	0								0.01	0.00
TOTAL		0.06	0.00	0.00	0.00	0.00	0.00	0.00	0.07	1.96

NUMBER OF VALID CATEGORY OBSERVATIONS = 6

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-62

CODE#LSO-2 — FLORIDA POWER & LIGHT COMPANY — PAGE 43
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = G
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = A
PASQUILL G-A

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENE	3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
E	4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ESE	5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SE	6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSE	7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
S	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSW	9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SW	10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WSW	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
W	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WNW	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NW	14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

NUMBER OF VALID CATEGORY OBSERVATIONS = 0

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-63

CODE#LSO-2 — FLORIDA POWER & LIGHT COMPANY — PAGE 44
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = G
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = B
PASQUILL G-B

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENE	3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
E	4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ESE	5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SE	6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSE	7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
S	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSW	9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SW	10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WSW	11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
W	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WNW	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NW	14	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

NUMBER OF VALID CATEGORY OBSERVATIONS = 0

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-64

CODE#LSO-2 FLORIDA POWER & LIGHT COMPANY PAGE 65
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL
STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = G
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = C
PASQUILL G-C

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENE	3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
E	4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ESE	5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SE	6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSE	7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
S	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSW	9	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
SW	10	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
WSW	11	0.03	0.00	0.00	0.00	0.00	0.00	0.00	0.03	2.25
W	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WNW	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NW	14	0.01	0.01	0.00	0.00	0.00	0.00	0.00	0.02	3.90
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.09	0.01	0.00	0.00	0.00	0.00	0.00	0.10	2.62

NUMBER OF VALID CATEGORY OBSERVATIONS = 9

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-65

CODE#LSO-2 FLORIDA POWER & LIGHT COMPANY PAGE 66
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL
STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = G
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = D
PASQUILL G-D

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
NE	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENE	3	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
E	4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ESE	5	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
SE	6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSE	7	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
S	8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSW	9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SW	10	0.03	0.00	0.00	0.00	0.00	0.00	0.00	0.03	2.25
WSW	11	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
W	12	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
WNW	13	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
NW	14	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.02	5.55
NNW	15	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
N	16	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.17	0.03	0.00	0.00	0.00	0.00	0.00	0.21	2.90

NUMBER OF VALID CATEGORY OBSERVATIONS = 18

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-66

CODE#LSO-2 FLORIDA POWER & LIGHT COMPANY PAGE 67
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL
STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = G
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = E
PASQUILL G-E

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ENE	3	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
E	4	0.01	0.00	0.01	0.00	0.00	0.00	0.00	0.02	6.15
ESE	5	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
SE	6	0.01	0.01	0.00	0.00	0.00	0.00	0.00	0.02	3.90
SSE	7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
S	8	0.01	0.01	0.00	0.00	0.00	0.00	0.00	0.02	3.90
SSW	9	0.03	0.01	0.00	0.00	0.00	0.00	0.00	0.05	3.07
SW	10	0.08	0.02	0.00	0.00	0.00	0.00	0.00	0.10	2.98
WSW	11	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
W	12	0.05	0.01	0.00	0.00	0.00	0.00	0.00	0.06	2.91
WNW	13	0.09	0.00	0.00	0.00	0.00	0.00	0.00	0.09	2.25
NW	14	0.07	0.08	0.00	0.00	0.00	0.00	0.00	0.15	4.03
NNW	15	0.03	0.02	0.00	0.00	0.00	0.00	0.00	0.04	3.57
N	16	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.48	0.17	0.01	0.00	0.00	0.00	0.00	0.66	3.24

NUMBER OF VALID CATEGORY OBSERVATIONS = 58

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-67

CODE#LSO-2 FLORIDA POWER & LIGHT COMPANY PAGE 68
HUTCHINSON ISLAND SITE
PERIOD OF RECORD: 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL
STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = G
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = F
PASQUILL G-F

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
ENE	3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
E	4	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
ESE	5	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
SE	6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSE	7	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
S	8	0.03	0.00	0.00	0.00	0.00	0.00	0.00	0.03	2.25
SSW	9	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SW	10	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
WSW	11	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.01	5.55
W	12	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
WNW	13	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
NW	14	0.02	0.08	0.00	0.00	0.00	0.00	0.00	0.10	[illegible]
NNW	15	0.01	0.01	0.00	0.00	0.00	0.00	0.00	0.02	[illegible]
N	16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CALM AND VARIABLE	0								0.00	0.00
TOTAL		0.17	0.11	0.00	0.00	0.00	0.00	0.00	0.29	[illegible]

NUMBER OF VALID CATEGORY OBSERVATIONS = 25

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-68

CODE#LSD-2 FLORIDA POWER & LIGHT COMPANY PAGE 69.
HUTCHINSON ISLAND SITE
PERIOD OF RECORD 3/ 1/71 TO 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL AND HORIZONTAL
STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

VERTICAL STABILITY AS DEFINED BY DELTA-T = G
HORIZONTAL STABILITY AS DEFINED BY SIGMA-THETA = G
PASQUILL G-G

SECTOR	SECTOR NUMBER	SPEED(MPH) ADJUSTED TO 10-METER ELEVATION 1-3	4-7	8-12	13-18	19-24	25-31	>31	TOTAL	MEAN SPEED
NNE	1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
NE	2	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
ENE	3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
E	4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ESE	5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SE	6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
SSE	7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
S	8	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
SSW	9	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	2.25
SW	10	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
WSW	11	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
W	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
WNW	13	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.02	2.25
NW	14	0.03	0.00	0.00	0.00	0.00	0.00	0.00	0.03	2.25
NNW	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
N	16	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CALM AND VARIABLE	0								0.70	0.41
TOTAL		0.15	0.00	0.00	0.00	0.00	0.00	0.00	0.84	0.81

NUMBER OF VALID CATEGORY OBSERVATIONS = 74

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 6704

TABLE 2.3-69

St. Lucie Plant SITE
AVERAGE ANNUAL RELATIVE CONCENTRATIONS
AS A FUNCTION OF WIND DIRECTION AND DISTANCE

PERIOD OF RECORD: 03/01/71 TO 02/29/72

DISTANCE IN METERS AND MILES

SECTOR	201.13 0.125	402.25 0.25	604.50 0.5	1554.29 0.97*	2413.50 1.5	4022.50 2.5	5631.50 3.5
NNE	2.52D-05	6.89D-06	1.96D-06	6.35D-07	3.16D-07	1.49D-07	8.89D-08
NE	3.40D-05	9.29D-06	2.64D-06	8.53D-07	4.24D-07	2.00D-07	1.10D-07
ENE	4.25D-05	1.16D-05	3.28D-06	1.06D-06	5.24D-07	2.47D-07	1.47D-07
E	4.59D-05	1.24D-05	3.49D-06	1.12D-06	5.56D-07	2.61D-07	1.55D-07
ESE	5.61D-05	1.58D-05	4.50D-06	1.46D-06	7.26D-07	3.41D-07	2.03D-07
SE	5.74D-05	1.56D-05	4.42D-06	1.43D-06	7.11D-07	3.34D-07	1.99D-07
SSE	5.53D-05	1.52D-05	4.37D-06	1.42D-06	7.08D-07	3.34D-07	1.98D-07
S	5.47D-05	1.51D-05	4.35D-06	1.42D-06	7.04D-07	3.31D-07	1.96D-07
SSW	4.53D-05	1.25D-05	3.60D-06	1.17D-06	5.81D-07	2.73D-07	1.62D-07
SW	6.71D-05	1.84D-05	5.32D-06	1.73D-06	8.60D-07	4.04D-07	2.40D-07
WSW	4.68D-05	1.29D-05	3.71D-06	1.21D-06	6.02D-07	2.85D-07	1.70D-07
W	3.79D-05	1.04D-05	3.01D-06	9.81D-07	4.89D-07	2.31D-07	1.36D-07
WNW	4.21D-05	1.16D-05	3.33D-06	1.09D-06	5.42D-07	2.57D-07	1.53D-07
NW	5.28D-05	1.45D-05	4.20D-06	1.37D-06	6.84D-07	3.25D-07	1.94D-07
NNW	3.29D-05	9.07D-06	2.61D-06	8.52D-07	4.25D-07	2.02D-07	1.20D-07
N	3.10D-05	3.50D-06	2.43D-06	7.88D-07	3.939-07	1.86D-07	1.11D-07

* RESTRICTED DISTANCE

DISTANCE IN METERS AND MILES

SECTOR	7240.5 4.5	8046.50 5.0**	12067.5 7.5	24135.0 15.0	40225.0 25.0	56315.0 35.0	72405.0 45.0
NNE	5.93D-08	4.96D-08	2.83D-08	1.10D-08	5.53D-09	3.56D-09	2.56D-09
NE	7.95D-08	6.65D-08	3.79D-08	1.40D-08	7.45D-09	4.81D-09	3.50D-09
ENE	9.77D-08	8.16D-08	4.66D-08	1.81D-08	9.12D-09	5.88D-09	4.25D-09
E	1.03D-07	8.61D-08	4.92D-08	1.91D-08	9.54D-09	6.22D-09	4.53D-09
ESE	1.55D-07	1.13D-07	6.43D-08	2.31D-08	1.27D-08	8.19D-09	5.96D-09
SE	1.52D-07	1.11D-07	6.31D-08	2.47D-08	1.25D-08	8.10D-09	5.91D-09
SSE	1.32D-07	1.10D-07	6.27D-08	2.44D-08	1.23D-08	7.91D-09	5.75D-09
S	1.30D-07	1.09D-07	6.18D-08	2.39D-08	1.19D-08	7.65D-09	5.54D-09
SSW	1.07D-07	8.96D-08	5.09D-08	1.96D-08	9.76D-09	6.24D-09	4.52D-09
SW	1.59D-07	1.33D-07	7.57D-08	2.94D-08	1.47D-08	9.47D-09	6.87D-09
WSW	1.13D-07	9.46D-08	5.39D-08	2.11D-08	1.06D-08	6.82D-09	4.95D-09
W	9.16D-08	7.69D-08	4.37D-08	1.71D-08	8.60D-09	5.54D-09	4.02D-09
WNW	1.02D-07	8.56D-08	4.87D-08	1.91D-08	9.59D-09	6.17D-09	4.48D-09
NW	1.29D-07	1.08D-07	6.15D-08	2.41D-08	1.21D-08	7.79D-09	5.65D-09
NNW	8.03D-08	6.75D-08	3.83D-08	1.50D-08	7.51D-09	4.83D-09	3.50D-09
N	7.42D-08	6.21D-08	3.54D-08	1.38D-08	6.93D-09	4.46D-09	3.23D-09

** LOW POPULATION ZONE DISTANCE

TABLE 2.3-70

F L O R I D A   P O W E R   &   L I G H T   C O M P A N Y

CODE*LSD-5

ST LUCIE SITE

AVERAGE ANNUAL RELATIVE CONCENTRATIONS

AS A FUNCTION OF WIND DIRECTION AND DISTANCE

PERIOD OF RECORD: 01/01/72 TO 12/31/72

DISTANCE IN METERS AND MILES

WIND SECTOR	201.13 0.125	402.25 0.25	804.50 0.5	1554.29 0.97*	2413.50 1.5	4022.50 2.5	5631.50 3.5
NNE	2.79D-05	7.61D-06	2.17D-06	7.05D-07	3.50D-07	1.65D-07	9.80D-08
NE	3.29D-05	8.91D-06	2.51D-06	8.06D-07	4.00D-07	1.87D-07	1.11D-07
ENE	4.37D-05	1.19D-05	3.35D-06	1.08D-06	5.37D-07	2.50D-07	1.48D-07
E	5.15D-05	1.40D-05	3.94D-06	1.27D-06	6.28D-07	2.92D-07	1.73D-07
ESE	5.14D-05	1.40D-05	3.93D-06	1.29D-06	6.39D-07	2.97D-07	1.76D-07
SE	5.49D-05	1.49D-05	4.24D-06	1.38D-06	6.83D-07	3.19D-07	1.89D-07
SSE	4.85D-05	1.33D-05	3.83D-06	1.25D-06	6.20D-07	2.89D-07	1.71D-07
S	4.57D-05	1.29D-05	3.74D-06	1.22D-06	6.04D-07	2.83D-07	1.68D-07
SSW	4.73D-05	1.30D-05	3.79D-06	1.23D-06	6.13D-07	2.88D-07	1.71D-07
SW	5.29D-05	1.45D-05	4.20D-06	1.37D-06	6.80D-07	3.20D-07	1.90D-07
WSW	4.86D-05	1.33D-05	3.54D-06	1.25D-06	6.22D-07	2.94D-07	1.75D-07
W	4.19D-05	1.15D-05	3.33D-06	1.09D-06	5.42D-07	2.56D-07	1.52D-07
WNW	5.26D-05	1.45D-05	4.19D-06	1.37D-06	6.85D-07	3.25D-07	1.95D-07
NW	5.23D-05	1.44D-05	4.15D-06	1.35D-06	6.75D-07	3.20D-07	1.91D-07
NNW	3.03D-05	8.34D-06	2.40D-06	7.82D-07	3.90D-07	1.66D-07	1.11D-07
N	2.95D-05	8.10D-06	2.33D-06	7.55D-07	3.76D-07	1.78D-07	1.06D-07

DISTANCE IN METERS AND MILES

WIND SECTOR	7240.50 4.5	8045.00 5.0**	12067.5 7.5	24135.0 15.0	40225.0 25.0	56315.0 35.0	72405.0 45.0
NNE	6.52D-08	5.45D-08	3.10D-08	1.20D-08	6.03D-09	3.87D-09	2.81D-09
NE	7.40D-08	6.17D-08	3.52D-08	1.36D-08	6.85D-09	4.41D-09	3.21D-09
ENE	9.85D-08	8.22D-08	4.69D-08	1.82D-08	9.13D-09	5.88D-09	4.25D-09
E	1.15D-07	9.58D-08	5.47D-08	2.10D-08	1.05D-08	6.78D-09	4.93D-09
ESE	1.16D-07	9.70D-08	5.57D-08	2.14D-08	1.07D-08	6.93D-09	5.05D-09
SE	1.25D-07	1.05D-07	5.97D-08	2.33D-08	1.18D-08	7.60D-09	5.54D-09
SSE	1.13D-07	9.45D-08	5.37D-08	2.08D-08	1.04D-08	6.70D-09	4.87D-09
S	1.11D-07	9.28D-08	5.26D-08	2.03D-08	1.01D-08	6.45D-09	4.66D-09
SSW	1.13D-07	9.44D-08	5.37D-08	2.06D-08	1.03D-08	6.55D-09	4.74D-09
SW	1.26D-07	1.05D-07	5.97D-08	2.32D-08	1.16D-08	7.45D-09	5.40D-09
WSW	1.16D-07	9.73D-08	5.54D-08	2.17D-08	1.09D-08	7.05D-09	5.13D-09
W	1.02D-07	8.50D-08	4.84D-08	1.89D-08	9.53D-09	6.14D-09	4.46D-09
WNW	1.30D-07	1.09D-07	6.21D-08	2.43D-08	1.22D-08	7.86D-09	5.70D-09
NW	1.27D-07	1.07D-07	6.07D-08	2.37D-08	1.19D-08	7.67D-09	5.57D-09
NNW	7.41D-08	6.21D-08	3.54D-08	1.38D-08	6.96D-09	4.47D-09	3.24D-09
N	7.03D-08	5.88D-08	3.35D-08	1.30D-08	6.49D-09	4.16D-09	3.01D-09

NOTES: 1) A * (NEXT TO MILES IN TITLE) IS THE RESTRICTED DISTANCE
2) A ** (NEXT TO MILES IN TITLE) IS THE LOW POPULATION DISTANCE
3) THE LETTER "D" IN THE ABOVE NUMBERS INDICATES THE NUMBER IS IN EXPONENTIAL FORM. (THE VALUE OF "D" IS 10)

TABLE 2.3-71

FLORIDA POWER & LIGHT COMPANY

CODE*LSD-5

ST LUCIE SITE

AVERAGE ANNUAL RELATIVE CONCENTRATIONS
AS A FUNCTION OF WIND DIRECTION AND DISTANCE

PERIOD OF RECORD: 01/01/73 TO 12/31/73

DISTANCE IN METERS AND MILES

WIND SECTOR	201.13 0.125	402.25 0.25	804.50 0.5	1554.29 0.97*	2413.50 1.5	4022.50 2.5	5631.50 3.5
NNE	1.76D-05	4.85D-06	1.48D-06	5.69D-07	3.07D-07	1.52D-07	9.33D-08
NE	2.24D-05	6.26D-06	1.89D-06	7.16D-07	3.83D-07	1.89D-07	1.16D-07
ENE	2.12D-05	5.84D-06	1.75D-06	6.69D-07	3.60D-07	1.77D-07	1.09D-07
E	2.99D-05	8.20D-06	2.44D-06	9.43D-07	5.08D-07	2.50D-07	1.53D-07
ESE	3.88D-05	1.06D-05	3.21D-06	1.24D-06	6.73D-07	3.32D-07	2.04D-07
SE	3.45D-05	9.49D-06	2.84D-06	1.10D-06	5.95D-07	2.94D-07	1.80D-07
SSE	2.92D-05	8.01D-06	2.45D-06	9.62D-07	5.24D-07	2.59D-07	1.59D-07
S	2.94D-05	8.09D-06	2.52D-06	9.76D-07	5.27D-07	2.60D-07	1.60D-07
SSW	2.88D-05	7.95D-06	2.45D-06	9.24D-07	5.01D-07	2.50D-07	1.55D-07
SW	2.67D-05	7.38D-06	2.26D-06	8.70D-07	4.70D-07	2.33D-07	1.44D-07
WSW	2.23D-05	6.14D-06	1.86D-06	7.11D-07	3.85D-07	1.92D-07	1.19D-07
W	2.53D-05	6.98D-06	2.12D-06	8.03D-07	4.36D-07	2.18D-07	1.35D-07
WNW	3.20D-05	8.81D-06	2.68D-06	1.02D-06	5.51D-07	2.75D-07	1.70D-07
NW	3.68D-05	1.02D-05	3.09D-06	1.17D-06	6.29D-07	3.13D-07	1.94D-07
NNW	1.98D-05	5.48D-06	1.67D-06	6.28D-07	3.39D-07	1.68D-07	1.04D-07
N	1.88D-05	5.20D-06	1.60D-06	6.04D-07	3.24D-07	1.60D-07	9.83D-08

DISTANCE IN METERS AND MILES

WIND SECTOR	7240.50 4.5	8045.00 5.0**	12067.5 7.5	24135.0 15.0	40225.0 25.0	56315.0 35.0	72405.0 45.0
NNE	6.35D-08	5.34D-08	3.10D-08	1.23D-08	6.23D-09	4.03D-09	2.94D-09
NE	7.88D-08	6.63D-08	3.85D-08	1.53D-08	7.76D-09	5.03D-09	3.67D-09
ENE	7.41D-08	6.24D-08	3.62D-08	1.44D-08	7.33D-09	4.77D-09	3.49D-09
E	1.04D-07	8.72D-08	5.05D-08	2.00D-08	1.02D-08	6.65D-09	4.87D-09
ESE	1.39D-07	1.17D-07	6.77D-08	2.69D-08	1.37D-08	8.91D-09	6.51D-09
SE	1.23D-07	1.03D-07	5.98D-08	2.38D-08	1.21D-08	7.88D-09	5.77D-09
SSE	1.08D-07	9.06D-08	5.24D-08	2.07D-08	1.05D-08	6.82D-09	4.98D-09
S	1.08D-07	9.12D-08	5.28D-08	2.08D-08	1.05D-08	6.77D-09	4.93D-09
SSW	1.06D-07	8.96D-08	5.22D-08	2.08D-08	1.05D-08	6.82D-09	4.96D-09
SW	9.79D-08	8.25D-08	4.79D-08	1.90D-08	9.67D-09	6.27D-09	4.57D-09
WSW	8.15D-08	6.89D-08	4.01D-08	1.61D-08	8.24D-09	5.36D-09	3.91D-09
W	9.28D-08	7.85D-08	4.58D-08	1.84D-08	9.37D-09	6.08D-09	4.43D-09
WNW	1.17D-07	9.88D-08	5.76D-08	2.31D-08	1.18D-08	7.65D-09	5.58D-09
NW	1.33D-07	1.13D-07	6.58D-08	2.63D-08	1.34D-08	8.71D-09	6.35D-09
NNW	7.14D-08	6.03D-08	3.51D-08	1.40D-08	7.15D-09	4.63D-09	3.38D-09
N	6.70D-08	5.64D-08	3.28D-08	1.29D-08	6.55D-09	4.23D-09	3.08D-09

NOTES: 1) A * (NEXT TO MILES IN TITLE) IS THE EXCLUSION DISTANCE
2) A ** (NEXT TO MILES IN TITLE) IS THE LOW POPULATION DISTANCE
3) THE LETTER "D" IN THE ABOVE NUMBERS INDICATES THE NUMBER IS IN EXPONENTIAL FORM. (THE VALUE OF "D" IS 10)

TABLE 2.3-72

FLORIDA POWER AND LIGHT COMPANY
PERIOD OF RECORD: 3/1/71 to 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL STABILITY CATAGORIES BY WIND DIRECTION AND WIND SPEED

PASQUILL A

SECTOR DIR	SPEED(MPH) ADJUSTED TO 10 METER ELEVATION 1-4	4-8	8-13	13-19	19-25	25-31	31+	TOT	AVG
NNE	0.000	.331	.742	.091	0.000	0.000	0.000	1.164	9.205
NE	.023	1.038	.445	.011	0.000	0.000	0.000	1.518	6.895
ENE	.068	1.483	.593	.057	0.000	0.000	0.000	2.202	7.097
E	.046	2.613	1.095	.148	.011	0.000	0.000	3.914	7.199
ESE	.023	1.689	1.107	.057	0.000	0.000	0.000	2.875	7.453
SE	.011	1.278	2.716	.285	0.000	0.000	0.000	4.290	9.056
SSE	.011	.205	.502	.308	0.000	0.000	0.000	1.027	10.711
S	.023	.126	.240	.126	0.000	0.000	0.000	.513	9.946
SSW	.023	.217	.217	.057	0.000	0.000	0.000	.513	8.418
SW	.034	.274	.422	.160	.046	0.000	0.000	.936	9.948
WSW	0.000	.194	.285	.080	.023	0.000	0.000	.582	9.757
W	0.000	.205	.137	.057	.023	0.000	0.000	.422	9.229
WNW	.046	.183	.217	.057	.034	0.000	0.000	.536	9.173
NW	.011	.171	.319	.137	0.000	0.000	0.000	.639	9.882
NNW	.011	.126	.228	.068	0.000	0.000	0.000	.434	9.413
N	0.000	.160	.742	.536	0.000	0.000	0.000	1.438	11.582
TOT	.331	10.292	10.007	2.236	.137	0.000	0.000	23.003	8.536
VARIABLE AND CALMS								.068	.090
TOTAL								23.072	8.511

NUMBER OF VALID CATEGORY OBSERVATIONS = 2024

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-73

FLORIDA POWER AND LIGHT COMPANY
PERIOD OF RECORD: 3/1/72 to 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL STABILITY CATAGORIES BY WIND DIRECTION AND WIND SPEED

PASQUILL B

SECTOR DIR	SPEED(MPH) ADJUSTED TO 10 METER ELEVATION 1-4	4-8	8-13	13-19	19-25	25-31	31+	TOT	AVG
NNE	0.000	.103	.057	.034	0.000	0.000	0.000	.194	8.639
NE	0.000	.148	.068	0.000	0.000	0.000	0.000	.217	6.971
ENE	.034	.160	.046	.034	0.000	0.000	0.000	.274	7.138
E	.011	.205	.057	.023	0.000	0.000	0.000	.297	7.060
ESE	.011	.114	.034	.011	0.000	0.000	0.000	.171	6.899
SE	.011	.114	.057	.034	0.000	0.000	0.000	.217	8.137
SSE	0.000	.046	.091	.046	0.000	0.000	0.000	.183	10.297
S	.011	.091	.034	0.000	0.000	0.000	0.000	.137	6.402
SSW	.034	.046	.046	.011	0.000	0.000	0.000	.137	7.057
SW	0.000	.091	.080	.046	.034	0.000	0.000	.251	10.982
WSW	.011	.080	.034	.023	.011	0.000	0.000	.160	8.850
W	.011	.034	.046	0.000	0.000	0.000	0.000	.091	7.387
WNW	0.000	.023	0.000	.011	0.000	0.000	0.000	.034	8.880
NW	0.000	.068	.023	.011	0.000	0.000	0.000	.103	7.659
NNW	0.000	.023	.046	.046	0.000	0.000	0.000	.114	11.715
N	0.000	.023	.103	.103	.034	0.000	0.000	.262	13.310
TOT	.137	1.369	.822	.434	.080	0.000	0.000	2.841	8.683
VARIABLE AND CALMS								.046	.110
TOTAL								2.887	8.547

NUMBER OF VALID CATEGORY OBSERVATIONS = 253

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-74

FLORIDA POWER AND LIGHT COMPANY
PERIOD OF RECORD: 3/1/71 to 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL STABILITY CATAGORIES BY WIND DIRECTION AND WIND SPEED

PASQUILL C

SECTOR DIR	1-4	SPEED(MPH) ADJUSTED TO 10 METER ELEVATION 4-8	8-13	13-19	19-25	25-31	31+	TOT	AVG
NNE	.011	.114	.114	.046	.023	0.000	0.000	.308	9.764
NE	.011	.205	.137	.034	0.000	0.000	0.000	.388	7.920
ENE	.091	.114	.103	.011	.034	0.000	0.000	.354	7.876
E	.011	.251	.091	.080	0.000	0.000	0.000	.434	8.249
ESE	0.000	.297	.160	0.000	0.000	0.000	0.000	.456	7.125
SE	0.000	.228	.126	.034	0.000	0.000	0.000	.388	7.889
SSE	.011	.160	.183	.114	0.000	0.000	0.000	.468	9.668
S	.034	.057	.251	.046	0.000	0.000	0.000	.388	9.020
SSW	.011	.171	.148	.137	.011	0.000	0.000	.479	10.099
SW	.023	.319	.262	.057	.011	0.000	0.000	.673	8.313
WSW	.023	.114	.057	.011	.011	0.000	0.000	.217	7.754
W	.023	.126	.023	.011	0.000	0.000	0.000	.183	6.327
WNW	.023	.148	.046	.011	0.000	0.000	0.000	.228	6.618
NW	.023	.114	.046	.011	0.000	0.000	0.000	.194	6.809
NNW	.023	.068	.114	0.000	0.000	0.000	0.000	.205	7.682
N	0.000	.103	.137	.080	.023	0.000	0.000	.342	10.748
TOT	.319	2.590	1.997	.685	.114	0.000	0.000	5.705	8.438
VARIABLE AND CALMS								.205	.040
				TOTAL				5.911	8.146

NUMBER OF VALID CATEGORY OBSERVATIONS = 518

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-75

FLORIDA POWER AND LIGHT COMPANY
PERIOD OF RECORD: 3/1/71 to 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL STABILITY CATAGORIES BY WIND DIRECTION AND WIND SPEED

PASQUILL D

SECTOR DIR	1-4	SPEED(MPH) ADJUSTED TO 10 METER ELEVATION 4-8	8-13	13-19	19-25	25-31	31+	TOT	AVG
NNE	.034	.228	.445	.251	.080	0.000	0.000	1.038	11.024
NE	.023	.262	.491	.468	.183	.011	0.000	1.438	12.499
ENE	.068	.456	.776	.445	.171	0.000	0.000	1.917	11.002
E	.068	.890	1.004	.468	.068	0.000	0.000	2.499	9.577
ESE	.046	.776	.536	.148	.011	0.000	0.000	1.518	8.140
SE	.068	.616	.388	.126	0.000	0.000	0.000	1.198	7.862
SSE	.011	.548	.810	.240	0.000	0.000	0.000	1.609	9.280
S	.011	1.061	1.381	.251	0.000	0.000	0.000	2.704	8.761
SSW	.046	1.164	.844	.365	.011	0.000	0.000	2.430	8.627
SW	.091	1.118	.548	.171	.046	0.000	0.000	1.974	7.884
WSW	.034	.468	.342	.034	0.000	0.000	0.000	.879	7.451
W	.091	.308	.148	.034	0.000	0.000	0.000	.582	6.767
WNW	.068	.399	.126	.205	0.000	0.000	0.000	.799	8.542
NW	.080	.571	.342	.137	0.000	0.000	0.000	1.130	7.895
NNW	.057	.274	.251	.228	0.000	0.000	0.000	.810	9.529
N	.023	.274	.559	.468	.034	0.000	0.000	1.358	11.245
TOT	.822	9.414	8.991	4.039	.605	.011	0.000	23.882	9.232
VARIABLE AND CALMS								.411	.020
				TOTAL				24.293	9.076

NUMBER OF VALID CATEGORY OBSERVATIONS = 2130

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-76

FLORIDA POWER AND LIGHT COMPANY
PERIOD OF RECORD: 3/1/71 to 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL STABILITY CATAGORIES BY WIND DIRECTION AND WIND SPEED

PASQUILL E

SECTOR DIR	SPEED(MPH) ADJUSTED TO 10 METER ELEVATION 1-4	4-8	8-13	13-19	19-25	25-31	31+	TOT	AVG
NNE	.103	.331	.377	.023	0.000	0.000	0.000	.833	7.451
NE	.183	.628	.491	.034	.057	0.000	0.000	1.392	7.604
ENE	.240	1.107	.719	.137	0.000	0.000	0.000	2.202	7.278
E	.262	1.187	.719	.194	0.000	0.000	0.000	2.362	7.372
ESE	.445	2.042	.901	.114	0.000	0.000	0.000	3.503	6.616
SE	.571	2.168	.571	.148	0.000	0.000	0.000	3.457	6.193
SSE	.422	1.917	.468	.034	0.000	0.000	0.000	2.841	5.919
S	.354	1.666	.468	.034	0.000	0.000	0.000	2.522	6.055
SSW	.365	.799	.365	.046	0.000	0.000	0.000	1.575	6.120
SW	.753	1.814	.297	.080	0.000	0.000	0.000	2.944	5.429
WSW	.525	.981	.240	.023	0.000	0.000	0.000	1.769	5.309
W	.411	.628	.148	0.000	0.000	0.000	0.000	1.187	4.970
WNW	.365	.707	.297	0.000	0.000	0.000	0.000	1.369	5.645
NW	.285	1.175	.673	0.000	0.000	0.000	0.000	2.134	6.526
NNW	.137	.513	.422	0.000	0.000	0.000	0.000	1.073	6.904
N	.103	.308	.319	.057	0.000	0.000	0.000	.787	7.669
TOT	5.523	17.971	7.474	.924	.057	0.000	0.000	31.949	6.349
VARIABLE AND CALMS								1.403	.080
			TOTAL					33.352	6.085

NUMBER OF VALID CATEGORY OBSERVATIONS = 2924

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-77

FLORIDA POWER AND LIGHT COMPANY
PERIOD OF RECORD: 3/1/71 to 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL STABILITY CATAGORIES BY WIND DIRECTION AND WIND SPEED

PASQUILL F

SECTOR DIR	SPEED(MPH) ADJUSTED TO 10 METER ELEVATION 1-4	4-8	8-13	13-19	19-25	25-31	31+	TOT	AVG
NNE	.148	.057	0.000	0.000	0.000	0.000	0.000	.205	3.167
NE	.126	.046	0.000	0.000	0.000	0.000	0.000	.171	3.130
ENE	.103	.046	.011	0.000	0.000	0.000	0.000	.160	3.751
E	.240	.057	.023	0.000	0.000	0.000	0.000	.319	3.396
ESE	.285	.171	.034	0.000	0.000	0.000	0.000	.491	3.328
SE	.342	.274	.023	0.000	0.000	0.000	0.000	.639	3.943
SSE	.285	.251	.011	0.000	0.000	0.000	0.000	.548	3.926
S	.183	.126	0.000	0.000	0.000	0.000	0.000	.308	3.800
SSW	.308	.331	0.000	0.000	0.000	0.000	0.000	.639	3.959
SW	.331	.148	.011	0.000	0.000	0.000	0.000	.491	3.558
WSW	.171	.091	0.000	0.000	0.000	0.000	0.000	.262	3.398
W	.262	.103	0.000	0.000	0.000	0.000	0.000	.365	3.177
WNW	.228	.388	0.000	0.000	0.000	0.000	0.000	.616	4.326
NW	.091	.342	0.000	0.000	0.000	0.000	0.000	.434	4.854
NNW	.194	.103	0.000	.011	0.000	0.000	0.000	.308	3.840
N	.080	.057	0.000	0.000	0.000	0.000	0.000	.137	3.625
TOT	3.377	2.590	.114	.011	0.000	0.000	0.000	6.093	3.787
VARIABLE AND CALMS								2.236	.330
			TOTAL					8.330	2.861

NUMBER OF VALID CATEGORY OBSERVATIONS = 731

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-78

FLORIDA POWER AND LIGHT COMPANY
PERIOD OF RECORD: 3/1/71 to 2/29/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL STABILITY CATAGORIES BY WIND DIRECTION AND WIND SPEED

PASQUILL G

SECTOR DIR	SPEED(MPH) ADJUSTED TO 10 METER ELEVATION 1-4	4-8	8-13	13-19	19-25	25-31	31+	TOT	AVG
NNE	.011	0.000	0.000	0.000	0.000	0.000	0.000	.011	2.250
NE	.034	0.000	0.000	0.000	0.000	0.000	0.000	.034	2.250
ENE	.034	0.000	0.000	0.000	0.000	0.000	0.000	.034	2.250
E	.023	0.000	.011	0.000	0.000	0.000	0.000	.034	4.850
ESE	.068	0.000	0.000	0.000	0.000	0.000	0.000	.068	2.250
SE	.011	.011	0.000	0.000	0.000	0.000	0.000	.023	3.900
SSE	.034	0.000	0.000	0.000	0.000	0.000	0.000	.034	2.250
S	.057	.011	0.000	0.000	0.000	0.000	0.000	.068	2.800
SSW	.057	.011	0.000	0.000	0.000	0.000	0.000	.068	2.797
SW	.148	.034	0.000	0.000	0.000	0.000	0.000	.183	2.867
WSW	.103	.011	0.000	0.000	0.000	0.000	0.000	.114	2.580
W	.091	.011	0.000	0.000	0.000	0.000	0.000	.103	2.617
WNW	.148	0.000	0.000	0.000	0.000	0.000	0.000	.148	2.250
NW	.137	.194	0.000	0.000	0.000	0.000	0.000	.331	4.187
NNW	.046	.046	0.000	0.000	0.000	0.000	0.000	.091	3.900
N	.057	0.000	0.000	0.000	0.000	0.000	0.000	.057	2.250
TOT	1.061	.331	.011	0.000	0.000	0.000	0.000	1.403	3.092
VARIABLE AND CALMS								.696	.410
TOTAL								2.099	2.203

NUMBER OF VALID CATEGORY OBSERVATIONS = 184

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8764

TABLE 2.3-79

FLORIDA POWER AND LIGHT COMPANY
ST LUCIE SITE
PERIOD OF RECORD: 1/1/72 TO 12/31/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

STABILITY A

SECTOR DIR	SPEED (MPH) ADJUSTED TO 10 METER ELEVATION 1-4	4-8	8-13	13-19	19-25	25-31	31+	TOT	AVG
NNE	.022	.547	.558	.068	0.000	0.000	0.000	1.195	6.538
NE	.216	1.48	.606	.148	0.000	0.000	0.000	2.450	6.909
ENE	.274	1.766	.717	.137	0.000	0.000	0.000	2.894	6.927
E	.273	2.552	1.322	.194	0.000	0.000	0.000	4.341	7.563
ESE	.091	1.653	.842	.046	0.000	0.000	0.000	2.632	7.440
SE	.022	.980	2.518	0.000	0.000	0.000	0.000	3.520	8.689
SSE	.011	.174	.590	.217	0.000	0.000	0.000	.992	10.426
S	.069	.046	.034	0.000	0.000	0.000	0.000	.149	5.900
SSW	.034	.102	.103	0.000	0.000	0.000	0.000	.239	7.309
SW	.091	.239	.456	.091	0.000	0.000	0.000	.877	8.920
WSW	.011	.296	.524	.057	0.000	0.000	0.000	.888	9.065
W	.022	.149	.227	.034	.011	0.000	0.000	.443	9.053
WNW	.022	.262	.197	.022	.022	0.000	0.000	.525	8.671
NW	.068	.479	.171	0.000	0.000	0.000	0.000	.718	6.809
NNW	.057	.183	.227	0.000	0.000	0.000	0.000	.467	7.475
N	.034	.148	.820	.102	.011	0.000	0.000	1.115	9.723
TOT	1.317	11.056	9.912	1.116	.044	0.000	0.000	23.445	7.860
VARIABLE								.011	1.800
CALM								.023	0.000
TOTAL								23.479	7.840

NUMBER OF VALID CATEGORY OBSERVATIONS = 2061

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8778

TABLE 2.3-80

FLORIDA POWER AND LIGHT COMPANY
ST LUCIE SITE
PERIOD OF RECORD: 1/1/72 TO 12/31/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

STABILITY B

SECTOR	SPEED (MPH) ADJUSTED TO 10 METER ELEVATION								
DIR	1-4	4-8	8-13	13-19	19-25	25-31	31+	TOT	AVG
NNE	0.000	.092	.023	0.000	0.000	0.000	0.000	.155	6.66
NE	.034	.092	.068	.023	0.000	0.000	0.000	.217	7.86
ENE	.034	.080	.045	0.000	0.000	0.000	0.000	.159	6.30
E	.023	.115	.125	.011	0.000	0.000	0.000	.274	7.95
ESE	.011	.115	.034	0.000	0.000	0.000	0.000	.160	6.29
SE	.034	.125	.068	0.000	0.000	0.000	0.000	.227	6.70
SSE	.011	.023	.092	.011	0.000	0.000	0.000	.137	9.67
S	.011	.034	.023	.011	0.000	0.000	0.000	.079	7.58
SSW	0.000	.034	.057	.011	0.000	0.000	0.000	.102	9.16
SW	0.000	.080	.092	.043	0.000	0.000	0.000	.215	9.80
WSW	.011	.080	.034	.011	0.000	0.000	0.000	.136	7.27
W	.023	0.000	.011	0.000	0.000	0.000	0.000	.034	5.10
WNW	.011	.034	.011	0.000	0.000	0.000	0.000	.056	5.58
NW	.023	.034	.034	0.000	0.000	0.000	0.000	.091	6.18
NNW	.023	.011	.057	.011	0.000	0.000	0.000	.102	9.30
N	.011	.023	.057	.034	.011	0.000	0.000	.136	11.93
TOT	.260	.972	.831	.166	.011	0.000	0.000	2.240	7.90
VARIABLE								.000	.000
CALMS								.023	.000
TOTAL								2.263	.782

NUMBER OF VALID CATEGORY OBSERVATIONS = 199

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8778

TABLE 2.3-81

FLORIDA POWER AND LIGHT COMPANY
ST LUCIE SITE
PERIOD OF RECORD: 1/1/72 TO 12/31/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

STABILITY C

SECTOR	SPEED (MPH) ADJUSTED TO 10 METER ELEVATION								
DIR	1-4	4-8	8-13	13-19	19-25	25-31	31+	TOT	AVG
NNE	0.000	.114	.045	.011	0.000	0.000	0.000	.172	6.780
NE	.067	.113	.079	0.000	0.000	0.000	0.000	.261	6.417
ENE	.033	.080	.057	.011	0.000	0.000	0.000	.182	7.594
E	.056	.138	.114	0.000	0.000	0.000	0.000	.308	7.300
ESE	.069	.114	.056	0.000	0.000	0.000	0.000	.239	6.300
SE	.011	.034	.056	0.000	0.000	0.000	0.000	.101	7.600
SSE	.011	.046	.091	0.000	0.000	0.000	0.000	.148	7.754
S	.023	.023	.114	.011	0.000	0.000	0.000	.171	9.780
SSW	.045	.080	.080	.011	0.000	0.000	0.000	.216	7.200
SW	.022	.138	.080	.011	0.000	0.000	0.000	.251	7.568
WSW	.045	0.000	.034	.023	0.000	0.000	0.000	.102	8.100
W	.034	.046	0.000	0.000	0.000	0.000	0.000	.080	4.886
WNW	.034	.034	0.000	0.000	0.000	0.000	0.000	.068	5.100
NW	.034	.023	.011	0.000	0.000	0.000	0.000	.068	4.950
NNW	.034	.046	0.000	0.000	0.000	0.000	0.000	.080	5.786
N	.023	.056	.093	.011	.023	0.000	0.000	.206	9.749
TOT	.541	1.084	.910	.089	.023	0.000	0.000	2.641	7.308
VARIABLE								.022	
CALMS								.022	
TOTAL								2.697	

NUMBER OF VALID CATEGORY OBSERVATIONS = 237

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8778

TABLE 2.3-82

FLORIDA POWER AND LIGHT COMPANY
ST LUCIE SITE
PERIOD OF RECORD: 1/1/72 TO 12/31/72

ANNUAL HOURLY PERCENT FREQUENCY OF VARTICAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

STABILITY D

SECTOR DIR	SPEED (MPH) ADJUSTED TO 10 METER ELEVATION 1-4	4-8	8-13	13-19	19-25	25-31	31+	TOT	AVG
NNE	.148	.411	.377	.102	.034	0.000	0.000	1.072	8.53
NE	.216	.318	.935	.490	.011	0.000	0.000	1.970	10.34
ENE	.228	.581	1.299	.262	0.000	0.000	0.000	2.370	8.92
E	.298	1.138	1.754	.319	0.000	0.000	0.000	3.509	8.80
ESE	.194	.809	.855	.137	.011	0.000	0.000	2.006	8.46
SE	.170	.662	.433	.056	.023	0.000	0.000	1.344	7.29
SSE	.137	.760	.800	.148	0.000	0.000	0.000	1.845	8.13
S	.125	1.094	1.435	.125	0.000	0.000	0.000	2.779	8.24
SSW	.171	1.162	1.139	.310	0.000	0.000	0.000	2.792	8.52
SW	.217	.878	.672	.148	0.000	0.000	0.000	1.915	7.40
WSW	.091	.332	.432	.011	0.000	0.000	0.000	.866	7.64
W	.171	.182	.160	0.000	0.000	0.000	0.000	.513	6.00
WNW	.251	.421	.148	.068	0.000	0.000	0.000	.888	6.54
NW	.205	.614	.422	.045	0.000	0.000	0.000	1.286	7.52
NNW	.114	.205	.444	.080	0.000	0.000	0.000	.843	8.45
N	.068	.468	.638	.422	.045	0.000	0.000	1.641	10.51
TOT	2.804	10.035	11.943	2.733	.124	0.000	0.000	27.639	8.46
VARIABLE								.125	1.727
CALMS								.159	0.000
TOTAL								27.923	8.38

NUMBER OF VALID CATEGORY OBSERVATIONS = 2451

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8778

TABLE 2.3-83

FLORIDA POWER AND LIGHT COMPANY
ST LUCIE SITE
PERIOD OF RECORD: 1/1/72 TO 12/31/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

STABILITY E

SECTOR DIR	SPEED (MPH) ADJUSTED TO 10 METER ELEVATION 1-4	4-8	8-13	13-19	19025	25-31	31+	TOT	AVG
NNE	.228	.492	.102	0.000	0.000	0.000	0.000	.822	5.074
NE	.296	.752	.613	.125	0.000	0.000	0.000	1.755	6.918
ENE	.376	1.536	.649	.138	0.000	0.000	0.000	2.699	6.581
E	.285	1.537	1.265	.034	0.000	0.000	0.000	3.121	6.890
ENE	.660	2.302	.843	0.000	0.000	0.000	0.000	3.805	5.779
SE	.752	2.596	.696	.011	0.000	0.000	0.000	4.055	5.642
SSE	.729	1.880	.490	0.000	0.000	0.000	0.000	3.099	5.303
S	.296	1.355	.478	.114	.023	0.000	0.000	2.266	6.513
SSW	.433	.900	.204	.068	0.000	0.000	0.000	1.605	6.680
SW	.661	1.391	.114	.011	0.000	0.000	0.000	2.177	4.732
WSW	.957	.934	.092	0.000	0.000	0.000	0.000	1.980	4.187
W	.763	.569	.057	0.000	0.000	0.000	0.000	1.389	3.796
WNW	.513	.808	.137	0.000	0.000	0.000	0.000	1.458	4.696
NW	.410	1.276	.263	.011	0.000	0.000	0.000	1.960	5.512
NNW	.250	.422	.148	0.000	0.000	0.000	0.000	.820	5.416
N	.170	.399	.274	.034	0.000	0.000	0.000	.877	6.653
TOT	7.778	19.148	6.392	.545	.023	0.000	0.000	33.888	5.854
VARIABLE								.182	1.487
CALMS								.627	0.000
TOTAL								34.697	5.602

NUMBER OF VALID CATEGORY OBSERVATIONS = 3046

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8778

TABLE 2.3-84

FLORIDA POWER AND LIGHT COMPANY
ST LUCIE SITE
PERIOD OF RECORD: 1/1/72 TO 12/31/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

STABILITY F

SECTOR	SPEED (MPH) ADJUSTED TO 10 METER ELEVATION								
DIR	1-4	4-8	8-13	13-19	19-25	25-31	31+	TOT	AVG
NNE	.115	.103	0.000	0.000	0.000	0.000	0.000	.218	3.37
NE	.091	.034	0.000	0.000	0.000	0.000	0.000	.125	3.30
ENE	.115	.056	0.000	.011	0.000	0.000	0.000	.182	5.16
E	.080	.070	.011	0.000	0.000	0.000	0.000	.161	3.82
ESE	.103	.068	.022	0.000	0.000	0.000	0.000	.193	4.52
SE	.171	.092	.011	0.000	0.000	0.000	0.000	.274	3.74
SSE	.193	.103	0.000	0.000	0.000	0.000	0.000	.296	3.48
S	.243	.237	.022	0.000	0.000	0.000	0.000	.502	4.00
SSW	.365	.046	0.000	0.000	0.000	0.000	0.000	.411	2.90
SW	.273	.228	0.000	0.000	0.000	0.000	0.000	.501	3.66
WSW	.354	.115	.011	0.000	0.000	0.000	0.000	.480	3.19
W	.240	.091	0.000	0.000	0.000	0.000	0.000	.331	3.03
WNW	.377	.250	0.000	0.000	0.000	0.000	0.000	.627	3.65
NW	.330	.342	0.000	0.000	0.000	0.000	0.000	.672	3.67
NNW	.172	.262	0.000	0.000	0.000	0.000	0.000	.434	3.48
N	.183	.138	0.000	0.000	0.000	0.000	0.000	.321	3.59
TOT	3.405	2.235	.077	.011	0.000	0.000	0.000	5.726	3.59
VARIABLE								.376	1.491
CALMS								.866	.000
	TOTAL							6.970	3.03

NUMBER OF VALID CATEGORY OBSERVATIONS = 611

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8778

TABLE 2.3-85

FLORIDA POWER AND LIGHT COMPANY
ST LUCIE SITE
PERIOD OF RECORD: 1/1/72 TO 12/31/72

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL STABILITY CATEGORIES BY WIND DIRECTION AND WIND SPEED

STABILITY G

SECTOR	SPEED (MPH) ADJUSTED TO 10 METER ELEVATION								
DIR	1-4	4-8	8-13	13-19	19-25	25-31	31+	TOT	AVG
NNE	.011	.012	0.000	0.000	0.000	0.000	0.000	.023	3.25
NE	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
ENE	.011	.011	0.000	0.000	0.000	0.000	0.000	.022	4.45
E	0.000	0.000	.022	0.000	0.000	0.000	0.000	.022	9.70
ESE	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
SE	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
SSE	.011	.011	0.000	0.000	0.000	0.000	0.000	.022	2.85
S	.046	.034	0.000	0.000	0.000	0.000	0.000	.080	1.98
SSW	.102	.011	0.000	0.000	0.000	0.000	0.000	.113	2.41
SW	.102	.046	0.000	0.000	0.000	0.000	0.000	.148	3.17
WSW	.045	0.000	0.000	0.000	0.000	0.000	0.000	.045	2.40
W	.103	.011	0.000	0.000	0.000	0.000	0.000	.114	3.21
WNW	.250	.079	0.000	0.000	0.000	0.000	0.000	.329	3.71
NW	.194	.149	0.000	0.000	0.000	0.000	0.000	.343	3.64
NNW	.057	.056	0.000	0.000	0.000	0.000	0.000	.113	3.81
N	.034	.023	0.000	0.000	0.000	0.000	0.000	.057	3.72
TOT	.966	.443	.022	0.000	0.000	0.000	0.000	1.431	3.45
VARIABLE								.273	1.48
CALMS								.262	0.000
	TOTAL							1.966	2.71

NUMBER OF VALID CATEGORY OBSERVATIONS = 173

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8778

TABLE 2.3-86

FLORIDA POWER AND LIGHT COMPANY
ST LUCIE SITE
PERIOD OF RECORD: 1/ 1/73 TO 12/31/73

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL STABILITY CATAGORIES BY WIND DIRECTION AND WIND SPEED

PASQUILL A

SECTOR DIR	SPEED(MPH) ADJUSTED TO 10 METER ELEVATION 1-4	4-8	8-13	13-19	19-25	25-31	31+	TOT	AVG
NNE	.011	.711	.229	0.000	0.000	0.000	0.000	.951	6.696
NE	.080	1.032	.539	.080	0.000	0.000	0.000	1.731	7.347
ENE	.172	.986	.367	0.000	0.000	0.000	0.000	1.525	6.368
E	.206	2.098	1.169	.011	0.000	0.000	0.000	3.485	7.116
ESE	.080	1.525	.871	.011	0.000	0.000	0.000	2.487	7.191
SE	.023	.837	2.419	.092	0.000	0.000	0.000	3.370	8.817
SSE	.011	.115	.757	.309	0.000	0.000	0.000	1.192	11.239
S	.023	.103	.103	.011	0.000	0.000	0.000	.241	6.836
SSW	.011	.034	.103	0.000	0.000	0.000	0.000	.149	8.280
SW	.011	.057	.229	.023	0.000	0.000	0.000	.321	9.722
WSW	.011	.160	.092	.023	0.000	0.000	0.000	.287	7.167
W	.011	.183	.023	.023	0.000	0.000	0.000	.241	6.900
WNW	.011	.126	.413	.034	0.000	0.000	0.000	.585	9.335
NW	0.000	.504	.252	0.000	0.000	0.000	0.000	.757	7.236
NNW	0.000	.229	.413	0.000	0.000	0.000	0.000	.642	8.181
N	0.000	.241	.860	0.000	0.000	0.000	0.000	1.100	9.103
TOT	.665	8.941	8.838	.619	0.000	0.000	0.000	19.062	7.881
VARIABLE								0.000	0.000
CALMS								0.000	
TOTAL								19.062	7.881

NUMBER OF VALID CATEGORY OBSERVATIONS = 1666

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8724

TABLE 2.3-87

FLORIDA POWER AND LIGHT COMPANY
PERIOD OF RECORD: 1/ 1/73 TO 12/31/73

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL STABILITY CATAGORIES BY WIND DIRECTION AND WIND SPEED

PASQUILL B

SECTOR DIR	SPEED(MPH) ADJUSTED TO 10 METER ELEVATION 1-4	4-8	8-13	13-19	19-25	25-31	31+	TOT	AVG
NNE	0.000	.080	.046	0.000	0.000	0.000	0.000	.126	4.909
NE	.080	.241	.195	.011	0.000	0.000	0.000	.527	6.907
ENE	.080	.206	.160	0.000	0.000	0.000	0.000	.447	6.715
E	.023	.252	.206	.011	0.000	0.000	0.000	.493	6.988
ESE	.057	.206	.160	0.000	0.000	0.000	0.000	.424	6.495
SE	.046	.252	.172	0.000	0.000	0.000	0.000	.470	7.200
SSE	0.000	.011	.034	.057	0.000	0.000	0.000	.103	10.557
S	.023	.023	.057	0.000	0.000	0.000	0.000	.103	7.000
SSW	0.000	.046	.069	.011	0.000	0.000	0.000	.126	8.649
SW	.034	.057	.069	.011	0.000	0.000	0.000	.172	5.712
WSW	.011	.023	.034	.034	0.000	0.000	0.000	.103	8.400
W	0.000	.023	.011	0.000	0.000	0.000	0.000	.034	5.100
WNW	0.000	.023	.023	0.000	0.000	0.000	0.000	.046	8.100
NW	.023	.057	.069	0.000	0.000	0.000	0.000	.149	6.902
NNW	.034	.069	.057	0.000	0.000	0.000	0.000	.160	6.208
N	0.000	.126	.126	0.000	0.000	0.000	0.000	.352	7.773
TOT	.413	1.696	1.490	.138	0.000	0.000	0.000	3.737	6.992
VARIABLE								0.000	0.000
CALMS								.011	
TOTAL								3.748	6.971

NUMBER OF VALID CATEGORY OBSERVATIONS = 327

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8724

TABLE 2.3-88

FLORIDA POWER AND LIGHT COMPANY
PERIOD OF RECORD: 1/ 1/73 TO 12/31/73

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL STABILITY CATAGORIES BY WIND DIRECTION AND WIND SPEED

PASQUILL C

SECTOR DIR	SPEED(MPH) ADJUSTED TO 10 METER ELEVATION 1-4	4-8	8-13	13-19	19-25	25-31	31+	TOT	AVG
NNE	0.000	.057	.011	0.000	0.000	0.000	0.000	.069	3.800
NE	.057	.103	.046	.023	0.000	0.000	0.000	.229	6.660
ENE	.069	.115	.023	0.000	0.000	0.000	0.000	.206	5.137
E	.034	.183	.034	0.000	0.000	0.000	0.000	.252	5.624
ESE	.023	.126	.046	0.000	0.000	0.000	0.000	.195	5.869
SE	.034	.183	.126	0.000	0.000	0.000	0.000	.344	6.707
SSE	.011	.023	.160	.023	0.000	0.000	0.000	.218	9.142
S	0.000	.023	.092	0.000	0.000	0.000	0.000	.115	7.884
SSW	.011	.011	.069	.011	0.000	0.000	0.000	.103	7.307
SW	.034	.069	.046	.023	0.000	0.000	0.000	.172	6.500
WSW	.011	.046	.011	0.000	0.000	0.000	0.000	.069	3.800
W	.011	.057	.011	0.000	0.000	0.000	0.000	.080	4.963
WNW	.034	.034	.046	0.000	0.000	0.000	0.000	.115	7.020
NW	.046	.057	.069	0.000	0.000	0.000	0.000	.172	7.098
NNW	.023	.080	.057	0.000	0.000	0.000	0.000	.160	6.429
N	.011	.115	.252	0.000	0.000	0.000	0.000	.378	7.670
TOT	.413	1.284	1.100	.080	0.000	0.000	0.000	2.877	6.612
VARIABLE								.0 11	1.8
CALMS								.023	
TOTAL								2.912	6.542

NUMBER OF VALID CATEGORY OBSERVATIONS = 255

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8724

TABLE 2.3-89

FLORIDA POWER AND LIGHT COMPANY
PERIOD OF RECORD: 1/ 1/73 TO 12/31/73

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL STABILITY CATAGORIES BY WIND DIRECTION AND WIND SPEED

PASQUILL D

SECTOR DIR	SPEED(MPH) ADJUSTED TO 10 METER ELEVATION 1-4	4-8	8-13	13-19	19-25	25-31	31+	TOT	AVG
NNE	.160	.802	.928	0.000	0.000	0.000	0.000	1.891	7.524
NE	.229	.344	.275	.046	0.000	0.000	0.000	.894	6.520
ENE	.218	.309	.573	.115	0.000	0.000	0.000	1.215	8.057
E	.080	.264	.287	.023	0.000	0.000	0.000	.653	10.635
ESE	.218	.527	.699	.057	0.000	0.000	0.000	1.502	7.835
SE	.264	.585	.573	.057	0.000	0.000	0.000	1.479	7.202
SSE	.149	.619	.493	.046	0.000	0.000	0.000	1.307	7.181
S	.057	.378	.940	.149	0.000	0.000	0.000	1.525	9.135
SSW	.115	.779	1.272	.126	0.000	0.000	0.000	2.293	8.618
SW	.138	.779	.963	.138	.011	0.000	0.000	2.029	8.362
WSW	.149	.562	.321	.138	0.000	0.000	0.000	1.169	7.438
W	.149	.172	.057	.011	0.000	0.000	0.000	.390	4.980
WNW	.149	.229	.069	0.000	0.000	0.000	0.000	.447	4.925
NW	.183	.516	.355	0.000	0.000	0.000	0.000	1.055	6.663
NNW	.172	.734	.722	.011	0.000	0.000	0.000	1.639	7.462
N	.115	.504	.332	0.000	0.000	0.000	0.000	.951	6.958
TOT	2.545	8.104	8.861	.917	.011	0.000	0.000	20.438	7.733
VARIABLE								.0 46	1.2
CALMS								.149	
TOTAL								20.633	7.663

NUMBER OF VALID CATEGORY OBSERVATIONS = 1799

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8724

TABLE 2.3-90

FLORIDA POWER AND LIGHT COMPANY
PERIOD OF RECORD: 1/ 1/73 TO 12/31/73

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL STABILITY
CATAGORIES BY WIND DIRECTION AND WIND SPEED

PASQUILL E

SECTOR DIR	SPEED(MPH) ADJUSTED TO 10 METER ELEVATION 1-4	4-8	8-13	13-19	19-25	25-31	31+	TOT	AVG
NNE	.573	.562	.115	0.000	0.000	0.000	0.000	1.249	4.300
NE	.481	.779	1.043	.011	0.000	0.000	0.000	2.315	6.849
ENE	.367	1.353	.883	.011	0.000	0.000	0.000	2.613	6.524
E	.653	2.040	1.192	.011	0.000	0.000	0.000	3.897	6.302
ESE	.711	3.072	1.364	.057	0.000	0.000	0.000	5.204	6.114
SE	.711	2.293	.963	.023	0.000	0.000	0.000	3.989	5.844
SSE	.722	2.224	1.089	.023	0.000	0.000	0.000	4.058	6.022
S	.504	2.121	.573	.092	0.000	0.000	0.000	3.290	5.917
SSW	.413	1.754	.275	.069	0.000	0.000	0.000	2.510	5.672
SW	.653	1.593	.229	.034	0.000	0.000	0.000	2.510	5.020
WSW	.894	.734	.080	.011	0.000	0.000	0.000	1.719	3.968
W	.825	.791	.080	0.000	0.000	0.000	0.000	1.696	3.958
WNW	.722	1.353	.206	0.000	0.000	0.000	0.000	2.281	4.671
NW	.630	2.338	.355	0.000	0.000	0.000	0.000	3.324	5.276
NNW	.459	.871	.252	0.000	0.000	0.000	0.000	1.582	5.042
N	.459	.550	.298	0.000	0.000	0.000	0.000	1.307	5.415
TOT	9.778	24.427	8.998	.344	0.000	0.000	0.000	43.547	5.638
VARIABLE								.5 27	0.000
CALMS								1.066	
TOTAL								45.140	5.452

NUMBER OF VALID CATEGORY OBSERVATIONS = 3938

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8724

TABLE 2.3-91

FLORIDA POWER AND LIGHT COMPANY
PERIOD OF RECORD: 1/ 1/73 TO 12/31/73

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL STABILITY
CATAGORIES BY WIND DIRECTION AND WIND SPEED

PASQUILL F

SECTOR DIR	SPEED(MPH) ADJUSTED TO 10 METER ELEVATION 1-4	4-8	8-13	13-19	19-25	25-31	31+	TOT	AVG
NNE	.160	0.000	.011	0.000	0.000	0.000	0.000	.172	2.828
NE	.138	.115	0.000	0.000	0.000	0.000	0.000	.252	3.349
ENE	.080	.103	.023	0.000	0.000	0.000	0.000	.206	3.776
E	.103	.103	.011	0.000	0.000	0.000	0.000	.218	3.298
ESE	.183	.046	0.000	0.000	0.000	0.000	0.000	.229	2.452
SE	.264	.218	0.000	0.000	0.000	0.000	0.000	.481	3.516
SSE	.218	.218	.011	0.000	0.000	0.000	0.000	.447	3.846
S	.206	.275	0.000	0.000	0.000	0.000	0.000	.481	3.743
SSW	.321	.309	0.000	0.000	0.000	0.000	0.000	.630	3.471
SW	.160	.218	0.000	0.000	0.000	0.000	0.000	.378	3.634
WSW	.149	.183	0.000	0.000	0.000	0.000	0.000	.332	3.124
W	.344	.057	0.000	0.000	0.000	0.000	0.000	.401	2.700
WNW	.367	.138	0.000	0.000	0.000	0.000	0.000	.504	2.968
NW	.241	.539	0.000	0.000	0.000	0.000	0.000	.779	3.934
NNW	.195	.172	0.000	0.000	0.000	0.000	0.000	.367	3.153
N	.172	.103	0.000	0.000	0.000	0.000	0.000	.275	2.798
TOT	3.301	2.797	.057	0.000	0.000	0.000	0.000	6.155	3.353
VARIABLE								.2 98	1.1
CALMS								.206	
TOTAL								6.660	3.154

NUMBER OF VALID CATEGORY OBSERVATIONS = 581

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8724

TABLE 2.3-92

FLORIDA POWER AND LIGHT COMPANY
PERIOD OF RECORD: 1/ 1/73 TO 12/31/73

ANNUAL HOURLY PERCENT FREQUENCY OF VERTICAL STABILITY
CATAGORIES BY WIND DIRECTION AND WIND SPEED

PASQUILL G

SECTOR DIR	SPEED(MPH) ADJUSTED TO 10 METER ELEVATION 1-4	4-8	8-13	13-19	19-25	25-31	31+	TOT	AVG
NNE	.034	0.000	0.000	0.000	0.000	0.000	0.000	.034	1.867
NE	.011	0.000	0.000	0.000	0.000	0.000	0.000	.011	0.000
ENE	0.000	.011	0.000	0.000	0.000	0.000	0.000	.011	0.000
E	.011	.023	0.000	0.000	0.000	0.000	0.000	.034	0.000
ESE	.034	0.000	0.000	0.000	0.000	0.000	0.000	.034	1.067
SE	.011	0.000	0.000	0.000	0.000	0.000	0.000	.011	0.000
SSE	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
S	.069	.023	0.000	0.000	0.000	0.000	0.000	.092	2.310
SSW	.138	.034	0.000	0.000	0.000	0.000	0.000	.172	2.252
SW	.080	.046	0.000	0.000	0.000	0.000	0.000	.126	3.309
WSW	.080	.046	0.000	0.000	0.000	0.000	0.000	.126	3.082
W	.103	.046	0.000	0.000	0.000	0.000	0.000	.149	3.108
WNW	.126	.160	0.000	0.000	0.000	0.000	0.000	.287	3.000
NW	.229	.103	0.000	0.000	0.000	0.000	0.000	.332	2.979
NNW	.057	.080	0.000	0.000	0.000	0.000	0.000	.138	3.192
N	.057	.011	0.000	0.000	0.000	0.000	0.000	.069	1.600
TOT	1.043	.585	0.000	0.000	0.000	0.000	0.000	1.628	2.683
VARIABLE								.0 92	1.3
CALMS								.092	
				TOTAL				1.811	2.481

NUMBER OF VALID CATEGORY OBSERVATIONS = 158

NUMBER OF TOTAL VALID OBSERVATIONS (ALL CATEGORIES) = 8724

TABLE 2.3-93

ANNUAL ST. LUCIE RELATIVE CONCENTRATION VALUES

FOR SELECTED WORST PERCENTAGES

Accident	Criteria	Distance	1971*	1972	1973
0-2 hours	worst 5%	1555 meters	8.55 D-05**	1.19 D-04	8.15 x D-05
0-2 hours	worst 50%	" "	8.58 D-07		
8 hours	worst	8045 meters	7.97 D-06	7.47 D-06	9.97 x D-06
8 hours	worst 5%	" "	4.83 D-07	4.96 D-07	5.59 x D-07
16 hours	worst	" "	4.26 D-06	3.73 D-06	5.86 x D-06
16 hours	worst 5%	" "	4.48 D-07	4.68 D-07	4.28 x D-07
3 days	worst	" "	9.63 D-07	1.06 D-06	1.70 x D-06
3 days	worst 5%	" "	3.14 D-07	3.42 D-07	2.87 x D-07
26 days	worst	" "	3.68 D-07	3.29 D-07	4.08 x D-07
26 days	worst 5%	" "	1.96 D-07	1.96 D-07	2.02 x D-07

*1971 Period of Record is 3/1/71 to 2/29/72

**D-05 denotes x 10^{-5}

DELETED

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

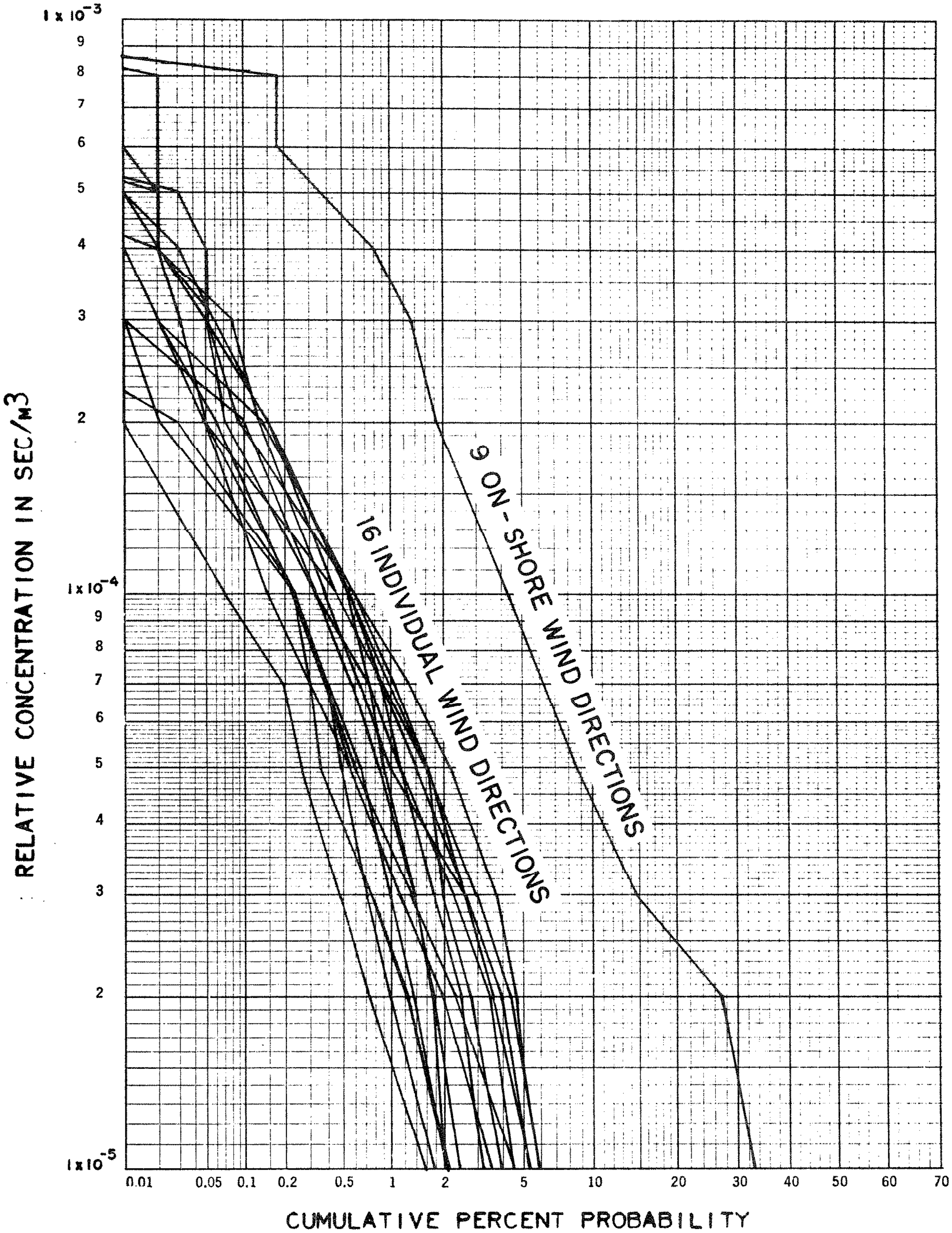
FIGURE 2.3-1

DELETED

EC246531

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

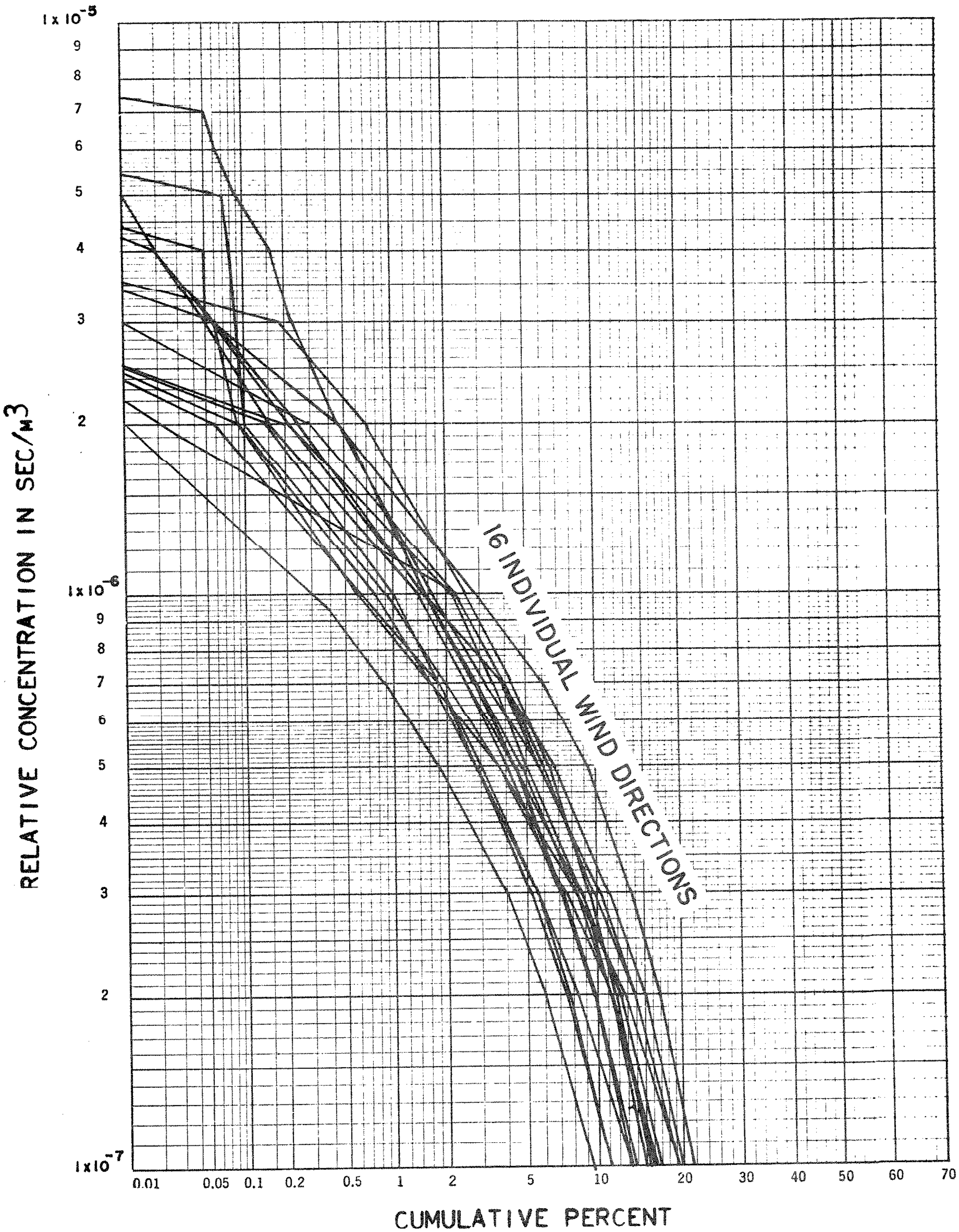
FIGURE 2.3-2



CUMULATIVE PERCENT DISTRIBUTION OF RUNNING HOURLY X/Q VALUES (SEC/M^3) FOR THE TIME PERIOD OF 0 TO 2 HOURS AT THE RESTRICTED DISTANCE OF 1555 METERS. PERIOD OF RECORD: MARCH 1, 1971 TO FEBRUARY 29, 1972.

St. Lucie Plant

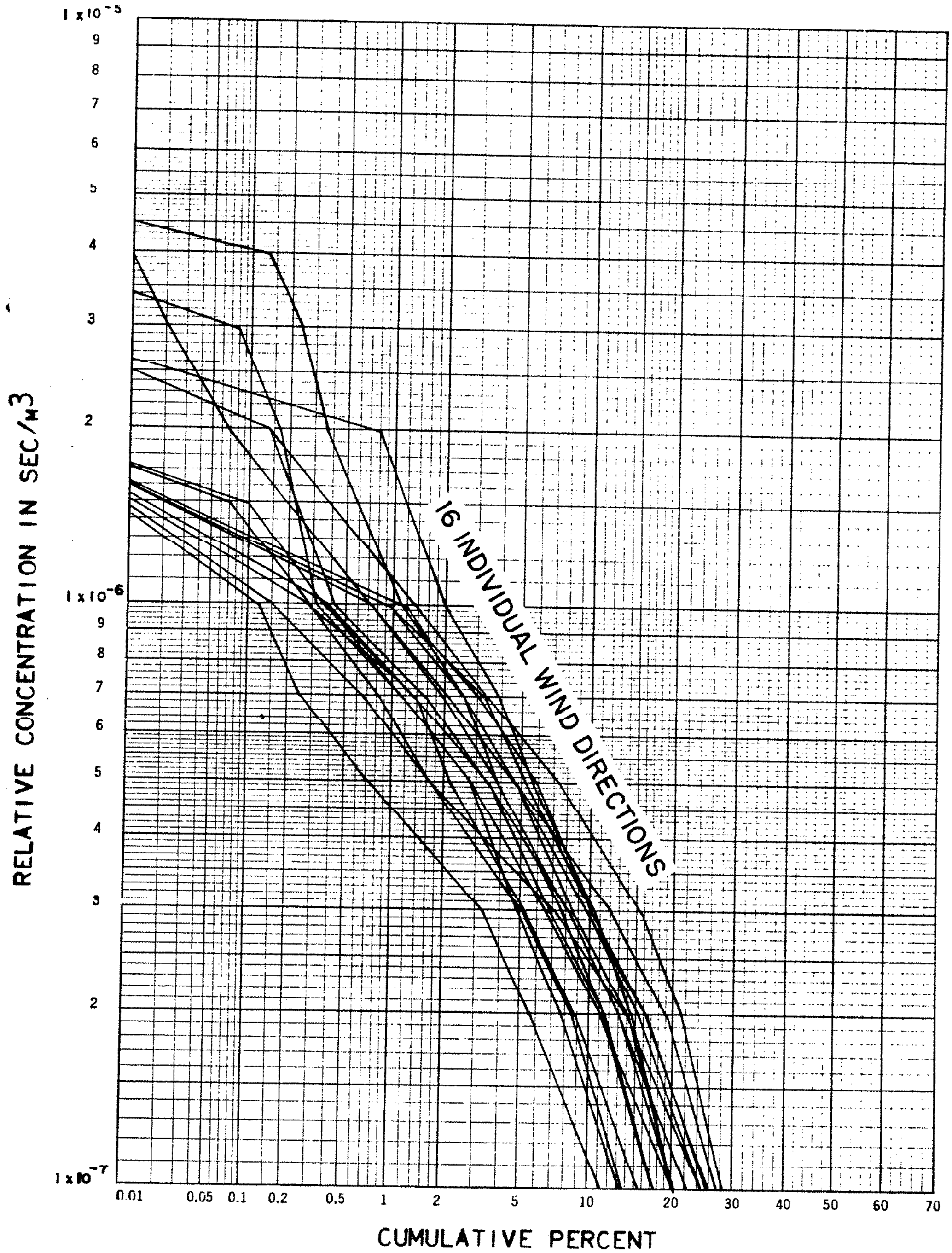
FIGURE 2.3-3



CUMULATIVE PERCENT DISTRIBUTION OF RUNNING HOURLY X/Q VALUES (SEC/M^3) FOR THE TIME PERIOD OF 0 TO 8 HOURS AT THE LOW POPULATION DISTANCE OF 5 MILES (8047 METERS) PERIOD OF RECORD: MARCH 1, 1971 TO FEBRUARY 29, 1972.

St. Lucie Plant

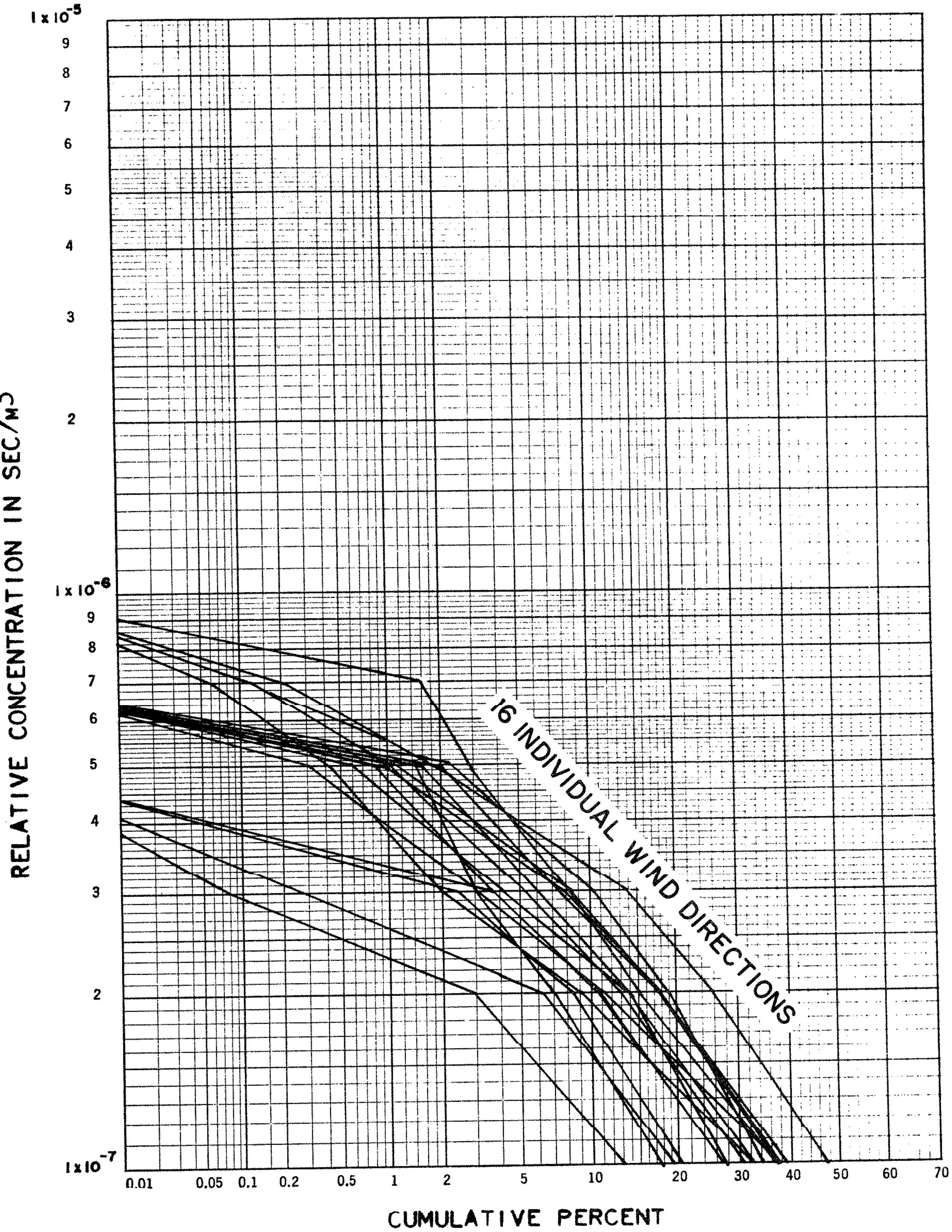
FIGURE 2.3-4



CUMULATIVE PERCENT DISTRIBUTION OF RUNNING HOURLY X/Q VALUES (SEC/M³) FOR THE TIME PERIOD OF 0 TO 16 HOURS AT THE LOW POPULATION DISTANCE OF 5 MILES (8047 METERS) PERIOD OF RECORD: MARCH 1, 1971 TO FEBRUARY 29, 1972.

St. Lucie Plant

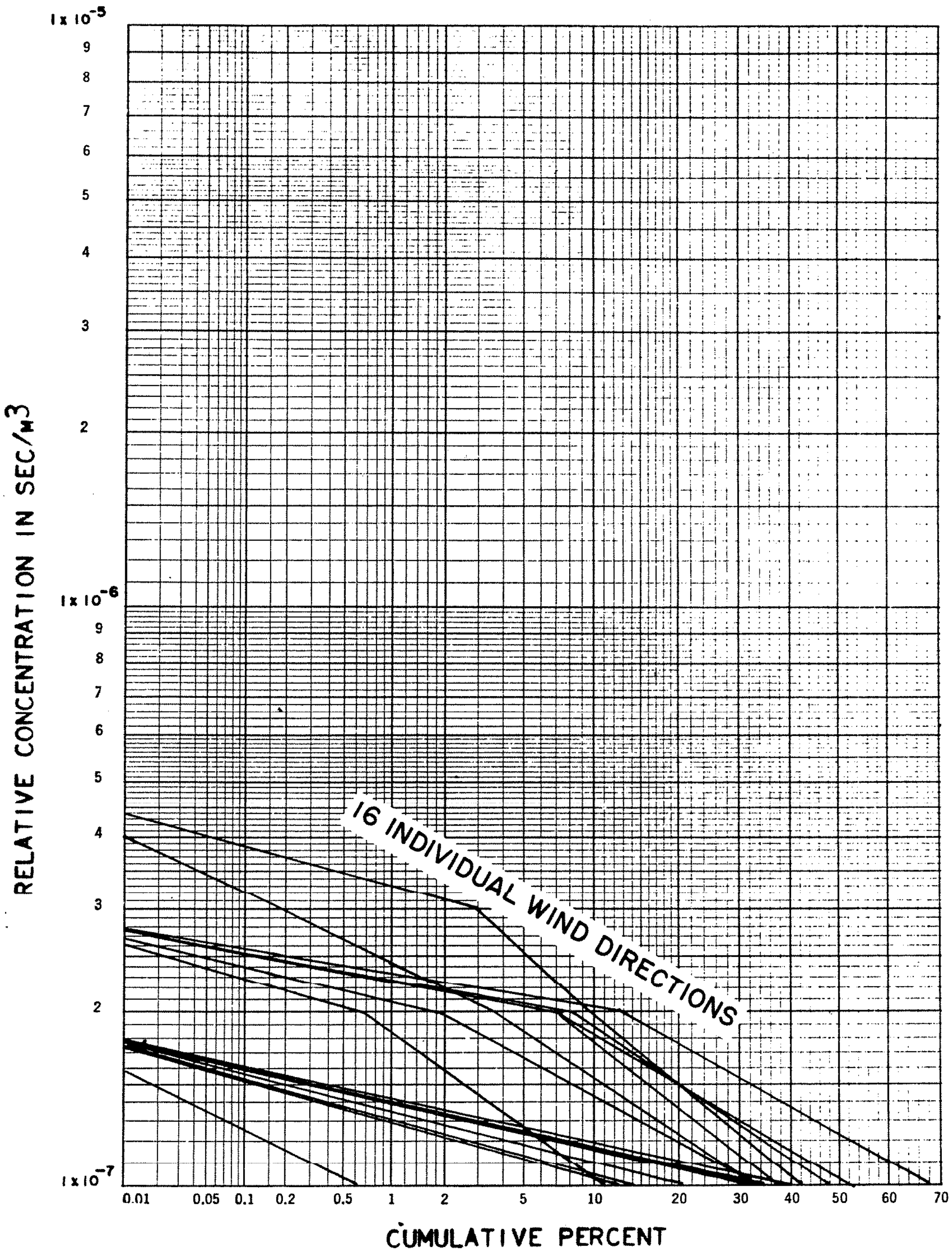
FIGURE 2.3-5



CUMULATIVE PERCENT DISTRIBUTION OF RUNNING HOURLY X/Q VALUES (SEC/M^3) FOR THE TIME PERIOD 0 TO 72 HOURS AT THE LOW POPULATION DISTANCE OF 5 MILES (8047 METERS) PERIOD OF RECORD: MARCH 1, 1971 TO FEBRUARY 29, 1972.

St. Lucie Plant

FIGURE 2.3-6

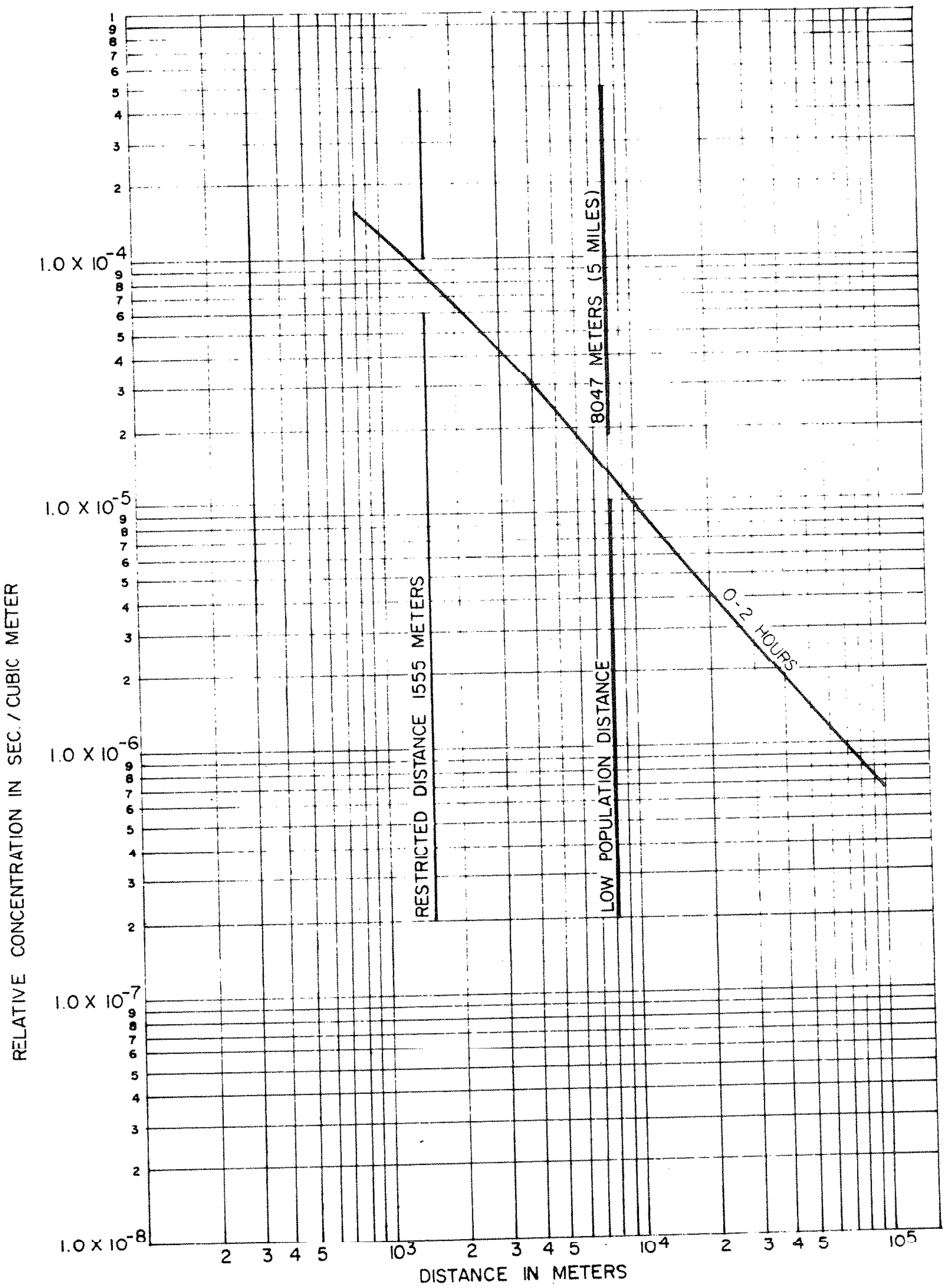


CUMULATIVE PERCENT DISTRIBUTION OF RUNNING HOURLY X/Q VALUES (SEC/M^3) FOR THE TIME PERIOD OF 0 TO 26 DAYS AT THE LOW POPULATION DISTANCE OF 5 MILES (8047 METERS) PERIOD OF RECORD: MARCH 1, 1971 TO FEBRUARY 29, 1972.

St. Lucie Plant

FIGURE 2.3-7

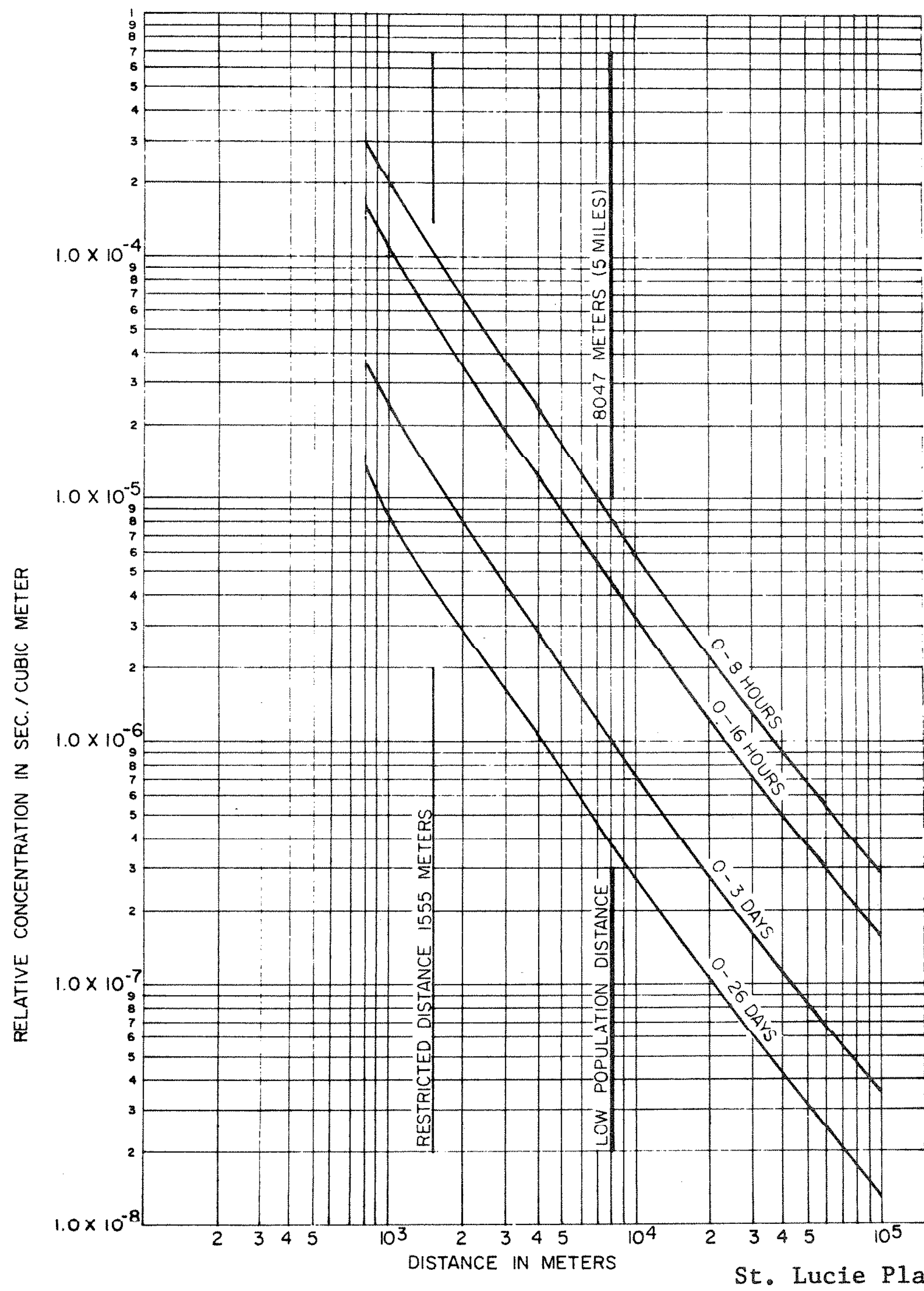
RELATIVE CONCENTRATION IN SEC./CUBIC METER AS A FUNCTION OF DISTANCE DURING THE 0-2 HOUR FIVE PERCENTILE CONDITION



St. Lucie Plant

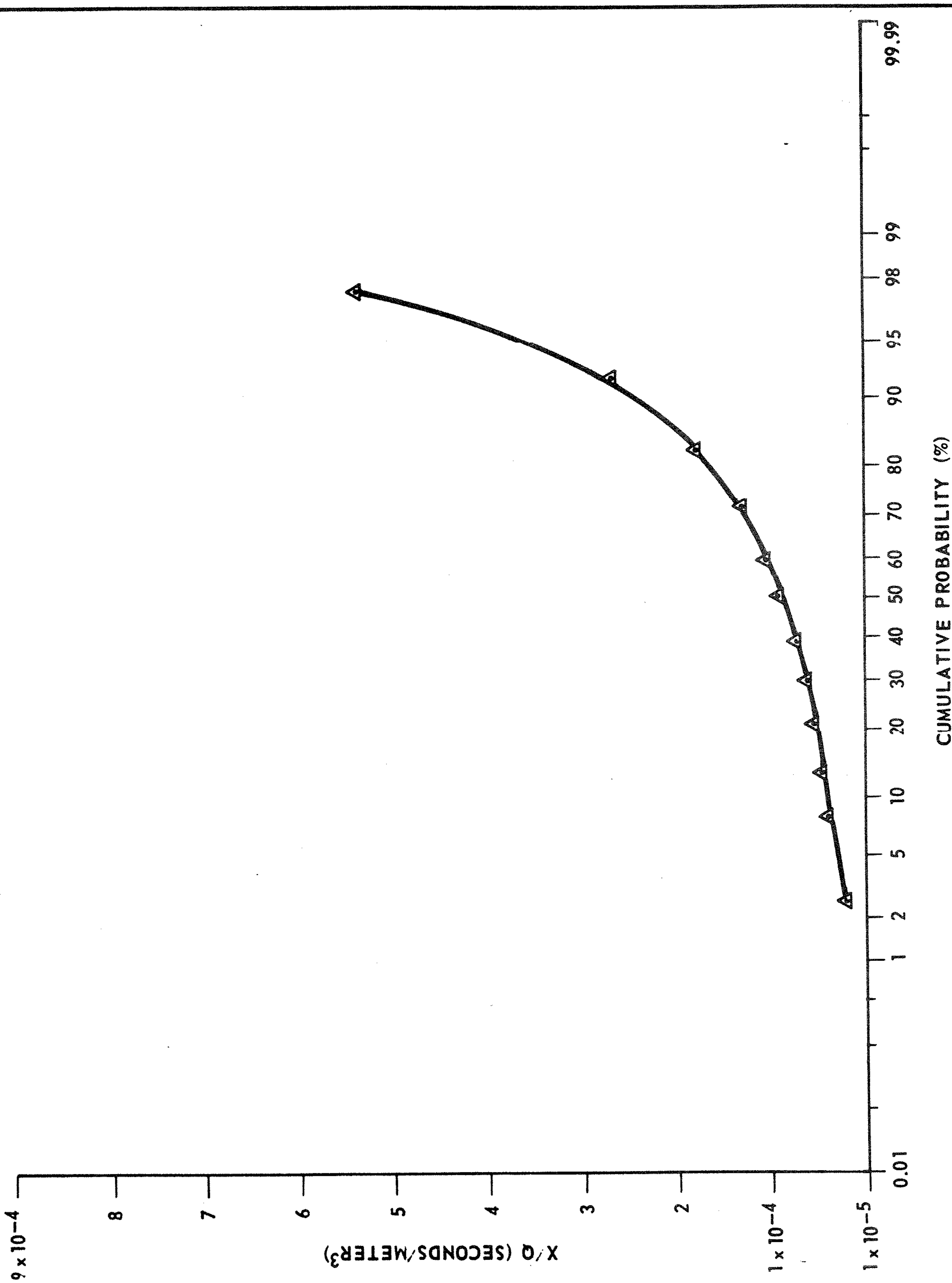
FIGURE 2.3-8

RELATIVE CONCENTRATION IN SEC./CUBIC METER AS A FUNCTION OF DISTANCE DURING THE WORST METEOROLOGICAL CONDITIONS FOR FOUR TIME PERIODS



St. Lucie Plant

FIGURE 2.3-9



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

CUMULATIVE PERCENT DISTRIBUTION OF X/Q VALUES FOR FUMIGATION CONDITIONS

FIGURE 2.3-10

2.4 HYDROLOGY

2.4.1 HYDROLOGIC DESCRIPTION

2.4.1.1 Site and Facilities

Figure 1.2-1 shows the completed facility natural topography and drainage features.

2.4.1.2 Hydrosphere

The surface hydrologic boundaries of the site are the Atlantic Ocean to the east and Indian River to the west. The climate of the site area is humid, sub-tropical. The average annual temperature is 75°F; the average annual precipitation at Fort Pierce is about 62 inches. The average annual water loss due to evaporation in the area is estimated to be 40 to 45 inches[(1)]. Rainfall is seasonal and about 65 percent of the annual amount occurs during the rainy season from June through October. During this period the rain usually occurs in the form of localized heavy showers. High soil permeability provides significant ground water recharge from local precipitation, with little runoff.

According to the information obtained from the Mosquito Control Commission at Fort Pierce, some areas of Hutchinson Island, including the site, were flooded six years ago by means of an eight foot high road-dike, and culverts with flap-valves were provided through the dike. Between the months of September and March the high tide automatically provides the necessary water cover. However, water during the remaining months of the year from the Indian River has to be pumped into the areas.

During our field investigations the water level at the site was approximately one foot above the ground surface. A continuous body of water was found at the site down to the depths investigated.

The United States Coast and Geodetic Survey maps indicate the average range of ocean tide in this area to be approximately 3 feet, and that the Indian River tidal range is approximately 1 foot.

The four county area immediately west of the site was studied by means of aerial photographs and air reconnaissance to determine if there were any surface indications of geologic structure. The counties involved were Indian River County, St. Lucie County, Martin County, and Okeechobee County. The entire region was examined by means of index mosaic sheets. The immediate vicinity of the site (within a radius of about 5 miles) was studied stereoscopically. The island and the adjacent areas were also covered by direct visual observation from the air.

The land-form of the four county area varies from flat to slightly rolling, typical of an emerging coast. The off-shore sand islands extending along the ocean shores of Indian River, St. Lucie, and Martin counties have a flat topography with a maximum elevation of about 15 feet. Across the Indian River on the mainland a narrow beach ridge extends along the shoreline with a maximum elevation of about 40 feet. Behind the ridge, long, shallow, swampy, mucky-bottom swales are found. These swales are called "the Savannas," and they parallel and lie west of the narrow beach ridge bordering the Indian River. Beyond the Savannas, the land is generally flat with a maximum elevation of about 30 feet extending for approximately 20 miles where a gentle slope paralleling the coast line brings the ground elevation to approximately 70 feet within a distance of about 2 miles.

The surface drainage of the region was studied in the following manner:

a. A detailed map of the surface drainage paths was prepared from the photo mosaic (See Figure 2.4-2).
b. A study was made of the low and possible subsidence areas and their intensities.
c. An effort was made to delineate the areas and study the effect of artificial drainage works on the natural drainage patterns.

West of the Savannas and the beach ridge, the following general observations can be made on the surface drainage characteristics of the four county area:

a. Internal (vertical) drainage is the most prominent feature.
b. The two primary surface drainage ways in the area are the St. Lucie River (North Fork in St. Lucie County, South Fork in Martin County) and the Kissimmee River.
c. Sandy ridges and bluffs paralleling the coast line are found inland at several locations.
d. Man-made works such as canals and spoil banks have distorted the natural drainage patterns in some areas.

The primary and secondary drainage paths were located and defined as the first phase of the air photo study. Tertiary (the rill and gully size) drainage paths were found to be almost non-existent. As the second phase of the study, subsidence areas were classified into various categories according to their densities. The shapes of the subsidence areas were noted to vary from perfectly circular to elongated, to long. In general, the subsidence areas form a drainage pattern which can be described as a combination of collinear or parallel channelless basins and swallow holes reflecting past littoral action and the limestone solution action of the flat-lying limestone, respectively.

The St. Lucie River North and South Forks have a braided drainage pattern upstream and a slightly reticular drainage pattern near the mouth. Along the drainage paths of these rivers numerous marsh areas were noted. The densities of the tributaries on the west banks were observed to be higher than those on the east banks. The slight drainage patterns of these rivers indicate a low depth to width ratio and a coarse grained unconsolidated surface soil along their courses. The slight reticular drainage pattern indicates fine grained materials along the lower basins subject to tidal effects. The average range of tide is approximately 3 feet along the ocean and approximately 1 foot in the Indian River by the U.S.G.S. topographic maps.

Elongated to irregular shaped basins without scar channels indicate coarse grained, unconsolidated poorly graded, low density sands. Near the Indian River, areas of elongated bays with inter-related series of channels and oval lakes were observed along with swamps or marshes. Long axes of the ovals were generally found to be parallel to each other and to the coastline.

Some ovals were not drained by surface channels. The indications are that the materials on the edges of these bays are unconsolidated, coarse grained and the materials in the bays are generally soft, unconsolidated silt, peat, or other highly organic matter. These appear to be swales in the original beach deposits.

Several sand ridges were observed inland in the area under study. A well-drained sand slope runs parallel to the coastline along the northeast corner of the Okeechobee County into the western half of Martin County of Lake Okeechobee. This slope represents the limit of the Penholoway Terrace postulated by Cooke (See Regional Geology Paragraph 2.5).

In general, the intensity of the basins were noted to increase midway between these ridges. Otherwise, the basins were found to be randomly distributed. Even though a general southeasterly drainage direction is indicated for the primary drainage ways, the secondary drainage paths showed a random distribution of drainage directions. The local nonsymmetrical tributary distribution of the secondary and primary drainageways appear to point to the influence of the sand ridges. The secondary drainage paths exhibit a generally dendritic drainage pattern.

Topographic and drainage patterns were found to have strong lineations parallel to the coast line (See Figure 2.4-2). The directions of the elongation of great number of basins are also an indication of this general lineation. In Indian River county, the vicinity of the Blue Cypress Lake also very strongly shows a topographic as well as drainage lineation supplementing this same general observation. This particular lineation appears to approximate an extension of the major sand ridge traversing the area of study. This ridge also forms the drainage divide between the Kissimmee River and the St. Lucie River basins. Lineations indicated by the secondary drainageways are considered unreliable since artificial efforts have altered natural drainage patterns extensively at this level of drainage, sometimes re-establishing minor drainage divides.

In conclusion, the air photo studies do not indicate any geologic structures that cannot be related to an emerging coast. The oval shaped swales, topographic and drainage lineations found are typical features of such a coast and the associated past littoral action. The circular subsidence areas (swallow holes) are probably a reflection of the limestone solution activity. None of the features mentioned exist at the site.

2.4.2 FLOODS

2.4.2.1 Flood History

No floods have occurred at the site location according to the specific definition of "flood" in the SAR Format and content guide. Surge flooding is addressed in Sections 2.4.5 and 2.4.6.

Normal tides in the area are semidiurnal having two highs and lows roughly every 23½ hours, of approximately equal magnitude. Information contained in References 2 and 3 gives normal and spring tide ranges at Fort Pierce and St. Lucie Inlets (jetties) and at locations within the Indian River. Those data are given below.

	Mean Range (ft.)	Spring Range (ft.)	Mean Tide Level (ft.)
Fort Pierce Inlet (Breakwater)	2.6	3.0	1.3
Fort Pierce (City Dock)	0.7	0.8	0.3
Jensen Beach	0.5	-	-
Port Sewall	0.5	-	-
St. Lucie Inlet (Jetty)	2.6	3.0	1.3

The average time difference between high water at the jetties and in the Indian River is +1 hour and 51 minutes; between low water occurrences it is +2 hours and 11 minutes, based on Fort Pierce City Dock. In the plant site area high and low tides occur 5 to 10 minutes later. A graphical representation of normal tide relations between the ocean and Indian River is shown on Figure 2.4-3. The equivalent elevations between datums are given by the following relationships:

+1.3 MLW = 0.0MSL (Atlantic Ocean)
+0.3 MLW River = 0.0MSL
+1.0 MLW = 0.0 MLW (Indian River)
0.0 MLW = -1.85 US Coastal & Geodetic Survey (USCGS)
18.0 MLW = +16.15 USCGS

Plant datum has been referred to USCGS bench mark D-34 near Stuart, Florida. Hydrographs for Fort Pierce and St. Lucie Inlets together with those for Indian River at Fort Pierce (City Dock) and in the vicinity of the plant site can be seen on this figure.

2.4.2.2 Flood Design Considerations

Reconnaissance of the beach area fronting the plant site in 1975 indicated conditions of beach slope, dune elevation, and general topography to be approximately as shown on Figure 2.4-4. The dune width is relatively narrow, being on the order of 5 feet wide at its present crest elevation which varies from about 8 to 14 feet MLW ocean, and some 40-50 feet wide at the elevation of the old beach road, remnants of which still exist behind and along the dune line. Beach and mangrove surveys will be conducted in accordance Chapter 13 Section 8.2.3. Permanent monuments will be used to establish a survey baseline behind the beach dune between the North and South FP&L property lines. The perpendicular distance from the survey baseline monuments to the mean high water level will be measured using the stadia survey technique or, as required by field survey conditions, an equivalent technique yielding the same degree of accuracy. The first mean high water beach survey at the St. Lucie site will be completed by June, 1976. The almost 1200 foot salt marsh area between the beach and State Road A1A is covered with dense mangroves. The elevation of State Road A1A

fronting the plant site for a distance of nearly one mile has been raised to between 16 and 18 ft. MLW.

During the Probable Maximum Hurricane, P.M.H., occurrence overtopping of the dune and beach by combined tide and wave action would occur for a period of about 4-1/2 hours duration (T-1-1/2 to T+2 hours). During the period prior to T-1-1/2 hours the erosive effect of waves breaking and overtopping the dune can be expected to lower the dune about 3 feet depending on variations in crest elevation, type of underlying material and other factors. The overflow water would move westward across the adjacent marsh and into Indian River, adding to its general water level. A maximum length of overflow of 6 miles along the beachfront is assumed, based on reconnaissance and the probable tide height distribution alongshore. Overflow would begin along an assumed 1-mile length of dune at time T-1-1/2 becoming progressively longer as tide and wave action builds up to a peak at T+0 hours.

An accurate determination of discharge over a weir under normal conditions is admittedly somewhat complex because the flow pattern varies with changes in head (pp. 5-1, 5-2 Ref. 4). The number of variables involved in any particular problem further adds to this complexity so that a purely analytical approach is extremely difficult at best. The conditions surrounding determination of weir discharge over a reach of beach or dune having varying widths and elevation, which is being eroded by the action of overtopping waves and tidal overflow and thus is constantly changing in shape with time, are many times more complex. The effect of hurricane winds of 110-140 mph, not only upon the nappe but also on water levels in the. discharge area of the weir further complicate resolution of the problem. During the initial overflow periods when the "theoretical" depth of nappe is small it is conceivable that hurricane winds and especially wind gusts would blow the weir surface "dry" at times. The effect of those winds on water levels in Indian River, especially along its eastern shore, will be such as to create a "setdown" condition alongshore lowering water levels in the vicinity of the weir discharge area. In attempting to establish the type weir flow that would be expected to prevail during the P.M.H. occurrence, which in turn would dictate the formula and procedures to be applied, research was made of available literature, model tests, etc. but failed to reveal any such determinations for conditions even remotely similar in complexity to those described above. Based on an evaluation of probable wind effect on weir flow plus the effect of setdown in the discharge area, it was decided that "free flow" would best describe the type flow condition that will occur. For a condition of "submerged flow" to occur it would be required that water levels in the discharge area (Indian River, at or near shore) be sufficiently high so that the discharge is partially under water and that the weir be submerged, or drowned. For the purpose of this report overflow, computations were made using the broad crested weir formula,

$$Q = C\ L\ H^{3/2}$$

Where C is an empirical coefficient
L is the length of overflow section transverse to the flow, and
H is the head on the weir, i.e., the depth of water at shore above the weir crest elevation

Reference 4, pp. 5-24, indicates that for round crested weirs with rounded corners the value of C is at a maximum of 3.087. Computations were made by 1/2 hourly intervals beginning at T-1 1/2 hours, when the P.M.H. tide elevation along the beach in the vicinity of the plant site reaches 10 ft. MLW Ocean, and were extended through the period T+2. After that time tide buildup in Indian River in the overflow area will extend ocean tide elevations and flow to the ocean from the river will take place. An overflow volume hyrograph for overflow from the ocean to the river is shown on Figure 2.4-5. The maximum flow rate across the 6 mile length of overflow section would be about 1.9 million cubic feet per second at about T+0 hours. The maximum overflow velocity in that period would be on the order of 8 feet per second.

2.4.2.3 Effect of Pooling on the Southern Site Property

During a rain fall event, water drainage from the power block occurs through drain lines toward the West and East Storm Water Basins. These basins will accumulate water, gradually drain toward the South Overflow Basin and eventually drain to the Intake Canal through the flood control structures. The peak basin water levels are dependent on their initial level, the rain fall amount, the duration of the rain fall event, and the position of the control structure gate. In general, heavy intense rain fall events occurring over a very short time frame result in higher water levels within the West and East Storm Water Basins.

UFSAR Figures 2.4-8A - 8b depict surge hydrographs for PMH cases. The figures depict ocean elevations which do not necessarily reflect water levels expected on the plant property. For instance, other than affecting drainage of rain fall from the site property, the basin water levels are only affected by ocean level increases if the their level exceeds the intake canal berm height. As the ocean level subsequently recedes, pooling will occur on the southern plant property until drainage occurs over many hours to the intake canal through the flood control structures.

Due to the elevation of buildings housing safety related equipment and/or the sealing of lower elevation penetrations, water levels in the storm basins will not adversely affect safety related SSCs. Plant procedures address potential backflow from these basins through underground systems (e.g., condenser pit, ECCS Tunnel, underground duct bank system).

2. 4. 3 PROBABLE MAXIMUM FLOOD (PMP) ON STREAMS AND RIVERS

2.4.3.1 Probable Maximum Precipitation (PMP)

The storm drainage system is not connected to any Class I structure or safety related item such that flooding of the drainage system as described below could result in flooding of safety related equipment.

In the West Palm Beach area, the maximum twenty-four hour total rainfall was 15.23 inches recorded in April, 1942. Short period rainfall amounts of 6.0 inches in one hour have been recorded west of the West Palm Beach area.

The probable maximum precipitation is 32 inches and would occur over a small 10 sq. mile area during a 24 hour period.

During the PMH, a combination of heavy rain and wave runup could fill the storm drainage system to capacity. For the one hour period when wave runup reaches the catch basins on the plant island, each wave runup will flood the plant area momentarily before running off around the periphery of the plant island. Only the water which is trapped within the high points of each catch basin drainage area (crown of loop roads is El +19.0 ft; top of most catch basins is El +18.0 ft) will flow through the storm drainage system. Since the bottom of doors are at El + 19.5, any water trapped within the individual drainage areas cannot enter the building.

The shield building, reactor auxiliary building and fuel handling building are not connected to the storm water system except for the roof leaders. The diesel generator building is located above the wave runup height.

The roof leaders are designed for a rainfall intensity of 6 inches per hour. Short periods of more intense rainfall would result in water running off the edges of roofs with no adverse effects to safety related equipment. No water buildup on the roofs in excess of 2 inches is possible except for the shield building dome which is surrounded by a 1'-6" high parapet. None of the above conditions adversely affects the structures or safety related equipment.

The roofs of buildings housing safety related equipment handle the runoff of rain in the following manner:

a) Shield building - dome roof with parapet. Drainage by three exterior leaders from parapet to storm water drainage system.

b) Reactor auxiliary building - sloping roofs to area drains. Drainage by various area leaders to storm water drainage system.

c) Fuel handling building - roof slopes from west to east to a gutter and exterior leaders.

d) Diesel generator building - peaked roof slopes to north or south where water runs off the edges and into catch basins at plant grade.

2.4.3.2 Probable Maximum Flood Flow

Due to the extremely shallow nature of the estuarine river adjacent to the site-and to the flat terrain, the hydrological flood flow characteristics are dominated by surge flooding and the associated hurricane winds, detailed in Section 2.4.5.

2.4.4 POTENTIAL DAM FAILURES (SEISMICALLY INDUCED)

No dams are located within the hydrological influence of Hutchinson Island, and none are proposed.

2.4.5 PROBABLE MAXIMUM SURGE AND SEICHE FLOODING

2.4.5.1 Probable Maximum Winds and Associated Meteorological Parameters

The Probable Maximum Hurricane (P.M.H.), by definition, represents an event approaching the physical upper limit of hurricane intensity considered reasonably probable of occurrence meteorologically in a general geographic area. The transportation of that storm on an exact path and forward speed, plus coincidence of maximum hurricane wind-induced tide with peak astronomical tide, assuming the most critical superposition of those and other related components, will result in a wind-tide (or surge) elevation that is considered to be conservatively representative of the maximum-hurricane tide level probable of attainment at a specific location.

Selection of the parameters defining the size, intensity, and forward speed of the probable maximum hurricane was based on data contained in Reference 5. Path of the storm was based on the most critical approach direction for generation of peak hurricane tides at shore, considering the peak hourly average wind pattern in superposition with an offshore depth profile normal, or near normal, to shore. Definition of each of those parameters, which collectively provide the most critical combination for a probable maximum hurricane, is given below.

a) Location - The plant site is located at a latitude of 27 degrees 21 minutes in zone 1 as shown in Figure 1 of Reference 5.

b) Central Pressure (po) - A Central Pressure Index (CPI) value of 26.28 inches of mercury was selected from Table 1 of Reference 5.

c) Asymptotic Pressure (P_m) An asymptotic pressure value of 31.28 inches of mercury was selected from the PMH envelope curve from Figure 6 of Reference 5.

d) Radius to Maximum Winds (R) - A mean radius of maximum winds (RM) value of 10 nautical miles and a large radius of maximum winds (RL) value of 18 nautical miles were selected from Table 1 of Reference 5.

e) Forward Translation Speed (T) - A moderate translation speed (MT) value of 10 knots and a high translation speed (HT) value of 17 knots were selected from Table 1 of Reference 5.

f) Maximum Wind Speed V_x - Maximum wind speeds were selected from Table 1 of Reference 5 for various combinations of radius of maximum winds and translation speeds. The following maximum wind speeds were used in the surge analysis:

1) 154 miles per hour for RM = 10 nautical miles and MT = 10 knots.

2) 158 miles per hour for RM = 10 nautical miles and HT = 17 knots.

3) 154 miles per hour for RL = 18 nautical miles and MT = 10 knots.

4) 158 miles per hour for RL = 18 nautical miles and HT = 17 knots.

Calculation of the maximum wind speed for each combination by use of equations 2 and 3 of Reference 5 resulted in maximum wind speeds of slightly lower values (1 to 2 miles per hour in each case). The effect of this difference on the surge analysis would be insignificant.

g) PMH Path - In order to generate critical tides along the open coast the path of the PMH was selected so that the wind direction of the maximum isovel would be oriented normal to shore and the offshore depth contours. The seabed profile is shown in Figure 2.4-6.

h) Astronomical Tides - An astronomical tide of 3.7 feet was used in the surge analysis. This value will be exceeded by only 10 percent of the spring tides at the Miami Harbor Entrance.

i) Initial Surge - An initial surge of 1.5 feet which was conservatively estimated for this region by CERC, was used in the surge analysis.

j) Bottom Friction Factor - A bottom friction factor of 0.0025 was used in calculation of the surge analysis for the four combinations of translation speed and radius to maximum winds.

k) Parametric Relationships - The isovel pattern and basic wind field data for each PMH combination were computed by methods outlined in Reference 5. Graphical representation of the wind field data (wind speed-theta-radius curves) for the four combinations of translation speed and radius to maximum winds can be seen on Figures 2.4-7(a-d). The data for each case is used as input to the hurricane surge program, "Quasi-Two-Dimensional Open Coast Storm Surge Program," written by B. R. Bodine of the Coastal Engineering Research Center (CERC). The programs were run on a Burroughs 6700 computer.

l) Rainfall - Hurricane-associated rainfall in the east coastal areas of Florida have ranged from very light to extremely heavy. Amounts shown in Table 2 of Reference 6 range from 21.6 inches at Hypoluxo, Fla. in the hurricane of October 11-18, 1910 to 7.7 inches at that station in the storm of September 7-8, 1931. For the PMH a rainfall total of 17 inches was postulated to occur within the 6-hour period just prior to peak tide occurrence. In this manner the maximum effect of that rainfall in Indian River could occur coincident with the other hurricane tide effects of wind, pressure, and wave action.

Following passage of the hurricane center overland, winds over the coastal area will shift from the NE to the SE direction as shown on Figure 2.4-10. At the hour T-10, a wind speed of 51 fps will blow from a northerly direction along the Indian River, while at the hour T+10, a wind speed of 79 fps will blow from a southerly direction along the Indian River. During those times the mean water level in the Indian River may be reasonably estimated from Figure 2.4-8a to be 6 feet MLW ocean, and hurricane tides will build up in each of the reaches between the Waveland, Jensen Beach, and Fort Pierce Bridges. Interchange of tidal flow between those reaches will result as wind setup and setdown occurs at each of the bridges. It is quite probable that a steady state condition would not be attained in any of those reaches due to changing wind intensity and the effect of lateral overflow of both the mainland and beach areas as tides build up to overflow elevations. Evaluation of these conditions in terms of the location of the plant site indicates that the site would be located near the midpoint of the available windtide fetch between Jensen Beach bridge and the Fort Pierce bridge. Therefore, the magnitude of additional rise in water level at the plant site, over and above the 6 feet mean water level in the river, is expected to be small and limited to about 1 foot, due primarily to wave effect plus some cross-wind effect. To illustrate, theoretical wind-tide computations were made for that reach, assuming it to be isolated from adjacent reaches and not subject to the effects of tidal inflow, outflow, and overflow, i.e., it was treated as an isolated and enclosed body of water. Average wind speeds over the reach at the hour T+10 of 53.4 mph (corrected) from the south were used. A step integration computation procedure was employed for the 15.5 mile available fetch using the formula and procedures developed in Reference 8. The resulting theoretical wind-tide profile in that reach is shown on Figure 2.4-9, illustrating the assertion of minimal tide effect at the plant site. As can be seen from this figure the tide at Fort Pierce bridge would reach 8 feet MLW.

2.4.5.2 Surge History

Historic accounts of early hurricanes affecting Florida date back to the 1880's and provide a valuable source of information on hurricane occurrence, intensity, and relative frequency. Although the early records are not believed to be complete some 56 hurricanes have been noted to have passed within a 150-mile radius of the site area since 1876. Forty-two of those have occurred since 1900. Considering a radius of 50 miles of the area, some 20 hurricanes since 1900 have directly affected the island, and give a relative frequency of occurrence of about 1 storm every 3.4 years. Among the more severe storms of record are those of August 1949, and of September 1928, 1933, and 1947. Detailed accounts of those storms can be found in References 6 and 9. In general, record hurricanes passing over or near the study area have had central pressures of from 27.43 inches to 28.17 inches of mercury and peak wind speeds over the ocean approaching 100 mph. The forward speed of storms in Florida latitudes has ranged from 5 to 20 mph, with a few isolated occurrences of looping and stalling on recurving. Northeast storms along the oceanfront of Hutchinson Island have accounted for considerable beach erosion. They are almost an annual occurrence. The most severe recent northeaster occurred in March 1962. Unusually high waves broke on and over the already eroded shoreline causing vertical benching in the dunes of 10 feet in some places. The greatest horizontal recession occurred between Fort Pierce Inlet and Lions Beach Park and has reported to have been about 27 feet (9). No known historical breaching of the dune or highway in the region has occurred.

Recorded tide height for the Fort Pierce and Indian River areas are sparse. In the September 1947 hurricane 6 to 8 foot tides were noted in Indian River in the vicinity of Fort Pierce, Florida. In the August 1949 hurricane a tide height of 8.5 ft Mean Sea Level was recorded at Stuart, Florida, near St. Lucie Inlet. In the September 1928 hurricane, which also affected Fort Pierce a tide of 10.1 ft Mean Sea Level was recorded at Palm Beach, Florida some 50 miles to the south of Fort Pierce. Fort Pierce has sustained frequent floods from hurricane tides generated in the shallow water of Indian River. Detailed accounts of the magnitude of tidal flooding from the above storms and others can be found in References 6 and 9. In general, it appears that a tide elevation on the order of 8+ feet has been the highest observed in Indian River, both at the inlets and at opposite ends of the reach of river.

2.4.5.3 Surge Sources

During passage of the P.M.H. on its selected path, conditions relating to wind directions over the Indian River area between Fort Pierce Inlet and St. Lucie Inlet will be such that tidal inflow will be occurring at the former while tidal outflow is taking place at the latter. While some obstruction to tidal flow will be offered by the bridges across Indian River at the communities of Jensen Beach and Waveland it is felt that this will be minor in comparison to the overall volume of flow in the river. The cross-sectional area of the two inlets is very nearly the same, about 10,000+ ft.^{2} at MLW ocean. Approximately the same normal tide range occurs at Fort Pierce City Dock as occurs near the mouth of St. Lucie River. Therefore, for the purpose of this determination it was assumed that tidal inflow through Fort Pierce Inlet to and through the Fort Pierce bridge to the south would be roughly balanced by tidal outflow through Jensen Beach and Waveland bridges and St. Lucie Inlet.

Water levels in Indian River during the P.M.H. are the sum of normal tide, hurricane rainfall, net tidal inflow and outflow, and overflow of the beach island by hurricane tide and wave action. An evaluation of the change in mean water level in the river was made beginning with a stage of 0.4 ft. MLW river at time T-2 hours (Figure 2.4-3) and adding the assumed incremental hourly rainfall amounts and the computed hourly overflow volumes. For the total storm rainfall contributing to a rise in water surface elevation in Indian River in the storm period a value of 1.42 feet (17 inches) was assumed. The resulting mean water level hydrograph for Indian River is shown on Figure 2.4-13; the peak level reached in the river would be about 10.26 feet MLW river at about time T+2 hours.

The basic parameters defining the P.M.H. as contained in Reference 5 and used in this report, are reasonable and applicable to the Hutchinson Island, Florida area for computing tides and wave action.

The peak P.M.H. tide elevation along the beachfront of Hutchinson Island would be 16.22 ft. MLW ocean (14.92 ft. MSL), and its conservative concurrence along the coast with peak astronomical tide. The ambient tide conditions assumed for the PMH estimate are attributed to two components:

a) Initial water level of 1.5 feet which is conservatively estimated by CERC. Previous studies and calibration of hurricane occurring in the Gulf of Mexico indicate initial surges of greater than 2.0 feet. It is generally accepted that initial surges accompanying hurricanes striking the East Coast of Florida will be less than those on Florida's Gulf Coast. Average figures range from 1.0-2.0 feet, so a value of 1.5 ft was used in the surge analysis.

b. Astronomical high tide of 3.7 feet which is taken in phase with maximum level of the storm surge for PMH was estimated. This value will be exceeded by only 10 percent of the spring high tides at the Miami Harbor Entrance.

The PMH wind tide effect in Indian River at the plant site would be on the order of 1 foot. However, the peak level in the Indian River can reach up to 10.26 feet. MLW due to the overflow of ocean tide at approximately T+2 hours.

2.4.5.4 Wave Action

Estimates were made of wave conditions expected to occur in the PMH as it approaches the site area. An hourly average wind speed of 115 knots was applied in Deep Water Wave Forecasting Relationships contained in Reference 8. A significant wave height of 23.6 feet with a wave period of 9.8 seconds could be generated at the coastline. Waves of that magnitude would break approximately 0.3 mile offshore on reaching a depth of about 30.7 feet. However, the probable wave height that would break about 150-200 feet from shore, runup and overtop the beach, would be on the order of 12.6 feet with wave period of 7.6 seconds. Both the wave height and period were calculated assuming that the dune did not exist.

During a PMH occurrence wave action will be approaching the plant site area from a NE to SE ocean quadrant. Three critical wave-approach directions shown in Figure 2.4-12a were selected for wave computations. For conservatism, the dunes with an average width of 5 feet at their present crest elevation, which varies from about 8 to 14 feet MLW, are assumed to be breached and overtopped by the combined wave and surge action. Basic data relevant to plant layout, fill grade, road elevations, adjacent topography, etc., were taken from Ebasco Drawings Nos. 8770-G-058 and 059, and SK-8770-CH-121. According to those drawings, finished plant grade is elevation 18.00 feet MLW at the lowest point, with paved areas and crown of roads at elevation 19 feet MLW. The plant road completely encircles the containment building.

The magnitude of waves and wave runups reaching the plant fill embankment is a function of various factors, e.g., the elevation grade of State Road A-1-A, the presence and characteristics of native vegetation behind the beach and ahead of the State Road A-1-A, the surge level and the wave-approach angles to the plant fill. According to drawing SK-8770-CH-121 at least a 700-foot wide stretch of native mangrove and marsh vegetation will be allowed to remain between the State Road A-1-A and the beach front. That vegetation which is fairly dense varies from 4 to 7 feet in height above ground which is at elevation 0-2 feet MLW. Again for conservatism, the depth of tide over the vegetative area is estimated to be 13.22 feet (16.22 feet MLW - 3.0 feet MLW).

This is the breaking depth d_b and the corresponding breaking wave across this vegetative area will be 13.22 x 0.78 = 10.15 ft. Since the-State Road A-1-A fronting the plant fill will be raised up to about 18.00 ft. MLW, the breaking wave will runup along the 1 on 3 embankment slope of State Road A-1-A and overtop the road crown. The wave transmitted to the west side of road will be relatively small because the shallower depth of tide on the west side of road cannot sustain a high wave without further breaking. However, this transmitted wave analysis will not be considered due to the fact that the highway embankment may be eroded and breached during severe storms. Assuming conservatively an average erosion rate of 50 yd^3/ft. or 150 ft of horizontal erosion at 16.22 ft. MLW level (Reference 23) for the PMH, we have concluded that wave runups along the three wave-approach directions can be estimated from Saville's method (Reference 22) which is one of successive approximations involving replacement of the actual composite slope by a hypothetical single constant slope obtained from the breaking depth.

2.4.5.5 Resonance

No resonance phenomena have been or are expected to be observed at the site. Neither bay, lake nor canal types of standing waves or seiches occur because of the open, shallow characteristics of the ocean and of the Indian River.

2.4.5.6 Runup

The discharge canal banks West of State Road A-1-A raised to a top-of-dike elevation of +18.00 feet and East of State Road A1A to +19.00 feet. This modification provided significant additional plant operational flexibility to offset some of the effects of pipe fouling problems caused by marine growth.

In the analysis contained in this section, the wave height applied to the discharge canal nose hurricane protection had been determined to be 3.87 feet; a new analysis was performed which resulted in a wave height of 3.99 feet. This small increase can be accommodated by the capability of the existing hurricane protection and, therefore, the dike modification does not affect this analysis.

Analysis A

Wave runup during an occurrence of the PMH can occur from both the ocean side and from the Indian River. The peak computed PMH ocean tide was 16.22 ft MLW*. Three critical wave-approach directions from the ocean side were selected for wave runup analysis on the site areas for Units 1 and 2 (Figure 2.4-12a). The actual topographic profiles for these three sections are presented in Figures 2.4-12b, and 2.4-12c, which justify using Saville's method (Reference 22) on wave runup on composite slopes. Procedures used to determine wave runup criteria along the hypothetical constant slope are those described in Reference 8. Correcting runup for model scale effect is not considered because of the milder slope. The results are summarized in Table 2.4-2 and indicate that the maximum wave runup including wave erosion considerations will not exceed the 18 ft MLW*, which is the lowest point on the plant fill grade.

* Reference Section 2.4.5.9 for updated surge levels and wave runup analysis.

During the PMH occurrence, one critical wave-approach direction from the ocean side to the switchyard is considered to be an angle 60 degrees counterclockwise from the east-to-west traverse line direction of the PMH storm. For such a maximum wind direction, the corresponding surge level is 13.7 feet MLW and the wave runup was calculated to be 3.3 feet by the Saville's method including the assumed erosion rate in Section 2.4.5.4. The topographic profile and the associated results are given in Figure 2.4-12c and Table 2.4-2 respectively. Since the embankment fill grade of the switchyard is at 18 feet MLW, the wave runup will not reach switchyard for this condition.

As shown in Section 2.4.5.3, the maximum water level in Indian River during the PMH is 10.26 ft MLW at the approximate time T+2 hours . If the passage of the PMH on its selected path is not changed, the maximum wind will not attack the plant fill from the Indian River. However, for a conservative demonstration, the PMH may be assumed to suddenly move northward at time T+2 hours, and its maximum wind would hit the plant fill from the Indian River. The significant wave height and wave period for a 140.6 MPH wind and 3 miles of effective fetch are estimated to be 10.8 ft and 5.5 seconds respectively. For a conservative estimate of the ground level and the vegetation height, waves of that magnitude would break and be on the order of 5.66 ft (0.78 x (10.26-3) = 5.66 ft) with a wave period of 5.1 seconds. The wave length is approximately 75 ft, derived by iteration solution of the well-known equation:

$$L = \frac{g\,T^2}{2\,\pi}\ \tanh\ \frac{2\,\pi\,d}{L}$$

Using Reference 8 relations for the Indian River direction:

$H = H_b = 5.66$ feet $T = 5.1$ seconds $L = 75$ feet

$\frac{d}{L_o} = \frac{7.26}{75} = 0.0928$ $\frac{d}{L} = 0.0527$ $\frac{H}{H'_o} = 1.014$

$H'_o = 5.58$ feet $\frac{d}{H'_o} = 1.37$

$\frac{H'_o}{T^2} = \frac{5.58}{26} = 0.215$

From Fig. 3-2 of Reference 8,

$\frac{R}{H'_o} = 1.0$ for a hypothetic constant slope of 15/80 resulting from the

Saville's method applied on the composite slopes near the switchyard.

R = 5.58 feet

Correcting R for model scale effect (9% increase) R = 6.1 feet. The runup elevation would be 10.26 + 6.1 = 16.36 feet MLW, which is less than the 18 feet MLW of the switchyard fill grade.

No flooding will occur as a result of wave runup generated in Indian River as well as from the ocean side.

Analysis B

The analysis for the probable maximum hurricane (PMH) indicated a peak flood surge elevation of 16.22 feet MLW*, and a maximum wave length of 12.6 feet (peak to trough). Figure 2.4-12d provides the site elevations west of highway A1A. On either side of the discharge canal the elevation north of the canal is 14.0 feet MLW and to the south 16.0 feet MLW. To assess the effect of a flood wave traveling up the discharge canal, the maximum wave length that can be supported in the canal, and to the north and south of the canal must be determined. To make this determination the following conservative assumptions are made:

a) The ocean dune is completely washed away along the entire length of the site property.

b) The A1A Highway bridge over the discharge canal is assumed swept away in a manner that would not interfere with a wave moving up the discharge canal.

c) The areas north and south of the discharge canal have been inundated and both are under 2.22 feet of water (this assumes that both the areas north and south of the canal are both at elevation 14 feet MLW).

d) The incident wave attacks the entire property line west of A1A, without any attentuation prior to reaching A1A, i.e., the wave entering the discharge canal at the intersection with A1A is 12.6 feet.

The wave length that can be supported at the center of the discharge canal is much greater than the wave that can be supported north and south of the canal. Thus, the wave as it moves down the discharge canal will spread laterally. The analysis that follows demonstrates the wave reaching the point where the Unit 1 and Unit 2 canals merge will not erode the Class I fill sufficiently to impact Class I structures.

The effect of wave runup along the discharge canal (see Figure 2.4-12e) is discussed as follows. Previous analysis indicated the peak PMH surge elevation of 16.22 ft MLW at the area in consideration. An incident wave height of 12.6 ft is also assumed possible in running up and overtopping the beach. Due to the geometry of the canal and the incident wave direction, the wave, in propagating along the canal, experiences considerable refraction. This is most prominent for waves on top of the inclined dike wall since the water depth gradient there is parallel to the wave front. To investigate the effect of refraction, the wave ray curves are first derived as follow: (See Figure 2.4-12e for definition of terms.)

The wave refraction equation is (from Reference 25, page 258)

$$\frac{d\theta}{ds} = \frac{1}{C}\left((\sin\theta)\frac{\partial C}{\partial x} - (\cos\theta)\frac{\partial C}{\partial y}\right) \quad (1)$$

From Figure 2.4-12e

$$C = C(y) = \sqrt{g(ho + my)}$$

* Reference Section 2.4.5.9 for updated surge levels and wave runup analysis.

Hence

$$\frac{\partial C}{\partial x} = 0 \quad \text{and} \quad \frac{1}{C}\frac{\partial C}{\partial y} = \frac{m}{2}\left(h_o + my\right)^{-1}$$

Furthermore,

$$\frac{d\theta}{ds} = \frac{y''}{(1 + y'^2)^{3/2}} \quad \text{and} \quad \cos\theta = \frac{1}{(1 + y'^2)^{1/2}}$$

By substituting the above expression for $\frac{\partial C}{\partial x}, \frac{1}{C}\frac{\partial C}{\partial y}, \frac{d\theta}{ds}$

and cos θ into equation (1) , the equation eventually reduces to:

$$-2y_1y_1'' = 1 + y_1'^2 \qquad (2)$$

where

$y_1 \equiv y + (1/m)\,ho.$

The solution of equation (2) is

$$x = -Co^2\left(\sin^{-1}\sqrt{\frac{y_1}{Co}} - \frac{\sqrt{y_1}}{Co}\sqrt{1-\frac{y_1}{Co^2}}\right) + C_1 \qquad (3)$$

where Co and C_1 are constants to be determined.

For ho = 16.22 - 14 =2.22 ft, m = 1/3, the ray that starts at x = 0, y = 93 with y' = 0 will reach the position where y = 0 at x = 155.37 by equation (3).If m is changed to 1/6 , x will be 164.8 instead.

By tracing the wave ray originating at the intersecting line of the inclined dike wall and the flat bottom, with m = 1/3 and ho = 16.22-14 = 2.22, it is concluded that the ray only advances 155.4 ft longitudinally before it reaches the top of the dike (See Figure 2.4-12e) . A conservative assumption of m = 1/6 to allow for further erosion of the dike will result in a distance of 164.8 ft (previous paragraph). In either case, there will be ample distance for the above mentioned situation to develop (Figure 2.4-12e).

As the wave ray bends toward the top of the dike, it is expected to break since the supporting wave height at the top of the dike is not more than 1.56 x (16.22 - 14.00) = 3.46 ft even allowing for the maximum adjustment due to the slope (See Reference 24). On a slope of 1:3, most of the refracted wave energy will either be consumed in breaking or transmitted overbank with insignificant reflection. To deal with that portion of waves which originate on top of the flat bottom, it is assumed that the wave energy is conserved between two adjacent wave rays so that the variation of the wave height follows the relationship:

$$\frac{H}{Ho} = Kd \times Dd$$

Where $Kd = \sqrt{\frac{b_o}{b}}$, the refraction coefficient,

and $Dd = \sqrt{C_{GO}/C_G}$, the shoaling coefficient,

except where the supporting wave height controls. Note that b_o is the starting distance and b the resulting distance in consideration between two wave rays while C_{GO} and C_G are the corresponding group velocities. The two wave rays considered here start at the two intersecting lines of the flat bottom and the dike walls (Figure 2.4-12e).

So, b = 203 ft at section B-B and b_o = 20 ft are used.

The group velocities are $C_{GO} = \sqrt{gh_1}$ $\quad C_G = \sqrt{gh_2}$ and with h_1 = (16.22 + 17) = 33.22 and h_2. = [33.33 + (16.22 - 14)] / 2 = 17.72 where the water depth h_2 is taken as an average value at section B-

B. Therefore Kd = 0.31, Dd = 1.36 and H/Ho = 0.423. So H = 12.6 x 0.423 = 5.33 ft. On the other hand, the supporting wave height for the area west of the state road A1A is only (16.22-14) x 0.78 = 1.73 ft. For the wave height attacking the point where the Unit 1 and Unit 2 discharge canals join, the

refraction coefficient $Kd = \sqrt{b_o / b} = \sqrt{\frac{10x2}{198x2}} = 0.226$ is used instead, where 198 ft is the distance from point J to the wave ray considered as indicated in Figure 2.4-12f. Apparently, 2 x 198 = 396 ft is less than the length of the wave front between the two wave rays in consideration when it reaches point J. Therefore, this is conservative. The wave height can then be computed H/Ho = 1.36 x 0.226 = 0.308 and hence H = 12.6 x 0.308 = 3.87 ft. The wave runup distance is computed, based on the concept of composite slope (Reference 8), as follows:

H	= 3.87 ft.	T =	7.6 sec.	d = 3.87/0.78 = 4.96 ft.
L	= 100 ft.	$\frac{d}{L}$ =	$\frac{4.96}{100}$ = 0.0496	$\frac{d}{Lo}$ = 0.015
$\frac{H}{H'_o}$	= 1.307	H'_o =	2.96	H'_o/T^2 = 0.0513
R	= 18 - 16.22 = 1.78		$\frac{R}{H'_o} = \frac{1.78}{2.96}$ = 0.60	

It can be concluded that the composite slope is 1:16. So the horizontal distance from the point where the wave breaks to the maximum runup point is 16 x (18 - (16.22 - 4.96)) = 16 x 6.74 = 108 ft. With a 35:138 slope fronting the plant grade, this will be equivalent to 108 - (138 x 6.74/35)= 81.2 ft. of runup distance from the edge of the plant grade. Even allowing for a conservative value of 150 ft. of erosion on the plant grade, the nearest Class I structure, (the diesel oil storage tank), which is more than 300 ft. from the edge of the plant grade, will not be affected. Refraction pattern for wave rays starting at equal distances on the sloping canal bank is shown in Figure 2.4-12f. The erosion distance of 150 ft value is appropriate for the combined effect of erosion both during and after hurricane wave action.

In substantiating the estimate of the extent of erosion (150 ft) covering both during and after PMH, Table 2 of Reference 23 is used. The argument is as follows:

a) First assume that the maximum surge level of 16.22 feet MLW which results in a wave height of 3.87 feet in attacking point J as indicated in Figure 2.4-12e is sustained for 30 hours. (It must be noted that the peak surge would not last more than about 10 hours).

b) This is then assumed to be followed by a surge level of 7 feet MLW, which will result in a corresponding wave height of 7 x 0.78 x 0.308 = 1.68 ft following the same procedure in obtaining the wave height in (a), for 40 hours.

The durations of both surge levels in (a) and (b) above are conservative approximations derived from the four cases of PMH studies, as indicated in the PSAR at Section 2.4.5.1 and Figures 2.4-8a through 2.4-8d in the UFSAR.

The erosion attributed to (a) above can be estimated by using the test result for wave height = 5.5 feet and wave period = 11.3 seconds. This amounts to 70 feet of erosion by extrapolation.

The erosion attributed to (b), on the other hand, can similarly be estimated by using the test result for wave height = 2 feet and wave period = 16 seconds. This then amounts to less than 14 x 4 = 56 feet.

Overall, this 30 + 40 = 70 hours of wave action will cause less than 70 + 56 = 126 feet of erosion.

The above demonstrates that wave protection is not required at the intersection of the Unit 1 and Unit 2 discharge canal. Should the staff conclude that such protection is required, and state so in their SER, appropriate protection will be provided.* About 200 feet of the intersection (100 feet in each canal) would be so protected. This is considered adequate because the width of the attacking wave is limited by the width of the canal, with the peak limited by the width of the deep section (20 feet). Steam Generator Blowdown Treatment Facility construction and high water levels in the Unit 1 discharge canal during operation would make concrete paving of the discharge canal slopes difficult. Thus equivalent protection will be afforded by a sheetpile bulkhead which will be driven at the top of the slope as shown in Figure 2.4-12g.

Wave protection has been required by the Staff for the nose of the discharge canal created when the Unit 2 canal is cut into the Unit 1 canal. As a minimum the installation of this protection for the Unit 1 portion of the discharge nose will be completed no later than June 1, 1977.* This will not impact the initial commercial operation of Unit 1 because of the following two reasons: i) the probability of a severe or "great" hurricane striking the plant site from a critical angle of approach during the intervening 1976 hurricane season is extremely small; and ii) even if such a hurricane were to occur the topography of the discharge canal "nose" area and physical characteristics of the Steam Generator Blowdown Treatment Facility provide considerable natural erosion protection. These two points are discussed in greater detail below:

According to a NOAA study (36), a "great" hurricane, i.e., maximum one minute wind ≥ 125 mph, striking the coastline, without regard to angle of

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* Condition of License Item G.1 required installation of erosion protection (wave protection) on the nose where the two discharge canals come together. Such protection was provided and reported to the NRC. See Reference 43.

approach, within 50 miles to the south of the St. Lucie site, has a probaability of occurrence of 5% within any given year. However, the probability is substantially less of a severe hurricane striking the coastline from the Atlantic Ocean at nearly a 90° angle, which is the critical angle for maximum storm surge. According to a compilation of severe hurricanes during the period 1873-1967 (42), 5 such storms have passed within 50 miles of the St. Lucie site. The following table shows the pertinent characteristics of these hurricanes.

Date of Occurrence	Angle of Approach w/ respect to coast	Location of Eye w/ respect to site
Aug. 21-26, 1885	0°	10-20 miles offshore
Sept. 6-20, 1928	45°	≤50 miles to south
Aug. 31 - Sept. 7, 1933	60°	≤50 miles to south
Sept. 4-21, 1947	90°	≤50 miles to south
Aug. 23-31, 1949	45°	≤50 miles to south

All of the above hurricanes had maximum sustained winds of approximately 125 mph (one minute average) or 104 mph (ten minute average). Of the 5 severe hurricanes, 4 approached the coast at an angle of 60° or less; these would result in substantially less than maximum storm surge. The hurricane of 1947 is the only severe hurricane that moved into the area just south of the site directly from the east. Over approximately a 100-year period, then, the probability of high storm surge at St. Lucie is approximately 1%. However, the 1947 storm (peak 10 minute winds near 104 mph) did not approach the criteria for the Probable Maximum Hurricane; 10 minute peak winds 158 mph.

In summary, it may be concluded that the probability, during the 1976 hurricane season, of a severe hurricane approaching the site from the critical direction is very small, on the order of 1%. Moreover, there is no record of any such hurricane approaching the severity of that necessary to cause the maximum storm surge, i.e., the Probable Maximum Hurricane.

Even without specific erosion protection of this area, considerable resistance to erosion is inherently provided by non-class structures and the plant arrangement. First, the distance from the "nose" to the nearest class I facility (the diesel oil storage tanks) is 400 ft. The height of this fill is +18 ft. or higher. Thus, erosion would have to advance over a 400 ft. distance and erode 8 ft of material at the diesel oil storage tanks before these Class I facilities could be threatened. Analyses do not indicate erosion of this magnitude. Second, the Steam Generator Blowdown Treatment Facility is in the path between the nose and Class I facilities along which erosion would proceed. The in situ structure provides a barrier to erosion.

The Steam Generator Blowdown Building is built on a 3 ft. thick reinforced continuous concrete mat about 80 ft. by 120 ft. in plan. The foundation is

founded in Class II fill at elevation +10. Two foot thick concrete perimeter walls extend from the top of the mat to elevation +19.5 ft.

Above this level, the north and west sides of the structure consist of an 8 inch concrete wall to Elev. +22.33 ft. Steel siding extends above this level on these two sides. The other sides of the structure consist of 2 foot thick concrete walls extending the elevation +65 ft. A steel superstructure extends to elevation +77 ft. Approximately 3500 cubic yards of concrete is used in the building's construction.

Since erosion analyses indicate that the PMH would not challenge Class I structures, the probability per year of a PMH striking the site at a critical angle is acceptably low, and the in situ structures provide considerable erosion protection, no undue risk results from the proposed schedule for implementing discharge canal nose augmented erosion protection.

2.4.5.7 Protective Structures

Reinforced concrete flood walls have been provided around structures in the plant to elevation +22 ft. MLW ocean. Erosion protection of embankments is predominately provided by gravel and/or articulating concrete block. However, certain areas which are subjected to turbulent conditions are paved with concrete. These areas are the canal slopes and bottom in front of the intake structures, the canal slopes and bottom around the sealwell and 400 ft. downstream of the structure. Additional erosion protection in the form of grout filled fabric is provided adjacent to the intake canal headwall and extending along the canal dike inner slopes 400 ft. downstream of the structure. Also, to satisfy condition of License Item G.1 which requires erosion protection of that side of the discharge canal peninsula associated with Unit 1, a sheet pile bulkhead with a bottom elevation of 31.5 ft. below MLW extending to 22.0 ft. above MLW is provided. All essential equipment on the intake structure is placed at elevation +22 ft. MLW ocean or higher.

In consideration of the conclusions stated in Section 2.4.5.6, there is no need to provide stop log flood protection for any of the plant safety related structures. The maximum calculated wave runup to an elevation of less than +18 ft.* MLW, coincident with the maximum peak surge level of +16.22 ft.* MLW is below the plant grade elevation of +18.5 ft. and well below the minimum elevation of +19.5 ft. of any building openings. Considerable additional flood protection is already afforded by virtue of the layout of the roads, buildings and tornado missile protective structures already permanently incorporated into the plant design. The minimum elevation of the crown of the perimeter plant road along the east face of the plant island is +19.0 feet. The in situ flood protection features are as described in the following paragraphs.

The structures along the immediate east face of the plant island form an effective concrete barrier with respect to inhibiting any wave runup. The barrier is located immediately east of the Reactor Auxiliary Building, and is illustrated on Figures 2.4-12h and 2.4-12i. All of the barrier structures are seismic Category Class I and have been designed to withstand hurricane and tornado wind loadings.

* Reference Section 2.4.5.9 for updated surge levels and wave runup analysis.

The two Diesel Oil Storage Tank foundations have a top elevation of +22.2 feet MLW and are surrounded by a 1 foot thick reinforced concrete retaining wall extending to elevation +24.5 feet. This provides a safety margin against the maximum calculated wave runup* of 6.5 feet (18 feet calculated versus 24.5 feet in situ).

The operating floor of the Diesel Generator Building is at elevation +22.67 feet, thus providing a safety margin of 4.67 feet against the maximum calculated wave runup*.

The Component Cooling Water Heat Exchanger and Pump Area is surrounded by a 2 feet thick reinforced concrete wall extending to an elevation of +23.5 feet. The base elevation of principal equipment is +24 feet leaving a safety margin of 5.5 feet against the maximum calculated wave runup*.

The Reactor Auxiliary Building has 10 ground level openings with a bottom elevation of +19.5 feet as shown on Figure 1.2-13. This provides a safety margin of 1.5 feet above the maximum calculated runup elevation*. Significant additional protection is provided as follows: The north side openings are protected by the Fuel Handling Building, Reactor Building and Steam Trestle. The west side is protected by the turbine building. The south side is protected by the Unit 2 Reactor Building, Steam Trestle, and Fuel Handling Building. The east side is protected by the effective barrier described above. The applicable margin of safety for the east side is therefore approximately 5.5 feet.

* Reference Section 2.4.5.9 for updated surge levels and wave runup analysis.

2.4.5.8 Stalled and Late Season Hurricanes

In response to the Directive of the ASLB in its Partial Initial Decision and questions put forth by the NRC Staff in their letters of 15 April 1975 and 27 June 1975 with respect to the impact of stalled or looping hurricanes on St. Lucie Unit No. 2, the Applicant has studied the possibility of the occurrence, the intensity and duration, and the erosional impact of such a hurricane on the St. Lucie site. Analyses were performed in light of both the historical record and theoretical models for the meteorological and erosional aspects of stalled hurricanes. The results of these analyses are presented in Appendix 2H and Supplement 1 to Appendix 2H to the St. Lucie Plant Unit No. 2 PSAR, Amendment 34, 9 May 1975 and Amendment 37, 22 August 1975. The NRC Staff is currently reviewing Amendment 37. Fuel loading of Unit 1 is not dependent on the results of the Staff's Unit 2 hurricane evaluation. Mangroves (which are not taken credit for in the Unit 2 analyses) afford considerable protection to the nuclear plant island. In addition, meteorological considerations indicate that erosional aspects of severe stalled hurricanes could not manifest themselves until the 1976 hurricane season. Thus there is ample time subsequent to Unit 1 operation to implement any changes that might be imposed by the Staff. It must be noted that Applicant's Unit 2 analyses and review by consulting experts indicate that modifications are not required.

A study has been performed to determine the frequency, intensity, direction of approach, and duration of storm surge of hurricanes affecting the Florida east coast in the vicinity of the St. Lucie Site during the calendar years from 1886 to 1974. The historical record was reviewed and the meteorological considerations involved in hurricane formation and maintenance were analyzed. The analysis presented herein shows that:

a) The frequency of a hurricane making landfall on the Florida East Coast between October 1 and June 30 is small (a factor of 4 less than in the three months of July through September). None have occurred from December 1 through June 30 in the 88 year period of record.

b) The intensity of hurricanes occurring during October and November is much less than in August and September, the primary severe hurricane months;

c) Conditions necessary for severe hurricane development and maintenance are such that a hurricane of PMH proportions is not likely during October or November.

d) The angle of approach of a hurricane during October and November is more likely to be from the southwest (which is of no concern to storm surge and erosion at the site) than the east

e) A late season hurricane moving from the east would very likely be of minimal intensity.

The above findings are physically supported by:

a) A decrease in sea surface temperatures to near the threshold for hurricane maintenance occurs in October. It is below this value until late June.

b) The origin of hurricanes in the late and early seasons in the Western Carribean and their recurvature at lower latitudes restricts the period of time the hurricane has to intensify. This causes most hurricanes which strike Florida from October through June to do so on the West Coast.

The analyses presented in Appendix 2H and Supplement 1 to Appendix 2H assumed no credit for the beach foredune or the dense stands of mangroves east of highway A1A. The mangroves were only assumed to maintain the base or non-eroding plane at the elevation of +4 feet MLW. Thus, no credit was taken for the effects that the dense mangrove stands would have in reducing wave heights and energy as the hurricane waves propogate towards the plant island and ultimate heat sink. The results of the highly conservative erosion analyses indicate ample margin around all critical plant structures. In order to preclude any adverse effects due to littoral drift along the north face of the plant island, three short steel sheetpile groins are provided in the emergency canal vicinity. This was based on the breaking wave heights presented in Figures 10 and 11 in the Erosion Estimates Section of Supplement 1 to Appendix 2H which did not assume any credit for the impact that the dense stands of mangrove trees would have in knocking down the attacking wave heights. It is reasonable to assume that the mangrove areas will remain viable for at least the next year or so. An analysis taking credit for the mangroves demonstrates that the existing mangrove areas to the north and east of the plant Island will by themselves adequately defend the emergency canal provided for initial commercial operation of Unit 1 from wave attack, i.e., the groins are not required to maintain the ultimate heat sink function. (Applicant's consultant Mr. J.M. Caldwell has performed an analysis on the effects of the mangrove areas to the north of the plant island on wave attack of the Emergency Canal. A summary of this analysis is presented herein.) Similar analyses would demonstrate the considerable protection afforded other areas by the site's dense mangroves stands.

<u>Hurricane Frequency</u>

Based on a review of Technical Paper No. 55, "Tropical Cyclones of the North Atlantic Ocean," (39) and, "Monthly Weather Review," (31) 68 hurricanes affected the state of Florida during the period 1886 to 1974.

Thirty-four hurricanes crossed the Florida east coast defined as the tip of southern Florida (excluding the Keys) to the northernmost coastline. Table 2.4-2A summarizes the frequency of occurrence of the 68 storms by month and by the following set of conditions: total number of hurricanes making landfall in Florida; total number of hurricanes with a Florida east coast landfall, hurricanes with an east coast landfall within ±100 nautical miles of the site; and finally hurricanes within ±100 miles of the site and with one minute wind speed greater than or equal to 125 mph (108 mph ten minute wind speed).

Historically, the frequency of all hurricanes hitting Florida (east and west coasts) is highest in September. This frequency decreases by 22% during October. The period November through May contains only 1 hurricane (1935). This is at least in part due to the climatalogical fact that during this period sea-surface temperatures are below the threshold value nominally considered necessary to the maintenance of hurricane strength.

The frequency of all hurricanes making landfall along the east coast shows a markedly different distribution. The decrease in frequency in October of hurricanes making landfall from the Atlantic on the east coast of Florida relative to August and September is quite large (57%) despite the fact that a secondary maximum occurs in early October for all of Florida. The principal cyclogenetic source region shifts to the southwest Carribean during this period and hurricanes in the area make landfall in the Florida Keys or the west coast and are of no concern as regards storm surge and erosion on the East Coast (see Figures 2.4-27 and 2.4-28.) No hurricane made east coast landfall from December 1 to June 30.

Only four hurricanes made landfall on the east coast of Florida for October through June during the 88 year study period and none of which had winds in excess of 105 mph. The last one to do so occurred in 1950. This compares with 14 which had occurred in the primary months of August through September. This decrease is related to a seasonal shift in the origin of the hurricanes. The frequency of storms making landfall within ±100 miles of the site show this same large decrease in October. All but one of these occured to the south of the site (where water temperatures are warmer in October).

Of some relevance in the process of estimating October 1975 to June 1976 hurricane occurrence are recent trends in Florida hurricane climatology. Hurricane genesis and the paths of hurricanes are highly variable due to the many dynamic and thermodynamic interactions of the atmosphere-ocean system. Since the early 1960's, Florida hurricane frequency has been substantially lower than during most earlier years due to a combination of poorly understood factors. This factor of persistence lends confidence to the belief that the 1975 Florida hurricane season will probably (though by no means certainly) continue recent trends and be less severe than the long-term average.

<u>The Influence of Sea Surface Temperature Upon Hurricane Intensity</u>

A factor important to hurricane maintenance is the sea surface temperature. Fisher (28), Miller (30), and Pearlroth (33, 34) have all shown that tropical cyclone intensity and sea surface temperature are closely

related. Fisher documented several cases where hurricanes diminished in intensity when moving over cooler water. These studies and those related to upwelling of cold water (see Q2.28 Appendix 2H) indicate that a hurricane responds rapidly to water temperature changes. Numerical hurricane models have also been used by Ooyama (32) and Rosenthal (35) to investigate the relationship between tropical cyclone intensity and sea surface temperature. Ooyama found up to a 50 percent decrease in kinetic energy of the low level winds when the sea surface temperature was suddenly decreased from 81.5° F to 78°F. Rosenthal also found large decreases in maximum surface winds when the energy release as a result of evaporation was suppressed. It can be concluded that the sea surface temperature is an important parameter related to the hurricane intensity, i.e., the higher the temperature, the potentially more intense the hurricane.

Gray (26) found the potential moist buoyancy ($\partial\theta e/ \partial P$) (which is the gradient in equivalent potential temperature with height) between the surface and middle tropospheric layers is primarily influenced by the change of sea surface temperature. Convection, thus tropical storm intensity, is directly related to this potential moist buoyancy. More intense convection results in stronger upward vertical motions which results in a greater mass flux into the hurricane and therefore a proportionately greater conversion of potential energy to kinetic energy. Sea surface temperatures in the vicinity of the site average 83.85°F in September and 80.3°F in October. This decrease in the mean monthly temperature of 3.55°F would imply a decrease of near 10°C in the θe (equivalent potential temperature) gradient between the surface and 600 mb (15,000 ft. level). Thus, hurricanes occurring in and after October should be less intense until the sea surface temperatures again approach the hurricane sea surface temperature threshold value. South of Miami, sea surface temperatures remain high in October, averaging 83.7°F. More intense hurricanes could be expected to occur in the Keys than in the vicinity of the site. Three of the four hurricanes making landfall on the east coast of Florida occurred to the south of the site in the region of this warm water. The one hurricane which did strike to the north of the site had 10 minute winds of only 66 mph.

The intensity of hurricanes making landfall on the Florida east coast supports the correlation between sea surface temperatures and hurricane intensity. The range of 10 minute wind speed for the storms making landfall along the Florida east coast during October to June was from 66 to 105 mph (average of 76 mph). The average 10 minute wind speed for 8 storms for which data was available that made landfall along the Florida east coast in late August or September was 106 mph (maximum was 146 mph for the hurricane of September 1945). This corresponds to a greater than 50% decrease in the average kinetic energy in the zone of maximum winds from September to October.

This decrease in intensity is also noted for the west coast landfalling hurricanes (which provides a statistically significant data base). The reduced sea surface temperatures near the site coupled with an origin nearer Florida in the late and early hurricane seasons which may not allow sufficient time for intensification leads to significant reductions in hurricane intensity when compared with the months of August and September. No "great" hurricanes (36) have occurred within ±100 miles of

the site during the October through June period whereas four have occurred in the two months of August and September. The great hurricane is defined as having 125 mph winds and again it should be emphasized that in terms of kinetic energy, this is about one half of what would occur with a PMH. The occurrence of the PMH during October through June is probably physically impossible. It is not coincidence that the most severe hurricanes on record (Labor Day, 1935 and Camille, 1969) occurred in September and August, respectively and moved over very warm waters in the vicinity (and to the south) of the Florida Keys where water temperatures frequently exceed 85°F during this time.

In his study of the relationship between hurricane strength and sea surface temperature, Fisher (28) found a strong suggestion that a hurricane will weaken and even dissipate if it moves over water whose temperature is below the threshold value of 79°F. Also, Palmen (40,41) has earlier demonstrated a well defined cut-off of tropical cyclone activity with the 79.7°F ocean isotherm. Sea surface temperatures in the vicinity of the site are below 79.7°F for the months of November through June and just above for October, 80.3°F. The reason water temperature increases of several degrees above 79°F are important to hurricane intensity is related to the non-linear increase in that amount of water vapor (latent fuel) which is evaporated into the air as it spirals inward. As mentioned earlier, an increase in the equivalent potential temperature (measure of the potential and latent energy flux) of 10° C results from an increase in sea surface temperature of 80.3°F to 83.35°F. The monthly average sea surface temperature for June is 79.2°F. (Annual sea surface temperature variation is illustrated by Figure 2.4-33.) It can be concluded that of the few hurricanes which could occur in the site vicinity during the October through June period, they would probably be of minimal, to at most, moderate intensity. A hurricane approaching from the east during October or June (when sea surface temperatures in the Gulf Stream near the site approach the threshold value) would move over cooler waters which should limit its intensity and possibly initiate dissipation.

<u>Direction and Duration of Hurricane Strike</u>

In his study of the tropical cyclone origins, Cry (27) found a shift in hurricane genesis from the lower latitudes of the tropical Atlantic in August and September to the Western Caribbean in October. Gray (26) has related this shift to changes in the Seasonal Genesis Parameter. This variable is a measure of the cyclone formation potential based upon both ocean and atmospheric conditions. A large percentage of the storms which form in the western Caribbean will strike the west coast of Florida as they recurve at lower latitutes. This is shown in Table 1 where 14 of 38 (37%) hurricanes made landfall along the Florida east coast in August and September, and decreased to 4 of 22 (only 18%) in October and November.

Gray's index shows a zero hurricane genesis potential for the months of October through December near the site. (See Figures 2.4-31 and 2.4-32) Thus, a hurricane moving into this region from the south should weaken. Furthermore, the occurrence of a hurricane approaching from the optimal angle for maximum storm surge (from the east) seems remote for anything but a minimal hurricane since it would necessarily have had a trajectory

over water which is of only marginal warmth for hurricane maintenance. This explains why 3 of the 4 hurricanes making landfall on the east coast had a southerly translational component. The one hurricane which had a northerly component only attained wind speeds of 75 mph. No hurricanes have approached the site (or the entire east coast) from the east from October through June. The hurricane occuring in the October through June period which most closely approached the site provides a good example of hurricane behavior during this period. (See Figure 2.4-29 Track 2). The hurricane of 1908 had winds in excess of hurricane strength as it approached the southern tip of Florida from the southwest. While executing a hairpin turn off the Georgia coast it moved over cooler waters and subsequently approached the site from the northeast with only tropical storm intensity. It totally dissipated over Florida.

In view of the reduced intensity and non-critical angle of approach of past October hurricanes, it is reasonable to assume that the potential October hurricane surge level would be substantially lower than that which would occur in September or August.

Hurricanes Making Landfall on the Florida West Coast

It was previously indicated that a hurricane making landfall on the Florida West Coast would not generate a serious surge or erosion threat to the site. This is true for the following 2 reasons:

a) the hurricane would weaken significantly while travelling over land, and

b) the surge resulting from a southwesterly approach overland is much less severe than an over water approach from the east.

A discussion of the effect of land on hurricanes with emphasis on Florida is given below.

Two hurricanes of special interest were investigated to determine the change in wind speeds as they passed across Florida. Hurricane Donna, September 1960, one of the most severe hurricanes to penetrate the Florida mainland, passed northward along the west coast of Florida making landfall at Ft. Myers and then traveled north-northeastward, emerging near Jacksonville. Donna was unusual in that the eye was extremely large, as much as 100 miles in diameter at one point over land. Maximum winds were 113 mph (10 min. average) as the hurricane approached landfall on the west coast and decreased to 98 mph (10 min. average) as it passed off the east coast into the Atlantic near Jacksonville, a 13% decrease in maximum wind speed. The relatively small decrease may be attributed to the exceptionally large eye which resulted in a larger portion of the circulation remaining over water than had the eye been closer to average size.

Hurricane Isbell was investigated because of her passage just 25 miles to the south of the site. Isbell moved in a typical path for October hurricanes, i.e., recurvature took place well to the south of Florida. The path of the hurricane over Florida was from a point between Everglades

and Ft. Myers on the west coast northeastward off the east coast near Juno Beach about 20 miles south of the site. Highest winds were near 90 mph on the west coast and decreased to near 75 mph near the east coast, resulting in a decrease of 17%.

Malkin (29), Figure 2.4-30, shows changes in central pressure values for 13 hurricanes following landfall. The average filling rate was 2.3 mb/hr during the first 12 hours or 30% per 12 hours. A moderately rapid moving hurricane would take about 12 hours to cross the peninsula. The effect of the water surrounding the peninsula upon hurricane filling is apparently minor since the filling rate of 4 Florida hurricanes which crossed normally was only 0.2 mb/hour less than those making landfall at non-peninsular areas of the Gulf or Atlantic coasts.

As has been discussed in Q 2.28, Appendix 2H to the St. Lucie Unit No. 2 PSAR, a PMH would necessarily have a small radius of maximum winds to maintain maximum intensity and would experience fully the effects of land upon its intensity. Since the maximum convection is located near the hurricane eye wall, the primary area of PMH energy inflow would be concentrated in a relatively small area (less than 100 miles in diameter) and would be relatively unaffected by the water surrounding the Florida peninsula during its over land passage.

It may be concluded that a servere hurricane for October through June, moving in a typical path from southwest to northeast would sufficiently deintensify and this coupled with a trajectory towards the Atlantic ocean, results in a lower surge than that caused by a moderate or average hurricane making landfall on the east coast.

PROTECTION OF EMERGENCY COOLING CANAL AFFORDED BY PRESENT MANGROVE SWAMP

Waves reaching the Ultimate Heat Sink during a PMH or stalled PMH would approach from the north quadrant (NW through N to NE). In making the approach, the waves at present would have to transverse extensive areas of vigorous mangrove swamps.

A study of the literature on dissipation of wave energy by vertical piling shows that the rate of dissipation is affected by the wave-height wave-length ratio of the wave and by the number of pilings (38). Assuming the mangroves to be similar to an array of pilings, it is possible to estimate the attenuation of the wave passing through the mangrove areas.

For the purpose of this analysis the mangrove trees were assumed to be growing on 10-foot centers, and a computation was made for waves approaching the area from the northeast quadrant. The waves transverse not less than 2500 ft. of mangrove swamp areas covered with thick mangrove growth standing up to elevation +20 and higher.

From the technical reference above (38), it can be computed that with the mangrove trees on 10-foot centers, incident waves would lose 10 percent of their height in traversing 480 feet of the mangrove area. In traversing 2500 feet of mangrove swamp, the waves would be reduced by the fifth power (2500 / 480 ≈ 5) of the reduction factor. Using the 10% reduction in height, the reduction factor would be 1.000 - 0.10 =

0.90, which raised to the fifth power is approximately 0.60. This would result in a reduction of the wave height to 60 percent of the initial height.

Applying the calculated 60 percent reduction to the waves from the north-east quadrant for the NRC stalled hurricane without full land deintensification (see Figure 11 of the Erosion Estimate Section Supplement 1 to Appendix 2H of the St. Lucie Unit No. 2 FSAR) and using the methods described in that section, it is found that the effects of the mangroves would be to reduce the littoral drift moving westerly along the north face of the plant island towards the ultimate heat sink from 17,000 cubic yards down to 5500 cubic yards. This 5500 cubic yards of littoral drift would not adversely impact the ultimate heat sink provided for the initial commercial operation of St. Lucie Unit No. 1. It can therefore be concluded that the existing mangrove trees to the northeast of the plant island would by themselves adequately defend the emergency canal provided for the initial commercial operation of Unit 1 during any of the various PMH cases.

2.4.5.9 Updated Surge Level and Wave Runup Analysis

As part of the Unit 2 FSAR Operating License (OL) review, an updated site specific analysis for determining flooding conditions resulting from surge level and erosion from wave attack was performed. The flooding analysis was based on the steady state Probable Maximum Hurricane (PMH) while the erosion analysis assumed a stalled PMH. This updated analysis showed that the beach dunes and mangroves were not needed to protect safety related structures and equipment from PMH surge and erosion damage. Since these analyses pertain to the plant site, they are applicable to Unit 1.

For the steady-state PMH, the maximum surge level is estimated to be 17.2 feet MLW (reference St. Lucie Unit 2 UFSAR) which is well below the flood protection level of 19.5 feet MLW. Considering the effects of wave runup for the maximum postulated surge level of 17.2 feet MLW, the maximum water elevation varies up to 18.8 feet MLW waves from the north breaking against eroded parking lot except for waves from the east over eroded areas (dunes and mangroves), which propagate up the discharge canal approaching the nose where Unit 1 and Unit 2 canals join, where a maximum water elevation (surge level and runup) of 28.0 feet MLW is postulated. The discharge canal nose area is protected by a steel sheet-piling barrier with its top at elevation 22 feet MLW. During the peak surge water level of 17.2 feet MLW, the refracted wave will break on the slope in front of the sheet piling and result in a wave runup of about 11 feet on a hypothetical extension of the slope of the canal nose. Overtopping of the barrier is expected and the resultant water behind the barrier will be drained off into the discharge canals. The temporary flooding around the nose is of no concern since there is no Category I structure located in that part of the plant site. For this surge and wave runup analysis, it is assumed that the foredunes are completely washed away along the entire east coast of the site and that the incident wave propagates from the ocean without any attenuation prior to reaching Highway A1A (reference St. Lucie Unit 2 UFSAR). Therefore, the presence of mangroves or the elevation of the beach dunes do not affect the analysis and neither is required to mitigate the consequences of the design basis steady-state PMH.

The St. Lucie Unit 2 Updated Final Safety Analysis Report, contains a re-examination of the stalled PMH for the Hutchinson Island site. The analysis assumed that the existing beach dune fronting the plant site would be eroded to the existing ground level elevation of 4 to 5 feet MLW during a severe hurricane once the surge level exceeded 8 feet MLW. The 4 to 5 feet MLW is the elevation of the root system of the dense vegetation which fronts the plant site. No credit was taken in the latest (Unit 2 UFSAR) analysis for the ability of the vegetation to reduce wave heights or rate of erosion, nor was any credit taken for dune elevations. Therefore, the erosion estimates are valid regardless of the condition of the beach dunes and/or dense vegetation.

The NRC staff previously reviewed the updated site specific analysis and issued the St. Lucie Unit 2 Safety Evaluation Report (NUREG-0843) which concluded in Section 2.4.7 that the analysis for the site is in conformance with the procedures in Regulatory Guide 1.59, "Design Basis Floods for Nuclear Power Plants," and that flooding does not present a credible threat to the plants with the exception of wave splash and spray at one entrance of the Fuel Handling Building. Also, the erosion analysis is considered to be conservative provided that the highway embankment and beach material assumed to exist and limit the breaching wave heights will be in place at the start of the storm. Note that these features are not required to maintain the Design Basis, only to maintain conservatism in the analysis.

2.4.6 PROBABLE MAXIMUM TSUNAMI FLOODING

No historically recorded or observed tsunamis have occurred along the Florida coastline.

2.4.7 ICE FLOODING

No sub-freezing air temperatures have occurred in the Hutchinson Island area of sufficient duration to cause ice formation on fresh or salt water bodies.

2.4.8 COOLING WATER CANALS AND RESERVOIRS

The intake and discharge canals are sized for a gross flow of 2300 cubic feet per second with low tidal conditions resulting in a maximum velocity of less than 1 foot per second. Both canals are diked. Refer to subsection 9.2.3.2 for further description.

During the P.M.H., wave runup from ocean generated waves will top those dikes located to the south and east. Canal dikes immediately to the west of the plant and in the vicinity of the intake structure will not be effected by ocean waves. Deterioration of the dikes will be relatively minor, less than a one foot reduction in height, because the natural dune line will absorb most of the force of the waves.

The canal slopes are compacted to 90 percent Modified AASHO Density and covered with gravel or are covered with an articulating concrete block and have a low slope (3 horizontal to 1 vertical) so that there is no chance of serious damage due to normal, wind generated wave action in the canals.

The canal is not seismic Class I except in the area of the intake structure. Refer to Section 2.5.5 for a discussion of behavior of the channel slopes under seismic loading.

No reservoirs are provided in the plant cooling water system.

2.4.9 CHANNEL DIVERSIONS

The intake canal takes water directly from the Atlantic Ocean through subaqueous intake water pipes which run under the beach to the east of highway A-1-A. In the event of blockage of the intake canal or pipes, emergency cooling water will be taken from Big Mud Creek through the emergency cooling water canal. This emergency source of water is designed to withstand the design basis earthquake or tornado conditions. The ultimate heat sink is described in Section 9.2.7.

2.4.10 FLOODING PROTECTION REQUIREMENTS

Flood protection criteria for safety related structures, systems and components are discussed in Section 3.4.

2.4.11 LOW WATER CONSIDERATIONS

2.4.11.1 Low Flow in the Indian River

The Indian River is generically a salt water estuary rather than a river, and as such, does not have a sustained flow. It is not used as a water source or outlet with respect to the St. Lucie Plant site, except for a possible short-term use under the emergency condition described in Section 9.2.7.

2.4.11.2 Low Water Resulting from Surges

Extreme low tides have been observed on numerous occasions in coastal bays and rivers bordering on or emptying into the Atlantic Ocean or the Gulf of Mexico. They are the result of hurricane passage far enough north or south of a coastal area (depending on whether east or west coast of Florida) to have sustained winds oriented either offshore (for bays and open coasts) or along the primary axis of coastal rivers. They may also occur as a result of a storm remaining stationary, or looping just offshore, so that sustained offshore winds of long duration prevail over the coastal area. For example, at Tampa, Florida tide levels at Seddon Island were lowered to -5.1 feet MSL during passage of the October 1944 hurricane. Similarly tides at Cedar Keys, Florida were lowered to -4.5 feet MSL as a result of the August 1949 hurricane, and to -3.1 feet MSL from the famed "Cedar Key Hurricane," which looped just offshore of the area.

At low tide (See Figure 2.4-3) the average depth in the river is about 4 feet. Under 50-60 MPH winds, water levels would be lowered to no more than -0.2 feet MLW at the plant site.

The worst condition could be hypothetically postulated as the PMH maximum wind (140.6 MPH) prevailing over the Indian River at the low tide level. It is estimated that an extreme low tide elevation of -3.0 feet MLW in the River can be expected to occur at the plant site.

For relatively straight coastlines such as Hutchinson Island, the offshore winds in the left quadrants will seldom provide sufficient offshore transport to depress the water level below the initial level because of the leading alongshore currents developed in the leading left quadrant of the storm. However, to estimate the possible extreme low water elevation due to a PMH it was assumed that the alongshore currents were negligible and did not offset the offshore transport. The winds and wind directions in the left quadrant of the PMH were estimated by construction of the entire isovel pattern. Since the hurricane is basically a low pressure system (causing a rise in the water surface) the pressure rise was subtracted from the total maximum wind stress associated with the winds of the left quadrant. Neglecting initial surge and astronomical tide it was found that the extreme low tide in the open coast due to the PMH was -3.0 feet below MLW. This will not affect pumping capabilities.

2.4.11.3 Plant Requirements

The required minimum cooling water flow varies with plant operating mode as follows:

a) Normal operation:

1)	Circulating water to main condenser	- 484,000 gpm
2)	Intake cooling water to component cooling heat exchangers	- 16,500 gpm
3)	Intake cooling water to turbine cooling and steam generator blowdown open cooling heat exchangers	- 12,500 gpm

b) Emergency conditions:

1)	Intake cooling water to component cooling heat exchangers	- 14,500 gpm

The intake structure bottom elevation is -31 ft. MLW. The circulating water pump suction elevation is -16 ft. The intake cooling water pump suction elevation is -18.5 ft. Minimum submergence is 6 ft. for the circulating water pumps. For the intake cooling water pumps, the minimum submergence is 4 ft. for 14,500 gpm flow. Two pumps are required for normal operation and one pump for emergency conditions.

The minimum water level therefore is -14.5 ft. for normal operation and emergency conditions based on the requirements of the intake cooling water pumps.

The intake canal is designed to provide an even distribution of intake water to all four bays of each intake structure. The effluent from the intake cooling water system is released to the discharge canal at elevation –1.5 ft. where it mixes with the circulating water flow and is discharged into the ocean. The pump arrangement is shown on Figure 2.4-26.

2.4.11.4 Dependability Requirements

Redundant seismic Class I level instrumentation is provided to alert plant operators of impending low water level in the intake canal. During operation a low level alarm is actuated in the control room whenever the level in the intake structure falls below -9 ft. MLW with permanent UHS barrier operation. Big Mud Creek serves as the source of emergency cooling water in the event water flow from the Atlantic Ocean through the intake canal is interrupted. Sufficient water is available from Big Mud Creek for long-term emergency cooling (see Section 9.2.7). To ensure a supply of water for safe shutdown or accident conditions, the emergency cooling water flow from Big Mud Creek is initiated from the control room via the permanent UHS barrier whenever the intake canal level falls below -10.5 ft. This provides a margin of 4 ft. about the minimum level for emergency operation of the intake cooling water pumps.

The analysis of availability of emergency cooling water from Big Mud Creek during extreme low water conditions, presented in Section 9.2.7.3, demonstrates that even during low water conditions the minimum submergence requirements of the intake cooling water pumps can be maintained. Since the suction bell of an intake cooling water pump is located at elevation -18.5 ft, there should exist no flow condition which could result in surging and consequent interruption of cooling water flow.

The following surge sources have been analyzed to determine their effect on the availability of water supply during periods of low water in the intake:

a) The stoppage of circulating water flow
b) The starting of the circulating water pumps
c) Wind-induced waves

The positive surge caused by the instantaneous stoppage of flow with water at a conservatively assumed initial elevation of -11.5 ft. has been calculated to be 1 ft. using equations available in Ven Chow's text "Open-Channel Hydraulics," McGraw-Hill, 1959. The negative surge caused by pump startup will be less than the positive surge determined for the above situation because the pump startup does not occur instantaneously.

Waves with a height of 3 ft. could develop in the intake canal due to 120 m.p.h. winds and a wind direction parallel to the canal. Because the intake structure is located to the side of the canal and not at the end, the maximum wave height that could reach the front face of the intake is approximately 2 ft. Surging in the intake would be further reduced due to the damping caused by the submerged inlet to the pump chamber and the divided bays of the structure.

No adverse conditions affecting the supply of water to the intake cooling water pumps are considered possible from these surge sources.

2.4.12 ENVIRONMENTAL ACCEPTANCE OF EFFLUENTS

2.4.12.1 Dilution of Circulating Water Discharged to the Environment

The circulating water discharge system downstream of the discharge canal consists of a 12 foot diameter pipe about 1500 feet long running easterly from the discharge headwall structure to the transition section of the "Y" port diffuser. Refer to Figure 9.2-1b. Each port of the diffuser is 7.5 feet in diameter and is designed for a flow of 575 cfs (13 fps discharge velocity). The ocean depth at the point of discharge is -18 feet (MLW). The discharge end is encased in tremie concrete and sheet piling. The center line of the discharge ports is at elevation - 34 feet MLW. A trench extending from the discharge port is excavated to elevation - 40 feet (MLW) to a layer of limestone approximately 5 feet thick that will not be scoured by the discharge jet.

This study of the dilution of the cooling water jet flows has been segregated into nearfield and farfield analyses. These two studies are described and their results are summarized in the discussions below.

2.4.12.1.1 Nearfield Studies

The nearfield area as discussed in this analysis is used to mean that region where the dilution is due largely to dynamic and buoyant forces. Both the physical (thermal-hydraulic) and analytic analyses were conducted to predict the nearfield concentration distribution above the "Y" discharge ports.

Physical Models

Florida Power and Light sponsored a physical (undistorted) model analysis of the outfall structure at the Coastal and Oceanographic Engineering Department of the University of Florida. The detailed 2 year analysis (12) showed that the discharge is diluted by approximately a factor of 4 to 5 at the surface of the ocean in the nearfield. In some thermal-hydraulic analyses (13), correction factors are applied to make the model more compatible with expected prototype behavior. For the St. Lucie Plant, these factors would reduce the maximum water temperature rise from 5.5°F to about 5°F in the nearfield. All analyses subsequent to the nearfield studies were conservatively based on a surface temperature of 5.5°F.

Mathematical Models

The physical model studies references above have shown that the flow from the "Y" ports could be treated as two separate jet discharges. The nearfield jet flow analysis was correlated using Koh and Fan's (14) mathematical models assuming two separate jets. This mathematical model predicts nearfield surface dilutions of about 4 or 5 and is in agreement with the physical models. Refer to Figure 2.4-13A. Baumgartner et al (15) have observed an average dilution factor of about 5 times greater than those predicted above. Nevertheless, all outfall dispersion calculations have been conservatively based on a dilution factor of 4 in the nearfield.

The mathematical modeling was updated to evaluate the effect of the additional waste heat from the EPU on the model results (45). The model used is MULTIDIF, which is the EnviroSphere version of the near-field Koh and Fan model.

2.4.12.1.2 Farfield Studies

The farfield study encompasses an analysis of the surface spreading and the surface concentration distribution of a jet being convected by ocean currents and mixed by ocean turbulence.

The surface distribution discussed in Section 2.4.12.1.1 provides the boundary conditions for the farfield dilution calculations. The farfield is assumed to have been reached when the mixing process exhibits no dependence on the initial mode of discharge and is only associated with ambient hydrodynamic considerations.

Brooks (16) and Ditmars (17) have both analyzed the concentration distribution in large flow environments for line sources of initial intensity, To, and finite width, b. The governing equation for this circumstance is:

$$v\frac{\partial t}{\partial x} = \frac{\partial}{\partial z}\left(K_z \frac{\partial t}{\partial z}\right) - kt \qquad (1)$$

where:

t = concentration at any point (x,z)
v = ocean current velocity
x,z = surface coordinates
K_z = lateral eddy diffusion currents
k = dissipation coefficient

The solution of equation (1) is given for the concentration along the axis z = o, Tm

$$\frac{Tm}{To} = \left(e^{-kx/v}\right) \text{ erf } \sqrt{\frac{3/2}{\left[\frac{1+8K_{z_o}x}{vb^2}\right]^3 - 1}}$$

where: K_{z_o} = lateral eddy diffusion coefficient based on the initial plume width b at x = 0, and

To = the concentration at x = o, z = o

Tsai (18) used the method of analysis above for a submerged diffuser at the James Fitz Patrick Nuclear Power Plant, Docket No. 50-333.

Baumgarten et al (15) used this method to analyze subaqueous multiport outfalls in the municipalities of Newport and Gardiner in Oregon.

The methodology presented above was used to estimate the farfield dilutions for the St. Lucie Unit No. 1. The initial width of the line source is assumed to be 500 ft. and the thickness of the thermal plume is assumed to be 6 ft. The lateral eddy diffusion coefficient is assumed to be 10^4 cm^2/sec. The results of the analyses show that a dilution of 20 is found at a distance of 13,200 ft. The area within which dilution is less than 20 covers 400 acres of ocean surface.

2.4.12.1.3 Effect of Thermal Effluent on Marine Ecology

A comprehensive evaluation of the marine ecology at both offshore and Indian River waters adjacent to Hutchinson Island has been made and is reported in Appendices 2 and 3 of the Hutchinson Island Plant Environmental Report. The results of that evaluation show that the plant will not affect any existing fisheries. Organisms with a known or potential for concentration are found well outside the 400 acre area where dilution is 20 or less. The St. Lucie heat dissipation system is regulated by the Florida Industrial Wastewater Facility Permit (No. FL0002208) and the discharge limit conditions. This permit describes an allowable mixing zone where the St. Lucie discharge can exceed the state water quality discharge standards. The results of the modeling show that the mixing zones for the EPU operation remain within the mixing zone limits already established (45). There are no anticipated impacts from the additional heat discharge on marine biota (45).

2.4.12.2 Recirculation of Discharge Water

The intake structure of St. Lucie Unit No. 1 is located 1250 feet offshore and about 2400 feet south of the discharge outfall. The top of the north intake is located at approximately -8 feet MLW, while the top of the south intake is located at approximately -9.33 feet MLW (as shown in Figure 2.4-13B). Recirculation of discharge water at St. Lucie is mainly a function of the currents and the effective dilution experienced between the ocean intake structure and the discharge outfall. No recirculation is expected with either northerly currents or slack water conditions. Some recirculation may occur under a prevailing southerly current.

2.4.12.2.1 Estimated Recirculation

The farfield studies have estimated that a surface temperature of 1°F above ambient could be expected above the intake point with a thermal plume depth of about 6 feet.

In addition, the field dye studies (12) conducted at the site in 1970 by the Coastal and Oceanographic Engineering Department of the University of Florida showed that under a southerly current the ambient surface water temperature rise in the vicinity of the intake would be less than 1°F. As described in the studies, the dye release point had approximately the same characteristics as those of the "Y" discharge ports.

In both the diverse methodologies discussed above the investigations support the conservatism of assuming a 1°F temperature rise above the plant intake structure. In terms of dilution, this represents a 1/24 dilution of the discharge water above the intake structure as the worst case.

The percentage of recirculation of discharge water is reduced even further due to the manner in which the intake structure functions. Water is withdrawn radially through the two velocity caps. The caps are always covered by a minimum of 8 feet of water (the thermal plume thickness is conservatively calculated to be 6 feet.). The withdrawal of water by the intake velocity caps results in a further dilution of the discharge water to approximately 0.5°F (1/48 dilution) above ambient. The basis for this conclusion is discussed below.

Craya (19) investigated the case of withdrawal from a bi-layered (stratified) system. He concluded that the critical Fronde number above which both fluid layers flow and below which only one fluid layer flows is 2.55 for a small circular opening. The equation for the critical Fronde number is:

$$F_c = \sqrt{\frac{Q_c}{g \frac{\Delta\rho}{\rho} h^5}} = 2.55$$

where:

F_c = critical Fronde number

Q_c = critical intake flow through the orifice

h = half-width of the withdrawal layer
ρ = density of sea water
g = gravitational constant

Gariel (20) performed experiments which substantiated the analytical findings of Craya.

The intake flow of 1150 cfs for St. Lucie Unit 1 results in a densiometric Fronde number greater than 2.55 and hence water will be withdrawn from both layers. The mixing of the thermal plume with the much cooler water below the plume results in the ultimate temperature rise of 0.5°F. This decrease in the temperature of the plume is determined by performing a single mass balance of the mixed seawater.

2.4.13 GROUND WATER

2.4.13.1 Description and On-site Use

[Note: A Site Conceptual Model (SCM) for the St. Lucie site was prepared in April 2008 to characterize groundwater flow and mass transport (Reference 44). The specific areas addressed in this report were as follows:

- Characterization of geologic and hydrogeologic conditions within the Owner Controlled Area, including subsurface soil types, and direction and rate of groundwater flow;
- Characterization of the groundwater/surface water interaction at the site;
- Discussion of the operation and design of the evaporation/retention basins (which receive flow from the storm water drainage system);
- Evaluation of groundwater quality at the site, including the vertical and horizontal extent, quantity, concentrations, and potential sources of tritium in the groundwater (including maps of mixed plume area);
- Definition of probable sources of tritium releases at the site;
- Evaluation of potential human, ecological, or environmental receptors of tritium that may have been released to the groundwater; and
- Recommendations for additional investigations and long-term monitoring.

The conclusions of the SCM were that tritium in groundwater at the St. Lucie site is unlikely to present an environmental or health risk on or offsite, and that none of the potential receptors identified are at risk of exposure to concentrations of tritium.]

The region under study is the area of St. Lucie, Martin, Indian River, and Okeechobee Counties. Two main aquifiers are found in this region: a shallow non-artesian or locally artesian aquifier, and a deep artesian aquifier.

The shallow aquifier is the principal source of fresh water supplies in the on-shore region. It consists of the Anastasia formation and extends to a depth of about 150 feet below the land surface. It is composed principally of sand but contains thin lenses of shell, limestone, or sandstone which are generally more permeable than the sand(10).

The shallow aquifier receives most of its recharge from rainfall in the immediate area. In general, surface water runoff is small. A small amount of recharge to the shallow aquifier comes from downward seepage of artesian water used for irrigation.

The discharge of the shallow aquifier is by flow into streams, or lakes, by direct flow into the ocean, by evapotranspiration, and by pumping from wells. Canals and ditches in the area carry some ground water away. The transmissibility of this aquifier in Martin County has been measured to be approximately 20,000 gallons per day per foot.(10).

The deep aquifier of the area and principal artesian aquifier of the region is the Floridian aquifier which underlies all of Florida and southern Georgia and consists mainly of permeable beds. The top of the Floridan aquifier in Martin County is usually between 600 and 800 feet below the ground surface and underlies the Hawthorne formation which is an aquiclude. The thickness of the Floridan aquifier in this immediate vicinity is unknown, since no well has completely penetrated it. It is estimated to be about 2000 feet thick. The artesian pressure head (piezometric surface) in the area of the site is estimated to be about 45 feet (See Figure 2.4-14).

The principal recharge area for the Floridan aquifier in this region is in and around Polk County where the limestone of the aquifier is overlain by semi-confining beds of the Hawthorne formation which are not impermeable and may permit downward leakage.

The points of discharge of the Floridan aquifier are springs and wells and where upward leakage occurs through the confining beds. There are no known natural springs in the region.

Underlying the 4 to 6 feet of surface peat on Hutchinson Island is the Anastasia formation which extends to about elevations (-) 135 to (-) 155 feet and consists of grey, slightly silty fine to medium sand with varying amounts of fragmented shells. It also contains discontinuous pockets of cemented sand with shells and sandy limestone. Occasionally, discontinuous thin plastic clay lenses are found in the upper part of the formation. The Anastasia formation is an unconfined or non-artesian aquifier.

Below the Anastasia, the upper 100 feet of the Hawthorne formation at the site consists of a green slightly clayey and silty very fine sand. Indications are that the top of this zone is a semi-confined aquifier. Below about 250 feet and extending to the 400 foot depth of boring termination were sandy clayey silts which form the principal aquiclude for the underlying Floridan artesian aquifier.

To determine the general ground water environment at the site, piezometers were installed in ten of the borings to measure ground water levels in the Anastasia and the Hawthorne formations (See Figure 2.4-15). Readings of water levels in the piezometers were taken periodically throughout the month of April, 1968. The data are presented in summary form in Figure 2.4-16.

The ground water table occurs very near or above the ground surface at the site due to the mosquito control ponds. A continuous body of water was found at the site throughout to the depths investigated. The slight artesian head of 0.5 to 1.0 foot was observed in piezometers P-1 and P-8 installed in the top of the Hawthorne formation (See Figure 2.4-16).

Ground water fluctuations in drill holes in response to tidal changes indicate relatively uniform transmissibilities of the sand deposits. Piezometer P-11 was installed near the Atlantic Ocean shore in the Anastasia formation and reflected tidal variations well. Piezometers along the Big Mud Creek (P-7, P-9, P-10) installed in the Anastasia formation indicated water levels less than 0.5 foot above the tidal range recorded in the Big Mud Creek. This is possibly due to the higher water level on the land side of the dike. Piezometer P-2 was installed partially into the Hawthorne formation but generally reflected the Anastasia levels.

2.4.13.2 Sources

Two piezometers were installed in borings B-17 and B-18 on the mainland and monitored for the two month period from the end of June 1968 to the end of August 1968. The locations of these piezometers are shown on Figure 2.5-4. Rainfall records for the same period were also kept. The results are presented in summary form on Figures 2.4-17 and 2.4-18.

Water samples were obtained from selected piezometers, the Indian River (Big Mud Creek), and the water at the site (swamp water). Chemical tests were conducted on these samples. The results of these tests are shown in Table 2.4-3.

The water temperature as measured at various depths of the piezometers indicated an average of 75.8 degrees F. with negligible variation.

In May 1968 the existing private wells within a six mile radius of the site were surveyed by Law Engineering Testing Company.

Two wells were discovered on Hutchinson Island. The first was located about six miles south of the plant site and east of highway A1A in the vicinity of the Jensen Beach causeway. Samples of water from this seven foot deep well contained chloride concentrations in excess of 1000 ppm. Investigation at the time of sampling confirmed that this source was not being used as potable water. The second well was found to be abandoned and capped. Its location was about seven miles north of the plant site near the Fort Pierce causeway. No attempt to take samples was made.

Data from on-site wells of both pre-construction and construction periods compare closely with regard to chloride content. Preconstruction piezometer readings indicated concentrations from 10,000 to 25,000 ppm. Information obtained from samples taken throughout the site during dewatering at an average depth of 90 feet had 10,000 to 23,000 ppm chlorides and 1,000 to 4,000 ppm sulfides.

In addition to Hutchinson Island, where only two wells were found, the area surveyed can be divided into two zones of well concentration:

1. Along State Road 707
2. Along U. S. Highway #1

In these zones there were many private wells. Therefore, no attempt was made to locate and record every well in the area. Approximate number of wells in the various communities are given in Table 2.4-4. Other data are presented below.

In general, every house was found to have one or more wells. Also, almost every well was found to be capped and attached to a pump so that measurements of water levels could not be made at the time of the survey. All shallow wells in the area are drilled and cased full depth with screen on the ends. In the area of Eldred and Eden, shallow wells are 20 to 40 feet deep. Further inland, in White City, 40 to 60 feet deep wells are common.

Only two roads extend west from SR-707 to U. S. #1 within the survey area. These are the Walton Road and the White City Road (SR-712). Along the former, there are no wells. South of SR-712 is a large tract owned by General Development Corporation (Indian River Estates). Presently, this development has 22 homes with expansion planned for the near future.

The cities of Fort Pierce and Stuart have water supply systems. On U. S. #1, the Fort Pierce water system extends to the State Farmer's Market. Water from Stuart is piped to Hutchinson Island. This system extends as far north as Holiday Out, a trailer resort located in St. Lucie County approximately 5 miles south of the site. The Port St. Lucie Community, which presently has 500 to 600 homes, also has a private water supply system.

On the mainland the piezometer readings were found to vary more than those on Hutchinson Island. The piezometers reflected recent rainfalls within 1 to 5 days after the rain had fallen. The average water levels were found to be generally decreasing after the large rainfall between July 5 and July 9. During this five day period 5.82 and 4.91 inches of rain were recorded at Stuart and Fort Pierce, respectively. The effect of the tidal variation was not evaluated.

The average dip of the water levels between the two piezometers was measured to be 0.03 percent towards the ocean. [The Site Conceptual Model (Reference 44) documented hydraulic gradients measured in August-September 2007 at the plant site; measured values ranged from 0.002 ft/ft to 0.02 ft/ft.]

All public and most domestic supplies of water in the region are obtained from ground water sources. Ground water is also used extensively for irrigation, stock watering and industry.

The cities of Fort Pierce and Stuart have public water supplied from wells developed in the shallow aquifier (See Table 2.4-5). The City of Fort Pierce water supply wells are 10 miles northwest of the site and Stuart wells are 11 miles southwest of the site. No large industrial water usage exists in the area. Irrigation and stock watering account for the largest withdrawals of ground water. Water from the shallow aquifier is for irrigation by farmers growing vegetables and citrus fruits, by ranchers for pastureland, stock watering, and feed crops. Many of the artesian wells were originally drilled for irrigating vegetable crops.

The total use of artesian water for irrigation may be about 10 million gallons per day during the dry season. During the rainy season most of these wells are not used.

Chemical analyses of water samples taken from the wells on the mainland indicate the water from the shallow aquifier to have a lower mineral content than the artesian water.(10).

When the area emerged from the ocean (See REGIONAL GEOLOGY SECTION 2.5) after the last major advance of the sea, all the land was saturated with salt water. Rain falling on the land and moving through the ground has gradually carried most of the salt water back to the ocean. Most of this Pleistocene sea water has been flushed from the shallow aquifier except at certain locations. The piezometric surface of the Floridan aquifier in Martin County is about 50 feet above mean sea level. This pressure head should be sufficient to insure at least 2,000 feet of fresh water below sea level. Artesian wells in Martin County down to about 1500 feet have shown high chloride content. It appears that this is due to contamination during the Pleistocene epoch rather than recent sea water encroachment(10).

The source of natural fresh water on Hutchinson Island is rain. Because of the low land surface altitudes, the permeable nature of the soils and the short distance to points of discharge, the water table is probably only a few inches above mean sea level. Wells in many places on the island are still in fresh water a foot or so below the water table. However, even moderate pumping allows salty water to enter the wells. At the site, the pumping of Indian River water into the pond has increased the mineral concentration of the ground water. Results of water chemical analyses on seven samples taken at the site are given in Table 2.4-3.

2.4.13.3 Accident Effects

Two types of pump-in, constant head type field permeability tests were conducted at two locations, as shown on Figure 2.4-19. These consisted of open-end pipe and well permeameter methods of testing, as outlined in the Bureau of Reclamation's "Earth Manual"(11). Field permeability tests were also planned at proposed locations of major structures in the swamp area and borings were made for this purpose. However, these holes were found to be clogged with organic fines. In an attempt to remedy this problem, the bore holes were re-drilled and cleaned, using clean swamp water and diluted hydrochloric acid. However, this attempt failed; therefore, tests in the swamp were abandoned.

A total of four test holes were drilled in the vicinity of Boring B-2, two for the open-end pipe (PC-1 and PC-2, fully cased) and two for the well permeameter method (PU-1 and PU-2, partially cased). Dimensions for these holes are shown in Figure 2.4-20. The subsurface conditions as determined by boring B-2 are given in the attached log, Figure 2.4-21.

In all test holes, casings were advanced by jetting or by washing and driving. All holes were carefully cleaned before testing was begun. After cleaning, cased holes PC-1 and PC-2, were completely filled with limestone ranging in size from 3/8 to 4 inches. All casings were sealed on the outside using expansive clay pellets.

In conducting all permeability tests clean water was pumped into the casing and a constant gravity head maintained in the casing during the test. The flow was measured with a water meter, the constant head in the casing being maintained by means of regulating valves.

Graphs of cumulative discharge versus cumulative time are shown in Figures 2.4-22 through 2.4-25. The coefficients of permeability were determined using the steady flow part of these curves for the two types of tests.

The field permeability values obtained for all four tests are shown in Figure 2.4-19. Laboratory permeability test results compare favorably with these values.

Field permeability tests made during this investigation have indicated a seepage rate of flow of about 15000 feet per year in the top 30 feet of the sand deposits at the site. Taking the highest permeability coefficient obtained and a hydraulic gradient of 100% any discharge introduced into the ground at the reactor site would reach the Indian River in about a day. The discharge would be greatly diluted immediately. Because of the proximity and width of the Indian River and the presence of slight flow of ground water toward the coast line, there is no possibility of sub-surface flow from the site to the mainlands.

An investigation of the ground water hydrology of Hutchinson Island permits the conclusion that the possibility of any intrusion of accidental releases of radioactivity into mainland ground water supplies is extremely remote.

2.4.13.4 Monitoring or Safeguard Requirements

Ground water samples will be taken quarterly from two locations as part of the operational radiological surveillance program (Table 17-1, Hutchinson Island Environmental Report, Supplement 2).

2.4.14 TECHNICAL SPECIFICATION AND EMERGENCY OPERATION REQUIREMENTS

Refer to Section 13.3 for the Emergency Plans outline for actions in case of emergencies.

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TABLE 2.4-1

PMH SURGE COMPUTATIONS *

						Peak Surge Windstress Component				
PMH	Translation Speed (Knots)	Radius to Max. Wind (N.M.)	Astronomical Tide (ft.) Above MLW	Initial Rise (Ft)	Pressure Rise @ Peak Surge	Sx (ft) (on Shore)	Sy (ft) (along shore)	Sx Max. Ft	Sy Max. Ft	Total Surge Ft. MLW
1	17	18	3.7	1.5	3.83	6.67	.52	6.71	1.34	16.22
2	10	18	3.7	1.5	3.87	6.25	.58	6.26	1.52	15.90
3	10	10	3.7	1.5	3.78	5.88	.62	5.88	1.37	15.48
4	17	10	3.7	1.5	3.75	6.40	.61	6.44	1.17	15.96

* Reference Section 2.4.5.9 for updated surge levels and wave runup analysis.

TABLE 2.4-2

WAVE RUNUP COMPUTATIONS

Profile Path	Section	Deepwater Wave Height H'_o (ft)	Wave Height H (ft)	T Sec.	L Ft.	Cot α = a/b	SWL	Runup R	SWL+R (ft. MLW)	Elev. at Plant & Switchyard (ft. MLW)
Unit 1 Parallel to traverse line	4-4	10.15	10.3	7.0	136.4	49.9 = 708/14.2	16.2	1.02	17.2	19.0
Unit 2 Parallel to traverse line	5-5	10.15	10.3	7.0	136.4	60.4 = 848/14.05	16.2	.85	17.1	19.0
Unit 1 θ = 30°	6-6	9.915	9.75	6.8	115.3	72.1 = 952/13.2	15.5	.70	16.2	19.0
Switchyard θ = 60°	7-7	8.93	8.346	6.25	109.4	14 = 210/15	13.7	3.3	17.0	18.0

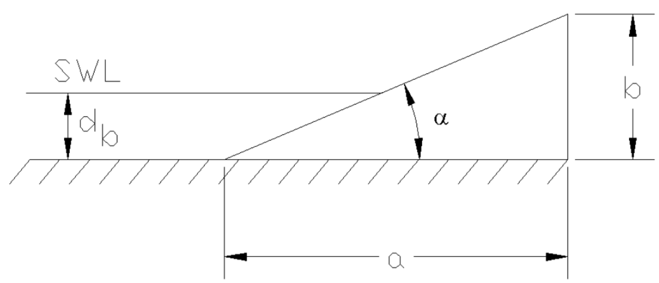


TABLE 2.4-2A

FLORIDA HURRICANES: PERIOD OF RECORD 1886 TO 1974

Month	Total Florida Landfalls	Total Landfall East Coast	Landfall East Coast Within ±100 NM of St. Lucie	Landfall East Coast Within ±100 NM of St. Lucie Wind Speed ≥ 125 mph (106 mph 10 min.)
Jan. - May	0	0	0	0
June	2	0	0	0
July	6	2	2	0
Aug.	11	7	5	1
Sept.	27	7	6	3
Oct.	21	3	2	0
Nov.	1	1	1	0
Dec.	0	0	0	0
Totals	68	20	16	4

TABLE 2.4-3
WATER QUALITY ANALYSIS

LAW ENGINEERING TESTING COMPANY

Lab. No................................	8231	8232	824.0
Sample Location	P-1	P-2	P-11
Date of collection......................	3-30-1968	3-28-1968	3-28-1968
Silica (SiO_2)...........................	15.1	10.1	8.4
Iron (Fe)..............................	1.4	1.2	4.1
Manganese (Mn)..........................	0.0	0.0	0.0
Calcium (Ca)............................	1364	1820	1204
Magnesium (Mg)..........................	781	190	173
Sodium (Na).............................	10000	11400	12700
Potassium (K)...........................	477	427	406
Bicarbonate (HCO_3)......................	256	659	573
Carbonate (CO_3).........................	0	0	0
Sulfate (SO4)...........................	2515	387	387
Chloride (Cl)...........................	20800	19200	18600
Fluoride (F)............................	21	6	1
Nitrate (NO_3)..........................	0.2	0.0	0.0
Dissolved solids			
Calculated.............................	36164	33928	33907
Residue on evaporation at 105 C.........	39840	44580	45760
Residue at 600° C.......................	35860	38960	40560
Organic solids..........................	3724	4961	4627
Hardness as $CaCO_3$......................	6620	5330	3720
Noncarbonate hardness an $CaCO_3$..........	6410	4790	3250
Alkalinity as $CaCO_3$, Phenolphthalein	0	0	0
Alkalinity as $CaCO_3$, Total..............	210	540	470
Specific conductance			
(micromhos at 25° C)...................	40000	38500	40000
ph.....................................	6.40	6.15	5.5
Color..................................			
pH saturation (calculated)..............	6.84	6.38	6.60
Langelier Index, I......................	-0.44	-0.23	-1.10
Salinity................................	33630	33780	32680
Specific gravity........................	1.024	1.027	1.028

NOTE: All analytical results are in mg/liter

TABLE 2.4-3 (Cont d)
WATER QUALITY ANALYSIS

LAW ENGINEERING TESTING COMPANY

Lab No.	8233	8234	8235	8236
Sample Location	P-3a	Big Mud Creek	Swamp Water	P-9
Date of collection.....................	3-31-1968	3-31-1968	3-31-1968	4-6-1968
Silica (SiO_2)..........................				
Iron (Fe).............................				
Manganese (Ma).........................				
Calcium (Ca)...........................	432	436	440	1800
Magnesium (Mg).........................				
Sodium (Na)............................				
Potassium (K)..........................				
Bicarbonate (HCO_3).....................	354	305	232	854
Carbonate (CO_3)........................	0	0	0	0
Sulfate (SO_4)..........................	2709	2902	3225	271
Chloride (Cl)..........................	17600	19400	20800	19200
Fluoride (P)...........................				
Nitrate (NO_3)..........................				
Dissolved solids Calculated 105° C.....................	45600	40540	44940	44880
Residue on evaporation at 600° C................................	30220	31140	34540	38740
Hardness as $CaCO_3$.....................				
Noncarbonate hardness as $CaCO_3$.........				
Alkalinity as $CaCO_3$, Phenolphtha-lein.................................	0	0	0	0
Alkalinity as $CaCO_3$, Total............	290	250	190	700
Specific conductance (micromhom st 25° C)...................				
pH....................................	7.10	7.40	7.70	6.30
Color................................				
pH saturation (calculated).............	7.25	7.28	7.42	6.24
Langelier Index,.......................	-0.15	+0.12	+0.28	+0.06
Salinity..............................	31230	34330	36630	33630
Specific gravity......................	1.018	1.020	1.024	1.026

NOTE: All analytical results are in mg/liter

TABLE 2.4-4

WELL LOCATION SUMARY

SURVEY AREA NO.	LOCATION	APPROXIMATE NUMBER OF WELLS
1	Ankona	65
2	Eldred	53
3	Eden	70
4	Indian River Estates	22
5	Port St. Lucie (west of US #1)	500-600
6	Walton	75
7	White City (east of US #1 and south of SR-712)	130
8	White City (other area within 6 mile radius)	100-150
9	Between south city limits of Fort Pierce and north town limits of Eldred along SR-707.	91
10	South of White City and north of Jensen Beach Road	50

TABLE 2.4-5

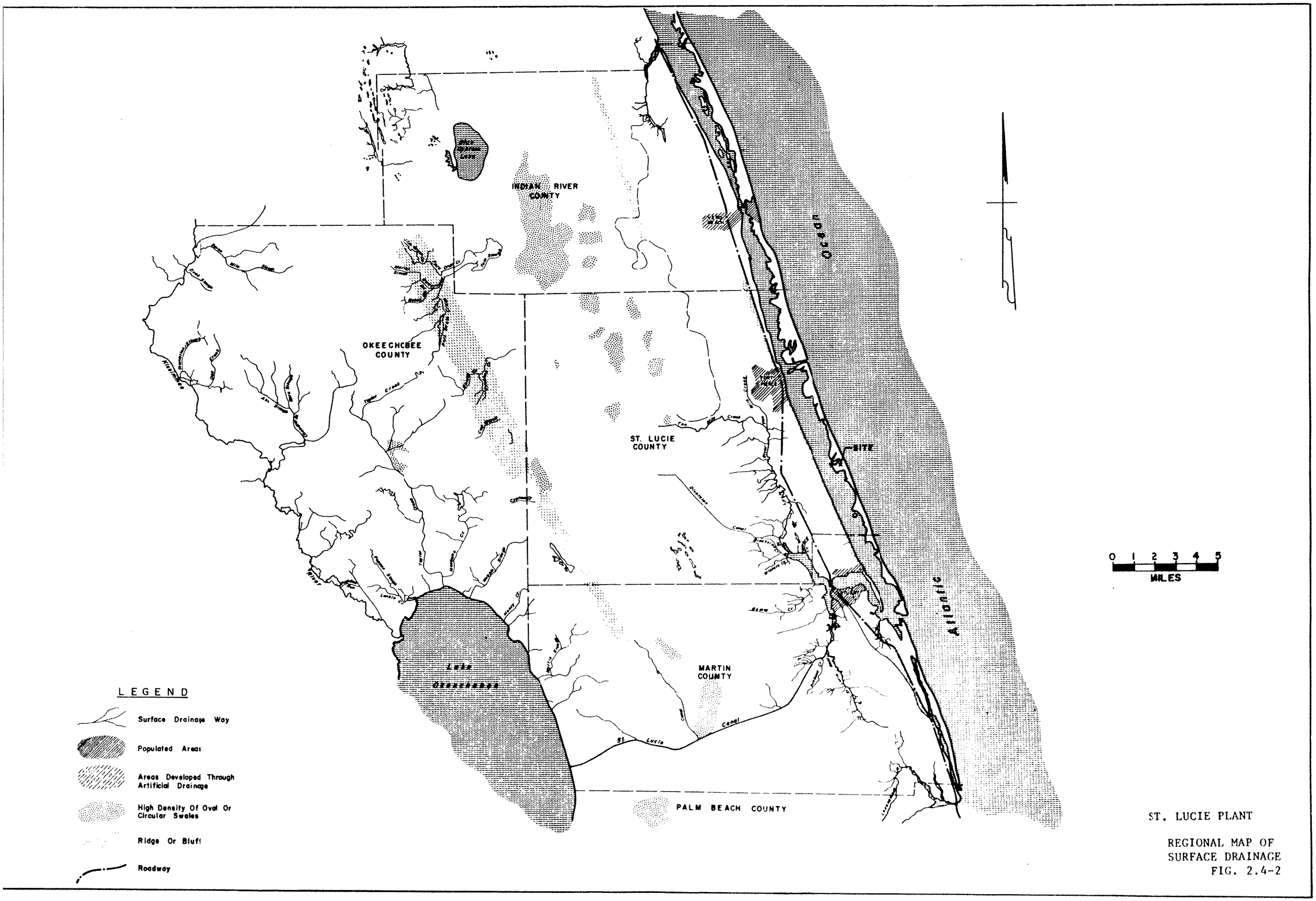
PUBLIC WELL WATER SUPPLIES

CITY	NO. OF WELLS	AVERAGE GROUND SURFACE ELEV.	AVERAGE DEPTH OF WELLS	AVERAGE GAL. PER DAY PER WELL
Ft. Pierce	17	+20	110 Ft.	500,000
Stuart	12	+15	105 Ft.	170,000

DELETED

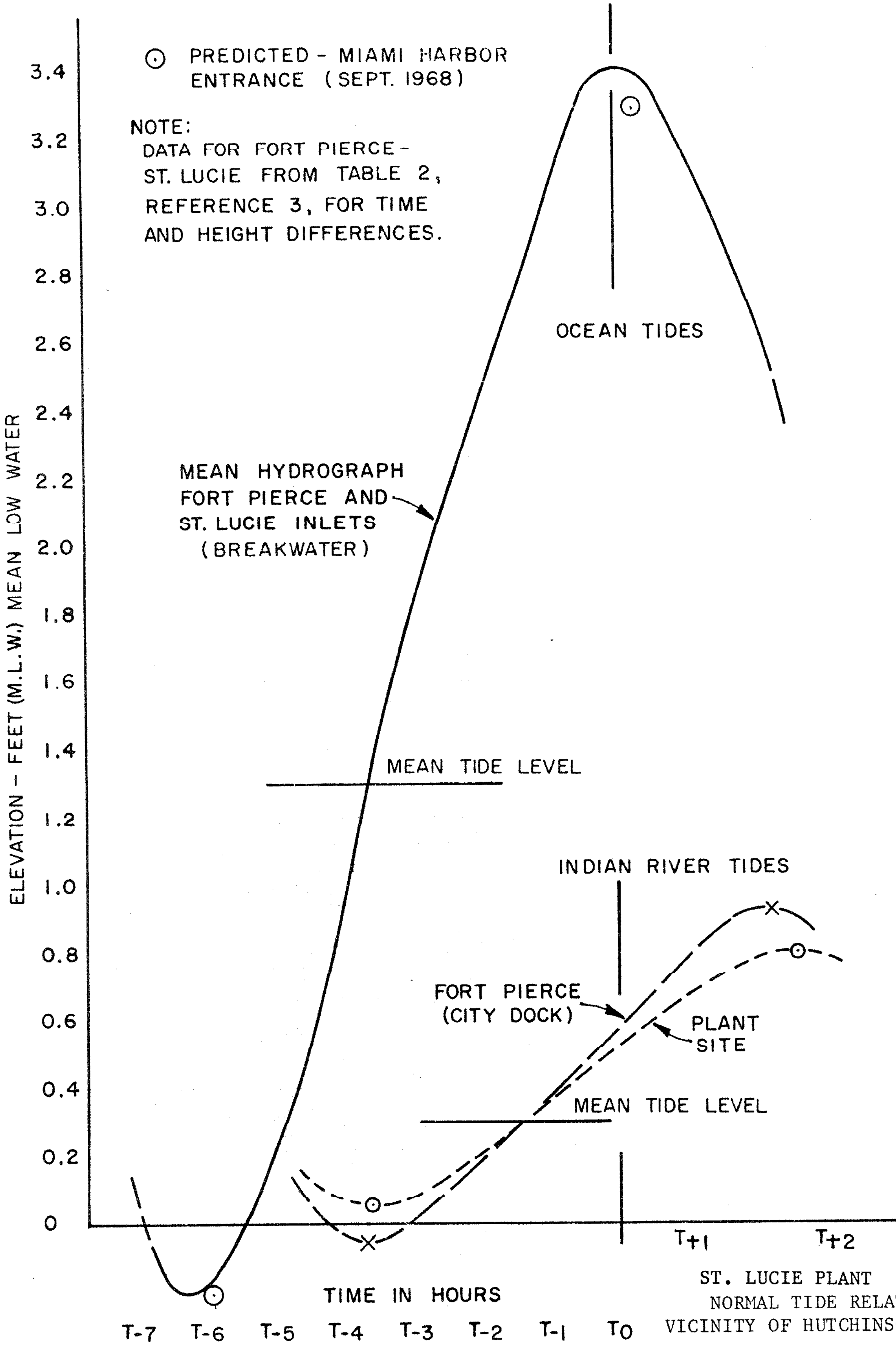
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

FIGURE 2.4-1



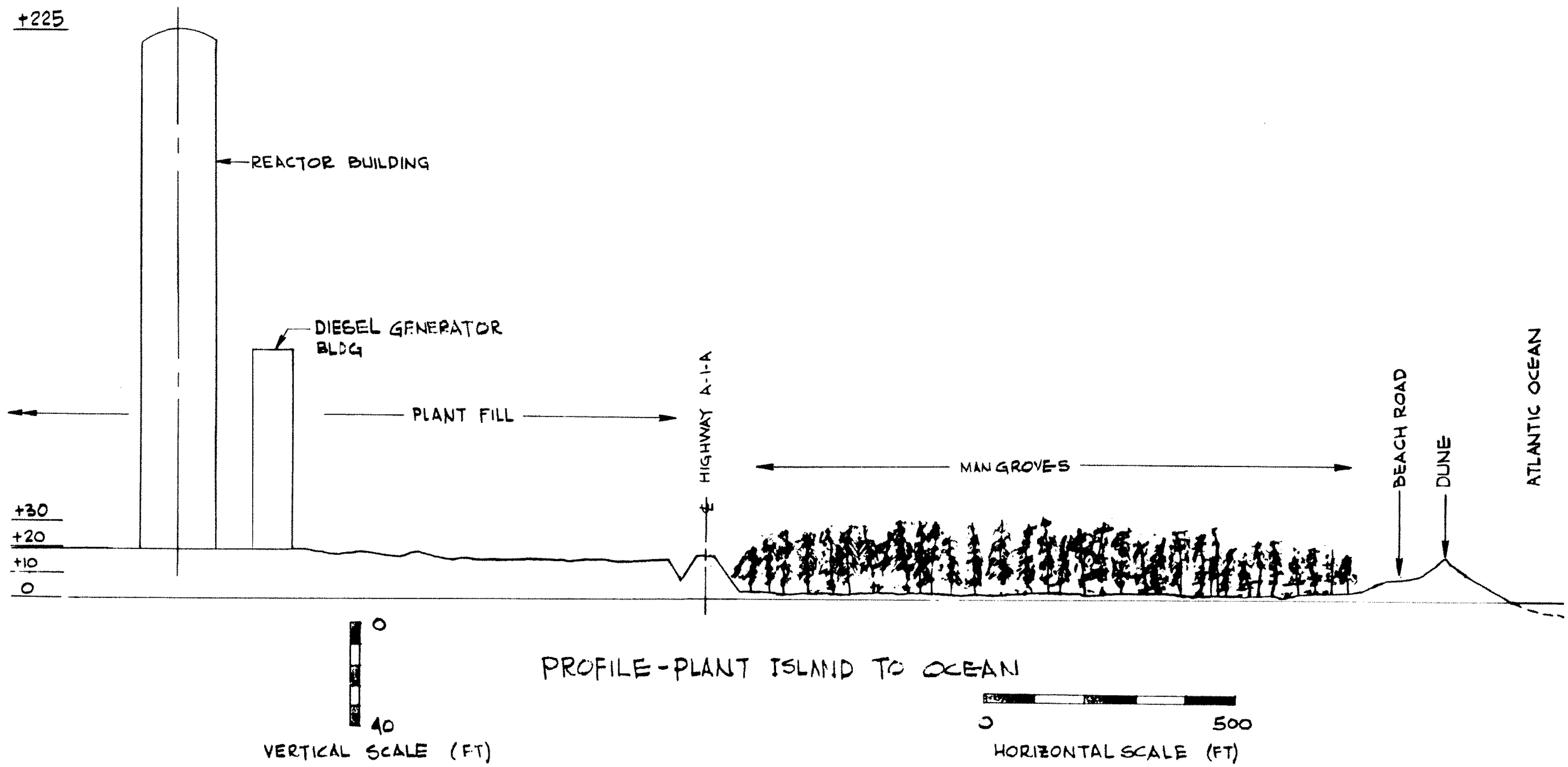
ST. LUCIE PLANT

REGIONAL MAP OF
SURFACE DRAINAGE
FIG. 2.4-2



ST. LUCIE PLANT
NORMAL TIDE RELATIONS
VICINITY OF HUTCHINSON ISLAND

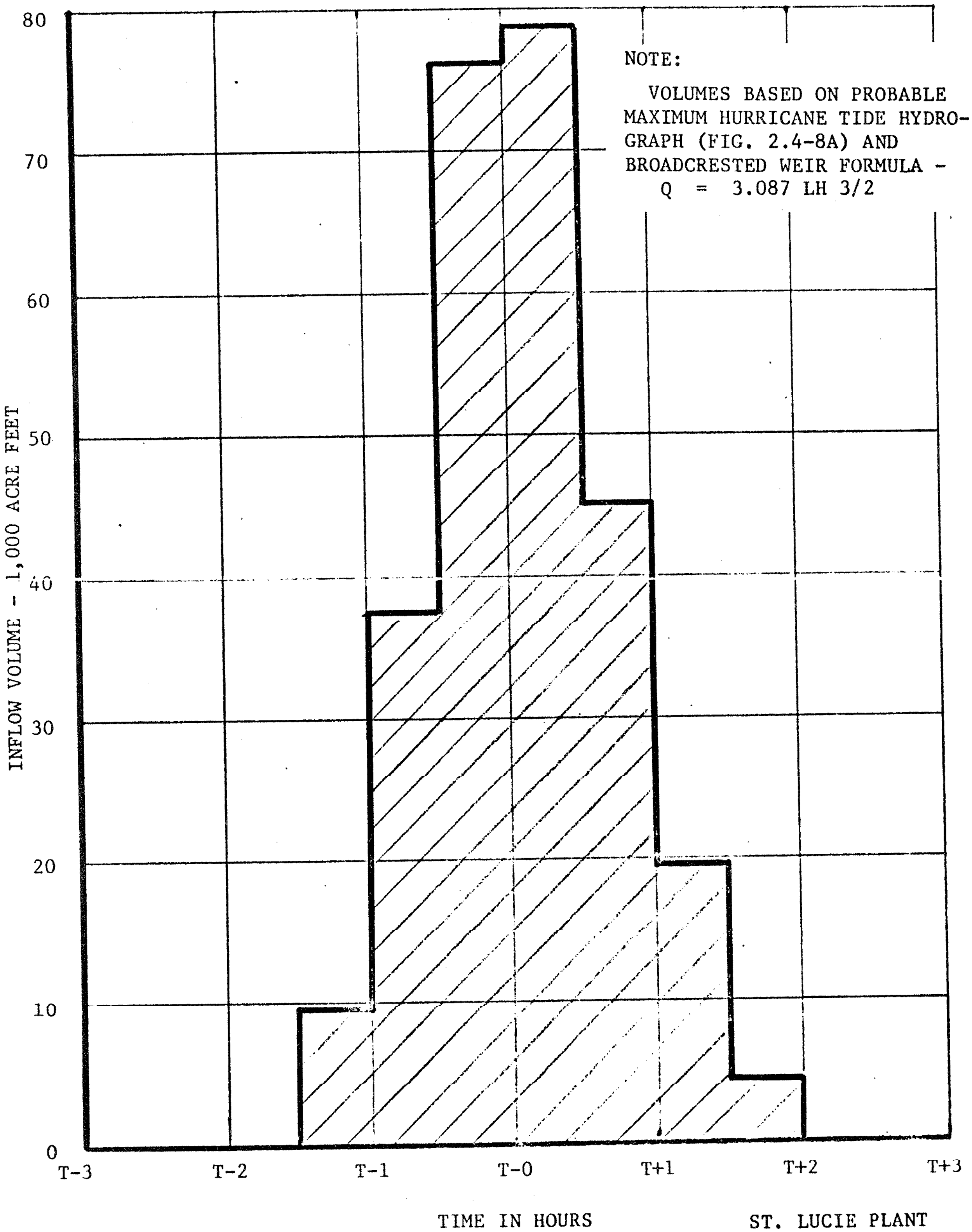
FIG. 2.4-3



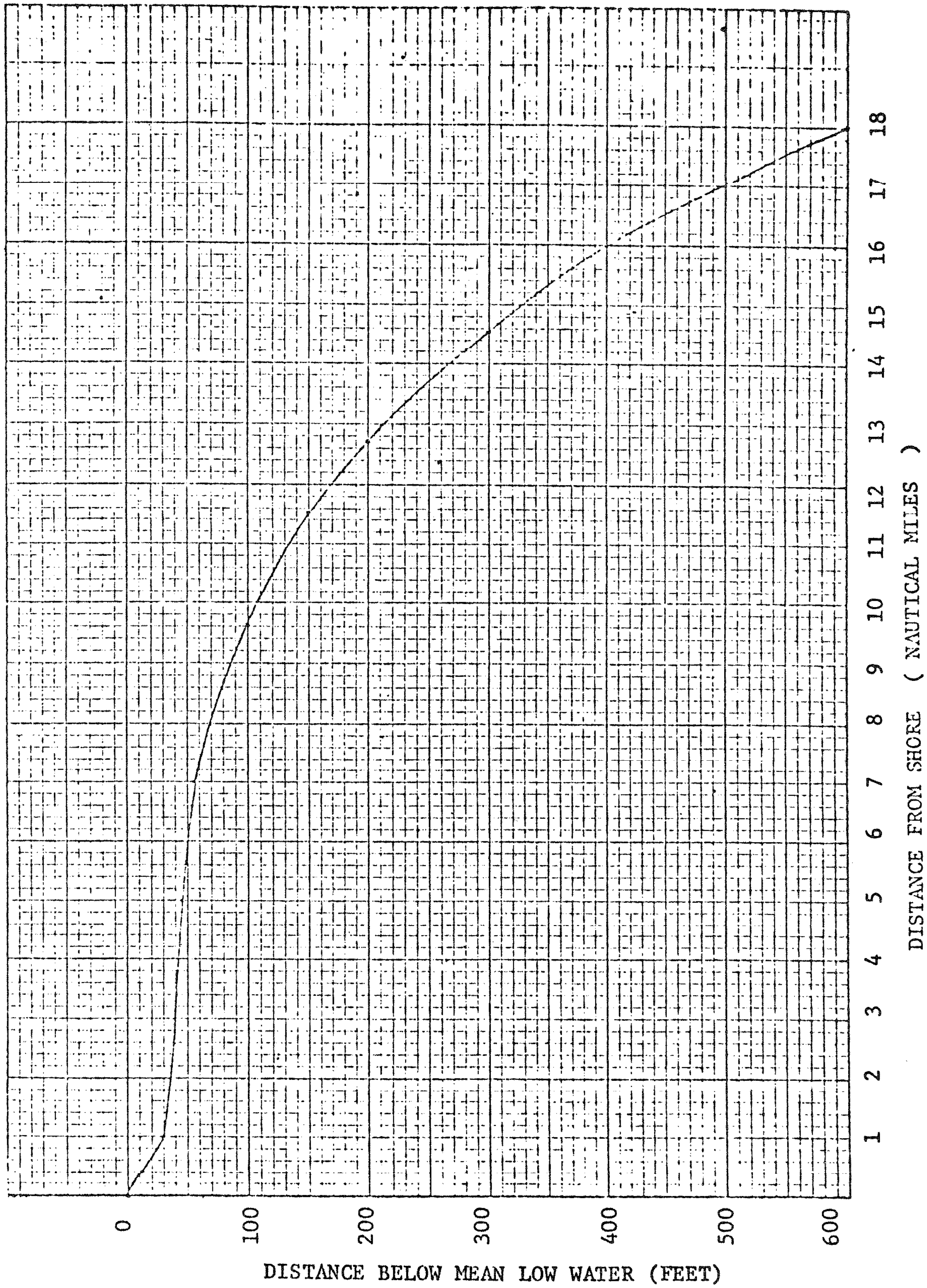
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

TOPOGRAPHY AND VEGETATION SECTION AT PLANT SITE

FIGURE 2.4-4



ST. LUCIE PLANT
PROBABLE MAXIMUM HURRICANE
OVERFLOW VOLUME HYDROGRAPH
FIG. 2.4-5



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SEABED PROFILE OVER THE CONTINENTAL SHELF

FIGURE 2.4-6

FIGURE 2.4-7

HAS BEEN INTENTIONALLY

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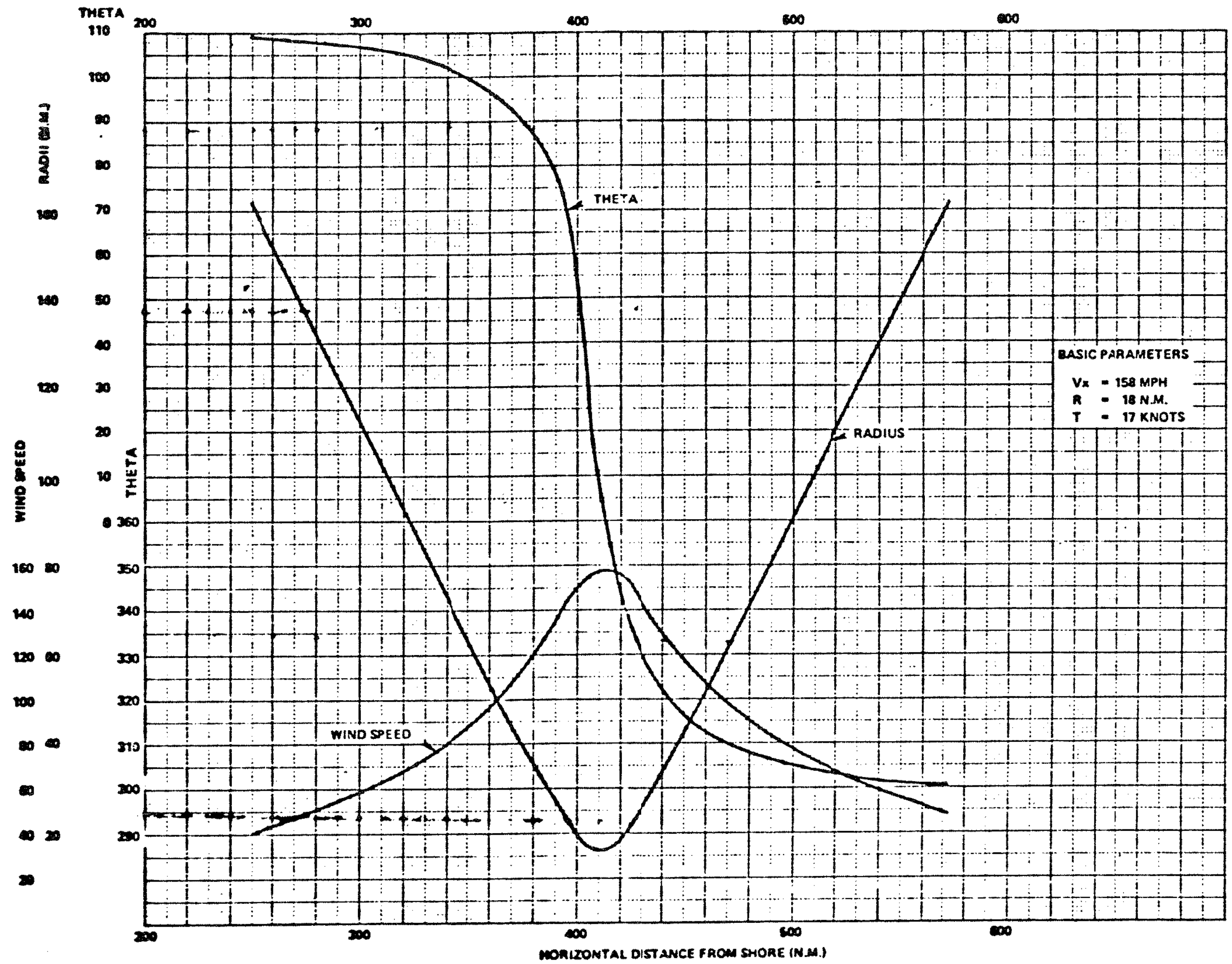

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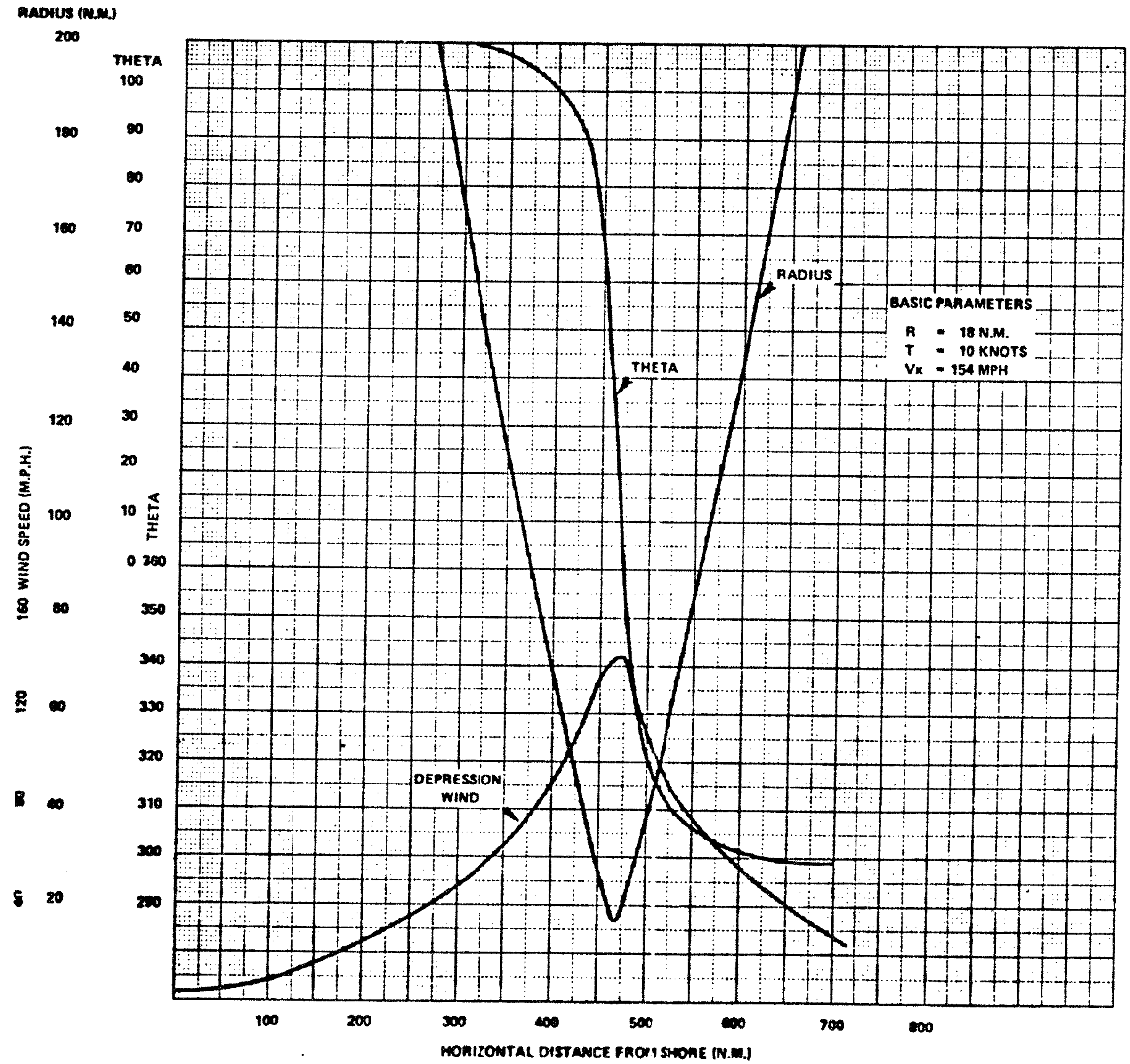
WIND FIELD - PMH - CASE 1

FIGURE 2.4-7a



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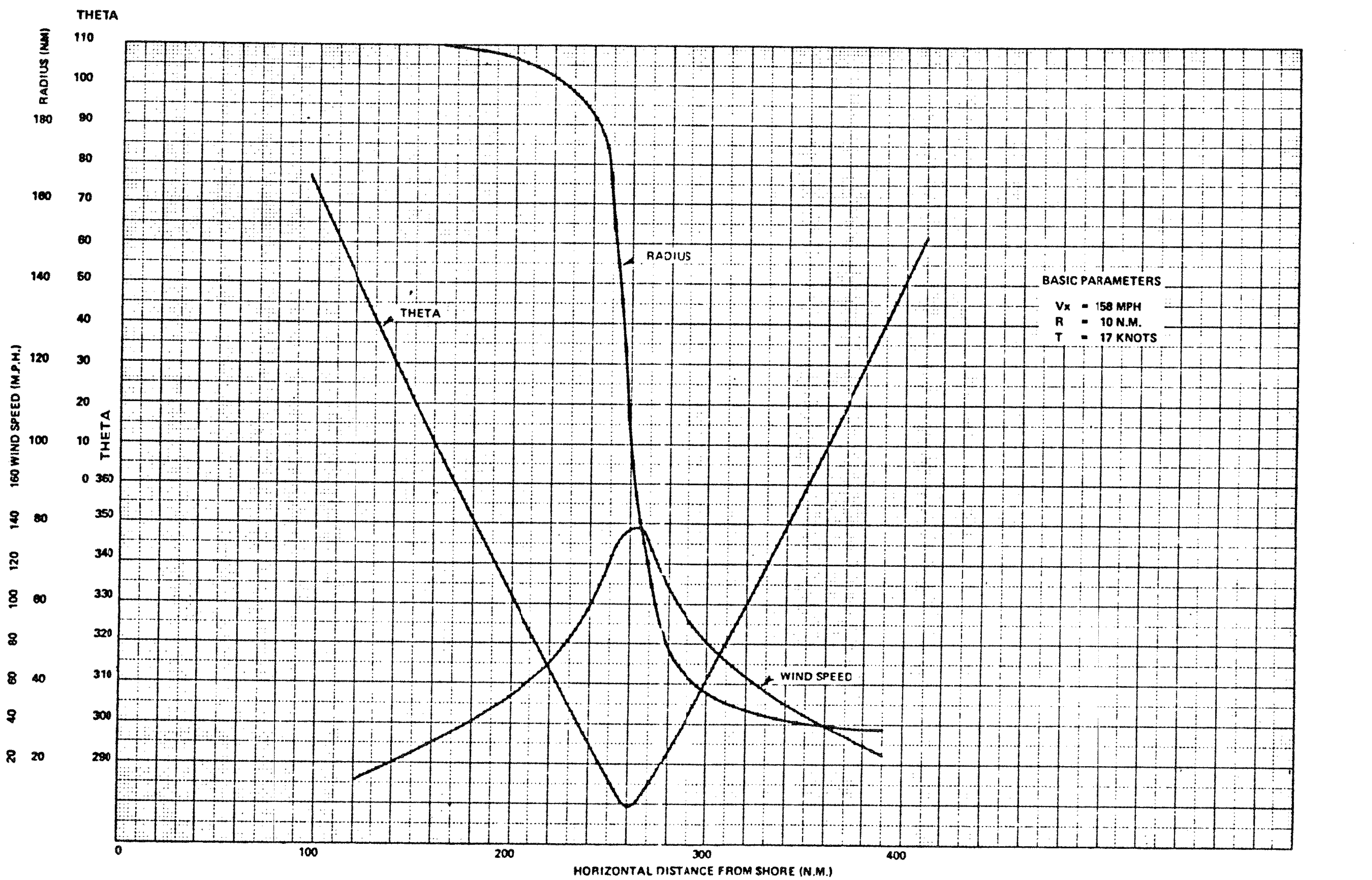
WIND FIELD - PMH - CASE 2

FIGURE 2.4-7b



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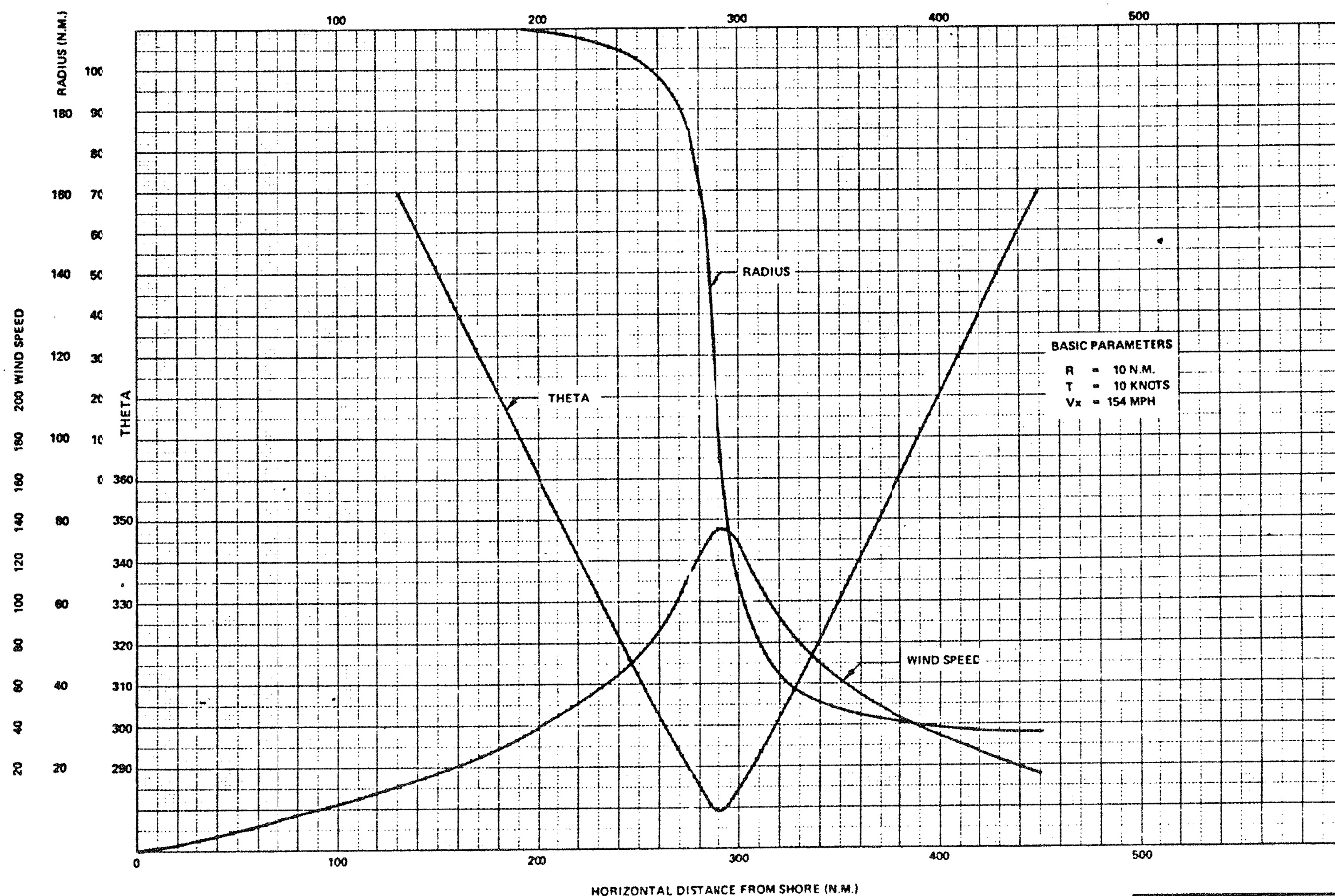
ST. LUCIE PLANT UNIT 1

WIND FIELD - PMH - CASE 4

FIGURE 2.4-7c



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ST. LUCIE PLANT UNIT 1

WIND FIELD - PMH - CASE 3

FIGURE 2.4.7d

FIGURE 2.4-8

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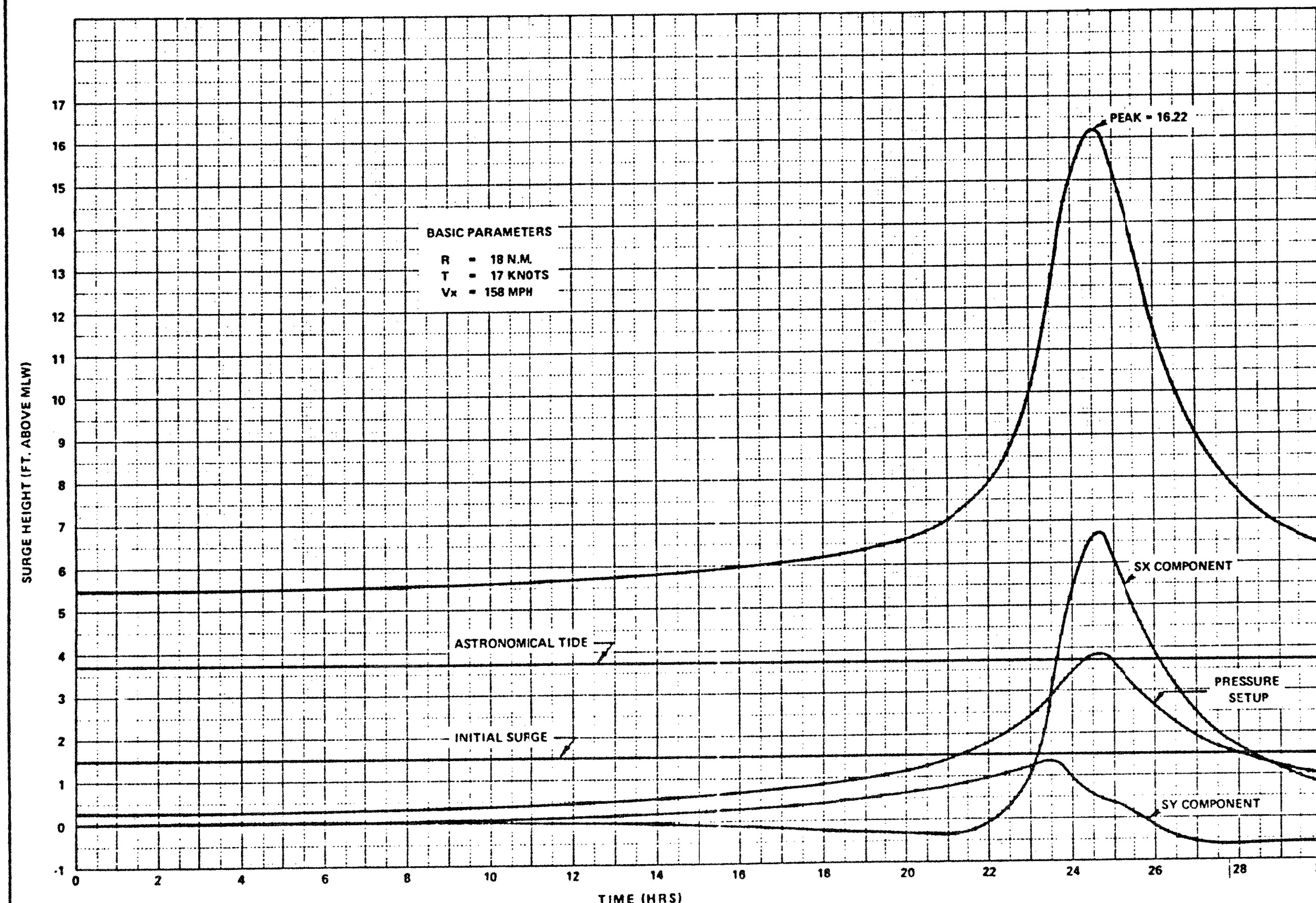
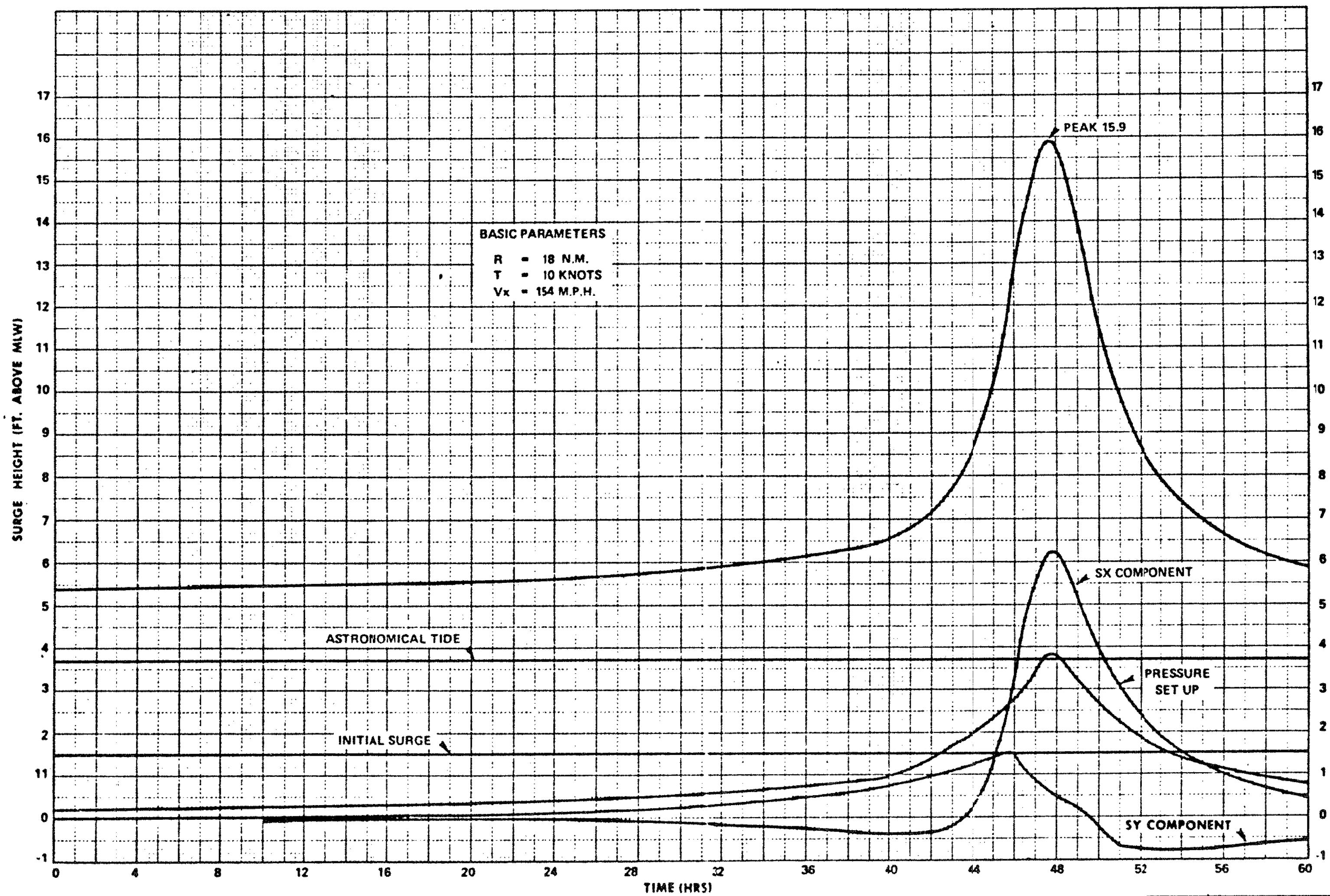


FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

TOTAL SURGE HYDROGRAPH
PMH - CASE I

FIGURE 2.4-8a



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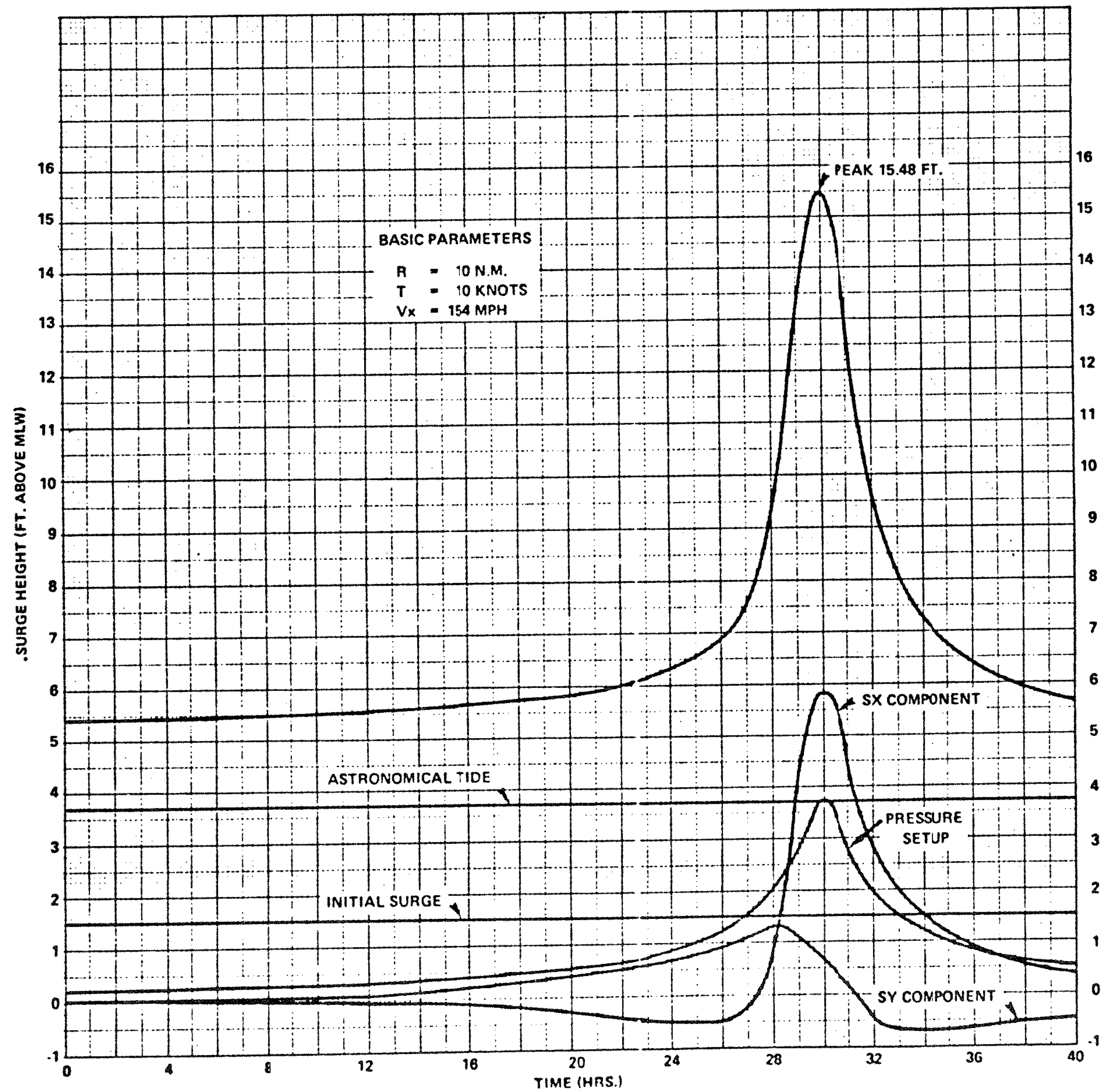
ST. LUCIE PLANT UNIT 1

TOTAL SURGE HYDROGRAPH
PMH - CASE 2

FIGURE 2.4-86



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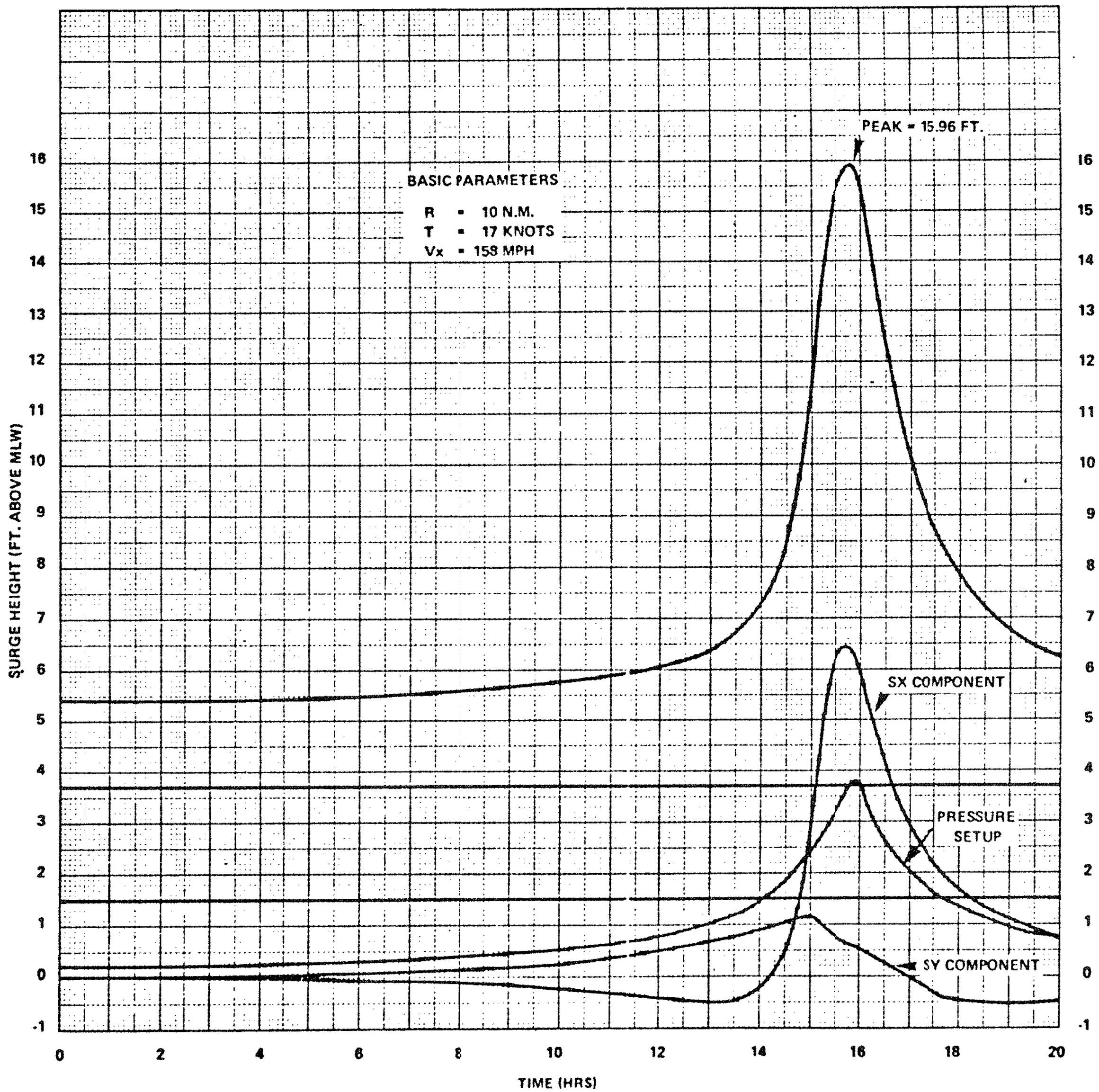
TOTAL SURGE HYDROGRAPH
PMH - CASE 3

FIGURE 2.4-8c



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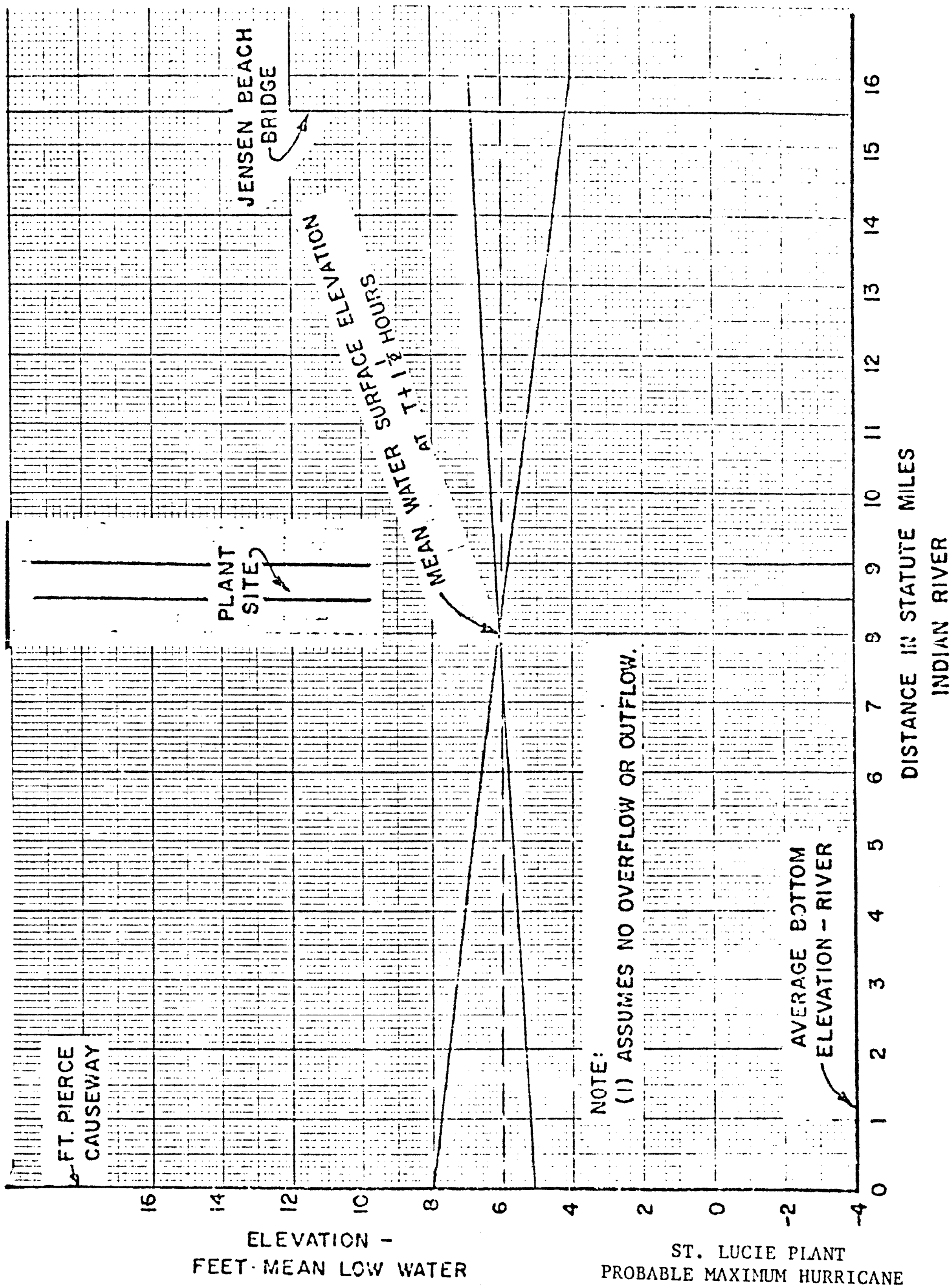
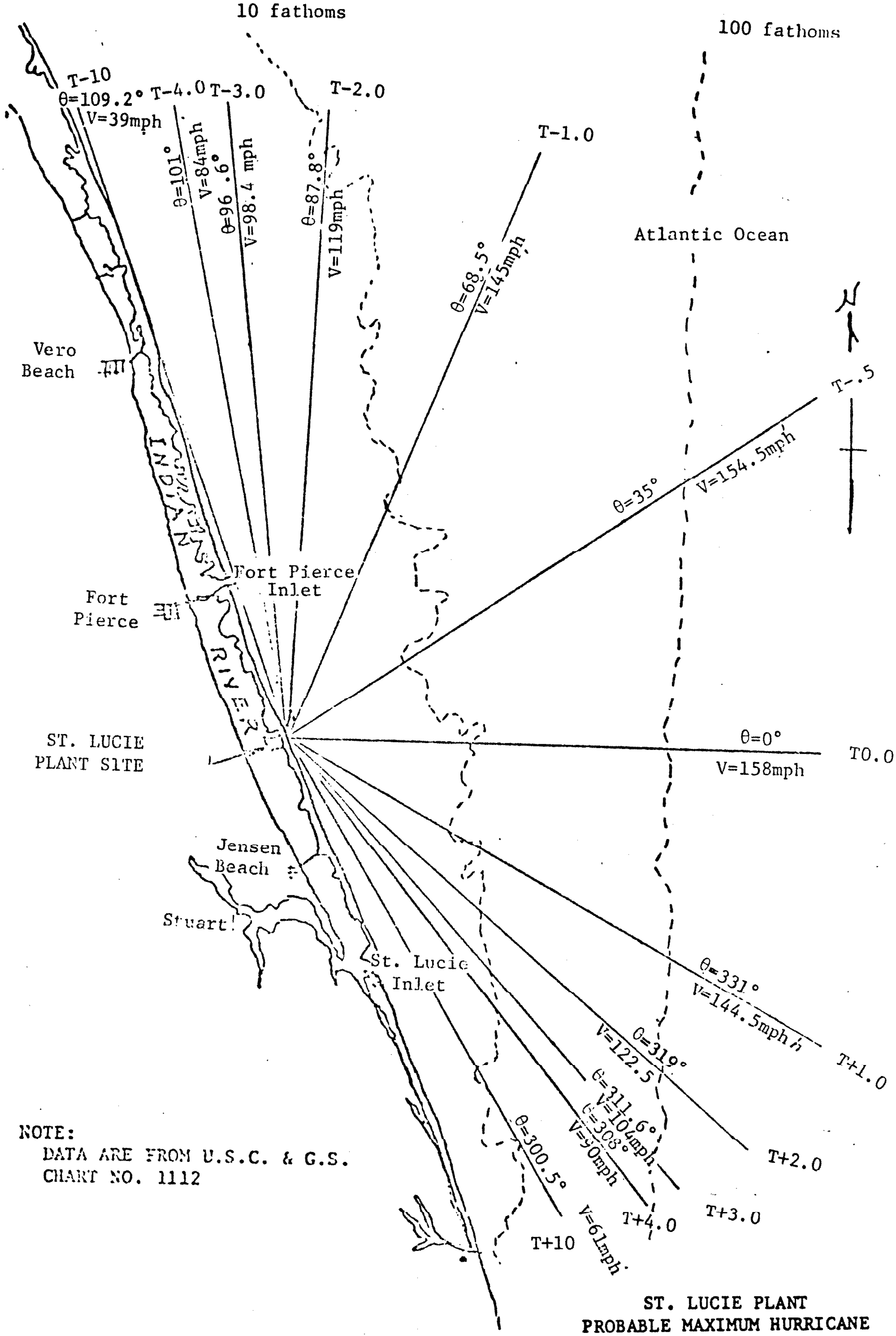
ST. LUCIE PLANT UNIT 1

TOTAL SURGE HYDROGRAPH
PMH - CASE 4

FIGURE 2.4-8d



ST. LUCIE PLANT
PROBABLE MAXIMUM HURRICANE
TIDE IN INDIAN RIVER
FIG. 2.4-9



NOTE:
DATA ARE FROM U.S.C. & G.S.
CHART NO. 1112

ST. LUCIE PLANT
PROBABLE MAXIMUM HURRICANE
HOURLY WIND-TIDE FETCHES

FIG. 2.4-10

FIGURE 2.4-11

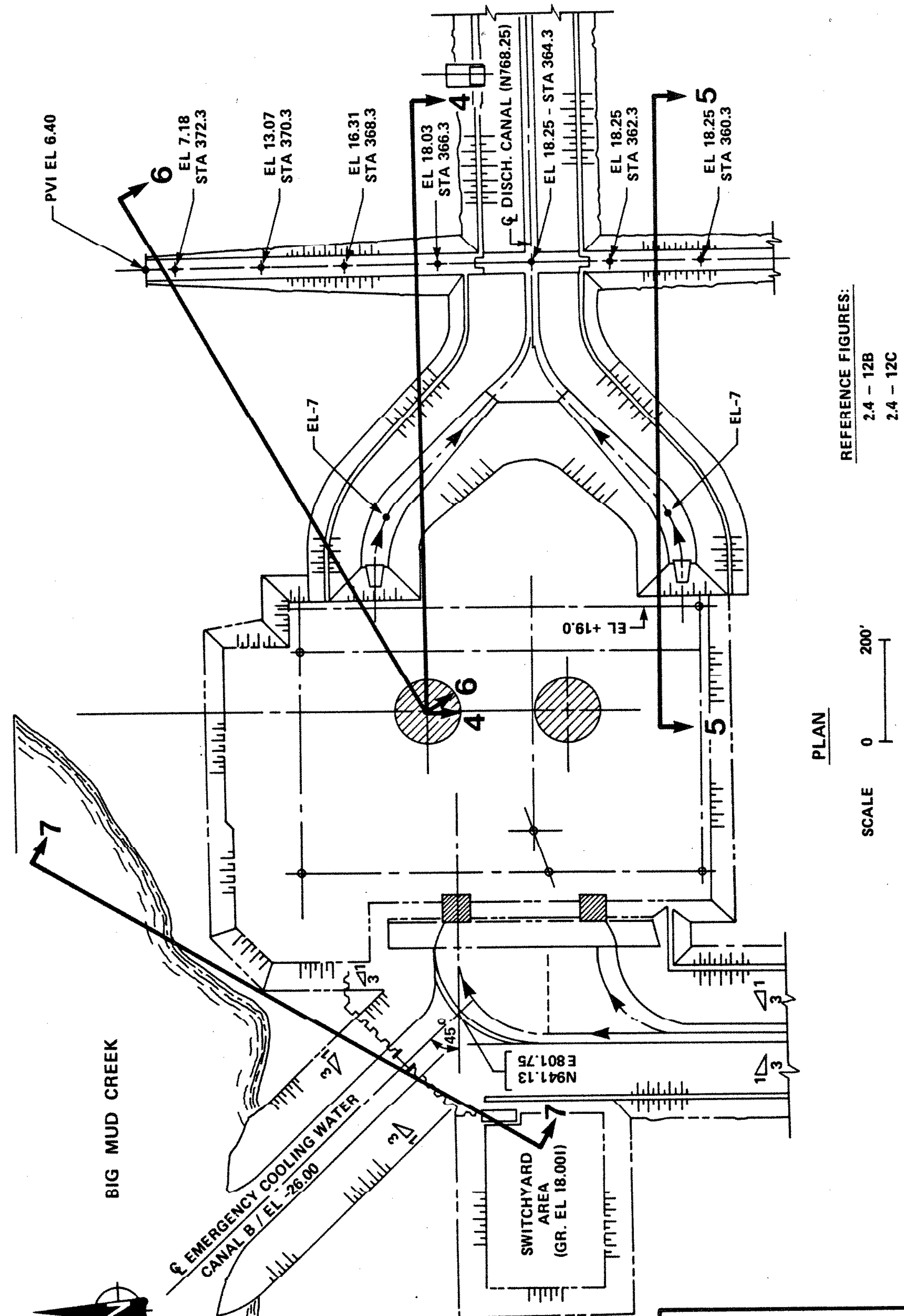
HAS BEEN INTENTIONALLY

DELETED

FIGURE 2.4-12

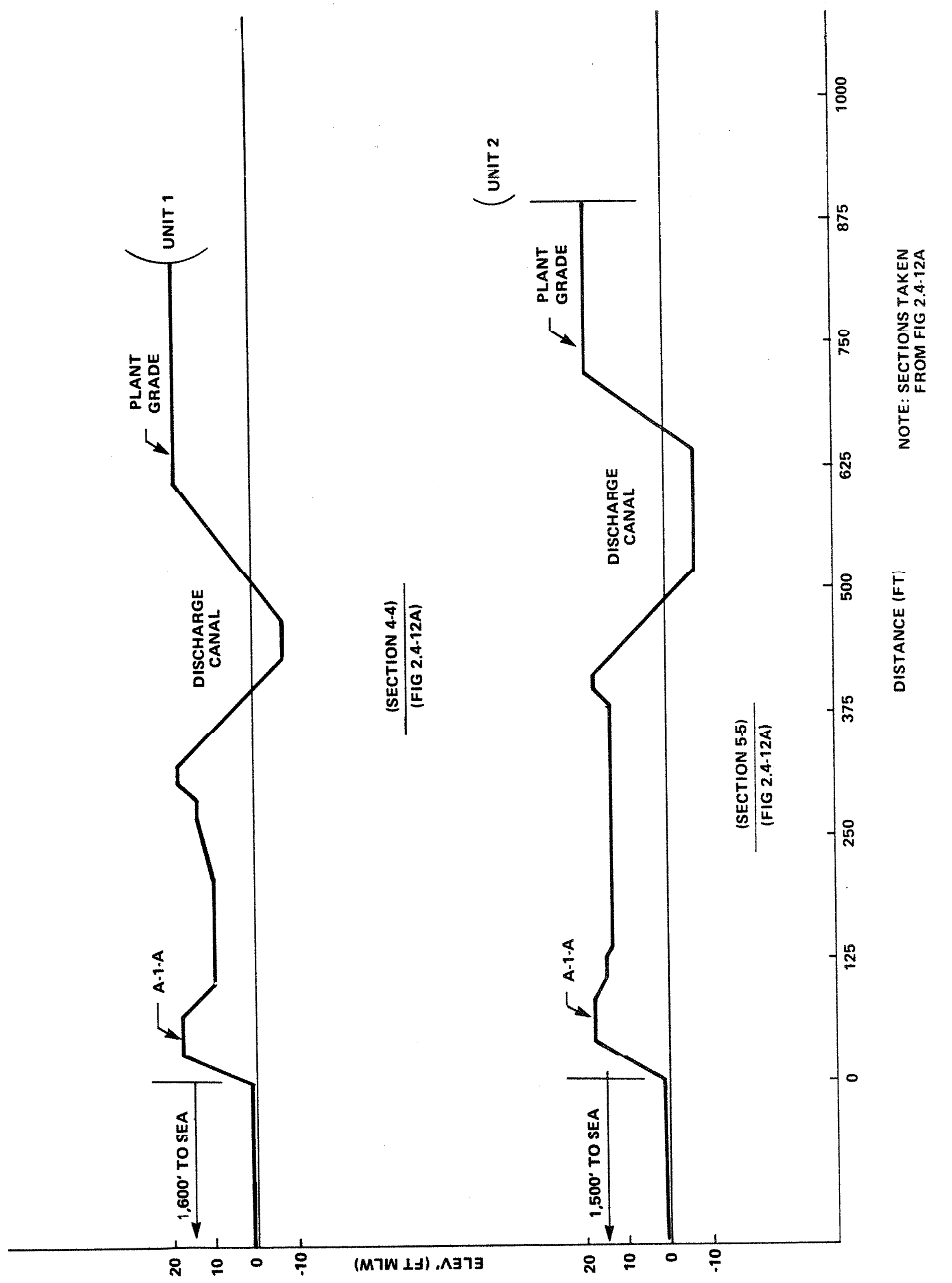
HAS BEEN INTENTIONALLY

DELETED



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

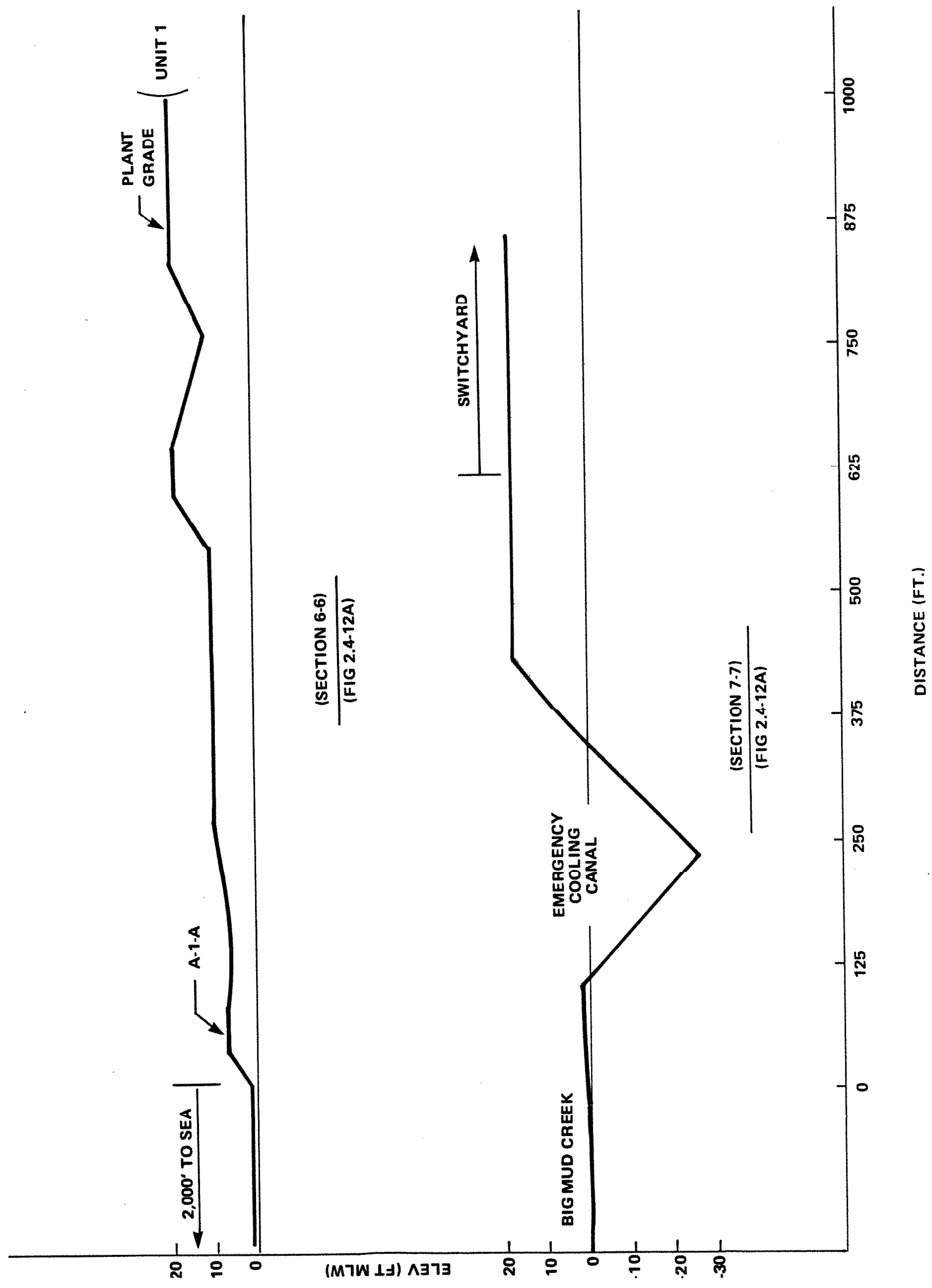
PROBABLE MAXIMUM HURRICANE PATH
THROUGH THE PLANT SITE
FIGURE 2.4–12a



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

PLANT PROFILES ALONG PMH PATH

FIGURE 2.4-12b



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

PLANT PROFILES ALONG
PMH PATH

FIGURE 2.4-12c

Security-Related Information
Figure Withheld Under 10 CFR 2.390

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

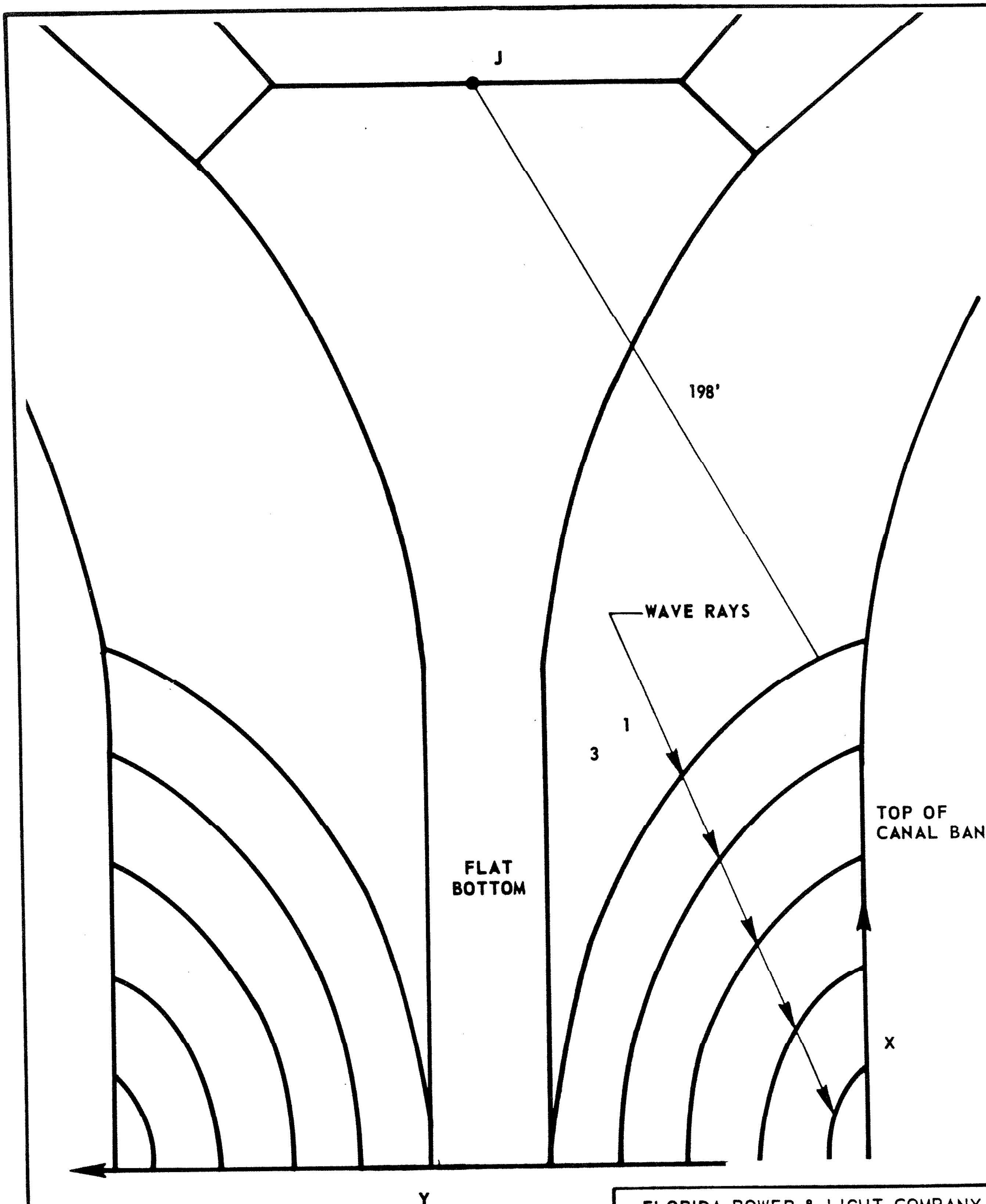
PARTIAL PLOT PLAN

FIGURE 2.4-12D

Security-Related Information
Figure Withheld Under 10 CFR 2.390

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

DISCHARGE CANAL BOTTOM PROFILE
AND AREA PLAN
FIGURE 2.4–12e



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

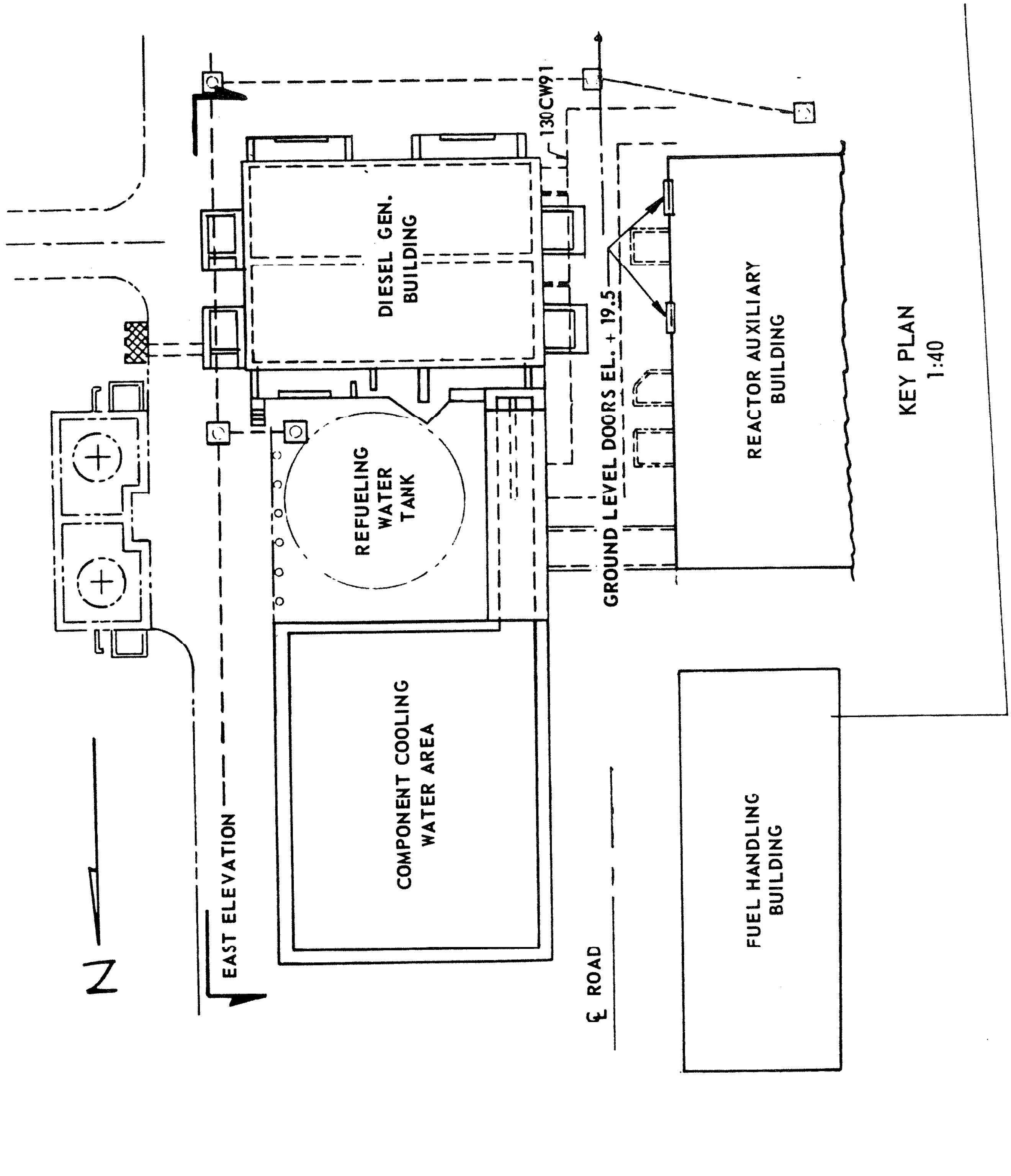
DISCHARGE CANAL WAVE RAYS

FIGURE 2.4-12F

Security-Related Information
Figure Withheld Under 10 CFR 2.390

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

DISCHARGE CANAL NOSE PROTECTION
FIGURE 2.4–12g



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

PHYSICAL ARRANGEMENT OF EFFECTIVE WAVE RUNUP BARRIER EAST OF REACTOR AUXILIARY AND FUEL HANDLING BUILDINGS
SHEET 1 OF 2

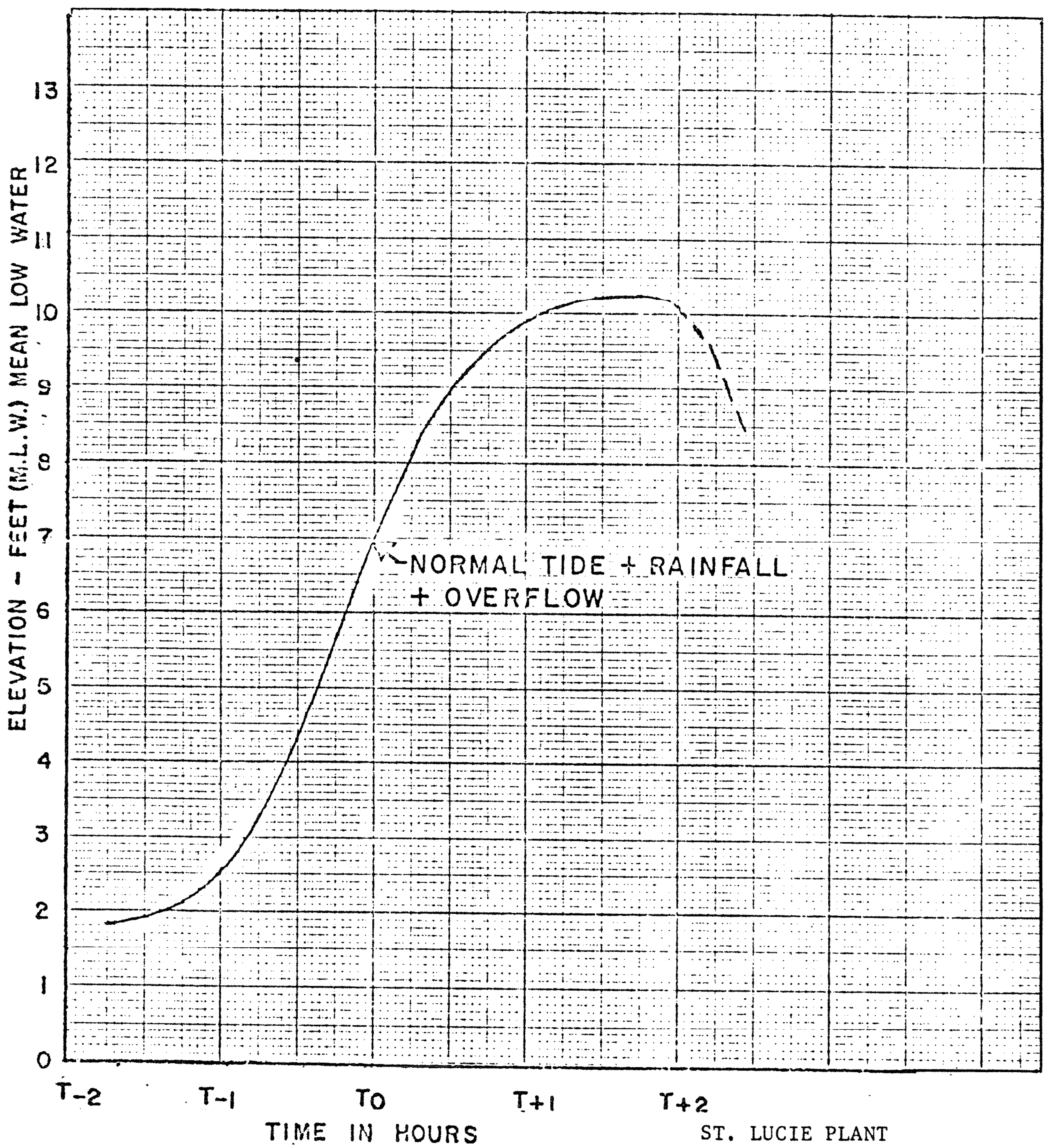
FIGURE 2.4-12h

Withheld Under 10 CFR 2.390

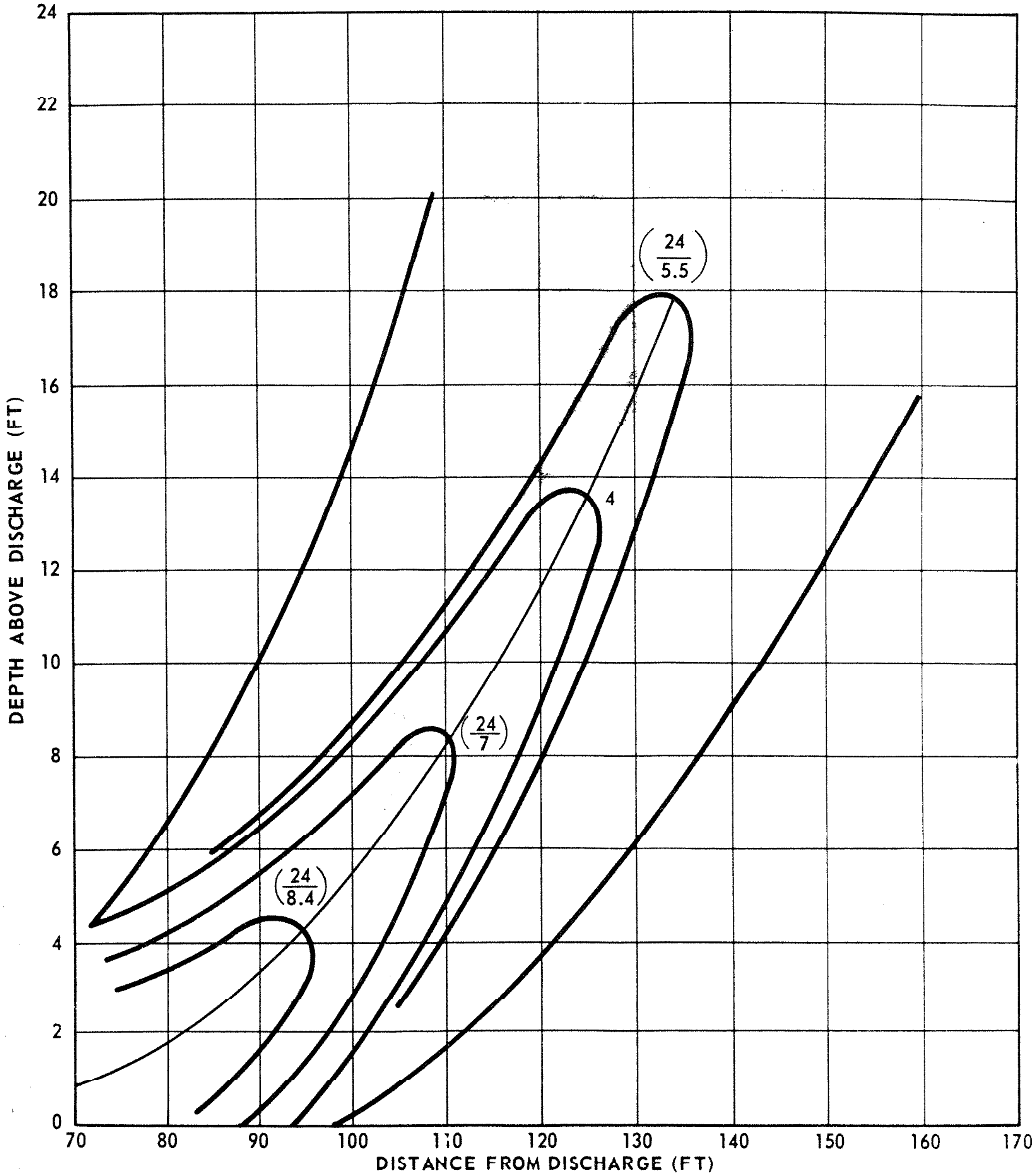
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

PHYSICAL ARRANGEMENT OF EFFECTIVE WAVE RUNUP BARRIER EAST OF REACTOR AUXILIARY AND FUEL HANDLING BUILDINGS
SHEET 2 OF 2

FIGURE 2.4-12i



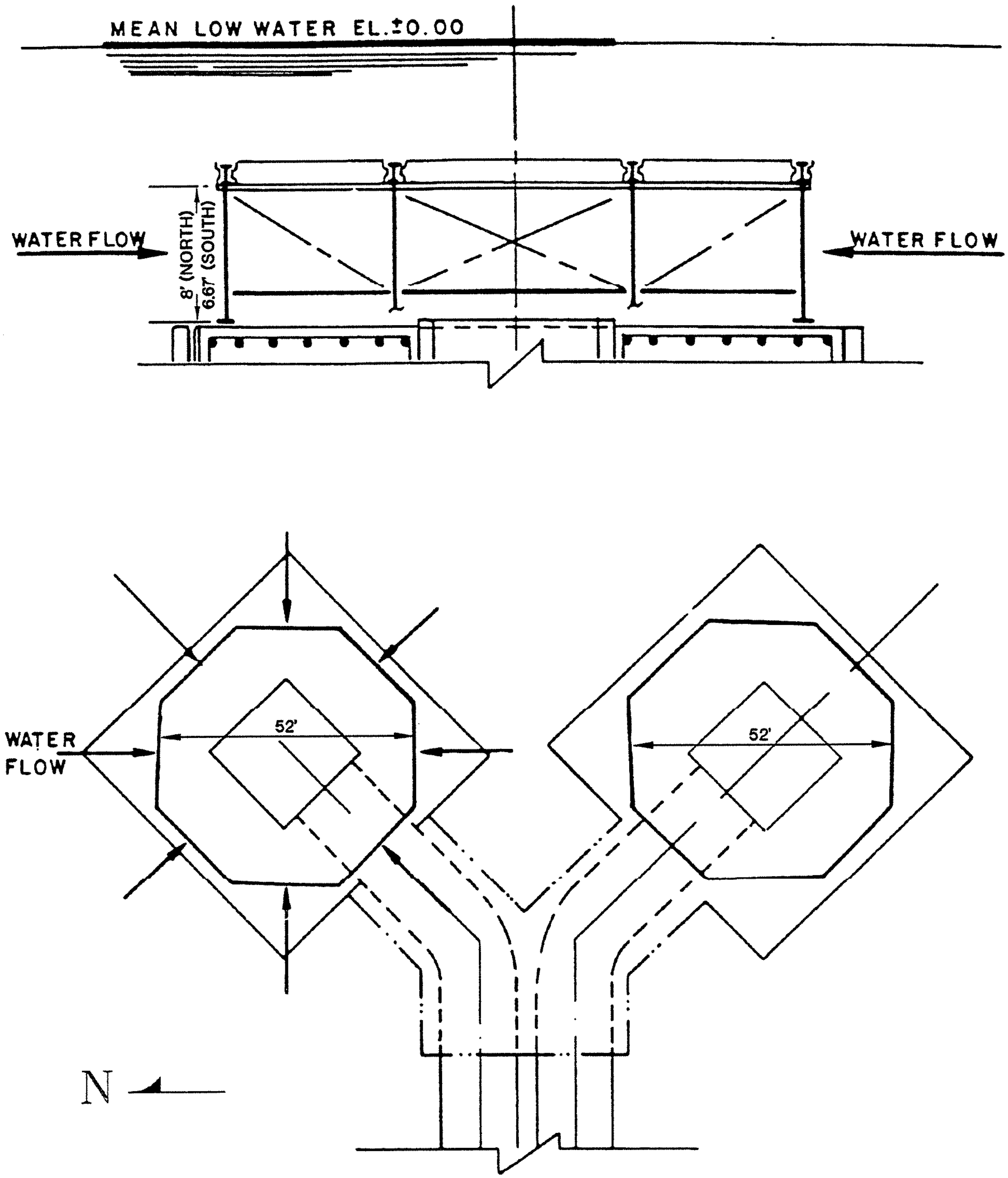
ST. LUCIE PLANT
PROBABLE MAXIMUM HURRICANE
INDIAN RIVER - MEAN WATER LEVEL
HYDROGRAPHS
FIG. 2.4-13



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

PLUME TEMPERATURE DISTRIBUTION
BASED ON MATH. MODEL

FIGURE 2.4-13A

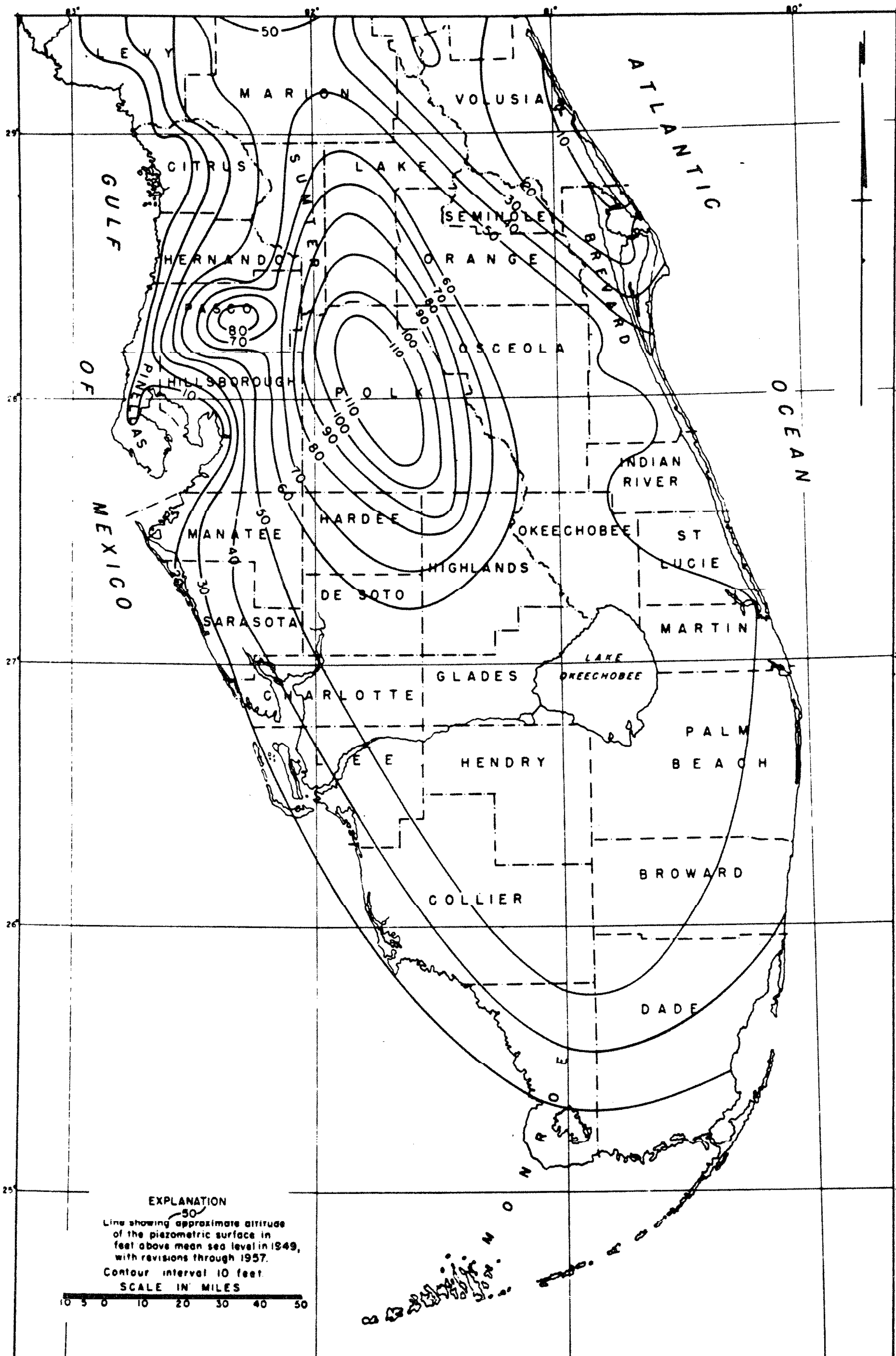


AMENDMENT NO. 12 (12/93)

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT - UNIT 1

DETAILED VELOCITY CAP FOR
COOLING WATER INTAKE

FIGURE 2.4-13B

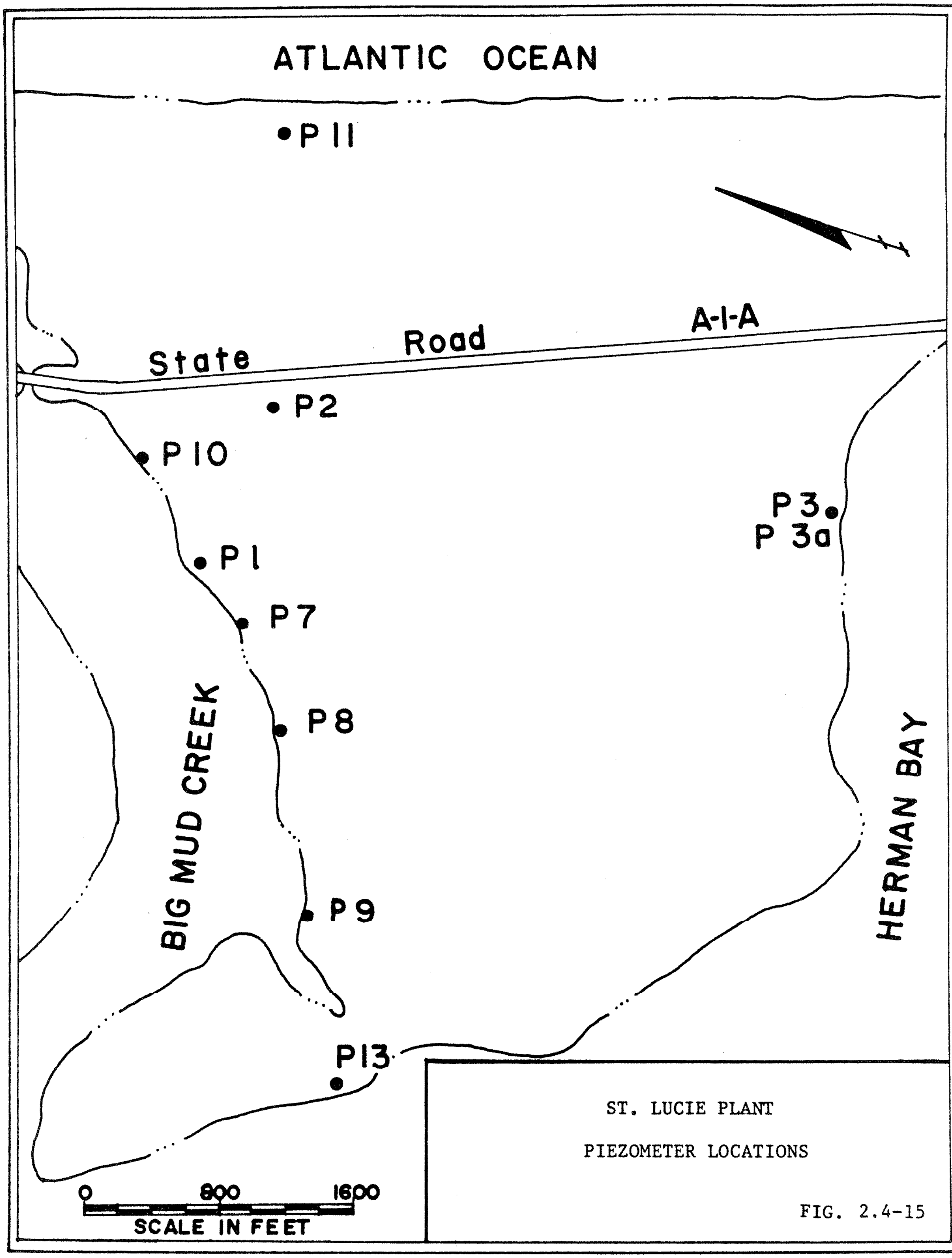


PIEZOMETRIC SURFACE OF THE FLORIDAN AQUIFER (AFTER LICHTLER - 1960)

ST. LUCIE PLANT

PIEZOMETRIC SURFACE OF THE FLORIDAN AQUIFER

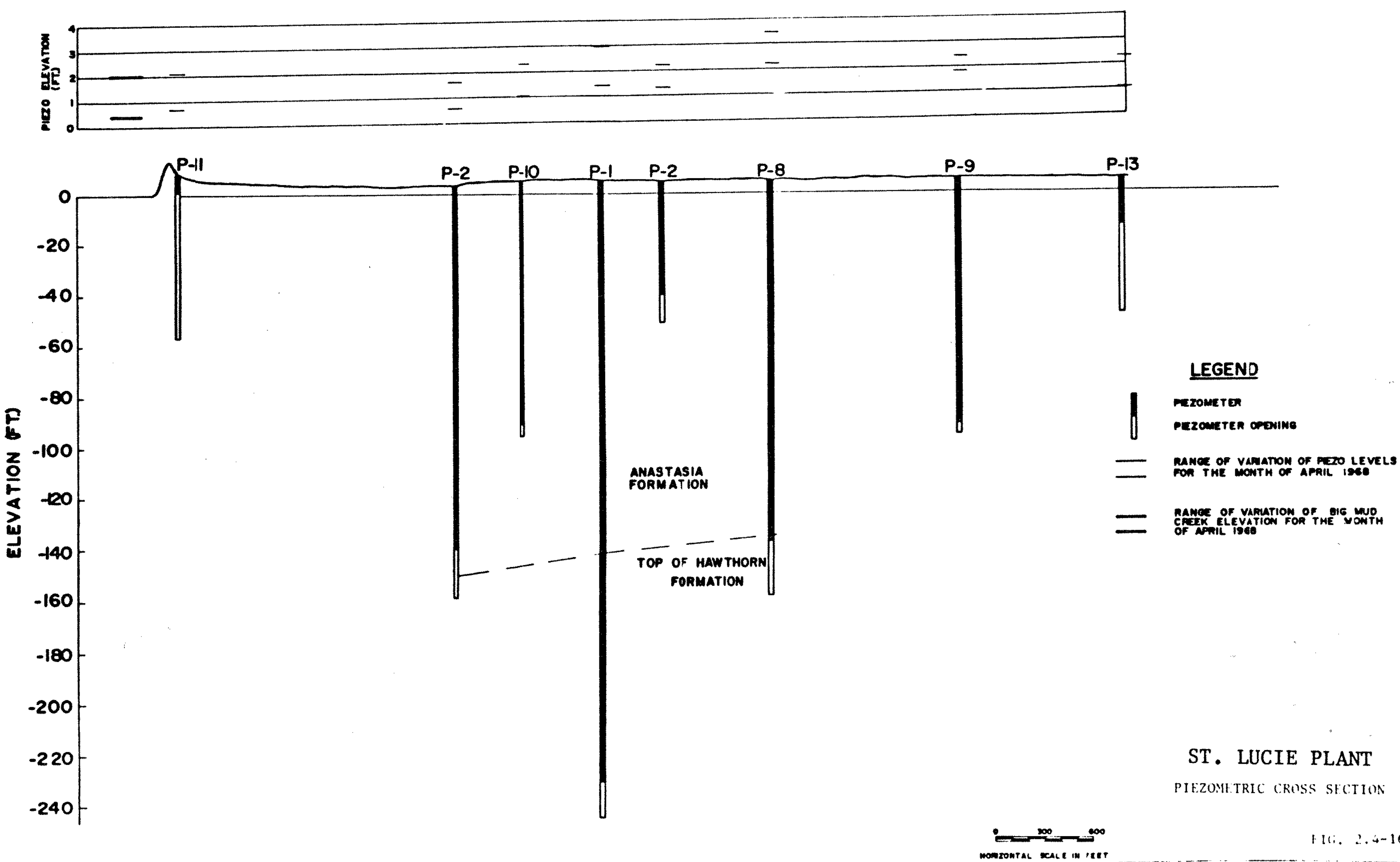
FIG. 2.4-14



ST. LUCIE PLANT

PIEZOMETER LOCATIONS

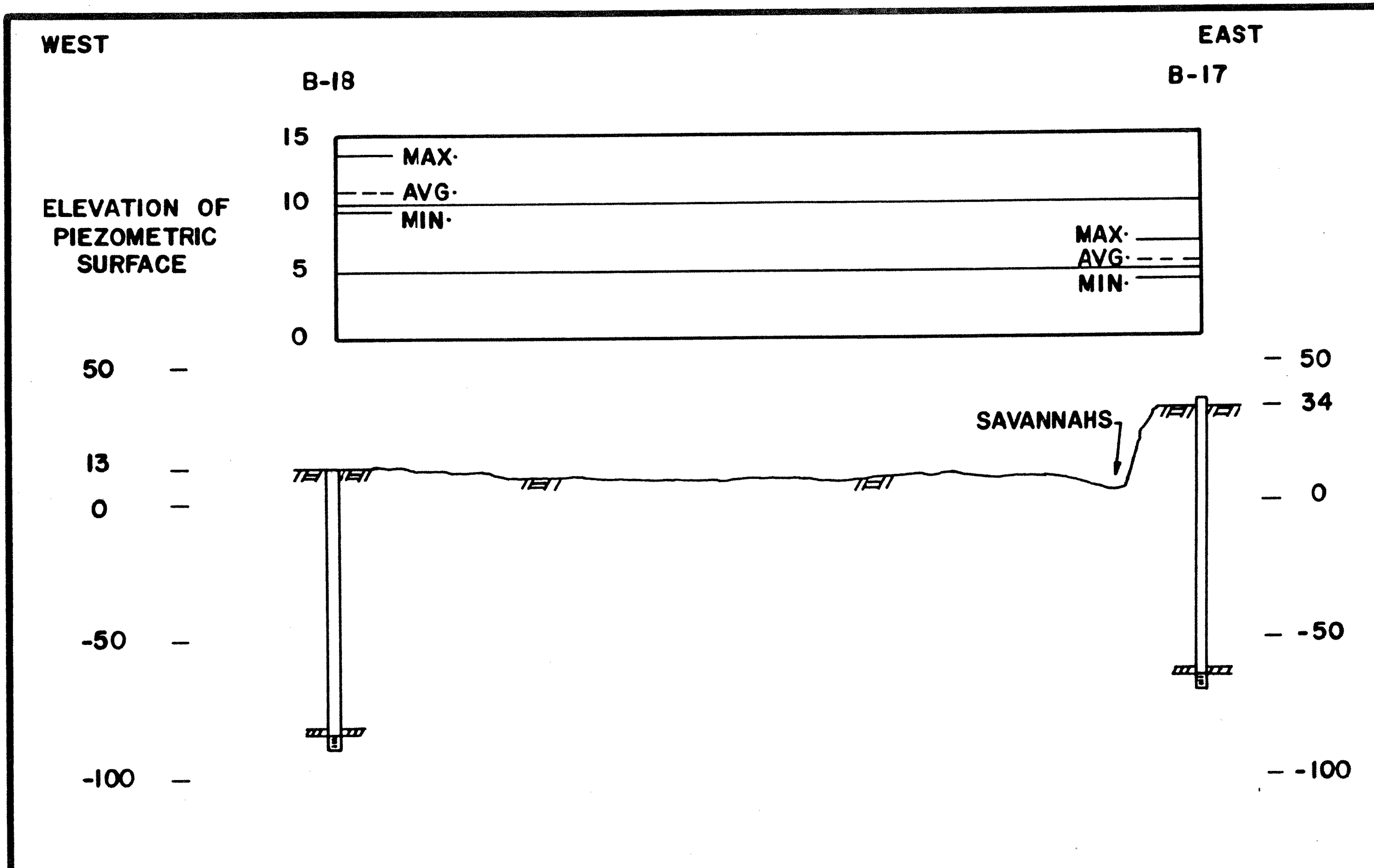
FIG. 2.4-15



ST. LUCIE PLANT

PIEZOMETRIC CROSS SECTION

FIG. 2.4-16

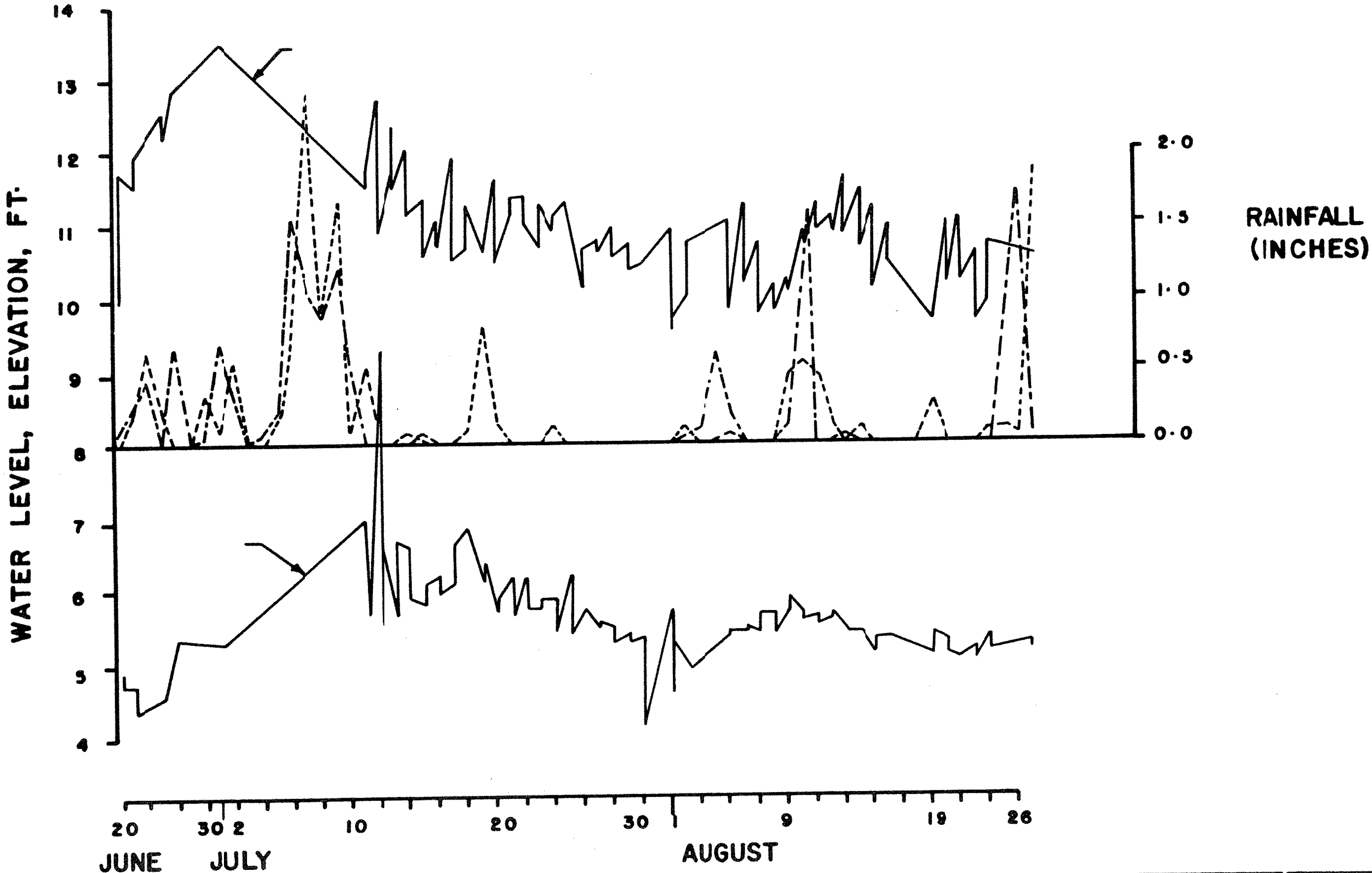


CLAY, PLUG
1" = 2500' HORIZONTAL
1" = 50 VERTICAL

ST. LUCIE PLANT

PIEZOMETRIC CROSS SECTION
BORINGS 17 AND 18

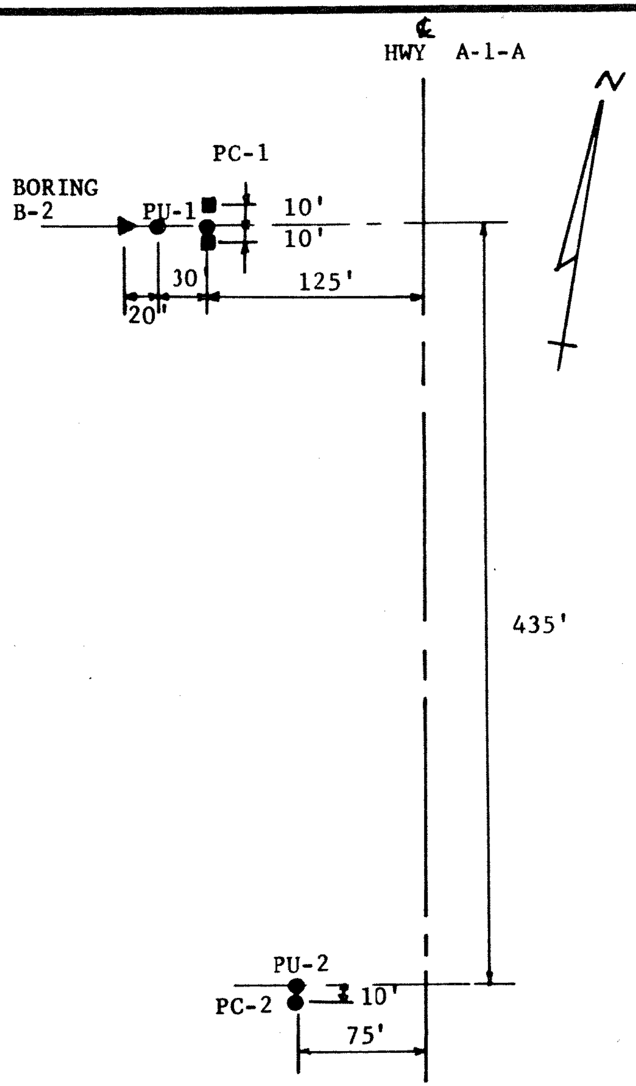
FIG. 2.4-17



— - — FT. PIERCE RAINFALL
- - - - - - STUART RAINFALL

ST. LUCIE PLANT
PIEZOMETRIC DATA
FOR P-17, P-18

FIG 2.4-18



TEST RESULTS

HOLE	PERMEABILITY , k (FEET PER SECOND)	
	OPEN PIPE METHOD	WELL PERMEAMETER METHOD
PC - 1	7.5×10^{-3}	
PU - 1		6.7×10^{-4}
PC - 2	3.4×10^{-5}	
PU - 2		4.9×10^{-5}

HUTCHINSON ISLAND - NUCLEAR PLANT
JOB NUMBER EC-163

LABORATORY PERMEABILITY TESTS

Permeability Tests were run on undisturbed samples from two locations. The samples were cut into discs 2.5 inches in diameter and 1 inch thick and sealed in closed chambers. They were then subjected to an unbalanced head of water and the rate of flow through the soil was measured.

RESULTS

BORING NO.	DEPTH	e	k	SOIL CLASSIFICATION
B-8	15'-16.5'	0.740	9.9×10^{-2} ft./min.	GREY SLIGHTLY SILTY FINE TO COARSE SAND (SHELL FRAGMENTS RETAINED IN SIEVE NO. 40)
B-13	35'-37'	0.915	2.8×10^{-2} ft./min.	GREY SLIGHTLY SILTY VERY FINE SAND WITH TRACE OF SHELLS

LEGEND

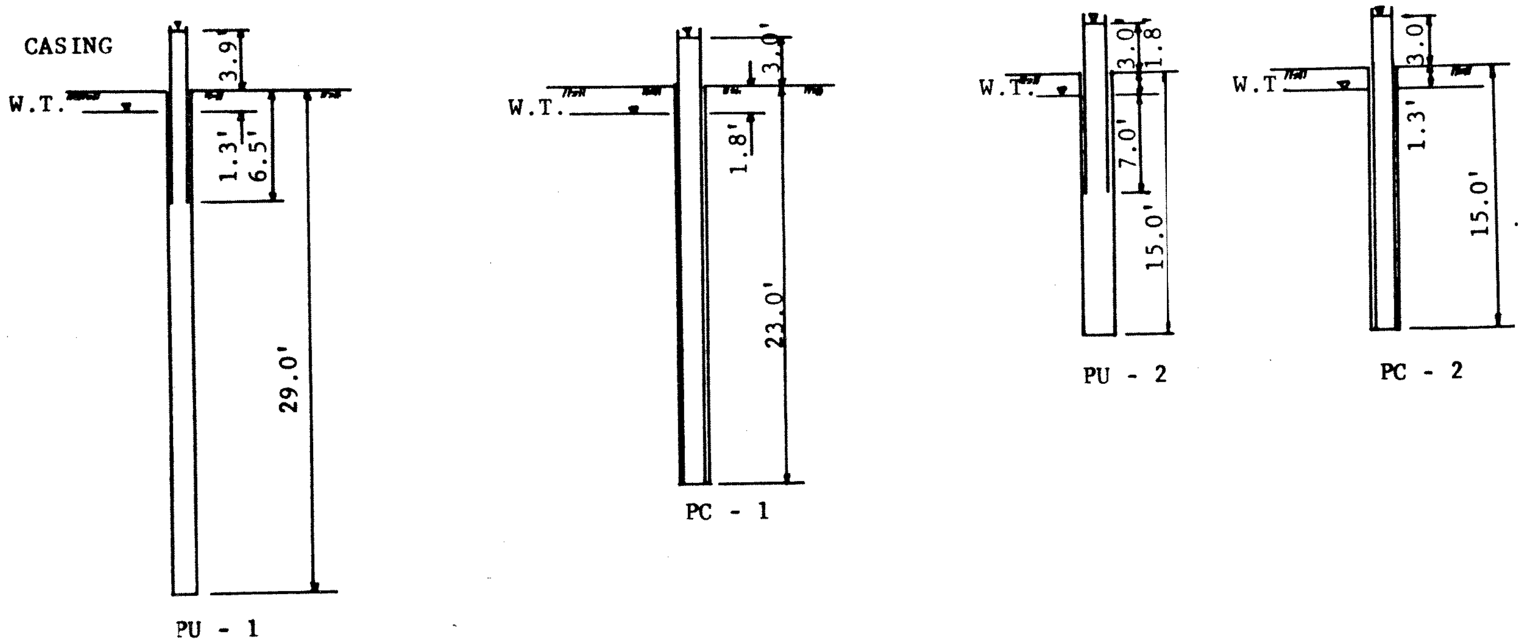
- ○ FIELD PERMEABILITY TEST HOLE
- ■ PIEZOMETER
- ▲ BORING B-2

SCALE

ST. LUCIE PLANT

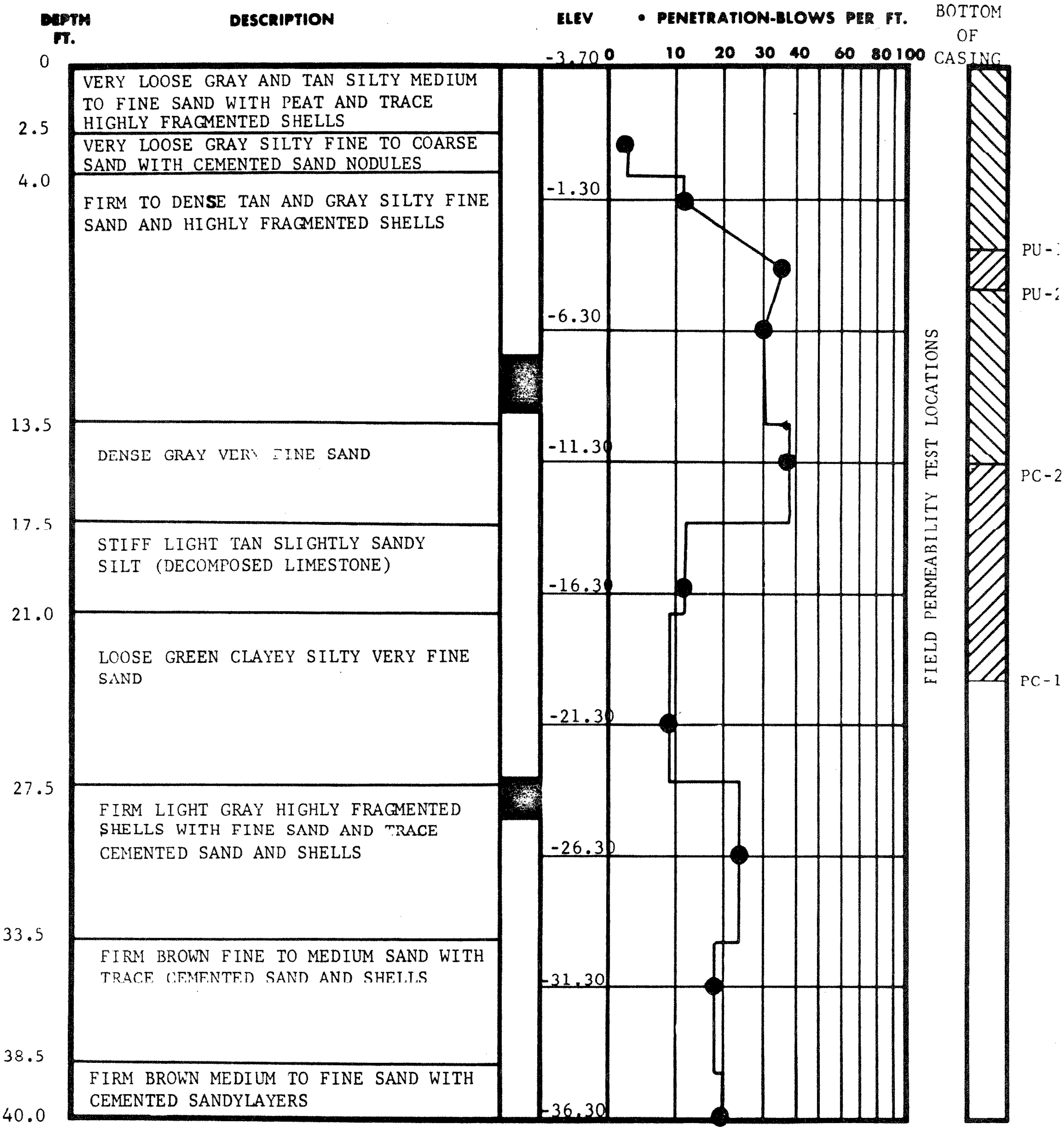
TEST BORING RESULTS

FIG. 2.4-19



NOTE: 4 inch I.D. casing used throughout.

LEGEND		ST. LUCIE PLANT
		CASING ARRANGEMENT
SCALE		FIG. 2.4-20



TEST BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

|50| % ROCK CORE RECOVERY

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

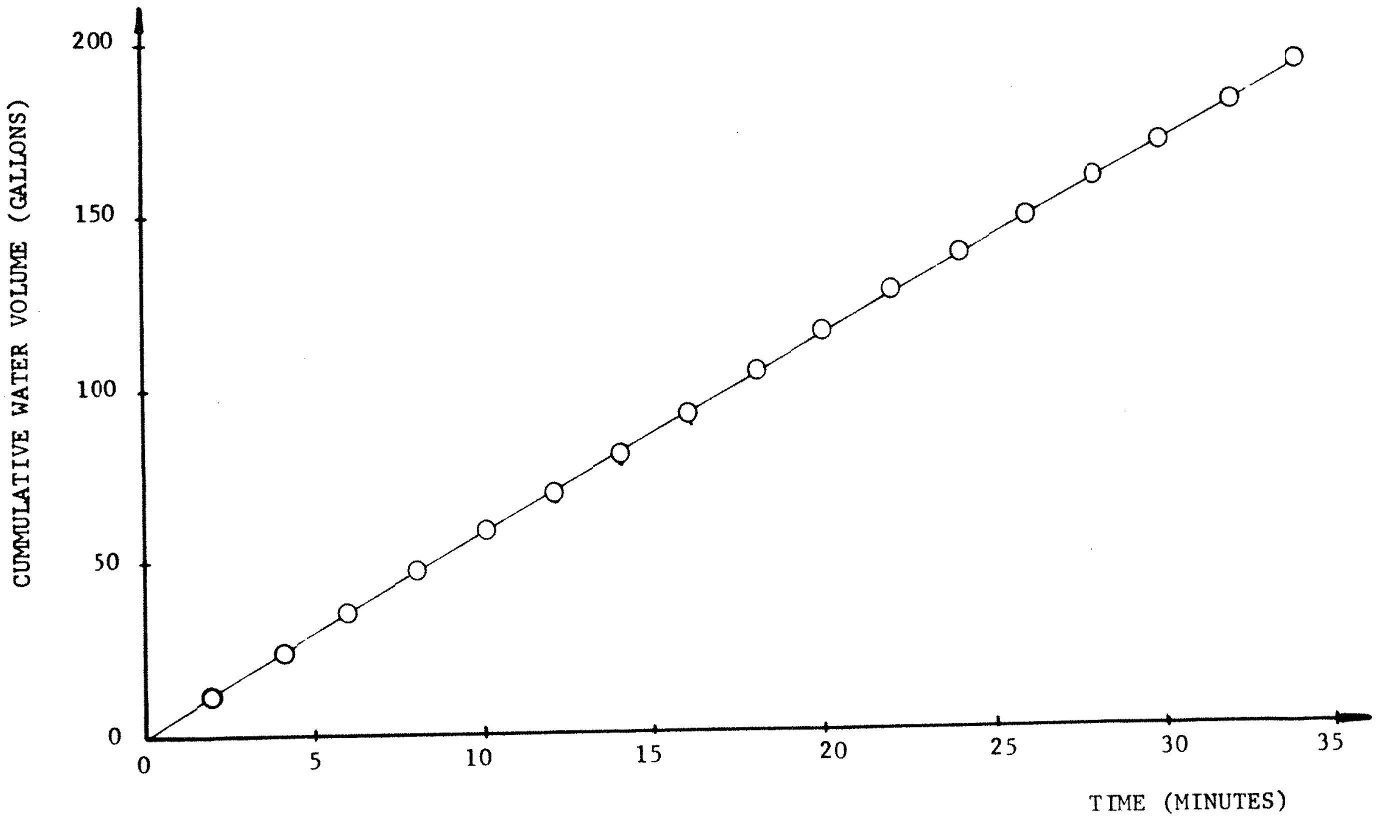
ST. LUCIE PLANT

TEST BORING RECORD

BORING B-2

FIG. 2.4-21

PU_1.



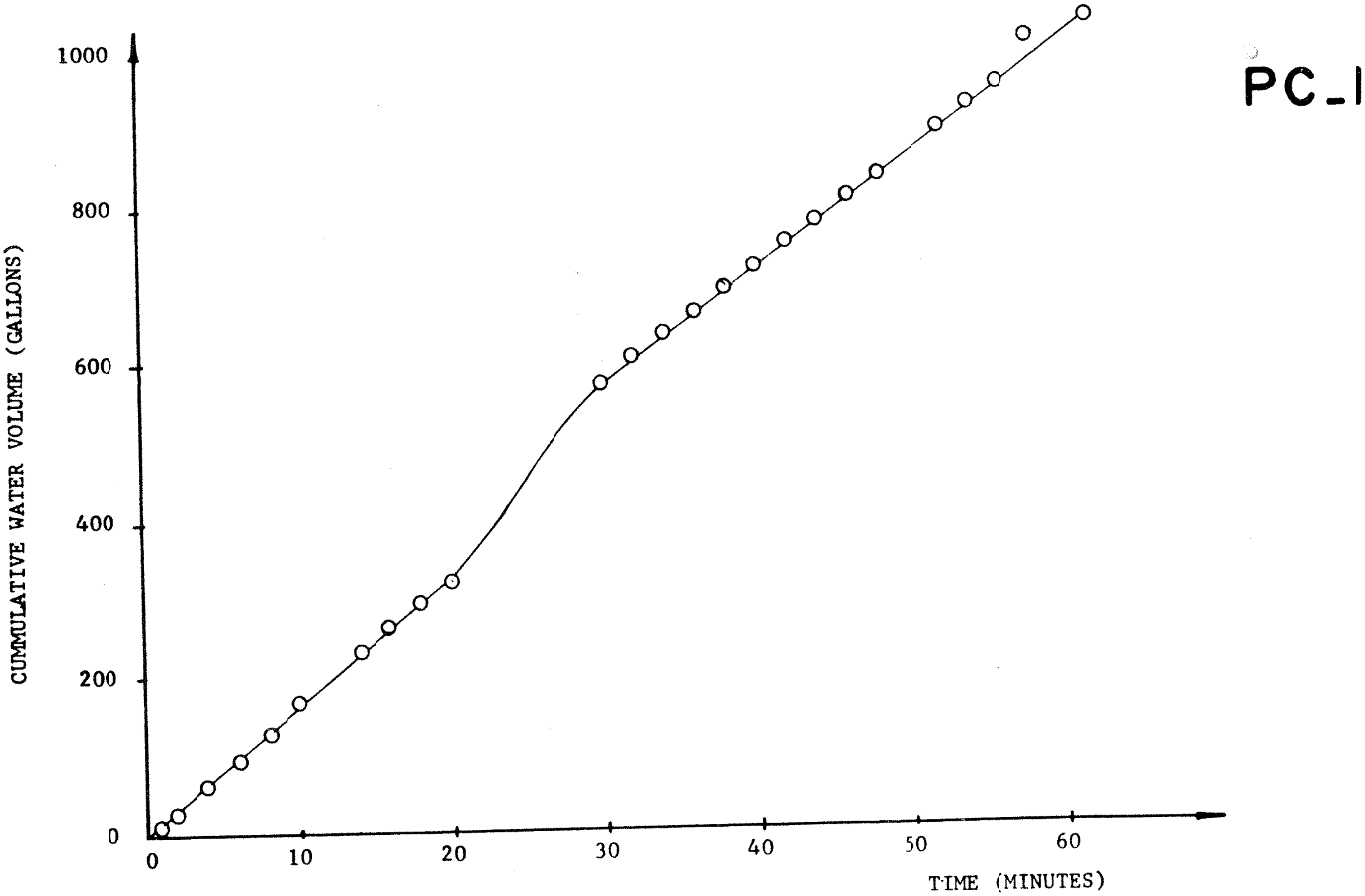
LEGEND

FIELD PERMEABILITY TEST
WELL PERMEAMETER METHOD

SCALE

ST. LUCIE PLANT

FIG. 2.4-22



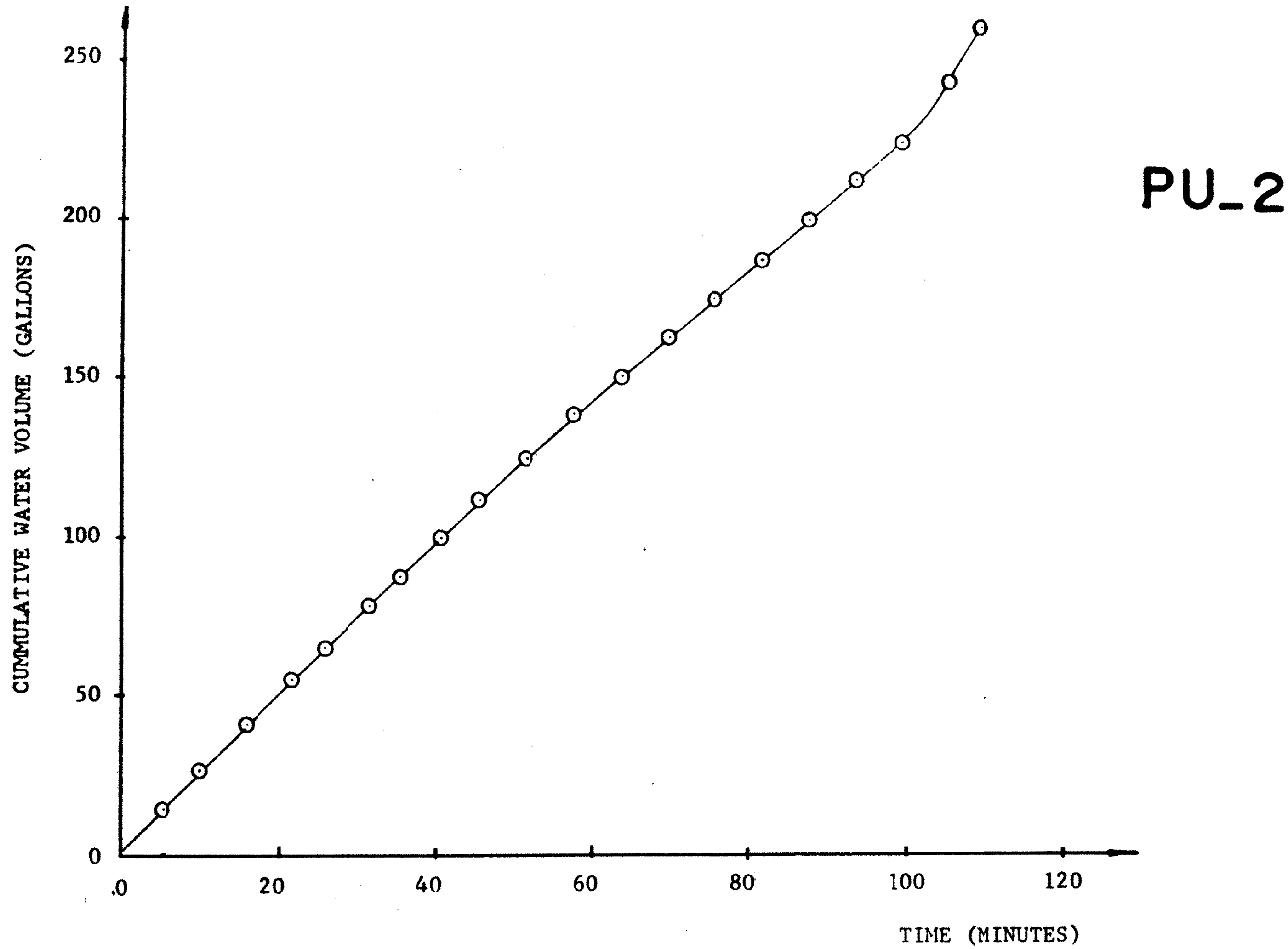
LEGEND

FIELD PERMEABILITY TEST
OPEN-END PIPE METHOD

SCALE

ST. LUCIE PLANT

FIG. 2.4-23



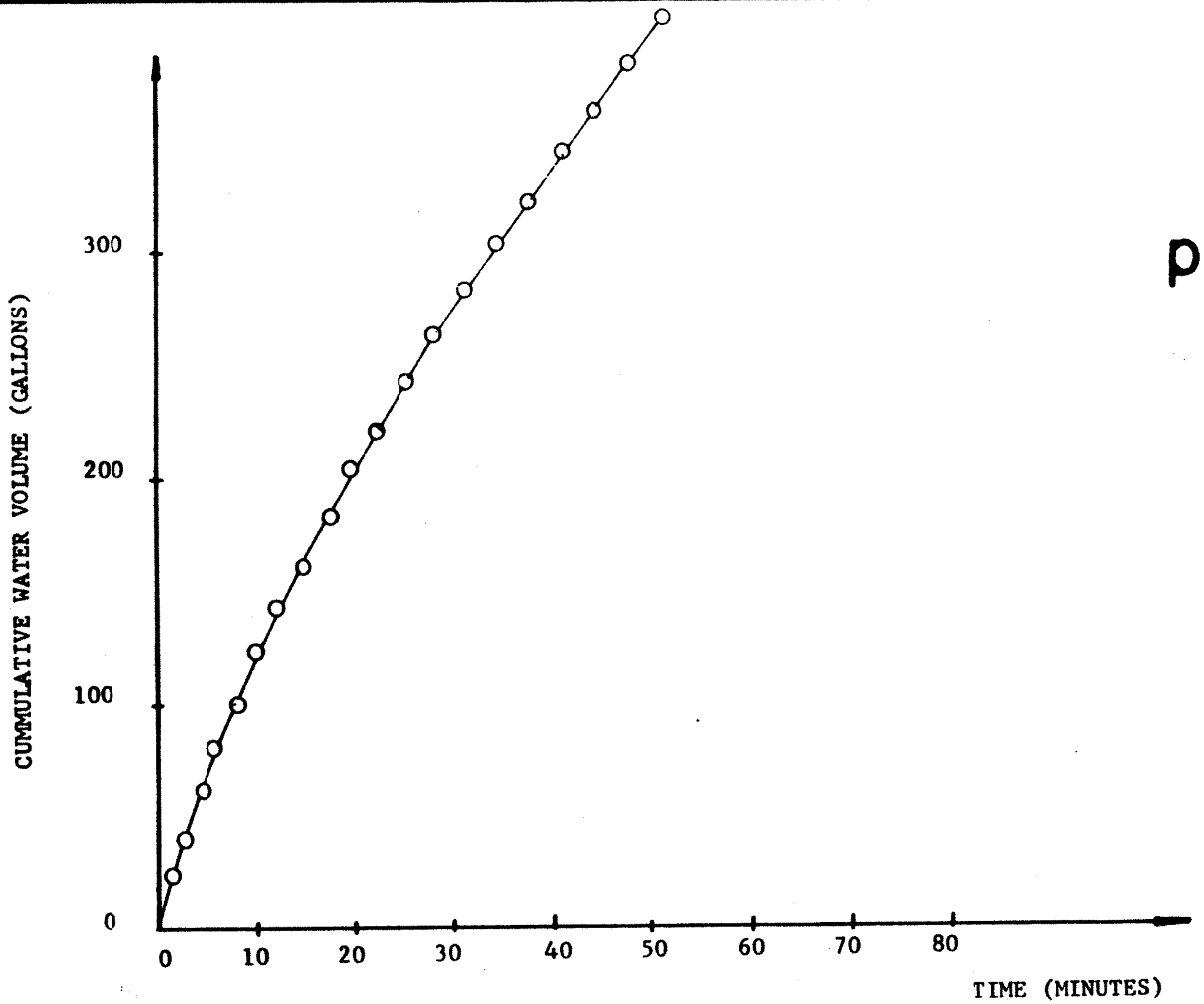
LEGEND

FIELD PERMEABILITY TEST
WELL PERMEAMETER METHOD

SCALE

ST. LUCIE PLANT

FIG. 2.4-24



LEGEND

FIELD PERMEABILITY TEST
OPEN-END PIPE METHOD

SCALE

ST. LUCIE PLANT

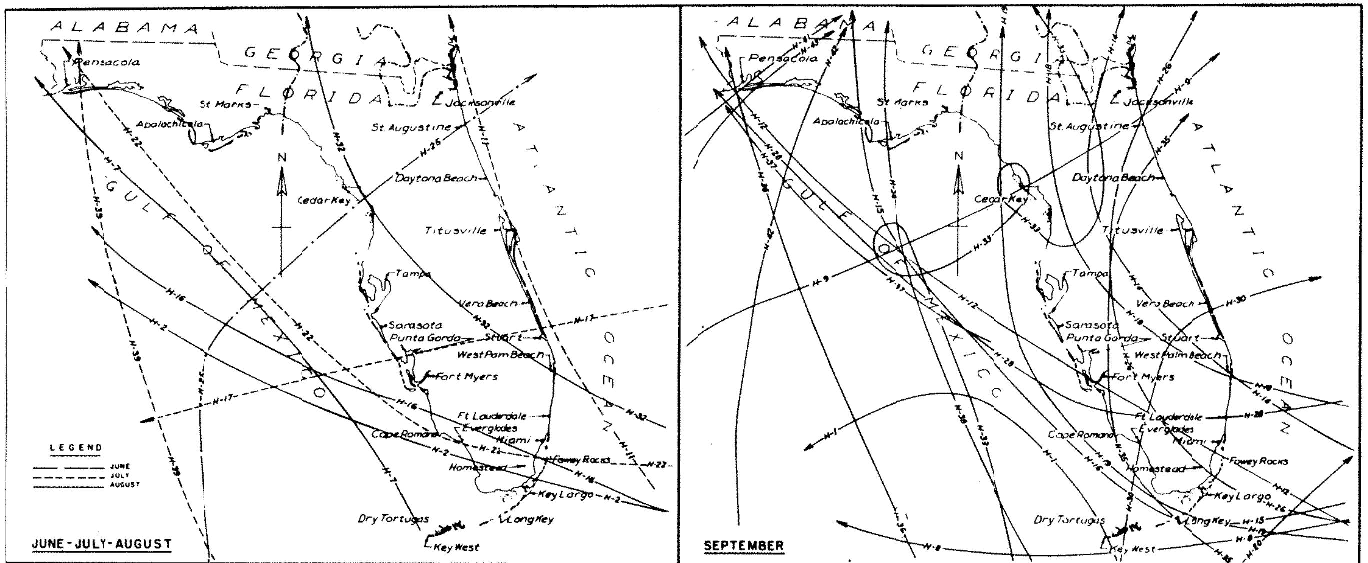
FIG. 2.4-25

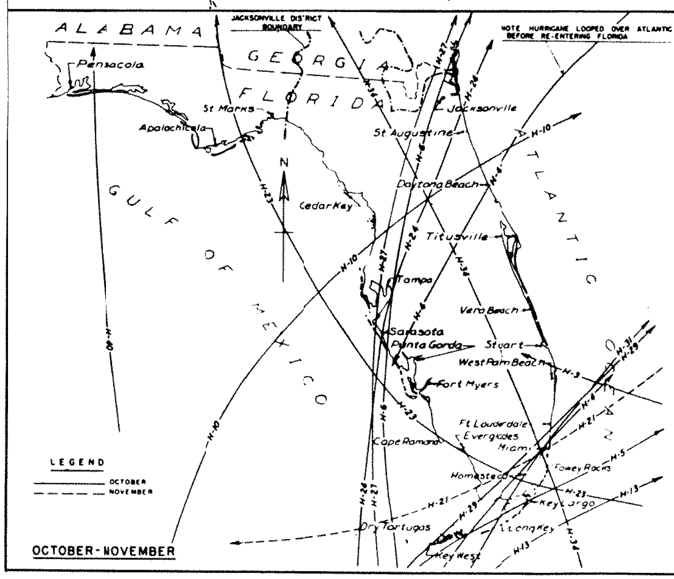
Withheld Under 10 CFR 2.390

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

INTAKE COOLING WATER
PUMP SECTIONAL ARRANGEMENT

FIGURE 2.4-26





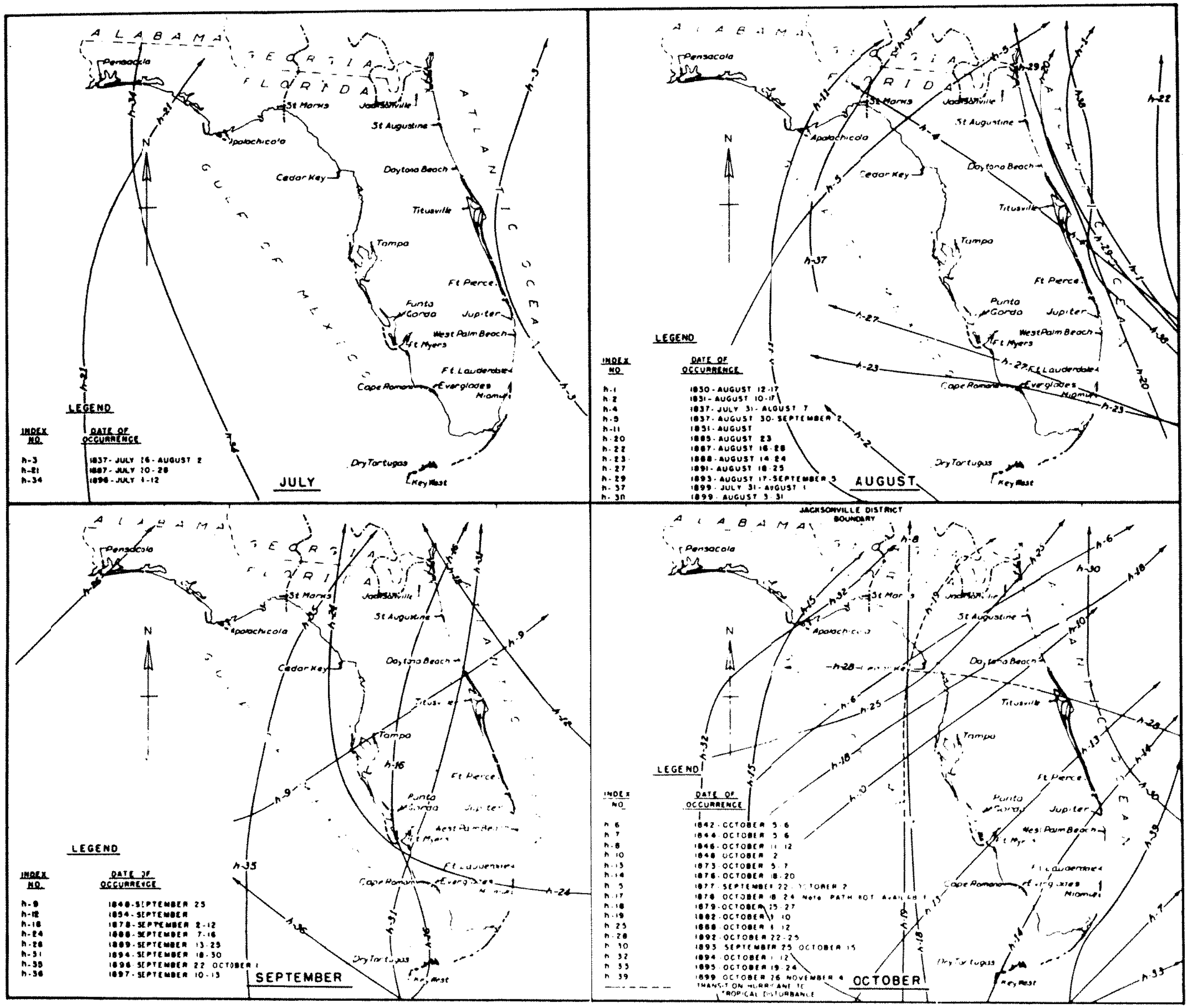
	INDEX NO	DATE OF OCCURRENCE
JUNE	H-25	1945 - JUNE 19-27
JULY	H-11	1926 - JULY 22 - AUGUST 2
	H-17	1933 - JULY 25 - AUGUST 4
	H-22	1936 - JULY 27 - AUGUST 1
	H-39	1916 - JULY 5
AUGUST	H-2	1901 - AUGUST 10-17
	H-7	1911 - AUGUST 9-14
	H-16	1932 - AUGUST 24 - SEPTEMBER 4
	H-32	1949 - AUGUST 24-29
SEPTEMBER	H-1	1900 - SEPTEMBER 5-7
	H-8	1919 - SEPTEMBER 2-14
	H-9	1920 - SEPTEMBER 27 - OCTOBER 1
	H-12	1926 - SEPTEMBER 6-22
	H-14	1928 - SEPTEMBER 6-20
	H-15	1929 - SEPTEMBER 22 - OCTOBER 4
	H-18	1933 - AUGUST 5 - SEPTEMBER 7
	H-19	1935 - AUGUST 31 - SEPTEMBER 8
	H-20	1935 - SEPTEMBER 23 - OCTOBER 2
	H-26	1945 - SEPTEMBER 12-19
	H-28	1947 - SEPTEMBER 11-19
	H-30	1948 - SEPTEMBER 19-25
	H-33	1950 - SEPTEMBER 1-7
	H-35	1960 - SEPTEMBER 3-13
	H-36	1906 - SEPTEMBER 10-30
	H-37	1912 - SEPTEMBER 11-13
	H-38	1915 - SEPTEMBER 3-4
	H-41	1917 - SEPTEMBER 21-29
	H-42	1953 - SEPTEMBER 23-28
	H-43	1956 - SEPTEMBER 21-30
OCTOBER	H-3	1904 - OCTOBER 10-23
	H-4	1906 - OCTOBER 11-20
	H-5	1909 - OCTOBER 6-15
	H-6	1910 - OCTOBER 11-18
	H-10	1921 - OCTOBER 21-31
	H-13	1924 - OCTOBER 14-29
	H-23	1941 - OCTOBER 4-12
	H-24	1944 - OCTOBER 13-21
	H-27	1946 - OCTOBER 7-9
	H-29	1947 - OCTOBER 9-15
	H-31	1948 - OCTOBER 4-8
	H-34	1950 - OCTOBER 15-19
	H-40	1916 - OCTOBER 12-21
NOVEMBER	H-21	1935 - OCTOBER 30 - NOVEMBER 8

SCALE IN MILES
0 25 50 75 100

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

PATHS AND MONTHLY DISTRIBUTION OF POST-1900 HURRICANES

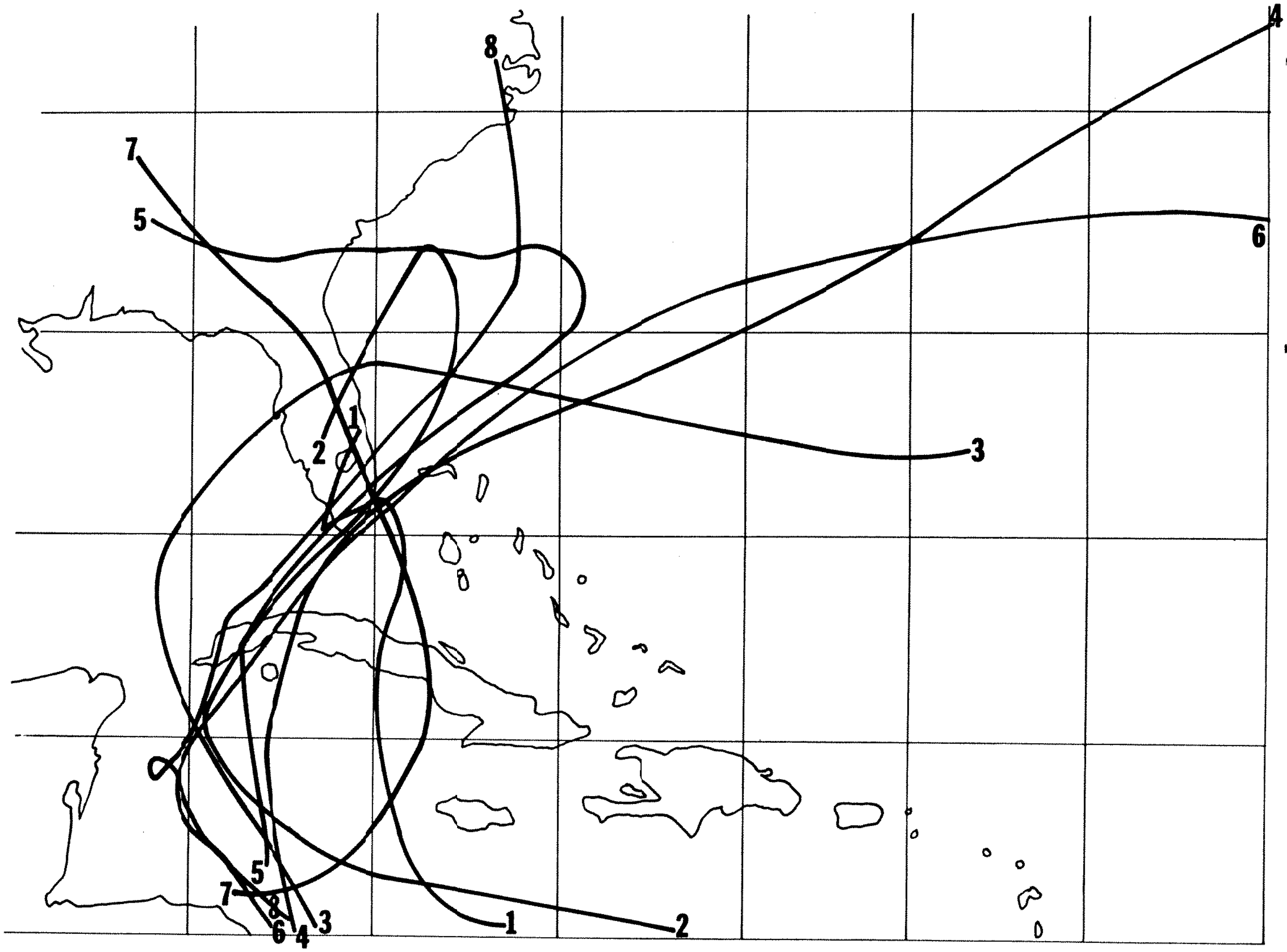
FIGURE 2.4-27



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

PATHS AND MONTHLY DISTRIBUTION OF PRE-1900 HURRICANES

FIGURE 2.4-28



MAP INDEX NUMBER	STORM NUMBER	YEAR	DATE	HOUR (EST)	MIN DISTANCE FROM SITE	HURRICANE OR TROP STORM	NAME
1 (1)	3	1904	OCT 20	0700	LT 25 W	H	
2 (2)	8	1906	OCT 21	2200	61 WNW	H	
3	4	1910	OCT 18	0900	78 W	TS	
4 (3)	6	1921	OCT 26	0200	97 NNW	H	
5	7	1924	OCT 21	0800	77 SSE	TS	
6 (4)	10	1926	OCT 21	0400	94 SE	H	
7	5	1927	OCT 2	1100	93 ENE	TS	
8	6	1938	OCT 20	0600	96 ENE	TS	
9 (5)	8	1947	OCT 12	0900	63 SE	H	
10 (6)	8	1948	OCT 5	2200	94 SE	H	
11 (7)	11	1950	OCT 18	0800	37 WSW	H	KING
12	8	1951	OCT 2	1200	LT 25 WSW	TS	HOW
13	10	1953	OCT 5	0200	88 ESE	TS	
14	12	1953	OCT 9	1500	26 NNW	TS	HAZEL
15	11	1959	OCT 18	1300	33 S	TS	JUDITH
16 (8)	11	1964	OCT 14	2200	27 ESE	H	ISBELL
17	10	1969	OCT 3	0800	41 WNW	TS	JENNY

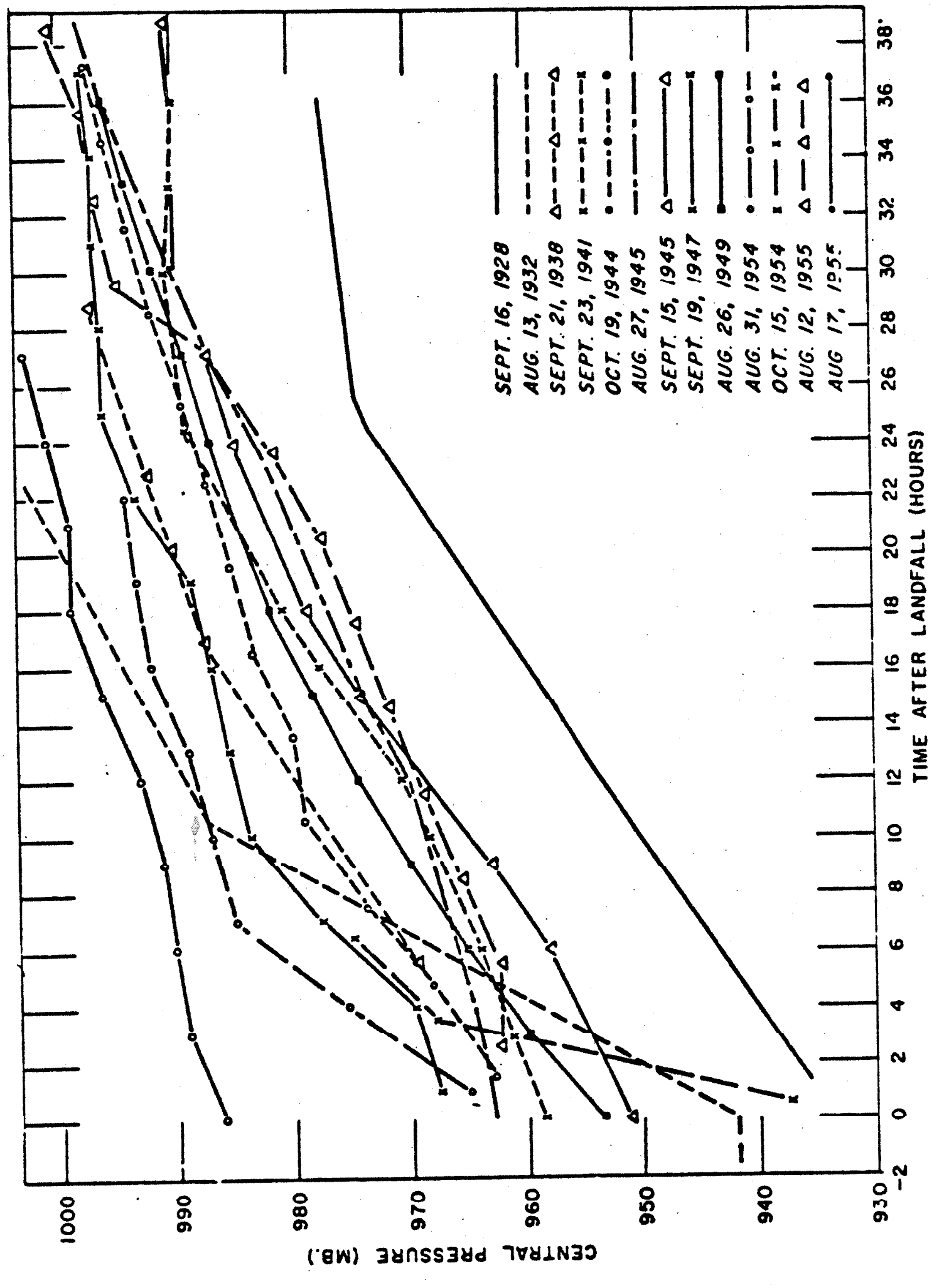
NOTES:

1. COLUMN 1.... NUMBERS IN PARENTHESES REFER TO INDEX NUMBERS AS PLOTTED ON MAP
2. COLUMN 5.... TIME WHEN STORM WAS CLOSEST TO FT. PIERCE.
3. COLUMN 7.... REFERS TO MAXIMUM INTENSITY WHEN STORM WAS WITHIN THE 100 MILE CIRCLE.

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

OCT. HURRICANES PASSING WITHIN 100 N.M. OF FORT PIERCE, FLA. – 1899-1974
Source: National Hurricane Center

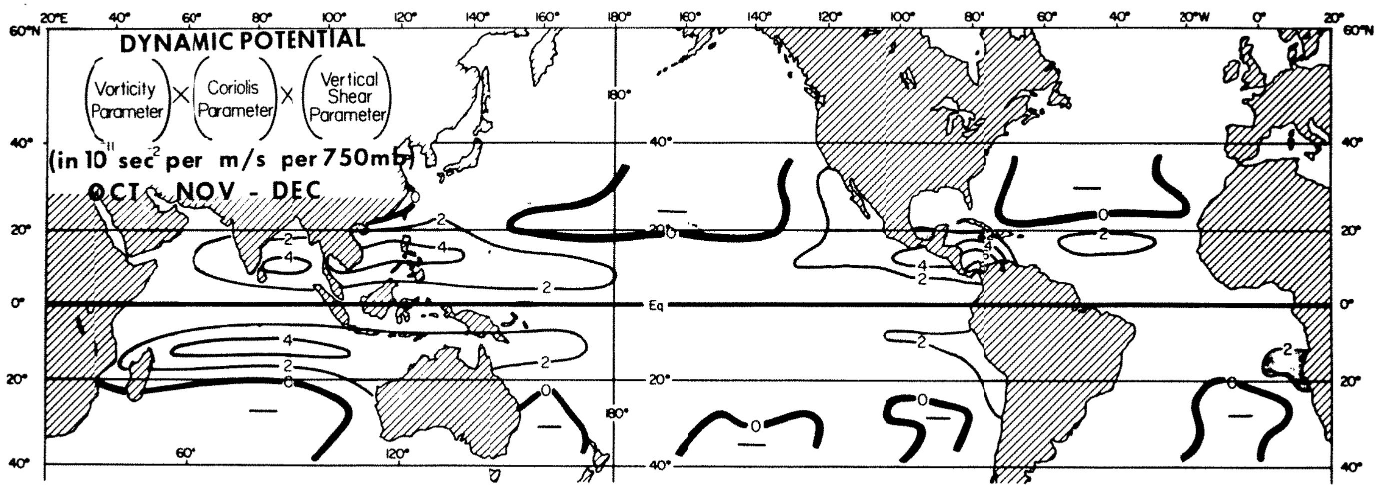
FIGURE 2.4-29



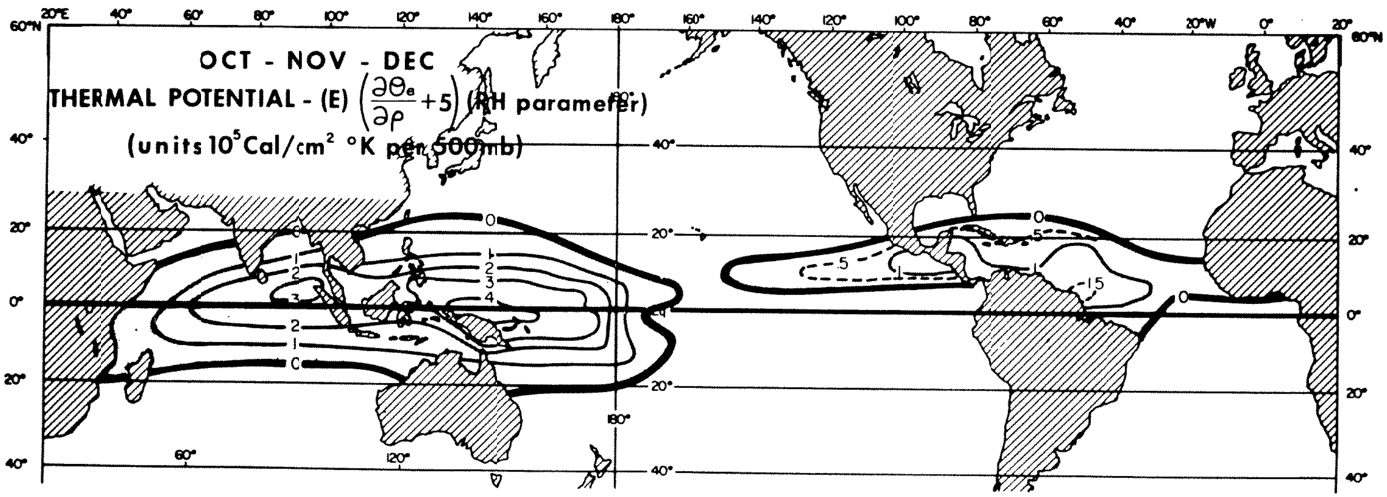
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

CENTRAL PRESSURE OF
13 HURRICANES AS A FUNCTION
OF TIME AFTER LANDFALL

FIGURE 2.4-30

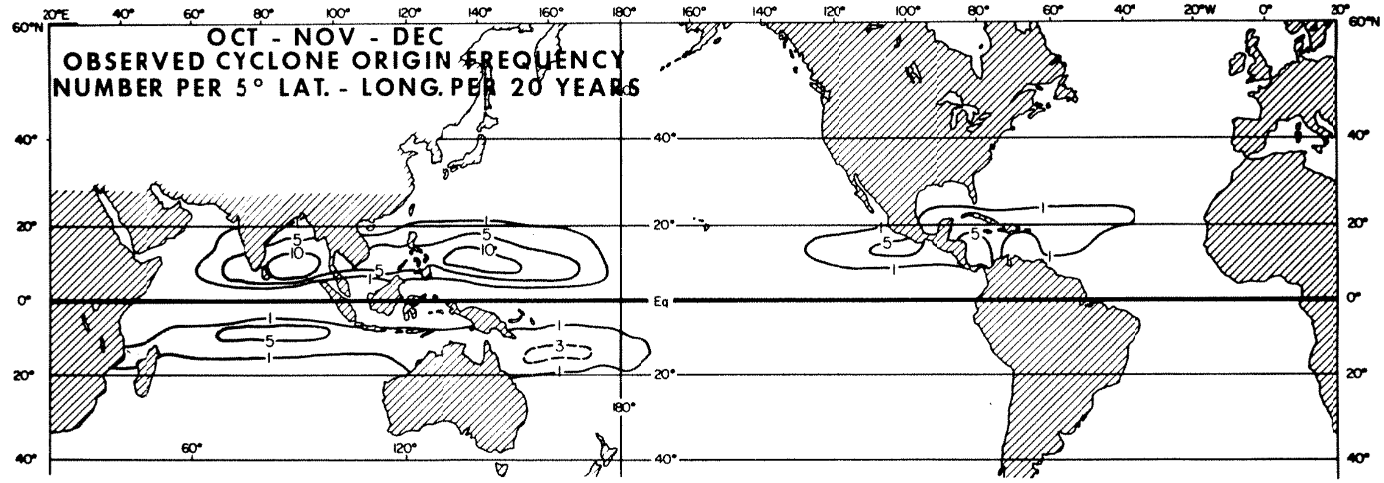


DYNAMIC POTENTIAL FOR HURRICANE GENESIS

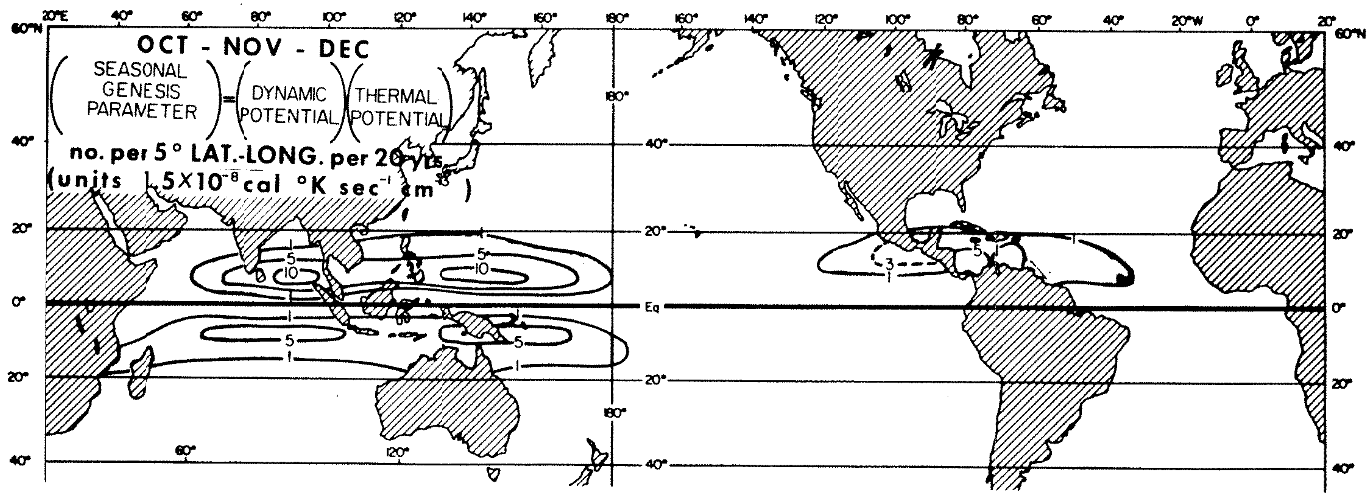


THERMAL POTENTIAL FOR HURRICANE GENESIS

FLORIDA POWER & LIGHT COMPANY ST. LUCIE PLANT UNIT 1
HURRICANE GENESIS POTENTIAL SHEET 1 FIGURE 2.4-31

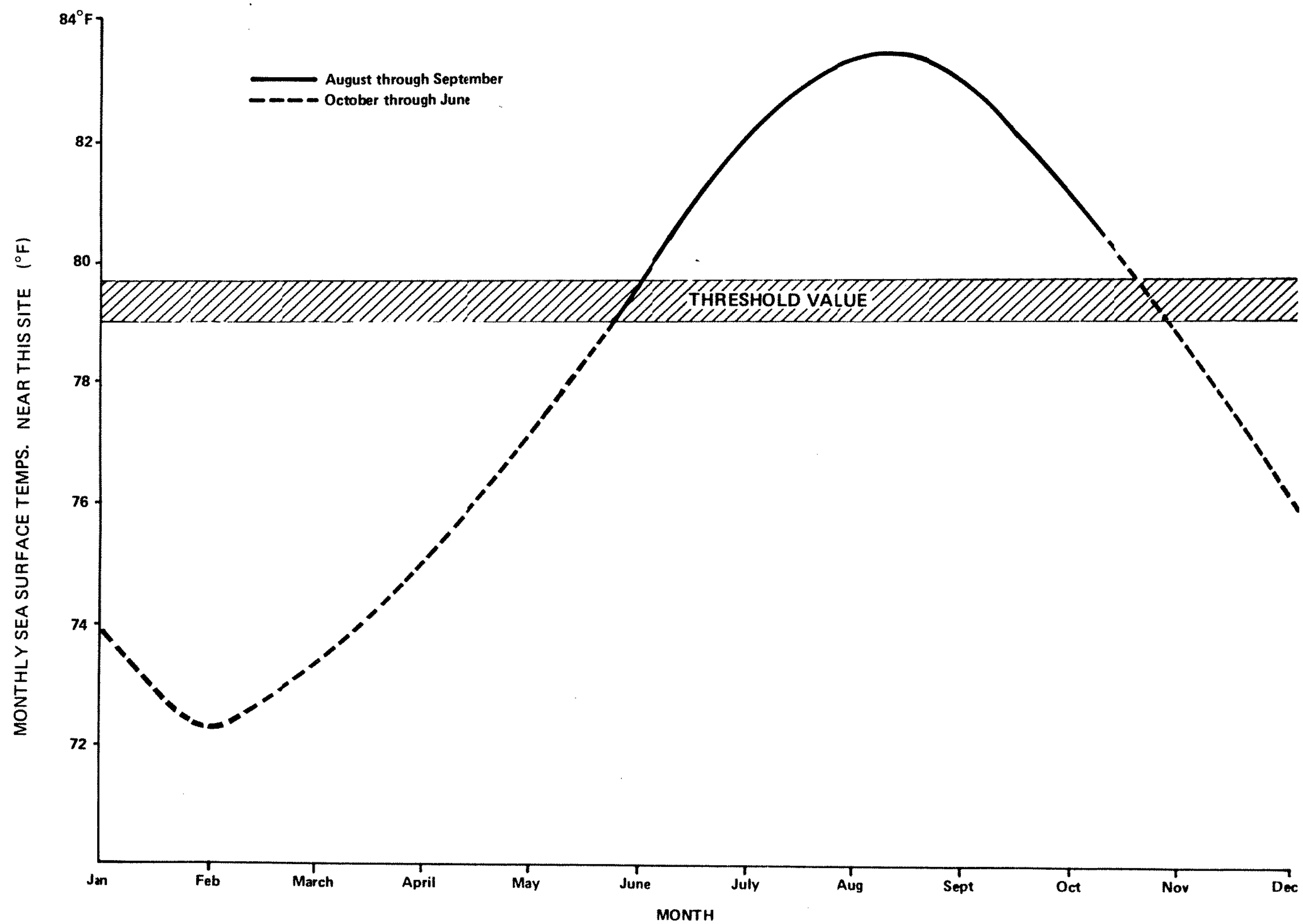


OBSERVED CYCLONE ORIGIN FREQUENCY FOR OCT.-DEC.



SEASONAL HURRICANE GENESIS PARAMETER FOR OCT.-DEC.

FLORIDA POWER & LIGHT COMPANY ST. LUCIE PLANT UNIT 1
HURRICANE GENESIS POTENTIAL SHEET 2
FIGURE 2.4-32



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

ANNUAL SEA SURFACE TEMPERATURES

FIGURE 2.4-33

2.5 GEOLOGY AND SEISMOLOGY

2.5.1 BASIC GEOLOGIC AND SEISMIC INFORMATION

The St. Lucie site is located on Hutchinson Island on the east coast of peninsular Florida, approximately 10 miles southeast of Fort Pierce, as shown on Figure 2.5-1.

Subsurface investigations were performed at the site during 1967 and 1968 in conjunction with the PSAR studies for St. Lucie Unit 1. The activities performed at the time included: literature studies, surface mapping, aerial photographic studies, geophysical studies, and 78 borings. In 1973 the Unit 1 FSAR and Unit 2 PSAR were submitted. Following AEC review in 1973, questions were posed concerning geology and seismology. Further investigations were performed during 1973 and 1974, including: air photo studies, additional literature review, 7 borings, and 50 miles of continuous seismic reflection survey (See Appendix 2E). This current report on the geology and seismology of the St. Lucie site incorporates all data obtained in the investigations and is presented as specified in the, "Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants," as revised February, 1974.

The original surficial deposits at the plant site were excavated to elevation -60 feet and backfilled with Category I fill. The Category I structures will be supported on mat foundations within this fill.

The fill is underlain by the Anastasia Formation, a sequence of partially cemented sands and sandy limestones, which extend to an average elevation of about -145 feet. The Anastasia is underlain to an elevation of about -600 feet to -700 feet by the partially cemented and indurated sands, clays and sandy limestones of the Hawthorne Formation. Underlying these surface strata are about 13,000 feet of Jurassic through Tertiary Formations, primarily carbonate rocks. These formations have a relatively gentle slope to the southeast.

2.5.1.1 Regional Geology

The following discussion of regional geology is based on a comprehensive review of available data, including published and unpublished reports and maps, and interviews with recognized authorities. The study region includes peninsular Florida with major emphasis placed on the subregional area, as shown on Figure 2.5-1.

2.5.1.1.1 Physiography

The Floridan Plateau is a partially submerged peninsula of the North American Continental Shelf. The peninsula of the State of Florida is the exposed portion above sea level of the Floridan Plateau, and lies entirely within the Coastal Plain physiographic province.

The major land forms are recognized from studies of aerial photographs and are evident on topographic maps. They are generally aligned in a northerly direction and may be grouped into three classifications: 1) highland

ridges, 2) interior plains and valleys, and 3) coastal lowland areas. The extent of these general land forms are shown on Figure 2.5-2.

The highland ridges are erosional remnants of Pliocene age deltaic and terrestrial stream deposits. These remnants are the most prominent topographic features in Florida. They occur as north-south aligned ridges located in central and western peninsular Florida with surface elevations generally ranging from 150 to 200 feet. These ridges may represent erosional remnants of a former broad plain (White 1970). Most of these ridges show evidence of advanced solution activity in the limestones below.

The remaining portion of peninsular Florida is mainly covered by Pleistocene age terraces or contemporaneous deposits. The portion of Florida covered by Pleistocene deposits can be subdivided into the interior plains and valleys and the coastal lowlands. The surface elevation of the interior plains and valleys ranges from about 40 to 150 feet, with the highest areas occurring adjacent to the erosional remnants of the highland ridges. This area is characterized by broad terraced plains and some solutioned interior valleys.

The coastal lowlands are characterized by relatively flat relief and swampy or marsh terrain. The lowlands along the east coast are low marine terraces at an average elevation of about 25 feet. The lowlands in south Florida are broad swamps and marshes. The lowlands along the west coast (generally north of Tampa) are characterized by relatively thin sand deposits overlying Tertiary limestones. This region displays numerous shallow depressions as a result of the solutioning of the relatively shallow underlying limestones. Along the east coast, the surface sands are thicker and shallow surface depressions are not as evident.

2.5.1.1.2 Geologic History

Paleozoic rocks underlie all of Florida, forming the basement complex on which younger formations were deposited. However, due to the lack of exposure of these older strata, hypotheses regarding Paleozoic Florida geological history have only recently been developed. Geologic and geophysical investigations for petroleum resources have provided the primary subsurface information upon which these hypotheses are based. A limited number of deep wells have extended into the Paleozoic basement rocks.

Based on deep well data, Applin (1951) defined the contact between two major rock types in the basement complex (Figure 2.5-3). The approximate zone of contact forms a line which extends northeasterly across Florida from about Tampa Bay on the west coast to about halfway between Jacksonville and St. Augustine on the east coast. Northwest of this contact zone are unmetamorphosed Paleozoic sediments; to the southeast are Paleozoic granites and volcanics.

Aeromagnetic surveys conducted by Gough (1967) in the eastern Gulf of Mexico show magnetic anomaly trends in Central Florida which tend to substantiate the Applin Boundary. This contact zone between Paleozoic sediments and Paleozoic igneous rocks may represent the southern limit of a structural basin, or a fault boundary (Vernon, 1951; Applin, 1965;

Rodgers, 1970). There is no evidence of displacements of the overlying formations.

The surface configuration of the basement is undulatory, featuring relative highs (archs) and lows (basins). The depth to the basement complex varies from about 2,600 feet in north central Florida to greater than 15,000 feet in south Florida (Vernon, 1970). In the St. Lucie County area it is at a depth of about 13,000 feet (Milton, 1972). The pre-Mesozoic structures which produced this undulatory surface are discussed in Section 2.5.1.1.3.1.

Late Paleozoic and early Mesozoic strata have not been identified in Florida, due to non-deposition or erosion. Overlying the basement complex are formations ranging in age from Jurassic through Recent. Applin and Applin (Vernon, 1964) indicate that: "....submergence of the Floridan Peninsula apparently began early in the Mesozoic Era...Jurassic(?) and Cretaceous sediments were deposited in a transgressive sea which encroached northward accompanied by progressive subsidence of the Coastal Plain floor." This subsidence is generally responsible for both the dip and general thickening of Mesozoic and Cenozoic formations away from the relatively stable Peninsular Arch. The formations are primarily shallow-water marine carbonate deposits, principally limestones and dolomites. Unconformities exist between some of the formations due to intermittent sea level fluctuations and resulting periods of erosion and non-deposition.

Within the Cenozoic Formations (primarily Eocene, Oligocene, and Miocene Formations) gentle warping (folding) has created localized areas of thickening and thinning sequences of sediments. Examples of variations of bedding thicknesses can be seen on the attached seismic reflection profiles (Figures 2.5-95 through 2.5-107). The time of warping varies from Eocene to middle Miocene, as evidenced in the relationship of the bedding in the Hawthorne Formation (Miocene) to the bedding in the underlying Suwannee and Ocala limestones (Oligocene and Eocene).

During the interglacial stages of the Pleistocene Epoch, the sea level was higher for a part of the time than at present and portions of Florida were submerged. Cooke (1945) postulated the existence of seven terraces which correlate with different levels of the sea during the Pleistocene Epoch. These can be more or less identified and correlated across north Florida, Georgia and the Carolinas.

During glacial stages of the Pleistocene Epoch, the sea level was lower than at present and some currently submerged portions of Florida stood above sea level. Evidence for this exists in the presence of a buried Pleistocene river channel discovered near the course of seismic reflection profiling in the St. Lucie site area. The buried channel is an incised river channel offshore of the town of Wabasso, just north of Vero Beach, Florida approximately 25 miles north of the site. The channels runs generally southeast and has eroded through most of the Hawthorne Formation. Offshore, the depth of this channel reaches approximately 500 feet beneath present sea level. As the Pleistocene channel approaches the present coastline, it becomes shallower. Reflection profiles crossing the channel in the Indian River show it to be approximately 200 feet below present sea level. This feature represents a major drainage feature during Pleistocene time which has since become filled with late Pleistocene and

Recent sediments. No trace of this river channel can be seen at the surface.

2.5.1.1.3 Regional Structural Setting of Peninsular Florida

Hypotheses with regard to the structure of the basement complex which underlies the Coastal Plain deposits in peninsular Florida are based on limited data - samples obtained from petroleum exploration wells and geophysical investigations; primarily aerial magnetic and gravimetric surveys. These data indicate that the basement complex is an undulatory surface, however data is insufficient to determine if elevation differentials are the result of warping of the rocks which comprise the basement or of differential erosion. Regardless of their mode of formation, the Peninsular Arch and the basing around it appear to be structures formed or initiated in pre-Mesozoic time. There is no structural activity hypothesized to have occurred in peninsular Florida during the Mesozoic and Cenozoic Eras. Mesozoic and Cenozoic sedimentary strata were deposited in Florida, with their distribution and lithologies controlled to a major degree by the pre-Mesozoic basement topography.

Vernon (1951) has hypothesized that the Ocala Uplift and associated local minor structures and faulting were produced in late Oligocene or early Miocene times by compressive forces acting from the west against the Peninsular Arch. These features are shown on Figure 2.5-3. Others have hypothesized that these features, if they exist, are the result of differential erosion, deposition and consolidation, principally during the Eocene-Miocene period.

Vernon (1951) also recognized numerous joints or fractures in Florida. Studies made by others, including Bermes (1958) have not shown a relationship between Vernon's joint patterns and faulting.

The structural features discussed above are presented in more detail in the following sections.

2.5.1.1.3.1 Major Tectonic Structure

A tectonic structure is considered to be a deformation of major scale in the earth's crust produced by major force systems in the mantle. It is a prominent regional feature. The major tectonic structures in peninsular Florida are the Peninsular Arch, the Southeast Georgia Embayment, and the South Florida Basin (Murray, 1961; Cohee, 1962). These features are shown in Figure 2.5-3. These major tectonic structures are attributed to warping or displacement of the basement surface prior to the Mesozoic Era.

The Peninsular Arch is a northwest - southeast trending subsurface high located in northern Florida and southern Georgia. It forms the axis of peninsular Florida and has influenced the subsequent deposition of Mesozoic and Cenozoic sediments.

The presence of the Arch was originally postulated by Applin and Applin (1944) based on a study of wells drilled through Coastal Plain deposits into the basement complex. Most deep wells have terminated upon encountering the basement complex, therefore, information regarding the structure

of the Arch is limited. The core of the Arch is composed of early Paleozoic rocks. Applin (1951) describes the arch as an anticlinal fold. Murray (1961) shows the Paleozoic rocks to be relatively undisturbed (flat lying) and unmetamorphosed, inferring that the Arch is an erosional feature.

Regardless of the origin of the Peninsular Arch, evidence indicates that:.

a) The Arch is a pre-Mesozoic structure comprised of Paleozoic rocks

b) Late Paleozoic and early Mesozoic rocks are not present due either to non-deposition or erosion

c) The Arch was a topographic high which was not covered over until the late Cretaceous, and

d) The presence of the Arch continued to influence deposition the Cenozoic period

The Southeast Georgia Embayment is located in coastal Florida, Georgia, and South Carolina, northeast of the Peninsular Arch. The Embayment is described as a shallow asymmetrical syncline, steeper on its southwestern flank and opening to the east (Murray, 1961). Along the Georgia coastline, it contains about 6,000 feet of Mesozoic and Cenozoic deposits. The thickness of the sedimentary sequence increases off-shore. Paleozoic, and possibly some Precambrian, sediments and crystalline rocks underlie the Coastal Plain deposits, forming the basement of the feature (Herrick and Vorhis, 1963).

The South Florida Basin is located southwest of the Peninsular Arch. Limited well data in south Florida indicates the axis of this structure is located off-shore in the Gulf of Mexico with a northwest trend, plunging northwest (Murray, 1961). The full sequence of Mesozoic and Cenozoic deposits have not been fully penetrated by a maximum 15,000 foot deep in south Florida. Paleozoic sediments and crystalline rocks apparently form the basin basement.

2.5.1.1.3.2 Minor Structure

The minor structures in peninsular Florida are of Tertiary age. They are the Ocala Uplift and associated Eocene-Miocene minor structures and local faults as shown on Figure 2.5-3. The closure or displacement of these structures is generally on the order of about 100 feet, with a maximum of about 400 feet, versus several thousands of feet for the tectonic structures previously discussed.

The Ocala Uplift is generally discussed as a northwest-southeast trending gently folded structure centered in northeast peninsular Florida. The highest point along its axis is located in Citrus and Levy Counties. In north central Florida along its axis, the uplift is about 230 miles long and 70 miles wide.

Two different hypotheses on the origin of late Tertiary structure in Florida are postulated by Vernon (1951) and Brooks (1974). Vernon indi-

cafes that wells drilled through the central axis of the Ocala Uplift penetrate the flanks of the Peninsular Arch west of the axis of the arch. He attributes the formation of the Ocala Uplift to late Tertiary forces, acting from the west against the Peninsular Arch. Brooks indicates that the Ocala Uplift is an erosional feature.

Vernon (1951) also hypothesized the existence of minor structures and local normal faults of late Eocene-early Miocene age in Florida. He attributes their development to the same forces which produced the Ocala Uplift. Most of the faults are oriented parallel to the crest of the Ocala Uplift or are postulated to form the boundaries of the other minor structures (Kissimmee Faulted Flexure, Sanford High, and Osceola Low). Vertical offsets on these faults are reported to range from less than 50 feet to up to about 400 feet, based on well cutting studies.

Other investigators have postulated the existence of additional late Eocene-early Miocene normal faults similar in origin to those described by Vernon. Among these are four faults in Indian River and Martin counties. Seismic reflection profiles made during the St. Lucie investigation indicates structure consists of warping rather than faulting. This is discussed in detail in Section 2.5.1.2.3.

Brooks (1974) has drawn contours on the top of the Ocala limestone which indicate that the Ocala Uplift is an erosional feature, probably produced as a result of natural erosion processes acting on a topographic high during Eocene-Miocene time. The minor structures and local faults associated with this feature, if they exist, are probably the result of differential consolidation created by gravitational forces acting on varying thicknesses of the sedimentary sequence.

Of these two different hypotheses, that of Brooks appears the more reasonable. There is no direct evidence of the forces that would be necessary, to produce the flexure postulated by Vernon. Further, because of the Ocala - Miocene interval, non-conforming erosion would be logical.

2.5.1.1.3.3 Joints

Vernon (1951) recognized a system of fractures or joints which he indicates can be traced from county to county throughout the state. He uses the terms "fractures" and "joints" interchangeably. The joints were mapped based on physiographic features (predominantly lineaments of stream and sink holes) recognized on aerial photographic mosaics. Most joints appear aligned in a northwest-southeast or northeast-southwest pattern. A secondary north-south, east-west pattern is also evident.

Vernon attributes the origin of the joints to structural movements during the late Tertiary. Specifically, he believes the joints are tension cracks developed parallel and perpendicular to the compressive axis of the Ocala Uplift. Straley (1968) has reported evidence for similar joints or fractures in the southern portion of the state. He associates these joints with the predominant structural trends in Florida.

An alternative interpretation for the development of the joint patterns is differential consolidation of relatively unconsolidated sediments as they were deposited in the structural lows or draped over structural highs of the stable basement complex.

Of the hundreds of "joints" mapped by Vernon in Florida, approximately 10 occur at locations coinciding with hypothesized faults. All of these occur in central and northern Florida, over 100 miles from the site. A geologic study was made by Bermes (1958) in Indian River County subsequent to Vernon's 1951 report. Bermes study did not show any relationship between Vernon's joint patterns and faulting.

2.5.1.1.4 Stratigraphic and Lithologic Setting of Peninsular Florida

Paleozoic igneous and sedimentary rocks form the basement complex in Florida. The igneous rocks range from granites to basalt flows and pyroclastics. The sediments are unmetamorphosed or weakly metamorphosed, relatively flat-lying, noncalcareous shales and sandstones (Applin and Applin in Vernon, 1970). The sediments range in age from early Ordovician to middle (?) Devonian.

The Paleozoic strata are overlain in peninsular Florida by from 2600 to over 15,000 feet of Mesozoic and Cenozoic sediments. The thinnest sequence of post-Paleozoic sediments is located in north central Florida, the thickest in south Florida. These strata form a seaward thickening wedge of southeastward gently dipping formations which extend off of the Piedmont physiographic province to form the Atlantic and Gulf Coastal Plains.

The Mesozoic strata in northern peninsular Florida are a mixed facies of clastics and silty and sandy carbonate rocks. To the south, they are predominantly carbonates (limestone and dolomites) with some evaporites.

Cenozoic rocks in Florida up through the Oligocene period, are basically, shallow-water marine carbonates and occur throughout most of peninsular Florida. Chen (1965) refers to this environment of deposition as the Florida Platform. Paleocene and lower and middle Eocene sediments are generally thick strata of dolomites with interbedded limestones. Late Eocene and Oligocene formations are mostly fossiliferous limestones.

The overlying Miocene strata contains some basal limestones, however, in most of Florida the Miocene is composed primarily of partially cemented and indurated sands and clays. The cementing agent is predominantly calcite. Pliocene deposits are generally partially cemented sands, shells and limestones.

A large portion of peninsular Florida is covered by a series of Pleistocene marine terraces comprised primarily of sands and shell fragments. The terraces were most likely deposited during interglacial stages when the sea level was higher than at present. Coastal currents transported the basically quartzitic sands which comprise the terraces down into Florida from the north along the Atlantic coast. The terraces are less distinct on the west coast of Florida, possibly due to the lack of source sand materials which were transported into Florida by Atlantic coastal currents. In south Florida, sandy limestones and calcareous sands were deposited

rather than terraces, indicating a lower land surface (deeper water environment) in this area. This indicates a continued influence of subsidence in south Florida through the Pleistocene epoch.

Figure 2.5-4 is a regional surface geology map showing outcroppings of Eocene to Recent deposits in Peninsular Florida. Figure 2.5-5 shows the top of the Avon Park Formation of Eocene age. A geologic section (Figure 2.5-6) shows the relatively flat lying Coastal Plain formations between Tampa and the St. Lucie site, and the influence of the basement Peninsular Arch on these sediments.

2.5.1.2 Subregional Geology

A specific geologic investigation has been made of the vicinity of the site, primarily Indian River, St. Lucie, and Martin Counties. This investigation included air photo studies, evaluation of existing well data, and geologic boring and continuous seismic reflection profiles. Particular emphasis was placed on defining stratigraphy in order to evaluate a previously hypothesized structure in the area and the validity of the extension of other structural features to points near the site.

2.5.1.2.1 Physiography

An investigation of land forms (shown on Figure 2.5-1) has been made using satellite photography and U.S.G.S. Topographic Quadrangle Maps. Figure 2.5-7 is a print made from a satellite photograph made in 1973. An overlay of Figure 2.5-7 (Figure 2.5-7A) shows land form subdivisions with delineations based primarily on topographic and drainage characteristics.

The site is located within the coastal lowlands. The land forms along this part of the east coast of Florida generally parallel the northwesterly orientation of the present coastline. This alignment of land forms reflects depositional processes similar to those which exist at the present. Recognizable land forms within the Coastal Lowlands are: 1) the present coastline depositional environment, 2) the Atlantic Coastal Ridge, and 3) a broad very shallow valley or swale which contains the headwaters of the St. John's River and the St. Lucie River. West of the Coastal Lowlands is a higher terrace of the interior plain and valleys which extends westward to the highland ridges.

The eastern-most land form within the coastal lowlands depositional environment includes the barrier islands and the lagoonal areas of the Indian River. The barrier islands, including Hutchinson Island, were probably formed as off-shore bars during a period of higher sea level. These islands vary in width from a few hundred feet to about one mile. Surface elevations on the barrier island are between sea level and about +15 feet, with the higher elevations along the present coastline. The western portions of the islands are primarily mangrove swamps.

The Indian River averages about two miles in width and is shallow (about 3 to 6 feet deep) except for a ten-foot deep dredged channel for the Inland Waterway.

The western bank of the Indian River is the Atlantic Coastal Ridge with surface elevations ranging up to a maximum of about +40 feet. The ridge is an almost continuous land form extending from the Sebastian River in northern Indian River County south to the St. Lucie River at Stuart, in northern Martin County. The average width of the ridge is about 1/4 to 1/2 mile.

West of the Atlantic Coastal Ridge is a broad essentially flat valley, which is approximately 25 miles wide. Surface elevations within the valley from southern Brevard County through Martin County generally range between +20 and +30. Originally the valley was probably a lagoon, similar to the environment of the present Indian River. It was later inundated by the sea and presently reflects Pleistocene deposition of the Pamlico Terrace. Studies of satellite photography and topographic maps indicate some remnants, through subdued, of swale and swell topography. These features suggest progradational beach ridges. However, relief is minor and most topographic expression has been lost, possibly due to leaching over a long period of time of the calcareous shell content of the sands (White, 1970).

There is a remnant of the Talbot Terrace in Indian River County, Figure 2.5-7A. Surface elevations on this terrace are about +40 feet. The terrace shows several northerly trending beach ridges.

West of the Talbot Terrace and the valley within the Coastal Lowlands is a higher terrace (Penholoway) at a level with surface elevations of about +60 to +70. The Penholoway terrace is the eastern portion of the major land form classification, interior plains and valleys. It is approximately 40 miles wide and the topography tends to rise to the north. The western boundary of this terrace is the Lake Wales Ridge, a north-south trending ridge of the Central Highlands, located about 80 to 90 miles inland from the present Atlantic coastline.

The eastern and southern portions of the Penholoway Terrace exhibit some distinctive beach ridges and inter-ridge swales. Where these features are present, they control drainage patterns. Inland of the beach ridges, the terrace exhibits little topographic relief and drainage is more random, except for the northwest-southeast trending Kissimmee River. The series of beach ridges along the eastern portions of the terrace probably mark an old coastline which may have created a barrier separating an interior lagoon from the ocean environment. White (1970), suggests that the Kissimmee River may have developed as a drainage feature of this interior lagoon as the sea level dropped. This mode of land form development is similar to that suggested in the valley immediately inland of the Atlantic Coastal Ridge.

In essence, the geomorphic features characterizing the east coast of southcentral Florida are postulated to be a series of beach ridges and inland lagoons, and environment of deposition not dissimilar to the present relationship between the barrier islands, including Hutchinson Island, and the Indian River.

2.5.1.2.2 Lithology and Stratigraphy

About 13,000 feet of Jurassic to Recent sedimentary formations overlie the Paleozoic crystalline basement complex in the sub-region (Figure 2.5-1). This thickness of sedimentary formations is based upon data from well number W-4086. The location of this well is shown on Figure 2.5-8 along with other well data points in the site area used for this report. The upper 600 feet of sediments are soft rock formations consisting of partially cemented and indurated sands and clays. Below 600 feet, these materials are primarily moderately hard to hard limestones and dolomites with some sandstones, shales, and anhydrates (Milton, 1972).

Lower Cretaceous and possible Jurassic formations extend upward from the basement to an elevation of about -7,000 feet. The Jurassic and lower Cretaceous formations pinch out against the flanks of the Peninsular Arch and dip in a southeasterly direction. These formations consist of limestones and anhydrites with some beds of clastic materials (Vernon, 1964). The clastic materials are most likely derived from erosion of the rock composing the Peninsular Arch.

In St. Lucie County, upper Cretaceous materials extend from about -7000 to -4200 feet (-4240 feet in well no. W-4086). The upper Cretaceous (Gulf Series) in southern Florida is basically thick sequences of carbonates (often non-fossiliferous, chalky) with some sandstones and shales. Beds of upper Cretaceous age also dip to the southeast in the site vicinity and thicken away from the crest of the Peninsular Arch.

Cenozoic formations representing Paleocene through Recent age underlie the site. The Paleocene and lower and middle Eocene formations are basically hard, crystalline dolomites with interbedded fossiliferous limestones. In well number W-4086 (located approximately 7 miles west of the St. Lucie site) these formations were encountered at the following elevations: Cedar Keys limestone (Paleocene), -3200 to -4200 feet; Oldsmar limestone (Paleocene), -2240 to -3200 feet; Lake City limestone (Eocene) -1340 to 2240 feet; and Avon Park limestone (Eocene) -800 to -1340 feet.

The upper Eocene in St. Lucie County is represented by the Ocala Group limestone. This formation is basically soft to moderately hard foraminiferal limestone. It is about 150 feet thick in well number W-4086 (elevations : -800 to -650 feet) and thickens to about 200 feet along the coastline. The large foraminifera, Lepidocyclina, is a diagnostic indicator of this formation.

Overlying the Eocene sequence, is the Suwannee Limestone of Oligocene age. The Suwannee varies from a hard, very fossiliferous, "coquinoid", limestone to a soft, granular, dolomitic, essentially non-fossiliferous limestone containing broken fragments of pectons, echinoids and barnacles. The lower Suwannee sequence appears to grade into the Ocala sequence.

The Suwannee averages about 135 feet thick in borings drilled by Law Engineering during its investigation of the site area. AG-104, AG-105, and AG-106 are located in St. Lucie County immediately west of the site as shown on Figure 2.5-9. Studies in Indian River and Martin Counties

indicate a thickening of the Suwannee to the east accompanied by an increase in dip. In northern and western Indian River County, the Suwannee is absent. The Suwannee Formation is considered to be the uppermost formation of the Floridan Aquifer in this region. The overlying Hawthorne Formation is a major confining unit (aquiclude) for the Floridan Aquifer.

The Hawthorne Formation of Miocene age unconformably overlies the Suwannee Limestone. This formation averages about 430 feet thick in borings AG-104, AG-105, and AG-106 located on the mainland west of the site. The lithology of the Hawthorne is basically green, clayey, very fine sands and very fine sandy clays. Phosphatic sand and clays and thin layers of sandy, phosphatic limestone are characteristic of the lower part of the formation. The formation is generally dense and indurated.

The contact between the Hawthorne Formation and the overlying Anastasia Formation is unconformable. Prior to deposition of the Anastasia Formation the Hawthorne was subjected to a period of erosion which left its surface essentially flat in the vicinity of the site, as shown in the geologic sections on Figures 2.5-9, 2.5-10, and 2.5-11.

The Anastasia Formation (contemporaneous) in the site area refers to all the material which overlies the Hawthorne Formation except for Pleistocene Terraces. The formation is of Pleistocene age. It is about 140' thick at the coast and thins to the northwest against older more steeply dippings formations. To the southwest it either pinches out or merges with the Ft. Thompson Formation. The Anastasia differs in composition from place to place, ranging from practically pure quartz to almost pure quartz sand.

There are generally thin discontinuous pockets and seams of sandy limestones or sandstones within the sequence, An example of these scattered discontinuous layers is exposed along the coast at St. Lucie Inlet 15 miles south of the site. The Anastasia is the main source of water supply for several towns and cities in this coastal area, including Fort Pierce and Stuart.

The Anastasia Formation is overlain in some areas by Pleistocene marine terraces. These terraces are composed of quartzitic sands and shell fragments, The Atlantic Coastal Ridge, which forms the western bank of the Indian River is a remnant of such a terrace.

2.5.1.2.3 Structure

No specific reports on the geology and groundwater resources of St. Lucie County have been published; however, geologic studies have been reported for Martin County, to the south, and Indian River and Brevard Counties to the north, Figure 2.5-12 is a map showing all hypothesized structures in the sub-region.

Bermes (1958) utilized data from wells drilled into Eocene strata to develop east-west and north-south geologic sections in Indian River County. These sections indicated the Eocene and Miocene strata sloped gently to the southeast in most of the county. Bermes reported an apparent change in dip from less than 5 feet per mile to greater than 70 feet per mile and the occurrence of Oligocene age beds near the eastern margin of the county. He postulates a somewhat complex system of three high-angle, normal faults essentially parallel to the coastline to explain the steepening dip and the occurrence and apparent thickening in Oligocene strata to the east. Strata on the east side of the faults were projected to be downthrown. The faults were inferred from a difference in elevation of about 225 feet of tho top of the Ocala Group for a horizontal difference in elevation of about 2.5 miles between the control points. The age of the faulting was not discussed by Bermes, but the fault traces shown in his geologic sections were terminated at the bale of Miocene strata, indicating an age greater than 20 million years.

Lichtler (1960) utilized electric logs and some cuttings from deep wells in a study of Martin County. Based on this study, Lichtler postulated a high angle northwesterly trending normal fault which is parallel to and about to 5 miles inland of the present coastline. The strata on the east side of the fault were projected as being downthrown by as much as 300 to 400 feet (Lichtler, 1960). The maximum displacement between control points (an electric log of well 443 and a study of cuttings from well 841) is a displacement of 270 feet for a horizontal distance of about 2.5 miles (Lichtler, 1960). Lichtler dated the faulting as offsetting strata at least as young as the Oligocene with some possible additional slippage during the early Miocene, He attributed the taulting to the forces which formed the Ocala Uplift, thus establishing an age of over 20 million years.

Vernon (1970) used hydrologic data to extend the Lichtler fault in Martin County through St. Lucie County (partly along the St. Lucie River) to a point at about the southern boundary of Indian River County. The shortest distance from this hypothesized fault to the site is 5 miles. His control

was the elevation of the top of the artesian aquifer (Floridan Aquifer) and not any particular geologic unit.

Our study of the regional geologic structure, particularly in-the vicinity of the St. Lucie site, included verification of existing well data previously used by Bermes and Lichtler in their hypotheses of area structure drilling deep geologic borings, the performance of approximately 50 miles of marine continuous seismic reflection survey, and meetings with Bermes, Lichtler, and other geologists familiar with the St. Lucie area.

Data from a number of previously drilled water wells to the south, west, and north of the site (including key data points used by Lichtler and Bermes in their reports on Martin and Indian River counties) were incorporated into the study. Prior to utilization of this data, samples from these wells on file at the Florida Geological Survey were studied.

Nine geologic borings were drilled outside of the immediate plant area on Hutchinson Island. The borings were drilled north, south, and west of the plant site at distances of from two to six miles. The locations of previously existing borings and additional geologic borings drilled by Law Engineering are shown on Figure 2.5-8.

The primary purpose of six of these borings was to verify the elevation of the top of the Hawthorne Formation at depths consistent with those found at and adjacent to the plant site. Figures 2.5-9, 2.5-10, and 2.5-11 show the top of the Hawthorne to be relatively flat surface, sloping slightly down to the east at about 3 feet per mile, as would be expected from the regional structure.

Three deep geologic borings (700 feet +) were drilled along a line west of the site to obtain data on both sides of the Vernon "fault" and on both sides of the hypothetical straight line extension of the bermes "fault" which would pass closest to the site (about 3 miles west of the site).

The location, spacing, and depth of the three deep geologic borings were dictated by the nature of the structure postulated by Vernon, and a straight line extension of the structure hypothesized by Bermes. Since these hypothesized structures are based on scattered well data several miles apart, the published locations of the structures are accurate only to within the limits of the distances between borings upon which they are based. Thus, the three borings drilled to investigate the structures were placed on two mile centers perpendicular to the hypothesized structure to insure that the structure was adequately bracketed. Since the postulated structures were based primarily on differences to top of limestone (top of Suwannee Formation) in scattered wells, the geologic borings were extended far enough into limestone such that specific geologic formations could be identified and correlated.

Within the depths drilled, three formational contacts were identified by lithology, electric and gamma-ray logs, and fossils. These contacts are the top of the Hawthorne Formation, top of the Suwannee Limestone, and the top of the Ocala Group Limestone. These are formations which have been hypothesized in the published literature to have been offset by faulting.

Figure 2.5-9 is a geologic section drawn through the borings where they cross the hypothesized Vernon fault and the closest extension of any of the Bermes faults to the plant site. This section shows flat lying strata with insignificant elevation variations across proposed or hypothetical faults.

It is concluded that no faulting exists at these locations. Our interpretation of 2 marker horizons (top of Ocala Group and Avon Park Limestone) are shown on Figures 2.5-13 and 2.5-14.

In order to more closely define structure in the vicinity of the St. Lucie site, and to further evaluate the existence of faults hypothesized in the literature, approximately 50 miles of continuous marine seismic reflection profiles were made. The survey was accomplished using a maximum 3000 joule sparker energy source firing at 1/1 second intervals and continuously monitored and recorded in strip chart format. This reflection surveying system was capable of detecting marker horizons within the upper 1000 feet of materials. The key marker horizons were correlated with deep boring data described.

As a part of the survey, the fault system hypothesized by Bermes in Indian River County was crossed at two locations where the structure trends across the Indian River into the Atlantic. The fault hypothesis by Lichtler was crossed at a location on the north fork of the St. Lucie river just north of the Martin County - St. Lucie County Line. Additionally, reflection profiling was accomplished immediately north, south, east, and west of the site, as well as adjacent to the Unit 1 reactor.

Within the depths penetrated, several key marker beds were identified. The most significant and the strongest reflecting horizon represents top of limestone (top of Suwannee Formation). Since differences in elevation of top of limestone as determined from scattered well data are the primary basis for postulated faults, the characteristics of this key horizon were studied in detail along the profile line locations. Generalized profile line locations are shown on Figure 2.5-92. Detailed line locations, showing navigational control points, are shown on Figures 2.5-93, 2.5-94, and 2.5-95. Sections of individual profiles are shown on Figures 2.5-96 through 2.5-110.

Three prominent seismic reflectors are present throughout the survey area. These have been correlated to drill hole AG-104, and represent the top of the Suwannee limestone (top of limestone in the site vicinity), and two distinct layers in the Hawthorne Formation. In addition, the top of the Hawthorne Formation was detected in a number of areas covered by the survey, as was the top of the Ocala limestone. All layers throughout the survey area are predominantly flat lying, with regional dip to the southeast being evidenced best by the top of the Suwannee reflector, which rises from approximately 750 feet near St. Lucie Inlet to about 300 feet Sebastian Inlet in Indian River County to the north.

Superimposed on the regional dip of about 10 feet per mile, are localized areas of warping (folding) in which dips on the order of 150 feet per mile or about 3 degrees exist. This type of warping was found to exist in Ft. Pierce Inlet, St. Lucie Inlet, in Indian River county near Sebastian

Inlet, and underlying the St. Lucie site.

These four areas illustrate the effects of localized warping on depth to top of limestone over relatively short horizontal distances. Under the Fort Pierce Inlet, a downwarp occurs as shown on Figure 2.5-95. Folding of the sediments has created a syncline with a closure of 150 feet in the limestone over a horizontal distance of approximately one mile (1.7°dip). The profile of St. Lucie Inlet (Figure 2.5-101) shows a similar synclinal downwarp with closure on the order of 200 feet over a horizontal distance of approximately two miles (1.4°dip). In the area just south of Sebastian Inlet, a monoclinal downwarp occurs as shown on Figures 2.5-104, 2.5-105, and 2.5-106. In this area, the top of limestone slopes downward from elevation -200 to nearly elevation -700 feet at a point two miles to the east (2.9°dip). Figure 2.5-98 shows a monoclinal downwarp under big Mud Creek adjacent to the site with closure on the order of 150 feet over a horizontal distance of one mile (1.7°dip).

Local variations in depth to the top of the limestone (Suwannee reflector) evidently was the basis for the hypothesized faults in Martin County and Indian River County. In Indian River County, one of the faults hypothesized by Bermes, and which showed apparent offsets in limestone on the order of 225 feet, was investigated by reflection profile lines crossing between three key boring locations. This area is described above as an area of warping near Sebastian Inlet. The reflection profile shows locally strong dips of up to 250 feet per mile in top of limestone. This variation in top of limestone is in agreement with logs of the three borings used by Bermes. However, the reflection profiles show continuous beds with no evidence of faulting.

The reflection profile made across Vernon's extention of Lichtler's fault in Martin County provides only sketchy data of subsurface structure in this area. However, the data which was obtained, as shown on Figure 2.5-109, has the same pattern of reflectors as that found throughout the seismic reflection survey. The decrease in elevation in the top of limestone to the southeast is typical of the regional dip, and shows an elevation decrease in top of limestone of about 100 feet eastward over a horizontal distance of approximately three miles. This is consistent with variations in elevation of the top of limestone found in the adjacent St. Lucie Inlet. It is concluded that faulting does not exist at this location.

Following collection of the data described above, meetings were held with Bermes and Lichtler to discuss the correlation and significance of this additional data relative to previously published structure. Both Bermes and Lichtler agree that no significant offsets exist anywhere along the approximately 50 miles of reflection profiles.

In summary, a series of roughly parallel layers in a fairly constant pattern with respect to depth has been traced from St. Lucie Inlet south of the St. Lucie plant, past and adjacent to the plant site, and north to Sebastian Inlet, approximately 30 miles north of the plant site. No faults of any kind were found in the sediment sequence. Several areas of localized and possibly connected warping were found, with most of these located under or near the present barrier island. Sediments under the Indian River (and at least 6 miles inland from the immediate site vicinity as determined

by the deep geologic borings) are nearly flat, as are those of the offshore area.

Time of warping apparently took place from the Eocene to middle Miocene Epochs, as evidenced in the various relationships of the bedding in the Miocene Hawthorne formation to the limestone surface below. There does not appear to be any evidence for warping at the present time, as the seismic reflector identified as top of the Miocene (Hawthorne) appeared to be nearly horizontal over the entire survey area.

Utilizing all of the data obtained, it is concluded that faulting does not exist within the upper formations in the vicinity of the St. Lucie plant.

2.5.1.2.4 Potential for Surface Subsidence

2.5.1.2.4.1 Solution Activity in Carbonate Terraines

Solutioning of carbonate rocks and resulting karst topography is well developed in some portions of Florida. However, a study of satellite photography shows no evidence of advanced solutioning of carbonate bedrock formations and resulting lake development within 50 miles of the site.

White (1958, 1970) has discussed solutioning in Florida and its mechanisms of development in detail. The site vicinity is not recognized as a high potential area for surface subsidence due to solutioning. Three factors which appear to be limiting influences on solutioning in this area are 1) the young age of the coastal land forms relative to the geologic time required to produce mature karst, 2) the depth to carbonate bedrock and 3) the thickness and clayey nature of the Miocene sediments which overlie the limestones.

White (1970) indicates that a principal mechanism for solution development is initiated in beach-ridge covered terraces where water is concentrated in the inter-ridge swales. These land form features are evident in terraces 20 miles west of the site where beach ridges, though subdued, are abundant.

2.5.1.2.4.2 Mineral Extraction

Several local communities obtain their water supplies from the Anastasia Formation. In order to assess the potential effect of water extraction on surface subsidence, officials of the two largest municipal users nearest the site, Fort Pierce and Stuart, were contacted.

According to Mr. C. W. Temby, Director of the Fort Pierce Utility Authority, Fort Pierce pumps a maximum of 10 MGD from about 30 wells installed to a depth of 120 to 130 feet. They project a usage of 24 MGD by 1980. Additional wells will be installed to the same depth as those in current use to provide this additional capacity.

In Stuart, Mr. D. A. Gale, Water and Sewer Superintendent, stated that Stuart pumps an average of about 2.5 MGD from 22 wells, 120 feet deep.

Additional wells will be installed to increase capacity to 6 MGD. Monitoring stations in the Stuart well field record minimal drawdown with maximum pumpage,

Both Messrs Temby and Gale state that there is no record of subsidence related to well field pumping in either Stuart or Fort Pierce. Since there is no subsidence associated with these two well fields, and since the St. Lucie site is separated from communities utilizing water from the Anastasia Formation by the Indian River (an infinite source of water), there is no anticipation of subsidence at the site related to ground water withdrawal.

Other than ground water, there is no mineral potential known in the site area. Several oil test wells have been drilled in St. Lucie, Martin, and surrounding counties. Most of these wells were terminated in the basement rock complex. No oil or gas "shows" were recorded in any of these wells, and all have been capped and abandoned.

2.5.1.3 Site Geology

2.5.1.3.1 Geologic History

The site is located on the East Coast of Florida on an offshore bar named Hutchinson Island, as shown on Figure 2.5-1. The East Coast of Florida is the emergent coast of Florida. Hutchinson Island has been developed and has remained exposed above MSL since some time during the Pleistocene age, approximately one million years ago. The Pleistocene sediments are slightly thicker at the site than at the mainland due to the method by which the island was developed.

2.5.1.3.2 Physiography

The topography of Hutchinson Island is a bar and inland swale type. There is a low bar near elevation +14 on the ocean side of the island. The surface of the island then slopes downward toward the Indian River to about elevation +4, generally forming a swale. To the north and west of the site, both Big Mud Creek and the Indian River are continuations of the swale and are very shallow, 5 to 10 feet deep. There is a dredged channel in the Indian River for the inland waterway. To the east, the Atlantic Ocean bottom slopes very slightly to the east to a depth of about 120 feet at a distance of about 15 to 20 miles from Hutchinson Island.

To the west, at the mainland, there is another bar (Atlantic Coastal Ridge) with a maximum elevation of about +40 parallel to the coast.

The site was originally covered by a mangrove swamp. A small road dike, for mosquito control, was constructed around the portion of the site that is adjacent to big Mud Creek and the Indian River. Within the site area at the time of initial field work, there was about 1 foot of standing saline water.

2.5.1.3.3 Stratigraphy and Lithology

There was 4 to 6 feet of peat and roots beneath the generally water covered surface. This material is a dark brown or black residuum produced by the partial decomposition and disintegration of trees, mangrove roots, and other vegetation. This peat was probably formed during the past several thousand years.

The Anastasia Formation of Pleistocene age underlies the peat. No differentiation has been made between the various Pleistocene deposits which extend to about elevation -135 to -155 feet below sea level. This material has been termed "Anastasia Contemporaneous". There is a suggestion from geologic and engineering evidence that the discontinuous cemented pockets are the erosional remnants of the Fort Thompson Formation. If this is true, then generally below elevation -35 to -60 are the Anastasia and Caloosahatchee Formations. The Anastasia Formation is quite possibly composed of marine, brackish and fresh water deposits. Petrographic studies have shown that at some places there is well rounded and frosted sand indicating a beach or marine depositional environment. At other places within the formation there is a blackening of shells which may indicate a reducing, or brackish depositional environment. Also identified was a fresh water gastropod from one of the partially cemented, discontinuous pockets,

The Anastasia Formation consists of gray, slightly clayey and silty, fine to medium sand with fragmented shells and, in places, fragmented shell beds with slightly clayey and silty, fine sands. There are also discontinuous pockets of cemented sand with shells and sandy limestone. These discontinuous cemented pockets are generally found from about elevation -35 to -60. Also occasional discontinuous plastic clay lenses were found in the upper part of the formation.

The Hawthorne Formation of Miocene age unconformably underlies the Anastasia Formation. There is evidence of this unconformity or erosional surface by possible "case hardening" of the contact of the Hawthorne and Anastasia. This occurs between elevations -140 and -157.

The upper 100 to 150 feet of the Hawthorne Formation consists of green, slightly clayey and silty, very fine sand. The lower part becomes generally more clayey. Geologic information obtained by seismic reflection pro files in Big Mud Creek show the Hawthorne formation as extending downward to about elevation -600 to -700 in this area.

This lithology of the Hawthorne Formation changes slightly to a gray white, phosphatic, sandy clay at this site below possibly elevation -450. The clays of the Hawthorne are unique in their content of the mineral Attapulgite. The x-ray diffraction analyses also indicate a high carbonate fraction (calcite and dolomite) in the Hawthorne Formation.

2.5.1.3.4 Structure

The structure of the geologic formations at this site is simple, as determined by borings (Figure 2.5-15) and reflection profiles (Figure 2.5-93). The younger, or Pleistocene and Miocene formations are nearly flat, dipping very slightly to the southeast at about 5 to 10 feet per mile. There is an

erosional surface or unconformity between the "Anastasia Contemporaneous" and the underlying Hawthorne formation. This has resulted in a slightly undulatory contact having minor irregularities (Figures 2.5-16 and 2.5-17).

The older or deeper formations (limestone formations) dip at somewhat greater angles. Seismic reflection profiles made adjacent to the site show dips increasing with depth, At a depth of approximately 800 feet, the limestone formations are dipping 150 feet per mile toward the southeast. This is consistant with regional and area structure,

2.5.2 VIBRATORY GROUND MOTION

2.5.2.1 Geologic Conditions of the Site

The St. Lucie site is located on Hutchinson Island, about 10 miles southeast of the central portion of Fort Pierce, Florida. Hutchinson Island was probably developed as an off-shore bar during one of the later interglacial stages of the Pleistocene epoch and was subsequently exposed as the sea level dropped in relation to the adjacent land surface. The surficial Recent and Pleistocene materials are underlain to an average elevation of about -145 feet by partially cemented sand and sandy limestone of the Anastasia Formation. The Hawthorne Formation, partially cemented and indurated sands, clays, and sandy limestones, underlies the Anastasia to an elevation of about -600 feet to -700 feet. These upper strata are underlain by about 13,000 feet of Tertiary to Jurassic formations, predominantly carbonate rocks. Paleozoic crystalline rocks form the basement complex on which these Mesozoic and Cenozoic Coastal Plain sediments were deposited.

The formations in the site area dip to the east and southeast as shown on the geologic sections presented as Figures 2.5-9, 2.5-10, and 2.5-11 and the contour maps presented as Figures 2.5-13 and 2.5-14.

These figures illustrate stratification that is typical of southeast coastal Florida. Our geologic studies of the area extending westward for a distance of seven miles from the site show no offset displacement of late Eocene or younger formations. We conclude that middle Tertiary faulting which has been hypothesized in this area does not exist.

The lithologic, stratigraphic and structural conditions of the site and the region surrounding this site, including its geologic history are described in Section 2.5.1.

2.5.2.2 Underlying Tectonic Structures

A summary description of the tectonic structures within the study region are given below along with an evaluation of their significance to the site. A detailed discussion of the tectonic structures in the region surrounding the site is given in Section 2.5.1.1.3.1, Major Tectonic Structure.

The major tectonic structures in peninsular Florida are the Peninsular Arch, the Southeast Georgia Embayment, and the South Florida Basin. These three features are attributed to warping or displacement of the basement surface prior to the Mesozoic Era (Murray, 1961, Cohee, 1962). Since the Mesozoic Era, these structures have influenced the deposition of later sediments throughout the Cenozoic Period. The closest of these basement tectonic structures to the site is the Peninsular Arch. Its influence on the deposition of sediments underlying the site is shown on Figure 2.5-6.

Minor structure in Eocene and Miocene sediments has been hypothesized to occur in nearby Martin, St. Lucie, and Indian River Counties as discussed in Section 2.5.1.2.2. These minor local structures have been investigated by borings and seismic reflection surveys. As a result of this investigation, discussed in detail in Section 2.5.1.2.3, it is concluded that faulting does not exist in the site vicinity.

2.5.2.3 Behavior During Prior Earthquakes

Peninsular Florida is underlain by great thicknesses of sedimentary rock formations, some of which have been only partially indurated. A simplified geologic column showing the significant formations is given in Figure 2.5-19. (For comparison the geologic column in the vicinity of Green Cove Springs is shown on the same figure as that for St. Lucie.) Since it is necessary to consider the response of the geologic column to shock and vibration, the following response analyses and observations have been made for the St. Lucie site.

Observations of the earthquake response of sites in other parts of the world have shown that sites immediately underlain by bedrock respond differently than sites underlain by soils. These differences occur because the reasonably elastic column which is analogous to the rock under a site is different from the quasielastic column which is comparable to soil beneath the same site. Those properties of the materials which affect the response to dynamic excitation (density, shear modulus of elasticity, and damping) are different for soil than for rock. Moreover, in soils these properties vary greatly with the confining pressure and with the magnitude of the strains resulting from dynamic excitation, whereas in rocks they are more nearly constant.

Numerous empirical studies have been made comparing the observed accelerations for sites underlain by soil with sites underlain by rock at the same distances from the epicenters of strong motion earthquakes. In many cases the accelerations for the sites underlain by soil exceed those for the sites underlain by rock. In other words, it can be said that the earthquake has been magnified by the soil. However, in other cases it has been found that the soil reduced the accelerations.

A mathematical analysis to evaluate the response of a level mass of soil to seismic excitation has been published by Idriss and Seed (1968). This analysis is based on representing the horizontal soil layers by elastic columns whose mathematical behavior can be represented by a series of masses interconnected with springs with viscous damping between them. Similar analyses have been published in which the quasi-elastic behavior of the soil can be represented by a large number of finite elastic elements whose combined behavior can be predicted mathematically (Idriss and Seed, 1967). Either method requires a knowledge of the basic soil parameters: density, shear modulus, and damping coefficient.

Data on the engineering properties of the soil materials and those of the Hawthorne Formation underlying the Hutchinson Island site are available for the depths drilled: 400 feet, the Hawthorne Formation, in which the deepest samples were obtained, extends down to approximately 600 feet; it was assumed that the engineering properties of the deeper materials in this

Miocene formation are the same as those at shallower depths.

No samples of the materials deeper than 600 feet have been obtained at the site. These are the limestones of the Suwannee and Ocala Formations as well as the older, more indurated sedimentary rocks throughout the entire sequence of formations above the crystalline basement. In other areas of Florida, the Ocala and Suwannee Formations are present closer to the ground surface. A search was made of the records of Law Engineering Testing Company for engineering data on the rocks of these formations. Additional data on the engineering properties of these and deeper limestones and other sedimentary rocks were obtained from the Florida Geologic Survey. These data indicate that the softer limestones below 600 feet have unconfined compressive strengths of about 2,000 pounds per square inch and elastic properties resembling those of concrete of comparable strengths. The deeper formations are harder and stronger.

An analysis was executed by Dr. R.B. Barksdale, Assistant Professor, Georgia Institute of Technology, and Consultant to Law Engineering, based on the work of Seed and Idriss (1968). This was used to determine the elastic response of St. Lucie (and Green Cove Springs) to a strong motion earthquake. Because no strong motion earthquakes have ever been recorded in the southeastern United States, it was necessary to utilize records of a California earthquake. The one selected was the 1940 earthquake which was recorded at El Centro, California. Although certainly the geology of the El Centro area is not the same as that of Florida, the blanket of sediments in the vicinity of El Centro has some resemblance to the body of sediments underlying Hutchinson Island. A number of different analyses were made. One analysis assumed that the appropriate elastic column beneath Hutchinson Island extends to the crystalline basement at a depth of approximately 13,000 feet. An analysis using the El Centro earthquake but about one-half the actual accelerations (which would be more nearly comparable to the strongest earthquakes observed in the Southeastern United States) shows that the accelerations are relatively constant from the crystalline basement upward through the rocks to the base of the Miocene or Hawthorne Formation. The acceleration is thus the same between depths of 13,000 and 600 feet, showing that there is no significant magnification in the indurated limestones below the Miocene Formations. The natural period for the entire column was found to be approximately 6 seconds, which is far slower than the natural period for most structures involved in nuclear power plants.

The 600 foot soil column above the rocks was analysed by inputting the earthquake motions at the top of the limestone formations. For these analysis the El Centro earthquake was normalized to a maximum acceleration of 0.05g. The initial analyses purposely selected low damping properties for the majority of the soils. This was done in order to assess the worst possible conditions of amplification. Subsequent to the analysis with low damping, additional analyses were made utilizing damping values that were more consistent with the material type and strain level. The two analyses show that the soil column will either attenuate or amplify the base motions, depending on the damping. Using the most reasonable values for damping compatible with the seismic strains the maximum surface accel-eration is about 1/2 the base acceleration. This indicates the analyses using

very low damping were probably over-conservative.

Recent studies performed using SHAKE, a computer program for earthquake response analysis of horizontally layered soil prepared by the College of Engineering, University of California at Berkeley and presented as report No. EERC 72-12. Actual shear modulus and damping values determined by field and laboratory testing, extended by Seed's empirical relationships have been input into SHAKE using the design earthquake time history at a depth of 600 feet with the soil layering disclosed by the test borings. These results indicate as noted for early studies that the soil column attenuates.

In order to ascertain the response of the soil column at the site due to a distant earthquake, the El Centro record was expended along its time base by factors of 3, 4 and 5, i.e., the duration of the earthquake was multiplied by these factors. The acceleration amplitude was normalized to a maximum acceleration of .015g in order to account for attenuation over a distance of 380 miles. These studies used values of damping consistent with the material type and strain level. These studies show that the maximum earthquake acceleration is amplified by up to a factor 3.5 for the maximum time expansion. Although this amplification results in a maximum surface acceleration of about .05g the over conservative nature of the assumed earthquake motion should be recognized. Expanding the time scale of the natural earthquake record results in a time history which has a predominant period of 2.5 seconds and a duration of 2.5 minutes.
Obviously, an actual earthquake would be of considerably less duration and shorter period; thus, the surface accelerations would be less than those calculated for this most severe case.

These studies show that for a distant earthquake whose predominant period exceeds about 1 second, the base maximum acceleration will be amplified. Figure 2.5-20 gives the approximate relationship between amplification and earthquake period. It should be recognized that the duration of the hypothetical earthquakes used in developing this figure are longer than those of the same magnitude that occur naturally; therefore, the amplifications shown are somewhat higher than they should be. This figure shows that the maximum rock acceleration from the Charleston earthquake, 380 miles from St. Lucie, may have been amplified about 1.6 times through the soil column.

As shown in Section 2.5.2.5, the intensity of the Charleston earthquake at the St. Lucie site was about V MM. This corresponds to a surface acceleration of about .02g according to the conservative Herschberger curve. According to the response relationships discussed above the maximum rock acceleration due to the Charleston earthquake was .0125g.

2.5.2.4 Engineering Properties of Materials Underlying the Site

The engineering properties of the materials underlying the site are discussed in detail in Section 2.5.4.

2.5.2.5 Earthquake History

Most of the major earthquakes of the world have been related to distinct structural features. The strains associated with the development of these features are the proximate causes of the earthquakes. In the Southeastern United States, with the absence of contemporary mountain building and continuing faulting, the present earthquakes are more difficult to explain. Although various hypotheses have been advanced relating the earthquakes to structural features in this region there is no direct evidence of their association. The geologic evidence suggests that southeast earthquakes are the result of minor adjustments from the residual strains associated with the earlier movements or with continuing warping.

A study was made of earthquakes of the Caribbean area, the nearest region of any activity. The data obtained have shown that there is no correlation between the Caribbean epicenters and activity in the Southeastern United States (Guttenburg, 1954). There is no evidence that any of the earthquakes which originated in the Caribbean area have been felt in the vicinity of the site. An earthquake whose epicenter was in Cuba was felt in the vicinity of Miami (location shown on Figure 2.5-18); however, this is the only one of many events that appears to have been noticed anywhere in the state.

The Caribbean epicenters describe a large oval whose northern boundary is the island of Cuba, whose eastern boundary is the chain of volcanic islands lying south of Puerto Rico, and whose western boundary is the mountain chain of Central America. This pattern of epicenters is sharply defined. It exhibits no lineation that could be extended toward the southeastern United States. Moveover, the pattern of epicenters coincides with the pattern of observed recent (geologically speaking) volcanic activity. No such pattern of volcanic activity (or any volcanic rocks younger than several hundred million years) are found in the Southeastern United States. Therefore, it is concluded that the Caribbean seismic activity is not related to potential activity in Florida.

The earthquake history of the Southeastern United States and the nearby West Indies has been reviewed in detail in search of data on earthquakes affecting Florida. The southern part of the peninsula has never experienced earthquake damage. The scarcity of damaging earthquakes in even the more extensive region studied has discouraged the installation of instruments to measure the occasional minor tremors that do occur. Therefore, it has been necessary to evaluate the earthquake intensities from the description of their behavior, correlated with the geology of the region in which they occur, and relate this data to the site on the basis of the underlying rock strata.

The earthquake shocks observed in Florida have been infrequent, of low to moderate intensity or with epicenters far removed from the site. Several small earthquakes have been observed in Florida in the vicinity of Green Cove Springs which is located more than 180 miles from the site. The other minor earthquakes within the state have been extremely scattered. There is no evidence that any are related to observed structural features. The

strongest earthquake felt in the state during the 200 years of historic record was centered about 380 miles to the north, at Charleston, South Carolina. There is no evidence of any structural feature which might project the effects of such an earthquake toward this site at some later time.

The uniform Building Code (1964 Edition, Volume 1, as approved by the International Conference of Building Officials) designates the vicinity of the site as Zone 0 on the map entitled, "Map of the United States Showing Zones of Approximate Equal Seismic Probability." The U. S. Coast and Geodetic Survey indicates Zone 0 as an area of no earthquake damage (United States Earthquakes, 1928-1965). Furthermore, an examination of the epicentral map (Figure 2.5-18) shows that there is a broad segment of south central Florida in which possible epicenters are entirely absent. The St. Lucie site is in the middle of this region.

Because earthquakes in Florida are so unusual, they have received more attention than tremors of similar intensity in regions of greater activity. As a result, the descriptions of earthquakes have been well documented. The locations of all epicenters within a radius of 200 miles of the site are shown in Figure 2.5-18. A regional earthquake summary listing data for historic earthquakes felt in Peninsular Florida is presented in Table 2.5-5.

Several of the Florida epicenters are widely scattered. However, it is significant that five of the estimated epicenters have occurred within about 50 miles of Green Cove Springs. This is a small town 25 miles south of Jacksonville. The nearest of these epicenters to the site is approximately 180 miles.

The origin and the depth of the focus of these earthquakes in the vicinity of Green Cove Springs have not been established. To our knowledge, no specific geologic or geophysical investigations oriented toward relating geology in the Green Cove Springs area to the earthquake epicentral concentration have been published (Oglesby, 1973; Faulkner, 1973).

Law Engineerings investigation of the Green Cove Springs area has consisted of a review of the published literature, personal contact with geologists knowledgeable of the Green Cove Springs area and a compilation of available well data from which several geologic sections have been made.

There have been six other shocks which probably were felt in Central Florida whose epicenters are more remote from the site. These include the Charleston, South Carolina earthquake of 1886; earthquakes originating at Savannah, Georgia and possibly some of the earthquakes in the Caribbean. The largest earthquake in this peripheral area was that of Charleston, South Carolina, in 1886; its estimated epicentral intensity was IX MM (Murphy, 1973).

The earthquakes which occurred in the vicinity of Charleston, South Carolina were centered approximately 380 miles from the Hutchinson Island site. There are no records of the effect of the Charleston earthquake at Fort Pierce. A study of the intensities that were observed and the effects recorded at other points in Florida suggest that this earthquake had an intensity of IV MM to V MM at the St. Lucie site.

A study was made of the newspaper records in Fort Pierce and Stuart, the cities nearest the site. Neither was established early enough to note the Charleston 1886 earthquake. The Fort Pierce record commences in 1913 and that in Stuart in 1903. Both papers quoted news dispatches from St. Augustine regarding the 1935 earthquake centered near Palatka (in the Green Cove Springs area). Neither paper mentioned the tremors in their respective vicinities. Neither paper contained any mention of the 1930 or 1942 tremors, whose epicenters were 100 miles southwest. The Fort Pierce paper quotes a dispatch from Hollywood, Florida, 85 miles south, describing a 1945 tremor at that city. However, there is no mention of this being felt at Fort Pierce. Considering the epicenter intensities of the other earthquakes in the region, it is unlikely that the others were noticed at Hutchinson Island or Fort Pierce.

On October 27, 1973, an earthquake occurred near Lake Harney, northwest of Titusville, Florida and approximately 115 miles north of the site. Preliminary data from the National Earthquake Information Center indicates this earthquake had a epicentral intensity of V MM. A separate investigation by Law Engineering was undertaken to determine epicentral intensity, approximate magnitude, and the felt area. During the investigation approximately 500 responses were obtained by questionaires or by personnal interviews in central and eastern Florida. In addition, a field seismic station was set up to monitor possible aftershocks, and records of 13 seismic stations which recorded the event were studied. On the basis of this data, and data collected from an independent investigation by Bollinger (1973), the following conclusions are made: the felt area of the earthquake is approximately 9,000 square miles. The event had a maximum intensity of V MM and an approximate magnitude of 3.9. Intensity V+ MM was felt over an area of 4,000 square miles. The earthquake was not felt at Hutchinson Island. A detailed report of Law Engineering's investigation of this earthquake is presented as Appendix 2D.

2.5.2.6 Correlation of Epicenters with Geologic Structures

The major tectonic structures which occur within about 200 miles of the site have been previously described in Section 2.5.1.1.3.1. The only major tectonic structure in east-central Florida is the Peninsular Arch. The Peninsular Arch is a deep structure in basement rocks which trends southeast from southern Georgia into central Florida. The Arch was produced prior to the Mesozoic (Applin, 1951) and has been covered with sediments. The early Cretaceous sediments above the Arch reflect its shape. However, succeeding sediments thin at the axis of the Arch and thicken along its flanks. Post-Cretaceous deposits above the Arch are practically uniform and do not reflect the undulating basement structure (Figure 2.5-6).

The Ocala Uplift is an anticlinal fold occurring in Eocene and Miocene rocks and is considered to be a minor structure. The uplift is superimposed above the Peninsular Arch, however, they are not structurally related. When the Ocala Uplift is compared to the Peninsular Arch, it is a relatively insignificant feature. The closure of the Peninsular Arch is over 5,000 feet compared to several hundred feet of closure in the Ocala Uplift.

The Florida Geological Survey and the U. S. Geological Survey have hypothesized a number of local faults along the Ocala Uplift. These faults generally have offsets on the order of 50 to 400 feet, as inferred from differences in levels of strata or marker beds at points 2 to 5 miles apart, as discussed in Section 2.5.1.1.3.2. Such minor differences can, of course, be attributed to other mechanisms, such as folding or erosional features. The investigations for the St. Lucie site, which were based on more comprehensive data than the original studies which resulted in the local fault postulations, result in the conclusion that local faults do not exist in the area of Hutchinson Island.

An attempt has been made to associate the hypothesized structures in Florida, previously described, with earthquake epicenters. Only a few earthquakes have been recorded in Florida over the 250 year period. However, of those which have occurred, the majority have an epicenter within miles of Green Cove Springs, Florida. No accepted hypotheses have been made which relate these earthquakes in the Green Cove Springs area to structure.

Where similar earthquake intensities and/or mechanisms can be defined, the region of similarity is referred to as a tectonic province. Within a tectonic province, the geology and general structure, together with the frequency and size of earthquake, are considered to be related. The current delineation of tectonic provinces in the Southeastern United States places the site within the Coastal Plain tectonic province (excluding the Charleston, South Carolina area).

2.5.2.7 Identification of Capable Faults

There is no geologic evidence of surface faulting within the Coastal Plain Province that is even remotely related to earthquakes that have occurred in historic time. This is supported in Bonilla's review of Historic Surface Faulting in the Continental United States and Adjacent Parts of Mexico (1967).

It is concluded that there are no identifiable capable faults that could be expected to produce surface displacements anywhere within the Coastal Plain Province, within 200 miles of the site.

2.5.2.8 Description of Capable Faults

There is no evidence for any capable faulting within 200 miles of the St. Lucie site which may be of significance in establishing the Safe Shutdown Earthquake.

2.5.2.9 Maximum Earthquake

The largest historic earthquakes which have occurred in Florida are in the vicinity of Green Cove Springs and have observed epicentral intensities of between V and VI MM. With rare exception, the Green Cove Springs area is the only area in Florida where earthquakes have occurred. In regions in which the geologic structure is homogeneous and in which a number of earth-

quakes have occurred, it has been observed that the earthquakes tend to approach a limiting magnitude. This limiting intensity or magnitude is logical in the light of the hypothesis of the generation of earthquakes by the release of strains that accumulate to produce local slippage along surfaces of weakness.

In accordance with the tectonic province concept, the largest earthquake which could have occurred in the vicinity of St. Lucie is of the same intensity as those which have occurred in the Green Cove Springs area (intensity VI MM).

In order to determine if the rock motions attendent to an intensity VI MM earthquake would produce different response at the Green Cove Spring site and the St. Lucie site, additional response analyses were performed utilizing the geologic column at Green Cove Springs. The sequence of formations at Green Cove Springs is similar to that at St. Lucie although the total sedimentary rock thickness at Green Cove Springs is substantially less. The upper most materials of lesser induration at Green Cove Springs have similar consistencies to those at the site, although the total thickness of the Miocene and younger formations at Green Cove Springs is approximately 300 feet compared with the 600 feet at Hutchinson Island. The response analyses conducted for the Green Cove Springs profile were identical to those previously described for the St. Lucie site. These analyses indicate that there is little change in the acceleration through the greater part of the limestone above the crystalline basement until the Miocene formations are reached. The amplifications through the Miocene and younger formations was calculated to be about 1.6 times greater than the amplifications through the Miocene and younger formations at the St. Lucie site. The primary reason for the greater degree of amplification at Green Cove Springs than at St. Lucie is due to the shorter soil column at Green Cove which has a fundamental period of about two seconds. The second mode of the Green Cove profile has a period of about .5 seconds which approximately corresponds to the predominate period of the input earthquake. Consequently the similarity between the dominant period of the earthquake and the major participating modes of the soil column result in some amplification. These analyses show that the same rock motions would produce higher surface accelerations at Green Cove Springs than at St. Lucie. Consequently the assumption that an intensity VI MM earthquake with an attendant surface acceleration of .03g is very conservative for the St. Lucie site.

2.5.2.10 Safe Shutdown Earthquake

Analyses to determine the maximum surface acceleration resulting from an intensity VI MM earthquake occurring adjacent to the site were performed as discussed in Section 2.5.2.3. The results of these analyses indicate that surface acceleration is partially dependant on the degree of damping applied to the upper 600 feet of sediments at the site. A conservative value of 0.05g resulted from these studies.

An intensity VI MM earthquake represents a surface acceleration of 0.05g according to Hershberger. This acceleration value of 0.05g was then increased to O.1g in order to satisfy SSE requirements.

Other intensity-acceleration relationships (numbering over 40) have been developed. The most recent intensity-acceleration relationship has been published by Coulter, Waldron, and Devine (1973). These relationships attempt to distinguish between foundation conditions which vary from rock to soft soil. The relationship of the intensity-acceleration curve published by Coulter, Waldron and Devine to the site is discussed in Section 2.5.4.9. This relationship also indicates a ground acceleration value below the minimum requirements which governed the selection of 0.1g. We consider the 0.1g maximum surface acceleration to be a very conservative figure, when compared with actual geologic and seismic conditions in peninsular Florida.

2.5.2.11 Operating Basis Earthquake

The Atomic Energy Commission criteria state the Operating Basis Earthquake (OBE) shall be specified by the Applicant and shall be defined by response spectra. If vibratory ground motion occurs which produces a maximum acceleration above the OBE, at any structure foundation, shutdown of the facility for inspection is required.

The OBE selected for foundations at the St. Lucie site is 0.05g. This is a conservative figure.

2.5.3 SURFACE FAULTING

2.5.3.1 Geologic Conditions of the Site

The geologic history of the site and the lithologic, stratigraphic and structural conditions have been described in the previous Section 2.5.1.2.

2.5.3.2 Evidence of Fault Offset

There is no evidence of fault offset at or near the ground surface at or near the site.

2.5.3.3 Identification of Capable Faults

No capable faults have been identified within five miles of the St. Lucie site.

2.5.3.4 Earthquakes Associated with Capable Faults

The scattered and infrequent moderate-intensity earthquakes in this area are not associated in any way with hypothesized faults or tectonic structures in the Coastal Plain.

2.5.3.5 Correlation of Epicenters with Capable Faults

Since there is no relation of earthquakes to capable faults, a correlation of epicenters with active faults can not be made.

2.5.3.6 Description of Capable Faults

As stated in the previous Section 2.5.3.5, no capable faults have been identified within five miles of the St. Lucie site.

2.5.3.7 Zone Requiring Detailed Faulting Investigations

As previously stated, no active fault exists within five miles of the St. Lucie site.

2.5.3.8 Results of Faulting Investigation

An investigation of the closest hypothesized fault to the site (approximately 5 miles west of the site) was made as described in Section 2.5.1.2.3 of this report. The investigation of this hypothesized fault concluded that this fault does not exist at its closest approach to the site.

2.5.3.9 Design Basis for Surface Faulting

As indicated in Section 2.5.3.2, a design basis for surface faulting does not need to be considered at this site.

2.5.4 STABILITY OF SUBSURFACE MATERIALS

2.5.4.1 Geologic Features

There is no evidence on-site or in nearby areas of potential subsidence, uplift or warping, of deformation zones, including shears, joints, fractures or folds, of alteration zones resulting from weathering or metamorphosis, of unrelieved residual stresses in bedrock, or of soil thixotrophy. For additional discussion refer to Sections 2.5.1.1 and 2.5.1.2.

For a detailed discussion of liquefaction potential refer to Section 2.5.4.8 and for a detailed discussion of consolidation characteristics and settlement refer to Section 2.5.4.10.2.

2.5.4.2 Plot Plan

Atlantic Ocean and the Indian River is shown on Figure 2.5-15. A larger scale drawing of the plant site area is presented on Figure 2.5-15A. The locations of all borings, proposed excavations, as well as plant structures (including Category I, structures, pipe lines and electrical ducts) are shown on Figures 1.2-1, 2.5-4, 2.5-15, 2.5-15A, 2.5-82, 2.5-83, 2.5-85, 2.5-86 and 8.3-13. Table 3.2-1 presents a list of all Category I structures. Subsurface cross sections of the plant area indicating the limits of proposed excavations are shown on Figures 2.5-21 and 2.5-22. The subsurface cross sections also show the locations of Category I structure foundations with respect to subsurface conditions.

Borings 1 through 20 as shown on Figure 2.5-23 were drilled in 1968 prior to site preparation. Cross sections through these borings are shown on Figures 2.5-24, 25, 26 and 27. Seven of these borings were performed in the plant area proper and the remaining thirteen borings were performed in the general site area. These borings were drilled to evaluate the general site geologic conditions and to provide preliminary subsurface information on static and dynamic soils conditions. Geophysical surveys were performed at that time.

In early 1969, Borings 101 through 146 were drilled. Twenty-seven of these additional borings were located in the immediate plant area to supplement the data obtained from the initial site boring program. The remainder were drilled at locations of site related facilities. The purposes of the supplementary foundation investigations were as follows:

a) To verify in further detail the subsurface conditions at the location of the structures

b) To establish the liquefaction resistance of the soils in greater detail

c) To develop additional foundation data to verify bearing capacity and estimated settlements

d) To determine the physical properties of the compacted soils for use in the dynamic evaluation of the soil foundation structural system

e) To locate suitable fill material

Additionally, at this time auger borings A-1 through A-4 were performed to obtain samples for laboratory investigation of the proposed fill.

Borings 147 through 173 and SB-1 through SB-5 were performed as continuing studies of site related facilities, such as ocean intake and discharge pipelines, bridge pier foundations (Highway A-1-A) and switchyard.

Additional geophysical surveys were performed in early 1973 to obtain shear wave velocities of materials within the construction area.

Graphical descriptions of the soils encountered in the main borings are shown on the logs of borings presented in Appendix 2A. Geophysical survey test results are presented in Section 2.5.4.4. The field and laboratory testing procedures used in this investigation are discussed in Sections 2.5.4.3 and 2.5.4.4. Laboratory test data is presented in Appendices A and C for the St. Lucie Unit 1, PSAR.

2.5.4.3 Properties of Underlying Material

Samples for laboratory testing were obtained from the test borings in order that the proper ties of the underlying materials could be determined. The borings were advanced by a rotary drilling process which utilizes a viscous bentonite drilling fluid to flush the cuttings and stabilize the hole. At regular intervals, the drilling tools were withdrawn and soil samples obtained with a standard 1.4" I.D., 2.0" O.D., split-tube sampler. The sampler was initially seated 6" to penetrate any loose cuttings then driven an additional foot with blows of a 140 lb hammer falling 30". The number of hammer blows required to drive the sampler the final foot was recorded and is designated the "Standard Penetration Resistance".

The samples, as they were obtained, were classified in the field by an engineering geologist. Portions of each soil sample were sealed in glass jars and transported to our laboratory where they were examined by soils engineers and geologists.

Core drilling was performed generally in accordance with specification ASTM D 2113-70. Prior to initiating coring operations, 4-inch I.D. casing was installed to a sufficient depth to prevent caving of overburden soils into the hole. Boring was performed using an NX or HQ, double-tubed, swivel type core barrel. Upon completion of each run, the core was removed from the barrel and logged by a geologist. All core was carefully placed in wooden boxes and wrapped in plastic to prevent excessive moisture loss.

Undisturbed samples were taken of selected strata for laboratory testing. The samples were obtained by forcing 30” long sections of 3” O.D., stainless steel tubing into the soil. The sampling procedures were conducted in accordance with ASTM Procedure D 1587. The samples thus secured, were sealed with paraffin to prevent moisture loss and transported to the laboratory.

Bore hole logging of borings AG 104, AG 105, and AG 106 was performed upon completion of drilling. Logging was performed by personnel of the Division of Water Resources, State of Florida. A Gearhart-Owens Model 3200 portable logging truck mounted system was used to perform the work. Data was recorded on a Gearhart-Owens MRP 501 x-y recorder.

A single point electric probe was used to simultaneously record self potential and electrical resistivity. A gamma-ray record was also obtained for each boring.

The properties of underlying materials are identified with the major geologic formations encountered. The two major formations encountered and their general engineering characteristics are discussed below.

The Anastasia formation occurs at the surface of the plant site and varies in thickness from 135 to 155 feet. The Anastasia formation consists of grey slightly clayey and silty fine to medium sand with fragmented shells and, in places, fragmented shell beds with slightly clayey and silty fine sands. There are also discontinuous pockets of cemented sand with shells and sandy limestone. These discontinuous cemented pockets are generally

found from about elevation (-) 35 to (-) 60, except at Boring B-1 (Boring logs are in Appendix 2A) where they extend to about elevation (-) 90. Also occasional discontinuous thin plastic clay lenses were found in the upper part of the formation.

The Hawthorne formation underlies the Anastasia formation. The upper 100 to 150 feet of the Hawthorne formation consists of a green slightly clayey and silty very fine sand. The lower part becomes generally more clayey. The published geologic information describes the Hawthorne formation as extending downward to about elevation (-) 600 in this area. This was partially substantiated by our deep geophysical exploration at the site. The Hawthorne formation was found to extend to a depth of at least 600 feet; no harder layers were encountered to this depth. The Hawthorne formation changes slightly to a grey white sandy clay at this site below possible elevation (-)450.

This formation is generally dense and indicated by both desiccation and cementation.

The borings indicated that the soil could be separated into three zones depending upon compactness. The first or Upper Zone 50 to 60 feet was a loose sand with small amounts of silt and clay, containing isolated pockets of shell fragments and limestone nodules.

As described in Section 2.5.4.8 the studies have shown that the Upper Zone was potentially subject to liquefaction. Therefore, this zone required remedial treatment beneath all critical structures as described in Section 2.5.4.5.

Beneath non-critical structures, the upper sands provide support for light structures. They have moderate strength and are relatively incompressible. Heavier structures could cause consolidation of the clayey lenses, and analysis is described in Sections 2.5.4.10. The recent organic or peat mat underlying the Upper Zone was unsuitable for foundation support, and therefore was removed.

An Intermediate Zone extends from about 60 feet to 150 feet in depth. The soil of the Intermediate Zone differs from the shallower soil in that it is denser, contains a greater percentage of fines (material finer than the number 200 sieve), and has very few pockets of limestone nodules and shell fragments. This zone is differentiated from the Upper Zone on the basis of consistency and grain size characteristics.

In this zone beneath the plant there do not appear to be any isolated pockets of limestone nodules and shell fragments. This finding coupled with the increased amount of fines in the zone from 60 feet to about 150 feet, separates it from the Upper Zone. The material in the Intermediate Zone has properties which will not adversely affect the foundation. It is strong enough to support the loads produced by surcharging and by the imposed weight of the structures.

The Deep Zone extends from 150 feet in depth to at least 400 feet. This material is considerably more clayey than the material above, does not contain pockets of shells and limestone, and is dense. The soils are normally consolidated under the existing overburden load.

Table 2.5-2 shows penetration resistance and percent fines of certain borings.

At depths greater than 150 feet the standard penetration test does not give reliable values. The consistency of this material was determined from triaxial test results, from torvane shear values, and from an approximate correlation of shear strength with the force required to push a standard split-spoon sampler into cohesive soils. The results of these tests are given in Table 2.5-4. This latter method was utilized since the drill rod and hammer weight was greater than 1000 pounds and significantly influenced the results of the standard penetration test. The approximate method of determining the shear strength of cohesive soil using a split-spoon sampler is shown in Figure 2.5-28. The soil consistencies were determined from the following table:

CONSISTENCY OF CLAY IN TERMS OF UNCONFINED COMPRESSION STRENGTH (Reference 30)

Consistency	Unconfined Compressive Strength (kg/cm^2)
Very soft	Less than 0.25
Soft	0.25 - 0.5
Medium (firm)	0.5 - 1.0
Stiff	1.0 - 2.0
Very Stiff	2.0 - 4.0
Hard	Over 4.0

In all cases, the consistencies were verified by visual examination. From analysis of laboratory test data it is concluded that this deep material does not present any foundation stability problem. It is anticipated that the minor amount of consolidation settlement will be such that the consolidation phase of the settlement is expected to be complete during the construction operation, or shortly thereafter.

Samples extracted from the borings were subjected to laboratory tests to determine the physical properties of the soils. Summaries of the laboratory data are given in Appendix 2A. Laboratory test data is presented in Appendices A and C for the St. Lucie Unit 1 PSAR. The complete laboratory program included the following tests:

a) Grain size analysis
b) Specific gravity
c) Moisture content and density
d) Maximum-minimum density
e) Proctor compaction test
f) Consolidation.

Additional laboratory tests were performed to determine the static and dynamic shear properties of the soils. These tests include the following:

a) Direct shear
b) Triaxial compression
c) Cyclic triaxial shear
d) Compression wave velocity.

Direct Shear and Triaxial Compression tests were performed on representative samples of the compacted material that was used for the plant backfill. These tests further established the soil parameters, such as the angle of internal friction and the effect of pore water pressure, which are utilized in bearing capacity evaluations.

2.5.4.4 Soil and Rock Characteristics

In order to determine the seismic effects on the plant structural foundations, the geophysical properties of the site were examined. The scope and mode of this examination is presented below:

a) Determination of Geophysical Parameters for Seismic Analysis of Structures

A refraction seismic exploration was undertaken at Hutchinson Island to determine the compression wave velocities under dynamic conditions and also to determine if there were any significant changes in the rigidity of the formations (which could be detected by seismic velocity) below the deepest borings made at the site. The strains that accompany this type of field test are very small, i.e., microstrains. The corresponding compression wave velocity represents an upper limit. The wave velocity for larger strains, such as those induced by strong motion earthquake, would be smaller. An additional use of the data from the geophysical exploration was to determine if there were any significant differences in elevation between the lithologic boundaries that might suggest structural displacements, in a wider area than that covered by the onsite borings.

Twenty-eight refraction lines of varing lengths were made in the site vicinity. The lines were spaced around the perimeter of the site, along State Road A1A to the east, and along the mosquito control dike south and west of the site.

In general, the seismic refraction profiles indicated there were no significant variations in the elevations of the boundaries between the strata at the St. Lucie site. A few of the seismic lines indicated materials with velocities of approximately 6660 feet per second exist between the surface and 150 feet. These higher velocity zones correlate with local concentrations of limestone nodules and cemented sand shell lenses found in the borings.

The instrument used was a Dresser Model RS-4, 12 channel, recording seismograph with surface geophones capable of recording compression waves. The charges of explosives detonated ranged from 2 to 15 pounds of gelatin dynamite placed both at the ground surface and in bore holes as deep as 70 feet.

In several instances, it was possible from the seismic refraction work to calculate velocities in the two principal formations underlying the site; the upper sands of the Anastasia Formation (to a depth of about 150 feet), and the deeper clayey Hawthorne Formation. In traverses where compression wave velocity could be differentiated between these two formations, it was found to be 5700 feet per second between 50 feet and 150 feet, and 6800 feet per second below a depth of approximately 150 feet. These two velocities are very similar, making a delineation of the Anastasia-Hawthorne contact difficult on the basis of the seismic refraction work alone. However, in those instances where velocity differences were obtained, the Anastasia-Hawthorne interface depth was found to be in agreement with borings drilled in the vicinity.

If it is assumed that each material is a homogeneous elastic mass, it is possible to calculate the elastic modulus from the density of this material and the compression wave velocity. Using density data available from the borings at the site, typical total densities were used in calculating the approximate dynamic modulus of elasticity. The upper limit elastic modulus for the sands of the Anastasia Formation was calculated to be approximately 210,000 psi, and for the deeper Hawthorne Formation, approximately 285,000 psi.

Due to the varying densities it was not possible to determine with reliability the compression wave velocity in the uppermost loose material above a depth of 50 feet. Further the water table was approximately at the ground surface. In saturated materials whose wave velocity in the dry state is less than the velocity of water, the indicated wave velocity will be that of the water in the voids rather than of the formation itself.

Thus, based on rigid control conditions for the backfilling operation (assuming uniform density) the elastic properties for the compacted fill foundation were calculated as follows:

Four cyclic triaxial compression tests were conducted to establish the compressional wave velocity of the compacted backfill. The tests were conducted on samples compacted to 85 percent relative density and at residual confining pressures developed during compaction in order to simulate the ground surface conditions. Crystal transducers were used for sending and receiving the ultrasonic waves, thus the induced strains were in the microstrain region as is the case with the field seismic work. Nine tests were made varying moisture content and percent fines and all tests confirmed a compressional wave velocity of 700 fps with results not varying more than 5 percent. Other investigators verify these results [9,33].

Assuming that the materials are a homogeneous elastic mass and utilizing the measured compressional wave velocity and a Poisson's ratio of 0.25 as determined from the above references, the near ground surface elastic modulus was calculated to be 12,000 psi. It has been shown by other investigators, including the ones referred to above, that the elastic modulus increases with depth and is approximately proportional to the square root of the depth. Thus at the level of the reported field test the calculated upper bound elastic modulus from laboratory data will be approximately twelve times the surface value or about 150,000 psi.

The comparison of the elastic modulus as measured in situ and as determined in the laboratory is reasonable. If consideration is given to the in-place soil formation and structure as compared to a remolded structure of a compacted soil, the comparison is quite favorable. This phenomenon of original soil structure or in-situ structure has long been realized to have an important effect on the characteristic of soils, in particular the elastic properties.

Since the strain conditions developed during earthquake shocks are much greater than the very small strains induced to determine the elastic properties as given above, consideration must be given to selecting parameters to be used in the elastic and shear moduli and Poisson's ratio.

Evaluation of the elastic moduli was carried out in two steps. First the modulus applicable to very small strains (microstrains) was determined. This is usually done by utilizing field or laboratory seismic tests as described above. Then reduction factors were used to adjust the modulus for the expected level of strain.

Once strains exceed the 10^{-5} to 10^{-4} level and non-linear effects become noticeable, there is no single universally accepted definition of velocity or modulus. As the peak strain involved in a repeated loading application increases, the modulus and velocity decrease. Some investigators have expressed this decrease by means of a reduction factor, which relates the ratio of the velocity for a level of strain to the velocity for very small strains(10). Another investigator, H.B. Seed(11) states that "...a change in strains from the magnitude associated with microtremors to the magnitudes associated with major earthquake may cause a 2 to 10 fold decrease in modulus... while microtremor effects can serve an extremely useful purpose in establishing one bound on the range of possible behavior patterns and in checking the applicability of a proposed analytical procedure, they do not appear to provide, in themselves, a full predictive capability for engineering purposes."

In order to evaluate the effects of the magnitude of the elastic constants on the dynamic analysis of the Class I structures, both the numerically larger modulus as determined from the field seismic work, and the smaller modulus as determined from the laboratory studies, reduced as described above, were considered. This approach served to establish an upper bound modulus, determined from microstrains, and a lower bound modulus, determined by reduction factors described above.

It was determined that the numerically large elastic constants would yield a condition in the range of the peak of the response spectra. Therefore, any other values of the elastic properties could only yield numerically smaller response characteristics. Field measurements indicated that the elastic modulus was 210,000 psi and laboratory measurements of the compacted fill indicated a ground surface elastic modulus of 12,000 psi.

Preliminary, separate dynamic analyses of the Reactor Building structure were run using elastic moduli ranging from 10,000 psi to 250,000 psi and it was found that the maximum response acceleration occurred at an elastic modulus of approximately 150,000 psi. These preliminary maximum acceleration values were used for the design of the structures. When the design

was completed the structures were again analyzed using the elastic moduli which resulted in the preliminary maximum acceleration. If a significant difference in acceleration resulted from the re-analysis, a range of elastic moduli were run again to determine the most conservative values. For the sake of conservatism, the numerically larger elastic moduli were utilized to determine the spring constants in the structural dynamic analysis. This approach of using the modulus determined from microstrain seismic methods is conservative. The method of seismic analysis of Class I structures is discussed fully in Section 3.7.

b) Determination of Geophysical Parameters for Seismic Analysis of Equipment

When the parametric values which were used for seismic Class I structure analysis were used for determining the floor response spectra envelopes for seismic Class I equipment analysis, the design of piping systems became unrealistic because of the wide range of periods for which peak resonant accelerations were indicated by the spectral curves. Therefore a further soils testing program was initiated to determine a more realistic value of soils modulus by determining the relationship with respect to strain rather than utilizing the parametric range for the building analysis.

The following laboratory program was set up to determine the applicable soils modulus.

1) Existing cyclic triaxial tests were studied to establish the moduli and strain relationships. Originally the data was not reduced to determine any stress-strain functions.

2) Existing computer studies of soil response had developed moduli and corresponding strains. These were reviewed in order to determine the compatability of the strains to the strains developed as a result of these additional investigations.

3) Additional cyclic tests were performed at stress and strain levels of interest using soil samples obtained from the field under proper relative densities and confining pressures.

4) The developed relationship of soils modulus versus strain was plotted and analyzed and is presented as Figure 2.5-32.

Two hundred pounds of sand were obtained from the site. Initially, a grain size test was performed on a representative part of the sample. This test was performed in accordance with ASTM standards.

Maximum and minimum density tests were also performed on a representative portion of the sample. The maximum density was determined utilizing a wet sample on the vibrating table. The minimum density was determined by pouring dry sand through a funnel into a known volume. A specific gravity test was also performed on a portion of the sand sample in accordance with ASTM D 854. The results of the specific gravity and density determinations were utilized for maximum and minimum void ratio calculations, which are:

Void Ratio, maximum - 0.987
minimum - 0.609

Dry Unit Weight, maximum - 104.0 pcf
minimum - 84.2 pcf

Specific Gravity - 2.68

Cyclic Triaxial Shear Tests were run on samples 2.8 inches in diameter by 5.6 inches long that were compacted to 85 percent relative density. The samples were then placed in a cyclic triaxial shear chamber and subjected to a confining pressure of 3,500 psf. Water was allowed access to the sample and backpressure of approximately 50 psi was applied to insure saturation. During the backpressure saturation, the effective confining pressure was maintained at 3,500 psf. Cyclic deviator stresses varying from 180 psf to 1,700 psf were applied with a bellowfram loading system at a frequency of 2 cycles per second. During cyclic loading, continuous record of load and deformation were monitored by calibrated strain gauge devices and recorded. Both load and deformation monitoring devices were mounted within the triaxial chamber.

The dynamic modulus of elasticity, "E", was calculated from the peaks of the load and deformation outputs. In effect, the calculated moduli are the secant values from peak to peak of the hysteresis loop. The dynamic shear modulus, "G", was calculated using the relationship:

$$G = \frac{E}{2(1+\mu)}$$

Poisson's ratio, "μ" was assumed to be 0.45 for saturated sand.

For low ranges of strains for which the concepts from the theory of elasticity are of use, Poisson's ratio (μ) varies with strain. The value has been shown by other investigators to be near 0.5 for very low strain values, on the order of 10^{-6} and to decrease to near 0.2 for strain levels near 10^{-3} and then increase to values above 0.5 at very high strains for dense sands. It is difficult to make an exact evaluation of the value of "μ" for use in solution to real problems; but for dense soils even the range from 0.5 to 0.2 has a relatively small effect upon engineering calculations and evaluations. The 0.45 value was utilized in this section to evaluate the value of the dynamic shear modulus of elasticity at strains of 10^{-5} in/in. This assumed value of "μ" has since been confirmed for this strain level using the values of the field compressional and shear wave velocities obtained from the cross hole technique discussed in Section 2.5.1.4(c), using the following relationship:

$$\mu = \frac{1/2(V_p/V_s)^2 - 1}{(V_p/V_s)^2 - 1}$$

where:

V_P = Compressional wave velocity

V_S = Shear wave velocity

Table 2.5-3 presents a tabulation of deviator stress, strain and moduli. Figure 2.5-11 is a graph of shear modulus versus shear strain.

The shear modulus of sand is dependent upon several variables. Among the most important variables, which are not functions of the soil properties, are the effective confining pressure and the range of strain to which the sand is subjected. Seed and Idress[11] have published a tentative relationship for a medium sand (D_r 75 percent) which accounts for both confining pressure and strain. When this data is extended to an effective confining pressure of 3,500 psf, close agreement occurs with the data developed by this study. At strains above 3 x 10^{-4} inches per inch the Hutchinson Island data is higher than the published data. This is partially explained in a somewhat stiffer material. Slight fluctuations in pore water pressure may also have influenced the Hutchinson Island tests but were not accounted for. At any rate, the data for Hutchinson Island agrees very well with the data presented by Seed and Idriss.

The tests for Hutchinson Island represent a single level of confining stress. The confining pressure used was selected as the average effective overburden pressure at the level of the Reactor Building foundation. In order to extend this data to other levels, the variation in confining pressure should be accounted for. The modulus of elasticity was found to vary exponentially with the confining pressure. Sowers and Sowers[12] state that:

$$E = C\sigma_3^n$$

where "n" varies from .35 to .82 depending on the principal stress ratio, Seed and Idriss state:

$$G = k_3\sigma_3^n$$

where n = .33

As previously mentioned, the test results for Hutchinson Island sand agree well with the data presented by Seed and Idriss, particularly in the strain range of 1 to 2 x 10^{-4} inches/inch. Therefore, it is concluded that the relationship developed by Seed and Idriss can be utilized to extend the Hutchinson Island data to confining stresses other than 3.5 ksf (24.3 psi). That is:

$$G_x - G_{3.5}\left[\frac{\sigma_{3x}}{24.3}\right]^{1/3}$$

Where:

G_x = Shear modulus desired at confining pressure

$G_{3.5}$ = Shear modulus from Figure 2.5-11

σ_{3x} = Confining pressure at level G is desired, (psi)

The response of the soils at Hutchinson Island was studied under varying base accelerations ranging from 0.059 to 0.10g. These studies utilized published relationships of shear moduli and strain for sands of 75 to 85 percent relative density[11]. The soil response studies utilized shear moduli ranging from 16,700 Psi to 14,000 psi for the various applied accelerations and resulting strain levels. The average strains calculated at a depth of 40 feet, (the level of the Reactor Building and Reactor Auxiliary Building foundation mats), ranged from 5×10^{-5} to 1.5×10^{-4} in/in for their respective induced acceleration levels.

The strain levels and respective soils moduli were obtained by shear beam type analysis of the site soil column and the soils modulus was iterated until compatible moduli and strain characteristics were obtained.

The nigher strain level commensurate with the SSE of 0.1g yields a strain of 1.5×10^{-4} which corresponds to a shear modulus of 14,000 psi (refer to Figure 2.5-11). The lower strain level on the order of 5×10^{-5} and commensurate with the OBE of 0.05g resulted in the use of a higher shear modulus of 16,700 psi (refer to Figure 2.5-11).

c) Determination of Geophysical Parameters by Field Seismic Measurements

The purpose of this survey was to measure the velocity of shear waves in the field using cross hole techniques, and to compare these geophysical parameters with values discussed in Section 2.5.1.4 a) and b).

Appendix 2C describes the procedure used for this seismic cross hole survey, includes the data obtained and presents an evaluation of that data. Figure 2.5-11 includes the data obtained and presents an evaluation of that data. Figure 2.5-32 includes the range of shear modulus for the compacted fill calculated from these seismic values. This data compares favorably with the previous data and indicates that the range of values for shear modulus used in our analyses are conservative.

2.5.4.5 Excavation and Backfill

2.5.4.5.1 General Description

The plant area grade will be established at an elevation of plus 18.0 ft. similar to unit number one. Foundations for all Seismic Category I and all non-seismic buildings within the Nuclear Plant Island Excavation will be founded on select engineered fill. The limits of excavation and backfill including sections showing the locations of the various Category I structural mats within the excavation and classes of engineered fill are shown on Figures 2.5-82 and 2.5-83. Subsurface profiles through the plant area are presented on Figures 2.5-21 and 2.5-22 which also show foundations with respect to elevation. Figure 2.5-29 presents an analysis of the stress conditions during construction (excavation and backfill) and Figure 2.5-30 presents the final loading conditions.

2.5.4.5.2 Gradation Limitations and Compaction Requirements for Engineered Fill

a) Class I Fill - (to be used for support of Seismic Category I structures within the Nuclear Plant Island) Clean sand and gravel with a maximum of 12 percent fines (where percent fines is defined as the percent of material passing the number 200 sieve). The source of this soil is the sands and gravels of the Anastasia formation obtained from the various plant area excavation or dredged from adjacent Big Mud Creek. All Class I soils will be compacted at proper moisture content to 98 percent Modified Proctor density, as determined by tests performed in accordance with the provisions of ASTM Specification D-1557-70, "Modified - Density Relations of Soils Using 10 lb Hammer and 18 in Drop." The tolerances and minimum density acceptance criteria are specified in the backfill specifications. Refer to Section 2.5.4.5.3.

b) Class II Fill - (to be used for nons-seismic structures) - Silty sand and gravel or clayey sand and gravel with a maximum of 40 percent fines. The sources of this soil are the sands and gravels of the Anastasia formation obtained from the various plant area excavations or dredged from adjacent Big Mud Creek. All Class II soils will be compacted at proper moisture content to 95 percent Modified Proctor density, as determined by tests performed in accordance with the provisions of ASTM Specification D-1557-70, "Modified - Density Relations of Soils Using 10 lb Hammer and 18 in Drop."

2.5.4.5.3 Excavation Backfilling and Compaction Specifications

Detailed excavation, backfilling and compaction specifications are presented as Appendix 2B of the PSAR. All compaction operations shall be closely monitored with field tests in accordance with the provisions of the Quality Assurance Manual established for the project.

2.5.4.5.4 Field Control of Foundation Treatment

In order to minimize the liquefaction susceptibility of the upper zone, the soils to a depth of 60 feet over the entire plant area for Units 1 and 2 were replaced with selected material and compacted to 85 percent relative density under controlled conditions.

To maintain this critical 85 percent relative density in the field a correlation was developed relating relative density to modified Proctor density, which could be directly measured in the field. Based on preliminary laboratory test results performed on typical in-situ soil samples from the plant area, 98 percent Modified Proctor was selected to conservatively yield the desired relative density. In analyzing the compacted backfill as a whole, a statistical approach was adopted based on Floe, written by R. A. Pettitt[34] and on a paper written by H. K. Cook[35].

This approach limited the in-place compacted backfill to a relative density of 85 percent with the maximum variation from this degree of compaction being one standard deviation less than 85 percent relative density.

Two standard deviations on each side of the mean were considered to comprise the entire set (or the entire number of samples). In this case, this criterion allows 16 percent of the samples or field tests to fall below 85 percent relative density which is a practical limitation in the field. A graphical statement of the above criterion is shown below in Figure 2.5-31(a).

The in-situ soils at the site were grouped into families based on Proctor density and grain size analysis. For each family typical samples were analyzed for maximum and minimum density determinations. From these tests, the field Proctor density required to yield 85 percent relative density was calculated from the following equation derived from an ASTM equation printed in the procedures for soil testing designation D2049-67 (Relative Density of Cohesionless Soils):

$$Dd = \frac{\gamma \max (\gamma - \gamma \min)}{\gamma(\gamma \max - \gamma \min)}$$

where

Dd = Relative Density Set at 85 percent.

γ max and γ min are determined by laboratory tests

γ = in-place field density required to yield 85 percent relative density.

Using the relationship stated above, curves were derived relating field density to relative density for each family. Entering these family curves with field densities, the corresponding relative densities were obtained. Using the relative density as the data to be studied, numerous statistical analyses were run.

The results of a typical study run in September of 1970 are shown in Figure 2.5-31(b).

Summary statistical analyses were made on all the Class I in-place density tests since the beginning of the job. The results of one of these studies are shown in Figure 2.5-31(c). It can be seen from Figures 2.5-31(b) and 2.5-31(c) that the amount of tests failing less than 85 percent relative density is much less than 16 percent. Therefore, the field compaction criterion based on modified Proctor density was shown to yield the quality backfill desired.

The results of all studies for Unit Number 1 and portions of Unit Number 2 are shown on Figure 2.5-31 (d).

Based on long term settlement criteria, compaction requirements and field workability, backfilling material was limited to no more than 12 percent silt content (finer than a number 200 sieve). To guarantee the fulfillment of both the compaction requirement and the silt content requirement, a soils laboratory was set up at the site as a control measure. The laboratory was placed under the jurisdiction of the Soils Engineer - Quality Compliance and run according to ASTM Procedures [36] . The soils laboratory performed all tests requested by the Construction Engineer - Soils, and any other tests deemed necessary by the Soils Engineer-Quality Compliance. All tests were located by the laboratory personnel, as directed by the Soils Engineer - Quality Compliance. The Soils Engineer Quality Compliance reviewed all field and laboratory tests and reported directly to the Ebasco New York office Chief Concrete-Hydraulic Engineer or his designated representative on all technical matters not covered in the specifications.

In addition to running all in-place field density tests (ASTM D-2167-63T), modified AASHO compaction tests (ASTM D-1557-64T) and grain size analyses (ASTM D422-63) the field soil laboratory was equipped to perform the additional tests according to the Applicable ASTM Procedure to aid in obtaining soil characteristics.

a) Material finer than a No. 200 sieve (Dll4O-54)

b) Over dry moisture content determination (D2216-63T)

c) Wet preparation for grain size analyses (D2217-63T)

d) Calibration of mechanical laboratory

e) Compactors (D2168-63T)

f) Minimum densities of granular soils by D. M. Burmister

g) Sand cone density method (Dl556-64)

h) Turbidity Test

A bimonthly laboratory equipment calibration program was set up as well as an independent comparison of test results to Ebasco Lab Test results on selectee samples to ensure the continuing quality of the Ebasco Testing Program.

Reporting and record keeping methods were established to ensure adequate documentation. Forms were developed for use in the field for recording calibration data as well as actual test data. A log of weather conditions, construction accomplishment, and other important factors was maintained by the Construction Engineer - Soils. The Soils Engineer - Quality Compliance retained all QC records of test and calibration results in the field. The carefully planned QC procedures and required documentation provide adequate assurance that the completed backfill operation resulted in foundation conditions which meet the requirements of the specification, Appendix 2B.

A minimum of one field density test was made for each 10,000 square feet of compacted fill material placed in each lift. A 50 foot grid system was established to locate and identify test samples and to provide reasonable confidence that a representative set of samples were obtained and tested for each lift. During the excavation and backfill cycle, it became evident that a good portion of the excavated material could not be used as backfill due to its fines content. To replace this unsuitable material, offsite sources of material were investigated, as well as other onsite or near site sources. Boring samples were analyzed as to percent of silt (finer than 200), cross sections were developed and quantities of usable backfill estimated. Economics in conjunction with availability, was the basis for selecting the dredge to evacuate the barge channel in Big Mud Creek as the major source of backfill material.

The governing criteria for the entire dredging operation was turbidity control. The turbidity of the discharging dredge water was limited to 50 Jackson units above the normal turbidity of the Indian River.

This was accomplished by allowing the dredge water to pond in several settling basins before it was routed back to the Indian River. By supplying sufficient retention time, enough settlement took place to keep the turbidity level of the returning water within the allowed 50 Jackson Unit rise.

To control the type of material pumped, borings of Big Mud Creek were used to locate pockets of silt and clay and these areas were avoided if possible. If not, they were pumped into a designated spoil area. In addition, all mud, silt, debris and other sediments on the bottom of the Creek were spoiled during an initial stripping cut. In this way, only usable sands were pumped into the storage basins.

As a final control, a wye valve was placed on the field end of the discharge dredge pipe and one line was run to a storage basin and the other to a spoil area. By visual inspection, a soils lab monitor directed the material to the appropriate area and thus compensated for any undetected pockets of silt, clay and any other undesirable material. For details

of the Big Mud Creek borings and laboratory tests see Borings 123 through 139, Appendix 2A.

Through the rigid control of the compaction program a structural backfill was obtained on which the plant foundations are supported directly on the compacted fill. This type of foundation was applied to all buildings and structures within the plant area except the switchyard structures and other non-critical structures on the periphery of the plant area.

As part of the St. Lucie Unit 2 Component Replacement Projects, Engineering Evaluation No. PSL-ENG-SECS-07-014 was performed to demonstrate the acceptability of using Controlled Low-Strength Material (CLSM) as Class I or lesser classification backfill material in restoring excavated areas. Use of CLSM is limited by PSL-ENG-SECS-07-014 to areas that do not serve as foundation support for any Seismic Category I structures or support for any Seismic Category I buried piping.

2.5.4.6 Groundwater Conditions

The history of groundwater fluctuations beneath the site is discussed in Section 2.4.13.

2.5.4.7 Response of Soil and Rock to Dynamic Loading

The response of the subsurface soils to dynamic loading is presented in Section 3.7. Dynamic soil properties applicable in the range of strains developed during strong motion earthquakes are presented in Section 2.5.4.4.

2.5.4.8 Liquefaction Potential

The geologic conditions underlying the site have been discussed in detail in Section 2.5.1. From discussions presented in Section 2.5.4.2, Properties of Underlying Materials, it is apparent that significant saturated sand deposits occur beneath the site. This section assesses the potential for liquefaction of the granular soils as a result of shear stresses induced by ground motion generated by the SSE.

The damage suffered by buildings at Niigata, Japan during the June 16, 1964 earthquake dramatized the need for evaluating the potential liquefaction of sand formations subject to seismic impulses. Although liquefaction has been observed or suspected in other earthquakes, the extensive studies of soil conditions at Niigata have made it possible to establish criteria for determining the possible liquefaction of similar sites underlain by sands.

The Niigata studies delineated areas of heavy, light and no damage.(27) Soil borings and laboratory tests in each area found that the degree of damage was related to the Standard Penetration Resistance and the percentage of fines (finer than a No. 200 sieve or 0.074 mm) of the underlying sands.

Frequency distributions of standard penetration resistance are presented in Figures 2.5-34 through 2.5-37. These figures show the relationship between the zones of no, light and heavy damage from the Niigata earthquake and the data obtained from the site. The curves show that the penetration resistances were greater at the site than those at Niigata for all three damage conditions to a depth of 10 meters feet). The curve of the material at the Hutchinson Island site is very similar to the curve for heavy and light damage at Niigata for the depth interval

from 10 to 15 meters or 33 to 48 ft. as shown in Figure 2.5-36. The only difference is the occurrence of some higher penetrations at the site. Figure 2.5-37 shows that there is nearly a uniform distribution of penetration resistances for the depth interval between 48 and 66 ft. or 15 and 20 meters. The penetration resistances in this zone are much greater than those at Niigata.

The Niigata penetration resistance frequency was compared to that at Hutchinson Island. The comparison indicates that liquefaction is possible between 33 to 48 feet below the ground surface at Hutchinson Island, if the ground were subjected to the accelerations of the Niigata earthquake.

Other empirical relationships have also been established from the Niigata earthquake. One of these is the grain size vs penetration resistance relationship. The liquefaction potential is influenced by the amount of fines present and the cohesion of the soil. As the amount of fines increases, the liquefaction potential decreases.

The percentage of material passing the Number 200 sieve (percent fines) was plotted as a function of the standard penetration resistance for the soils at Niigata. Grain size distribution curves are given in Appendix A. The data obtained from the Hutchinson Island site is superimposed on the Niigata data and presented in Figure 2.5-38 for the shallow zone. the shaded area indicate the critical combination of standard penetration values and the percent fines which resulted in liquefaction of the Niigata sands. Investigation data from Hutchinson Island indicates that the penetration resistance is too great for a given amount of fines or that the percentage of fines is too great for a given penetration resistance in all points except one. The one sample failing in the critical area is from a depth of 47 feet in Boring B-5. Several points are very close to the danger zone for liquefactions.

Figure 2.5-39 illustrates the same relationship for depths between 50 and 150 feet. This shows that all the data are further away from the POSSible area of liquefaction, using the Niigata criteria, than in the shallow zone, and the soil should not liquefy under earthquake loading.

Figure 2.5-40 presents two histograms showing the frequency of occurrence of the standard penetration resistance. This figure shows that the penetration resistances are much greater for a depth range of from 50 to 100 feet than for the first 50 feet. Since the penetration resistance is an index to the soil strength and density, it may be deduced from this diagram that the soil between 50 and 100 ft. below the surface is stronger than that soil found in the shallow zone.

Another approach to evaluating the liquefaction potential at the site is comparison of the shear stress resulting from an earthquake With the shear stress required to produce soil liquefaction. The shear stress was computed for different depths assuming an acceleration of 0.1g and the unit weights found from the laboratory tests. This is plotted on Figure 2.5-41 as the stress developed by the design basis earthquake.

The critical shear stress, required to produce liquefaction in cyclical loading was computed the relative density using a relation developed by Seed and Idriss[28] . The relative density was computed from the Standard Penetration Resistance using the method developed by Gibbs and Holtz[29] as well as a modification of the Terzaghi and Peck criteria[30]. In each case, the lowest value of the penetration resistance at each level for all borings below the plant was utilized in computing the relative density. Figure 7 of Reference 29 presents the Gibbs-Holtz curves and the modified Terzaghi-Peck curve. The original Terzaghi-Peck curve was modified by relating the descriptive terms of "very loose" to "very dense" to values of relative density. The Terzaghi-Peck data has always been referred to as modified Terzaghi-Peck data since it is recognized that the original data has been modified by several authors. The lower of the two relative densities found by the two methods was employed to compute the critical shear. These values are also shown on Figure 2.5-41.

The relative densities and the corresponding critical shear stresses were not computed below a depth of 60 ft. The relative density has little physical significance in the silty and clayey soils of the Intermediate and Deep Zones, and therefore any computation based on relative density would be invalid.

This plot shows that between about 25 ft and 45 ft the dynamic earthquake stress is greater than the shear stress required to produce liquefaction in the loosest materials. In fact the values of critical shear stress and available shear strength are comparable throughout the Shallow Zone.

Cyclical dynamic shear tests have been employed to establish the shear stress required to produce liquefaction of sands. Dynamic triaxial tests have been performed on selected samples to determine if their behavior is consistent with that of sands from previous laboratory investigations. The results of tests on soils in the Intermediate Zone confirm the conclusion that liquefaction will not occur under less than 10 cycles of a dynamic load.

A number of uncertainties are involved in such testing. First, in the laboratory tests, it is not possible to fully simulate the stress field and stress changes produced by earthquakes. Second, the number of cycles of dynamic loading likely in the design basis earthquake can only be estimated from past earthquakes in similar geologic regions. The absence of such records in the Eastern United States make it necessary to rely on records from California in which the geologic conditions are not comparable and generally much more severe. Finally, the laboratory tests cannot show the effect of continuing strain in cancelling an initial liquefaction.

Figure 2.5-42 relates grain size and standard penetration resistance for soils from the Intermediate Zone. This figure also shows that the upper and lower portions of the zone differ in character. The upper portion contains an average of 14 percent fines and the lower contains an average of 31 percent fines. Since the Niigata studies have established criteria

which indicate that materials containing low percentages of fines are potentially susceptible to liquefaction, only the upper portion of the Intermediate Zone need be considered. The data obtained from the Hutchinson Island site are superimposed on the Niigata data as shown in the shaded area. The shaded area indicates the critical combination of standard penetration values and the percentage fines which resulted in liquefaction of the Niigata sands. This investigation indicates that the penetration resistance is either too great for a given amount of fines or that percentage of fines is too great for a given penetration for all points except six. The six points within the shaded area are generally from elevations minus 70 to 85. From a percentage standpoint the six points comprise approximately 10 percent of the sampling shown on the figures; however, the laboratory grain-size analyses were selective and generally limited to samples with blow counts less than 30 blows per foot. If the entire 295 samples are considered, the percentage of critical combinations of standard penetrations and percent fines would be approximately 2 percent.

In order to further explore the susceptibility of the Hutchinson Island sands and particularly those samples which fall within the critical combination of penetration resistance and percent fines, a further comparison with the Niigata criteria was made. Studies of the Niigata sands have indicated a range of grain-size distributions which can be used as criteria for evaluating soils which are susceptible to liquefaction. Figure 2.5-43 is a grain size distribution chart indicating the band of susceptible soils within the shaded portion of Figure 2.5-42 and superimposed are the six soil samples considered to be critical with respect to blow count and percent fines. This figure illustrates that the sands considered do not conform to the range of susceptible Niigata sands and therefore, based upon this Niigata criteria, are not a potential liquefaction problem. Further laboratory tests and studies also confirm this conclusion and are presented below.

It was recognized in the early investigations that there were a few isolated points at which the penetration resistances were less than 15 blows per foot. In each of these cases the percentage of fines was greater than for the critical conditions established by the Niigata criteria. However, a major effort was directed to developing information to evaluate the occurrence, character and significance of these areas.

From the soil boring profiles presented in Figures 2.5-21 and 2.5-22, it was clearly revealed that these areas (12 to 20 blows per foot) or "soft zones" were completely random and isolated. This can be seen on the boring profiles by comparing the numbered borings with the suffixed "A" borings (i.e., compare 108 and 108A, 111 and 111A etc.). These additional borings were taken to better define the characteristics of the adjacent borings and to obtain additional samples; they were located approximately 3 to 100 feet from the adjacent boring.

In order to determine the liquefaction potential of the Intermediate Zone with these randomly located "soft zones" an analysis was performed with respect to material type and relative density from undisturbed

samples taken from the soft zone. The average relative density of 24 samples was determined to be 55 percent. This density compares very favorably with the relative densities calculated for the lowest 12 percent of the standard penetration resistances obtained. These penetration resistances were within the isolated soft zones described earlier. The results revealed that, even considering the loose condition to be predominant throUghoUt the Intermediate Zone, a margin of safety against liquefaction ranging from 1.7 to 2.4 are realized. Thus, from a standpoint of dynamic stability based on evaluation of liquefaction potential, the extent of the "soft zone" could exist throughout the plant site and still be acceptable. The detailed analysis from which the above results were obtained is now presented. The penetration resistances of these soft zones were converted to approximate relative densities using the method of Gibbs and Holtz[(29)]. The results are summarized below for the range of El -60 to El -150 feet.

PERCENTILE	RELATIVE DENSITY
Lowest 12 percent	55%
Lowest 20 percent	62%
Lowest 25 percent	68%
Median: 50 percent	82%

As a basis for design/ use of the median value is not conservative. most designers consider the lowest 20 percent or lowest quintile as the basis for a conservative design, which in this case corresponds to a relative density of 62 percent.

A number of undisturbed samples were obtained specifically in the loosest zones. These undisturbed samples were not considered representative of the entire mass but instead were intentionally Obtained in the very loose materials encountered. The average relative density of 24 samples was determines to be 55 percent, somewhat lower than the lowest quintile and equivalent to the lowest 12 percent.

Cyclic triaxial shear tests were run on those undisturbed samples of the loosest sands which showed no evidence of damage in the sampling operation. Because of the limited number of good undisturbed samples which could be shaped for laboratory testing, more tests were run on both denser samples, such as Boring B-20 at 137 ft. and reconstituted samples of the same materials at the same average void ratio (corresponding to a relative density of 55 percent) as the undisturbed samples. While it could be argued that the standard penetration tests in Boring B-20 at a depth of 137 are very high indicating a high relative density, the sample was selected as having representative grain size characteristics of the soils encountered at the site. This sample was tested both undisturbed and remolded. The undisturbed relative density was 65 percent to 83 percent and the remolded relative density was 68 percent to 81 percent. In any case the data was conservatively reduced to 55 percent for the purpose of the liquefaction analysis. The low standard penetration test data generally occurred in higher silt content material and we conservatively selected low silt content materials with D.50 sizes most likely to liquefy.

The dynamic test equipment and the procedures used were similar to those described by Seed and Lee[31].

In the older version of the cyclic triaxial machine the pneumatic device applies an approximate square shaped wave excitation to the test sample. Because of this, the peak load is not instantaneous but applied for a finite duration of time. The transition to the peak load is not smooth; in addition to this, the inertia of the complete system becomes significant and the load is applied in a non-uniform way. Since this sharp build-up and holding of load existed the best loading conditions were not met, and sample failure was premature when compared to a smoothly applied sinusoidal excitation.

The newer versions of the cyclic triaxial loading mechanism eliminates most of these limitations. The hydraulic system programmed applies a smoothly varying sinusoidal excitation where the peak load is gradually developed, held as an instantaneous peak and not applied for a finite time causing premature failure. In addition to the above conditions, the load pattern would be erratic and might begin at the desired deviator stress but, has in the past, due to equipment friction and inertia, been seen to vary up and even down during continued cycling. The load dropping off during continued cycling could lead to unconservative results at a high number of cycles of higher strain levels.

Data from Ebasco's experience with soils and recent cyclic triaxial tests were assembled and reviewed. Figure 81 shows the effect of equipment and procedures on cycles to produce initial liquefaction (i.e., Pore Pressure = Confining Pressure) and clearly shows the conservatism of older equipment and procedures. It is concluded that in spite of many variables coming into play in a liquefaction analysis, data conclusively prove the conservatism (lower stress ratio $\theta_{dp}/2\theta_a$ causing liquefaction of the older equipment and testing procedures. In fact, the increase in stress ratio at 10 cycles using today's techniques would be equal to 1.78 the ratio of $\frac{0.32}{0.18}$ (obtained from Figure No. 81)).

The soil sample was subjected to a uniform confining stress approximately equivalent to the overburden stress at the depth of concern. The sample was then subjected to cycles of axial loading. Tests were run at axial load magnitudes both larger and smaller than those that might be induced by earthquakes at the site. The cyclical or pulsating axial loads were maintained as constant as possible with a frequency of approximately one cycle per second (corresponding to the longer shear pulses of a strong motion earthquake) until the sample failed.

Failure generally occurred in two stages. The first indication of impending failure was an increase of the pore water pressure which momentarily equalled the confining stress on the soil. However, with the reversal of load the pore water pressure changed and the soil exhibited little change in resistance through the continuing load cycle. This "momentary liquefaction" is defined by the number of load cycles required to produce a momentary pore water pressure equal to the confining stress. This does not, however, mean that substantial permanent strains would occur in the soil.

In these tests, momentary liquefaction generally occurred at double amplitude strain levels between 0.5 percent and 3 percent, with one exception at 5 percent. These strain levels varied depending upon the relative density confining pressure and peak pulsating deviation stress.

With additional cycles of load, the duration of the high pore water pressure became progressively longer and the soil had little or no strength through most of the load cycle. The strain increased rapidly, until it became impossible to impose the full cyclic load on the sample.

At this point, the soil had lost its strength and liquefied. This point is defined as "liquefaction" or by some, "complete liquefaction". Generally, in these tests, this corresponded to strains of about 15 percent. This compares very well with the definition of "complete liquefaction" of 15 percent strain adopted by Seed and his co-workers[(28)] . The above definition of "momentary liquefaction" is more severe than Seed's definition of "initial liquefaction" at 5 percent strain.

From the test results, graphs were plotted depicting the shear stress at the time of both momentary and complete liquefaction as a function of the number of cycles of load required to produce liquefaction. Figure 2.5-44 shows the number of cycles required for momentary liquefaction; Figure 2.5-45 shows liquefaction or complete liquefaction. The test data, including the back pressure valves and "B" coefficient, is presented on Table 2.5-6.

The relative density was determined as follows.

A single maximum-minimum density determination was made on a composite sample. The composite sample was obtained by combining shelby tube samples obtained from boring B-106, 87 ft to 89 ft, and boring B-113, 87 ft to 89 ft and 97 ft to 99 ft. The maximum density of the sample was determined by the modified proctor test (ASTM D1557 - Method C), and the minimum density was determined by the dry funnel method such as described by ASTM 2049-64T (ASTM 2049-69). The results of the test are:

Maximum density	116.5 PCF
Minimum void ratio	0.500
Minimum density	88.9 PCF
Maximum void ratio	0.966

The weight-volume relationships were measured for each sample subjected to cyclic triaxial shear tests. The relative densities, therefore, are the relationship between maximum, minimum, and sample void ratio.

The scatter in the data is the result of using samples at different relative densities. Straight lines are shown on Figures 2.5-44 and 2.5-45 as conservative lower bounds of the data, with respect to a minimum relative density of 55 percent as discussed above. The data on page A-13 of the Unit No. 1 PSAR was originally replaced with Figures 2.5-44 and 2.5-45 after a review of the original data indicated errors in the testing and in the data on page A-13.

The results of the tests are expressed in terms of the cyclic deviator stress divided by the effective confining stress σ'. This is based on the findings of Seed et al[31] that the cyclic shear stress at failure is proportional to the effective confining stress. Therefore, tests run at one confining pressure can be used to analyze failures at other confining stresses. Seed has expressed the shear stress at failure, induced by 10 cycles of load by the expression: $\tau = \sigma' R_d/200$

In this expression:

τ = Shear stress for liquefaction in 10 cycles

σ' = Effective confining stress

R_d = Relative density

For cycles different than 10 the value of 200 would be replaced by a constant.

The shear stress is half the deviator stress so the expression could also be rewritten as follows for any given number of cycles:

$$\tau = \frac{\sigma d}{2} = \frac{\sigma' R_d}{\text{Constant}}$$

or

$$\frac{\sigma d}{\sigma'} = \frac{2 R_d}{\text{Constant}}$$

This expression also includes the relative density. For the intermediate ranges of relative density of more than about 30 percent and less than about 70 percent, the shear resistance increases in proportion to the relative density[31]. Therefore, tests run at one relative density can be converted to other relative densities by this procedure.

The above relationship using a relative density of 85 percent was used to develop the curve for the compacted fill in the liquefaction analysis shown on Figure 2.5-46. The results of actual laboratory cyclic triaxial tests used in the analysis are presented in Figure 2.5-55 and on Table 2.5-6.

The results of triaxial dynamic tests are somewhat overly optimistic in assessing the dynamic shear strength of soils[32] . Based on the results of simple shear tests and correlation with observed liquefaction of soils, Seed has suggested that the realistic dynamic shear strength of soils is 55 percent of the strength by triaxial testing.

Figure 2.5-47 shows the effective shearing resistance of the undisturbed soils with respect to liquefaction between El -60 and El -150 feet at the site.

The shear stress imposed by the design basis earthquake at any level can be found by the surface acceleration and the profile of acceleration versus

depth. This profile of the acceleration versus depth was developed during a comparative study of acceleration at Hutchinson Island and Green Cove Springs. In general, the profiles at Hutchinson Island show a decrease of acceleration with increasing depth. At a depth of 60 feet the acceleration is 0.86 times the surface acceleration. At a depth of 100 feet it is 0.79 times the surface accelerations. At a depth of 150 feet it is 0.78 times the surface acceleration.

The total overburden stress at any level is found from the weights of the soil in the overall area. These soil weights together with acceleration are utilized to calculate the dynamic shear stress at the appropriate level. These dynamic shear stresses are similarly plotted on Figure 2.5-47.

The margin of safety against liquefaction can be expressed by the ratio of dynamic shear resistance to the dynamic shear stress. The plots show that for complete liquefaction the ratio ranges from 2.4 at a depth of 60 feet to 1.7 at a depth of 150 feet. Such margins are considered more than adequate against liquefaction. It should be emphasized that these margins are based on test data on samples with 55 percent relative density, which corresponds to the lowest 12 percent of the samples. Moreover, an inspection of the boring records indicates that the possibility of liquefaction in the deeper portions of the zone between -60 and -150 is academic because of the scarcity of the looser zones within that range. Finally, experience of liquefaction in actual earthquakes suggests that liquefaction is confined to the upper soil layers and has never been observed deeper than 100 feet.

Figures 2.5-46 and 2.5-47 are not comparable as presented since the shear stress induced in the soil by the DBE for Figure 2.5-46 was calculated using the relationship $\tau = \frac{\gamma h}{g}$ a max and uses a datum of elevation +18.0

while Figure 2.5-47 uses a relationship of $\tau_{avg} = (.65)\frac{\gamma h}{g} a_{max} (Rd)$ and a datum of elevation +0.0. Each was prepared at different time frames in accordance with the state of the art and are not meant to be compared. Section 2.5.4.8.1 combines this data with depth using the equation

$$\tau_{avg} = (.65)\frac{\gamma h}{g} a_{max} (Rd)$$

In the above expressions:

τ = shear stress induced by earthquake

γ = the unit weight of the soil

h = depth

a_{max} = maximum ground surface acceleration

rd = depth stress reduction coefficient

Rd = depth stress reduction coefficient determined during comparative study

g = 32.2 ft/sec^2

0.65 = converts τ to τ_{avg}

Summary of Liquefaction Potential

a) Upper Zone

All of the data demonstrated that a part of the Upper Zone was potentially subject to liquefaction during earthquakes. The remedial treatment is described in Section 2.5.4.5.

b) Intermediate Zone

Within the Intermediate Zone there are a few isolated points at which the penetration resistances are less than 15 blows per foot. In each of these cases the percentage of fines is much greater than the critical conditions based on the Niigata criteria. A more detailed investigation was directed toward developing information to evaluate the occurrence, character and significance of this area. Based on this investigation and on the Niigata criteria, it was concluded that the soils in the Intermediate Zone between about 55 and 150 ft are not susceptible to liquefaction.

c) Deep Zone

The Deep Zone is clayey. Such materials are not susceptible to liquefaction.

2.5.4.8.1 Simplified Procedure for Evaluating Soil Liquefaction Potential - Applied to the St. Lucie Plant Site

To supplement the evaluation already presented and to be able to compare results from El +18 ft to -150 ft, the liquefaction potential was also analyzed on the basis of Seed and Idress' simplified method.[40]

The position of the water level is important in the analysis of liquefaction potential. Based on the information in Section 2.4, a water level of +2.0 has been used in the calculation presented. This is the mean high water level and is, therefore, a conservative water level to use in the liquefaction analysis. This is the highest water level expected at the site without assuming simultaneous improbable hydrologic conditions (e.g., due to hurricane or flood) with the DBE. However, any additional rise in water level due to such events would be a surface phenomenon, since the permeability of the soil would not noticeably raise the water level in the soil beneath the plant.

For this analysis, the stress ratio causing liquefaction was determined from cyclic loaded, dynamic triaxial tests conducted on representative samples of granular soils. Figures 6 and 7 from Reference 40 are reproduced and presented as Figure 2.5-48. Seed actually used data from the St. Lucie site. These points are indicated on the respective curves. In

Figure 2.5-48 the test results are presented in terms of 50 percent relative densities. The relative densities applicable to the St. Lucie site are tabulated below:

St. Lucie Soils

	Loosest Relative Density	Average Relative Density
Backfill	85	97
Natural Soils El. -60 to -100	55	85(+)
Natural Soils El. -100 to -150	55	68(*)

(+) Based on average standard penetration resistance, Figure 2.5-51.
(*) Based on average standard penetration resistance, Figure 2.5-52.

The data from Figure 2.5-48 must be adjusted with respect to relative density. The shear stress required to cause liquefaction is proportional to the relative density. This relationship is:

$$\frac{\sigma dp}{2\sigma_a(D_x)} = \left[\frac{\sigma dp}{2\sigma_a(D_{50})}\right] \times \left[\frac{x}{50}\right]$$

where:

$\dfrac{\sigma dp}{2\sigma_a(D_x)}$ = Stress ratio for sample at x relative density

$\left[\dfrac{\sigma dp}{2\sigma_a(D_{50})}\right]$ = Stress ratio for sample at 50 percent relative density

σdp = Cyclic deviator stress

σ_a = Triaxial confining pressure

Using the lowest value from each graph on Figure 2.5-48, very conservative stress ratios were selected. As the number of significant stress cycles resulting from the DBE (a_{max} = 0.1g) would be less than 10, ten cycles of strong motion were used as well as 30 cycles of strong motion, assumed to occur from a distant earthquake with a max = 0.05g. The stress ratio at various depths causing liquefaction for each case was computed from the following relationship:

$$\tau_1 = \frac{\sigma dp}{2\sigma_a}(\sigma_o')Cr$$

where:

τ_1 = In situ effective shear stress causing liquefaction

$\dfrac{\sigma dp}{2\sigma_a}$ = Stress ratio from test program causing liquefaction

σ_{dp} = Cyclic deviator stress

σ_a = Triaxial deviator stress

σ'_o = In situ effective overburden stress

Cr = A correction factor applied to laboratory triaxial test data to obtain stress conditions causing liquefaction in the field. The "Cr" correction accurately corrects for the effects from laboratory to field conditions by reducing the effective in situ shear strength. The dynamic stress distribution within the soil mass beneath and adjacent to the foundation structures is not different than the stress conditions under which triaxial testing or direct shear testing is usually carried out since the porous stones at the top and bottom of the laboratory samples simulate the foundation mat, and the sides of the direct shear apparatus or the confining membrane simulate rigid conditions similar to those adjacent to the foundation structures.

To assess the liquefaction potential of the site soils, the shear stresses causing liquefaction were compared to the average shear stresses induced by the DBE or the shear stresses induced by a distant earthquake causing a maximum acceleration at the site of 0.05g and 30 cycles of strong motion. Assuming that the site soil is a deformable body rather than a rigid body, the average shear stress induced in the soil was calculated by the following relationship:

$$\tau_{avg} = 0.65 \left[\frac{\gamma h}{g}\right][a_{max}][rd]$$

where:

τ_{avg} = average shear stress

γ = total unit weight of the soil (pcf)

h = depth (ft)

a_{max} = maximum ground acceleration

g = gravitational acceleration constant (32.2 ft/sec ^{2})

rd = stress reduction factor accounting for soil acting as a deformable body

0.65 = factor converting maximum shear stress to average shear stress

The average uniform shear stress, τ_{avg}, and the shear stress causing liquefaction, τ_1, shown on Figures 2.5-53 and 2.5-54 were compared and the safety factor against liquefaction was calculated by dividing the shear

stresses required to cause liquefaction by the shear stresses developed during the postulated seismic event. The minimum safety factor against liquefaction thus calculated assuming loose soil conditions as discussed in Section 2.5.4.8 was 2.16 for a_{max} = 0.1g and 10 cycles of strong motion. Also assuming loose soil conditions, a_{max} = 0.05g and 30 cycles of strong motion, the minimum safety factor against liquefaction was calculated to be 3.72. Also shown on Figures 2.5-53 and 2.5-54 are graphs comparing the data for average soil conditions as discussed above. This is a more realistic condition. The minimum safety factors from this data are 3.09 and 5.31. These high safety factors again demonstrate that liquefaction would not occur at the site under the postulated conditions.

It appears that the lowest safety factors occur at El -150 ft and that the safety factor might actually be lower than this if calculations were performed for deeper soil strata. This occurred because one of the conservative assumptions used in the calculations assumed a constant reduction factor, r_d, below a depth of 100 ft while actually it still decreases at a reduces rate. However, including this further reduction would increase the safety factors below 100 ft. In addition, no liquefaction has ever been observed at these depths and is precluded by the high confining pressures caused by the overburden soils.

2.5.4.9 Earthquake Design Basis

The selection of the earthquake design basis is presented in Section 2.5.2.10.

With respect to a paper regarding seismic criteria for nuclear power plants (Coulter, Waldron, Devine, "Seismic and Geologic Siting Considerations for Nuclear Facilities," 5th World Conference on Earthquake Engineering, Rome, 1973), these authors point out the subjective nature of intensity-acceleration relationships and provide one of the over 40 available relations. In order to apply the relationship between intensity and acceleration presented on Figure 1 in the Coulter, Waldron, Devine paper, the foundation conditions must be evaluated in accordance with general soil mechanics principles as well as dynamic or wave propagation considerations, that is, a conventional evaluation, in soils and foundation terms, must be made in order to place the site in one of the three categories, 1) below average, 2) average or 3) above average. Once the general category is determined it is a simple matter to obtain a ground acceleration.

Figure 2.5-33 presents the commonly accepted soil mechanic values for sand materials. The soil values presented in this Figure were compiled using data from the references listed. Relative to Figure 2.5-33 and using the values listed below:

Relative Density	85% above EL-60 62% below EL-60	lowest 20 percentile and conservative as discussed in sections 2.5.4.8 and 2.5.4.10
Angle of Internal Friction	40° above EL-60 34° below EL-60	conservative based on test values presented in Appendix A&W Vol 4 for the St. Lucie Unit 1 PSAR (entitled Hutch-inson Island Plant Unit 1 PSAR)
Standard Penetration	23.2 above EL-50 (50 + in plant area) 45.5 above EL-100 43.3 above EL-150	conservative as shown in Figs. 2.5-49, 50 and 51
Shear Modulus	$3x10^6$ psf	conservative as shown on Fig 2.5-32

It is obvious that the St. Lucie site fits into the middle range of the average zone. For an intensity VI MM, at the St. Lucie site, using the Coulter Waldron and Devine intensity - acceleration relationship, a value of 0.07 is obtained for the ground acceleration which is again below the minimum requirements which governed the selection of 0.1g. We consider the 0.1g maximum surface acceleration to be a very conservative figure, when compared with actual and seismic conditions in peninsular Florida.

2.5.4.10 Static Analyses

2.5.4.10.1 Bearing Capacity

The ultimate bearing capacities of the compacted backfill foundation as determined for the Class I structures by utilizing the Terzaghi and Peck Bearing Capacity Equations[30] . The soil parameters used were those determined by laboratory tests; an angle of internal friction of 40° with the compacted backfill at 85 percent relative density.

The ultimate bearing capacity for the containment structure is that of a circular foundation in a cohesionless soil and is calculated as follows[30]:

$$q_{ult.} = \gamma_1 D_f N_q + 0.6 \gamma_2 R N \sigma$$

Where: γ_1 = The effective unit weight of overburden soils (lbs/cu ft)

γ_2 = the effective unit weight of bearing soils (lbs/cu ft)

D_f = The depth of the foundation base below grade (feet)

$N_q N\gamma$ = bearing capacity factors dependent upon the angle of internal friction

R = the radius of the foundation (feet)

$$q_{ult} = (80.5 \text{ lb/ft}^3)(43 \text{ ft})(60) + 0.6(60 \text{ lb/ft}^3)(80.0 \text{ft})(90)$$

$$q_{ult} = 207{,}700 + 259{,}200 = 466{,}900 \text{ lb/ft}^2$$

This bearing value is extremely large and is that soil pressure necessary to cause a shear failure of this large diameter foundation resting on and embedded in, 43 feet of dense compacted soil. The utilization of soil pressure of even one-tenth of the above calculated value would yield intolerable settlements; however, the concept is presented to illustrate the fact that the utilization of allowable bearing values in the range of 10,000 to 12,000 lb/ft^2 are extremely conservative with respect to foundation bearing failure or instability. The criteria which govern the allowable bearing capacities were total and relative settlements among the various component plant structures.

The dynamic bearing capacity of the foundation materials was analyzed by subjected Class I structures to oscillations due to hurricane buffeting. This analysis considered the following: (1) the design hurricane winds of 130 mph sustained with gusting to 195 mph and reducing to 65 mph. (2) the frequency of one cycle every one to three minutes and a duration of 30 hours.

On the basis of the above criteria the imposed foundation level loadings

are as given in Figure 2.5-55. As can be seen, the sustained 130 mph wind imposes an additional 190 lb/ft^2 maximum edge pressure on the downwind side which is the result of determining the wind moment on the building from the sustained 130 mph wind and gusting; then using the

$$\sigma = \frac{P}{A} + \frac{Mc}{I}$$

equation

The gust to 195 mph raises the pressure to 420 lb/ft^2 and the reduction to 65 mph lowers the pressure to 50 lb/ft^2. For the analysis a deviation of $\pm$ 230 lb/ft^2 was utilized as the imposed foundation loading, and considered at a confining pressure of 3500 lb/ft^2 due to the vertical soil pressure at el -25 feet and adjacent to the shield building. The gusting frequency of one cycle/minute was conservatively considered since this yields 1800 total gust cycles over a 30-hour duration.

Four cyclic triaxial compression tests were conducted on materials used for plant area compacted backfill to demonstrate that the soil can withstand cyclic loading.

While it is true that diverse sources of Class I fill were used it is known that significant differences in soil strength properties would not be expected due to a slight change in average particle size (the D_{50} typically used to correlate materials) but rather is strongly dependent on the density. Since strict control was established on backfilling operations as can be seen from Figure 2.5-40D, which indicates the mean relative density to be 97 percent, no variation in dynamic soil properties can be expected at any one overburden pressure. These tests were conservatively run on samples compacted to only 85 percent relative density rather than 97 percent, since at that time sufficient actual field data was not available to justify a higher value and then subjected to various cycle deviation stresses at a constant confining pressure of 3500 lb/ft^2 . A plot of the results in terms of peak pulsating deviator stress and number of cycles to initial liquefaction is given in Figure 2.5-55. The tests were run at the normal one cycle/second frequency. This is conservative when used for a frequency of one cycle/minute. The curve appears to be asymptomatic to a deviator of 500 lb/ft^2 . There is also a line indicating the imposed stress level due to the wind gusting described above. Since this line is below the test curve it was concluded that the loadings developed in the foundation materials as a result of oscillations due to hurricane buffeting was not a problem regardless of the number of applied cycles or reasonable frequencies considered.

The 55 percent of the stress determined from the triaxial testing could be used to reduce the results as if the simple shear device was used, since this takes into account more correctly the dynamic stress distribution at the edges of the foundation. Since the line indicating the imposed stress level due to the wind gusting is still below the reduced test curve, it is similarly concluded that the loadings developed in the foundation materials as a result of oscillations due to hurricane buffeting, even considering a reduction in stress, is not a problem. If the cyclic triaxial tests were run at one cycle per minute, pore pressures could not build up (similar to static testing of granular materials) and

liquefaction could not occur. The whole purpose of the analysis as presented here is to indicate that even if ultra-conservative assumptions were used, foundation failure could not possibly occur even due to sustained hurricane buffeting.

Seed & Lee's, 1967 data Figure No. 6, for a sand with a relative density of 78 percent is plotted on Figure 2.5-55. As can be seen, the shape of the St. Lucie data is conservative and is probably asymptotic at a peak pulsating deviation stress higher than the specified value of 500 psf. Further conservatism can be shown on Figure 2.5-31D. The final statistical survey indicates a mean relative density of 97 percent and therefore the St. lucie curve as presented at 85 percent relative density should move upward and to the right. While it could be argued that a 20 ft head of flood induced pore pressures might reduce the asymptotic limit of 500 lb/ft^2, this event was not considered because:

a) due to the permeability of the controlled backfill the 20 ft use of flood buildup could not occur during the hurricane period.

b) the conservative assumption that the tests run at one cycle/sec are equivalent to one cycle/min which precludes the possibility of any damage from this event due to liquefaction.

Even if the site had earlier experienced an OBE, the data shown on Figure 2.5-55 would still be applicable under hurricane buffeting since

a) as discussed in Section 2.5.8 and 2.5.8.1 the site is safe against liquefaction due to seismic events

b) dynamic settlement is calculated to be less that 0.5 inches and is not a problem.

The overall dynamic stability of the compacted backfill was also explored with respect to safety factors.

Based on the cyclic triaxial compression tests mentioned above, it can be seen from Figure 2.5-55 that the pulsating deviator stress required to cause liquefaction in ten cycles is equal to 3,300 lb/ft^2 . The resistance in terms of shear stress is equal to one half the deviator stress of 1,650 lb/ft^2 .

The shear stress imposed by the design basis earthquake is a product of the ground acceleration and the total overburden pressure at the level in consideration. Since the surface acceleration is attenuated with depth; for simplicity the earthquake induced shear stress was conservatively calculated utilizing the surface acceleration of 0.1 g. Thus, the stress induced at the shield building foundation mat level is equal to 0.1 g times the total overburden stress at el -25 feet or 0.1 x 5215 lb/ft^2 = 521.5 lb/ft^2 .

The safety factor is defined as the ratio of available shearing resistance to the induced shearing stress and is equal to 3.2. This result and re-

sults of the liquefaction testing are in good agreement with the information presented in Figure 2.5-46.

If one considers the previously mentioned discussion relative to the utilization of 55 percent of the strength determined from triaxial testing and the 0.65 factor to correct maximum shear stress to the average peak shear stress, the 3.2 safety factor previously calculated is

$\frac{0.55}{0.65} \times 3.2 = 2.7$ and would be even higher if the acceleration to depth relationship were considered.

2.5.4.10.2 Settlement Analysis

A settlement analysis was performed for the critical plant structures to determine the anticipated relative vertical displacements for static conditions. As described in Section 2.5.4.5 and Figures 2.5-82 and 2.5-83 the excavation and backfill plan yielded surcharge pressures in the underlying deep strata which were comparable to the final loading conditions. The construction sequence of maintaining the groundwater table at El -60 feet during the backfilling operations and construction of the containment structure reduced the "seat of settlement" to within the dense sand strata above El 150 feet. The preloading also tended to more uniformly compress or "even-out" the variations in density; namely, reduce the effects of any possible soft zones with respect to static settlements. In addition the 50 feet of compacted backfill and dense insite sands between El-25 feet and El-75 feet bridged possible isolated soft zones and tend, once again, toward more uniform settlement characteristics.

For the purpose of conservative evaluation, an extensive loose zone extending from El-60 to El-100 was postulated. The lower limit of EL-100 feet was selected since very few indications of "soft zones" were obtained below El-100 feet. As a result an additional 0.5 inch of settlement was calculated. These settlements consider the net loadings in excess of the already imposed backfill surchange loads and the settlements would take place during the construction period.

Utilizing the laboratory triaxial and consolidation test data and considering only the additional loadings imposed by the building contents and superstructure to be the cause of the settlement, the following maximum settlements were calculated: (1) reactor structure -0.9 in.; (2) reactor auxiliary building -0.5 in. and (3) turbine pedestal mat -0.5 in. The above maximum settlements were calculated using the one-dimensional consolidation theory, as presented in Reference 30. The laboratory consolidation test data is presented in Appendices A and C for the St. Lucie Unit 1 PSAR (entitled Hutchinson Island Plant Unit 1 PSAR).

For this analysis, the structural foundation weights were considered to be equal to the weight of overburden adjacent to the structure at the time the backfill is completed to grade. The actual settlements resulted from the dead weight loading of structures and surcharging during construction. The longest calculated consolidation period was on the order of nine months to a year. This presented no problem since it was well within the construction period.

Table 2.5-1A provides a comparison of actual St. Lucie Unit 1 settlements with calculated settlements. A review of the settlement data indicates that settlement has essentially stopped. The actual settlements reflect the total settlement resulting from the designated structure as well as that contributed by the influence of adjacent structures.

Special cyclic triaxial compression tests were performed to assist in evaluating the effects of a dynamic load on the settlement characteristics of looser materials with the intermediate zone. The tests were performed on materials from the soft zones at 55 percent relative density and in the normal cyclic cell. The confining pressure was the calculated overburden pressure at El-80 feet and the pulsating deviator stress was that required to cause a pulsating shear stress equivalent to the design basis earthquake at El-80 feet. The tests were run at one cycle/second with strains and pore pressures being automatically recorded. After 15 cycles, the cyclic deviator was stopped and the sample was left to consolidate at the confining pressure for one minute, during which time the recording continued. After one minute, the pore pressure was relieved and the test completed by recording the final axial deformations and strains. The final strain was multiplied by various thicknesses of "soft zones" to obtain estimates of dynamic settlement. This information was utilized to compute the after earthquake settlements. A summary of this test data is presented in Table 2.5-1B.

The settlements were computed for various thickness of "soft zones" and the maximum additional settlement, as a result of a shock associated with the design basis earthquake for a 40 ft. thickness, is 0.5 inches. A 40 ft. thickness was selected as a conservative upper boundary of any possible "soft zone" as discussed above. This settlement would increase or decrease in proportion with the thickness of "soft zone" considered. Therefore, from the dynamic or "after-earthquake" settlement standpoint, the "soft zone" could exist throughout the plant site and still be acceptable.

Considering a localized "soft zone" or "pocket" of loose materials, roughly 20 feet by 20 feet and 5 to 10 feet in thickness, the bridging effect of the overlying dense formation will spread the loading and additional static and dynamic settlements will be negligible. This is believed to be reasonable and conservative based on the findings previously described.

Characterizing a hypothetical "soft zone", which would be critical from the standpoint of differential settlements, one must consider a 100 to 200 foot plan dimension, an extreme 20 foot thickness and located such that it would cause additional differentials between critical structures. The additional differential settlements would be one-half the values determined for the 40 foot thickness previously discussed. Therefore, additional differentials of 0.25 inch each would be experienced for static and dynamic settlements, which, even with these extreme assumed conditions, is the maximum calculated differential between adjacent critical structures and their electrical and mechanical interconnections.

2.5.4.11 Criteria and Design Methods

A comprehensive foundation investigation has beep performed at the site to develop detailed data pertaining to the selected soil foundation design based upon the results of the analyzed data.

A remedial foundation treatment program was established for the foundation of the reactor building, turbine building, reactor auxiliary building, fuel handling building, component cooling water pump and heat exchanger area, emergency diesel generator building, intake structure and certain other facilities.

The soils above elevation minus 60 feet were excavated and recompacted
to a relative density of 85 percent AASHO to provide satisfactory bearing, settlement and resistance to liquefaction characteristics. The existing grade around the unit at approximately elevation 0 was raised to elevation plus 18 with compacted fill. For details of excavation and backfilling procedures refer to excavation and Backfill Specification in Appendix 2B.

Plant structures are supported on reinforced concrete mat foundations ranging from elevation minus 29 to plus 15 ft. These supporting soils were also compacted to a relative density of 85 percent above elevation minus 60 ft.

The water table was originally above the existing grade. A dewatering system was installed and operates during the removal of the existing soils, during below grade construction of all concrete foundations and during the placing of compacted fill. The extent of the dewatering included all of the major structures and was accomplished through the use of 90 wells spaced approximately 50 ft. apart around the perimeter of the plant at elevation +4.0. The dewatering system.was designed to pull the water table down to elevation -65.0 mlw which required a pumping capacity of approximately 13,000 gpm. This elevation ensured that all construction operations were dry. A monitoring system was employed through the use of piezometers to indicate that the water table was maintained at the proper level. The water level was maintained below minus -60 ft throughout the construction period.

The final site grade is at elevation plus 18 ft. The ground water level was estimated to be the normal high water level in the Indian River at elevation plus 2 ft. The foundations of the reactor building and all other structures extending below the water table are designed to resist hydrostatic uplift pressures and lateral soil and hydrostatic pressures.

The reactor building foundation mat is located at elevation minus 25 ft. The reactor shield structure is cylindrical in shape and the outside diameter is 156 ft. The effective average dead load imposed on the supporting soils is approximately 8,500 lb per square ft. The effective average dead load is defined as the stress induced by the total dead load less the effect of the hydrostatic uplift due to the water table.

The turbine building is an outdoor type of building 260 feet by 130 feet in plan. The bottom of the foundation mat is elevation 0. The effective average dead load imposed on the supporting soils is 5,000 lb per square foot.

The excavation and backfill plan as outlined in Figure 2.5-29 yielded surcharge pressures to the underlying deep strata which are comparable to the final loading conditions. The construction sequence has consolidated the entire area and reduced if not eliminated any measurable post construction settlements. The final stress conditions at the minus 150 feet elevation are shown as Figure 2.5-30.

In order to study the effect of soil-structure interaction, seismic analyses have been performed utilizing a finite element approach for a plant nuclear island consisting of a reactor building, an auxiliary building and a fuel handling building similar to those at St. Lucie with comparable soil properties and building embedments (see Figures 2.5-90 and 2.5-91 for a comparison of plant layouts).

The shear modulus and damping values for this generic study were similar to those values at St. Lucie (see Figure 2.5-89 for a comparison of shear modulus). The soil depth considered was 600 feet which is the same as the soil depth of 600 feet which should be considered at St. Lucie, in accordance with prior discussions in Section 2.5.2.3. The maximum acceleration considered was 0.13g inputed at 600 feet (St. Lucie design is 0.1g), since some data was available for this study and would not have to be repeated.

The results of the finite element analysis using todays current methods which included the effects of both the soil-structure and building to building interactions were obtained. Floor spectra at foundation level of all three buildings in both N-S and E-W directions with different damping values were plotted and are shown in Figures 58, 60, 62, 64, 66, 68, 70, 72, 74, 76, 78, and 80.

For the purpose of demonstrating the adequacy of the seismic analysis and the conservatism of structural seismic design at St. Lucie the nuclear plant buildings as mentioned above were re-analyzed utilizing an identical method as used at St. Lucie plant; namely, lumped-mass cantilever model, as discussed in Section 3.7, with rocking and translational springs representing soil-structure interactions. Floor spectra at the foundation levels were also plotted and are shown as Figures 13 through 24 and 57, 59, 61, 63, 65, 67, 69, 71, 73, 75, 77, and 79. In comparing foundation level spectra, it was discovered that in all twelve cases studied the spectra based on the method used at St. Lucie generally envelop spectra based on the finite element method except at the very low frequency end (less than 1 Hertz). This lack of conservatism at the very low frequency range should not be considered significant since it is way out of range of interest of nuclear power plant structural and equipment design. To summarize, it is concluded that although the method of seismic analysis used at St. Lucie did not precisely represent the soil-structure on building to building interactions, the proper engineering judgment in selecting conservative structural parameters, along with the analysis approach, assured a reasonable and conservative analysis result as demonstrated by the comparison of

floor spectra. For all twelve cases studied, only in two cases (reactor building, N-S direction and fuel handling building, N-S direction) does the finite element method result in higher peaks at about 4~5 Herz. This peak response again is away from the general interest of piping or equipment design frequency which is normally at 7 Herz and higher.

2.5.4.12 Techniques to Improve subsurface Conditions.

Subsurface conditions at the site are such that special treatments such as grouting, vibroflotation, etc. are not required. Section 2.5.4.5 presented plans and profiles showing the extent of excavation and backfill planned at the site, which is the only foundation treatment necessary. Detailed specifications for excavation and backfill work is presented in Appendix 2B.

2.5.5 SLOPE STABILITY

The stability of the slopes that effect the plant are the intake and discharge canal side slopes, the emergency cooling water canal slopes and the slopes of Big Mud Creek. Since the slopes were originally protected by grass, and portions were later modified with gravel and articulating concrete block which have been proved to be an effective measure of erosion protection, the effect of erosion during plant life has been neglected in the subsequent analyses. In any event the intake canal is sufficiently over excavated such that if erosion of the grassed slopes was assumed, it still would not result in a canal cross section less than the design cross section required for emergency cooling water. 100 ft interval cross sections of the completed parts of the intake canal indicate this over excavation.

The analysis of slope stability is divided between those which are designed not to liquefy and those which may have a potential to liquefy.

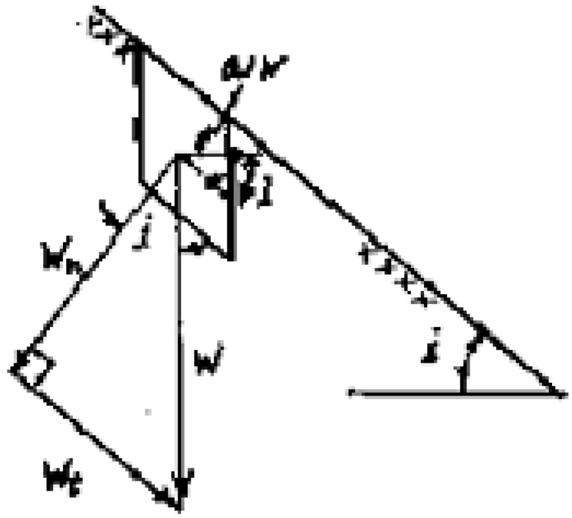
The portion of the intake channel immediately in front of the intake structures will involve approximately one hundred feet of channel within the plant area compacted backfill. Laboratory tests have been performed upon the proposed backfill materials at various relative densities to determine the angle of internal friction.

These tests show that an angle of 42° can be realized at 85 percent relative density. This measured angle was conservatively reduced to 40° for the analysis. Further conservatism is shown since the soils were actually compacted to a mean relative density of 97.01 percent, and if the tests were performed at this relative density an angle of internal friction greater than 42 degrees would have been obtained. Additionally, if the angle of internal friction had been based on the dynamic simple-shear device test, it would have been higher than in the static case. Thus, the effect of earthquake cyclic loading actually increases the angle of internal friction.

The soil data testing report and boring logs for Hutchinson Island are presented in Table 2.5-6 and in Appendix 2A at the end of this section, and in Appendices A and C in the Unit 1 PSAR (excluding Table A-11).

For a uniform slope in a cohesionless soil, the minimum factor of safety is obtained when failure parallel to the slope is considered. This is the

limiting case of a "slip circle analysis" utilizing an infinite radius. Thus, considering an infinite slope (which is conservative-since end
conditions are neglected) and analyzing a block of soil within the slope, the following conditions exist:



With:

W	=	the weight of the block of soil being considered
i	=	the slope angle
W_n	=	the component of the weight of soil normal to the sliding surface, equal to (W cos i)
W_t	=	the component of the weight of soil tangent to the sliding surface, equal to (W sin i)

The factor of safety for a static slope ($F.S._{(ST)}$) is defined as a ratio
of the forces resisting sliding to the forces driving the sliding, namely:

$$F.S._{(ST)} = \frac{\text{ResistingForces}}{\text{DrivingForces}} = \frac{W_n \tan\phi}{W_t}$$

where ϕ is equal to the angle of internal friction of the sloping material, in our case compacted sand is equal to 40° .By substituting the relationships described above we have for a 3:1 slope (which has been confined at the St. Lucie site by a subsequent contour survey)

$$F.S._{(ST)} = \frac{W_n \tan\phi}{W_t} = \frac{(W \cos i) \tan \phi}{(W \sin i)} = \frac{\tan \phi}{\tan i}$$

$$F.S._{(ST)} = \frac{\tan \phi}{\tan i} = \frac{\tan 40°}{\tan 18.5°} = \frac{0.840}{0.333} = \underline{\underline{2.52}}$$

Considering the slope during the earthquake condition and utilizing the design bases earthquake of 0.1 g, a relationship is obtained which considers the horizontal earthquake force on the sliding block of soil. Once again the safety factor during an earthquake (F.S.$_{(EQ)}$) is determined by a ratio of resisting forces to driving forces: thus:

$$F.S._{(EQ)} = \frac{\left(W\cos i - 0.1\,W\sin i\right)\tan\phi}{\left(W\sin i - 0.1\,W\cos i\right)} = \frac{\left(\cos i - 0.1\sin i\right)\tan\phi}{\left(\sin i + 0.1\cos i\right)}$$

By considering the compacted 3:1 slope and $\phi = 40°$ degrees we have

$$F.S._{(EQ)} = \frac{(\cos 18.5° - 0.1\sin 18.5°)}{(\sin 18.5° - 0.1\cos 18.5°)}\tan 40° = \frac{0.949-0.032}{0.316+0.095}(0.840) = \underline{\underline{1.87}}$$

Since the above safety-factors are much greater than unity, the channel side slopes for the portion in the compacted backfill are safe from sliding during both static and earthquake conditions. The remainder of the channels run through original ground which comprised the upper zone as described in Section 2.5.4.8. This zone was considered to be potentially susceptible to liquefaction and plant design is based on the assumption that this soil does in fact liquefy after a design basis accident.

The method of analysis of the canals and the Big Mud Creek portion of the channel is one of determining the final cross-section after a shock has liquified the channel side-slopes.

This analysis conservatively assumes that the channel side slopes liquefies. The degree of conservatism of this basic assumption should be realized, since, as discussed below, under this extreme assumption emergency cooling water can still be maintained.

The parameters utilized in this approach are: (1) the final liquefied slope conditions; (2) the balancing of displaced soils during the earthquake shock; and (3) the resulting available channel cross-section. The resultant after earthquake slope utilized in this analysis is twenty horizontal to one vertical (20:1). This slope is based on reports of resultant earthquake slopes reported during the recent Alaskan earthquake in areas where sand slopes or clay slopes with sand seams were considered to have liquefied (37, 38).

Based on the Alaskan earthquake where slopes after liquefaction experienced values of (8:1) to (10:1), (a detailed search of available literature does not indicate any flatter conditions), a conservative value of (20:1) was selected for the St. Lucie site. The data from the Alaskan earthquake is applicable since the soils there also consisted of sands.

These parameters have been applied to the sections shown on Figure 2.5-56. In cross-section A-A, which is a typical section across the Emergency Cooling Water Canal, the liquefaction slope is 1:20; balancing "cuts and fills" establishes the level of the 1:20 slope and results in a low point at the middle of the canal at elevation - 12.0. The "cut" material, or that material which will slide into the canal, is the area above the 1:20 slope line

and below the original ground line at +2.0. The direction of flow for the vertical cross section when Unit #1 is operating prior to Unit #2 construction is shown on Figure 2.5-87. The direction of flow for both units operating is shown in the Unit 2 PSAR Figure 2.5-87. The "fill" material, for the vertical cross section when Unit #1 is operating prior to Unit #2 construction, or the material which has slid into a previously open canal, is the area below the 1:20 slope line and above the original canal bottom. To assume that the 1:10 slope line is at the proper elevation, the total cut area must equal the total fill area. If the 1:20 slope line is drawn too low, an excess of cut area will result. This is corrected by raising the 1:20 slope line to a level where cut equals fill.

In Section B-B, the same procedure is used to establish a liquefaction slope with a low point elevation of -6 feet. From an examination of these two cross-sections, it is evident that the liquefaction will occur longitudinally along the canal, that is, material from the Section B-B will flow toward the area represented by A-A. Thus, a redistribution of slopes is made to account for this occurrence. The low point of slope becomes elevations -12 and -10 for Sections A-A, and B-B, respectively.

This remaining area is many times the area required to pass 130 cfs, the required emergency cooling water flow.

Erosion due to wave action associated with the PMH was discussed in Section 2.4.5.4. Referring to Figure 1.2-1, it can be seen that the critical Class I slopes forming the plant island are protected by Class II slopes sufficiently to protect the underlying Class I slope from erosion. Because the liquefaction analysis is based on actual measurements of failed slopes, the effect of seismically-induced water waves is implicitly included. By virtue of the 2 to 1 safety factor used (measured slopes of 8:1 and 10:1 versus-the design value of 20:1) and the fact that all Class I slopes are protected from wave action by either a concrete retaining wall (as in the case of the intake area) or concrete slope paving (both at the intake and discharge seal well), the conclusion that safety-related slopes cannot be eroded by wave action of any kind is justified.

The effect of liquefaction on the amount of sediment carried into the intake structure and consequently to the pumps has been examined.

Sediment in channels may be transported either by rolling or sliding along the bed (bed load) by bouncing along the bed (saltation load), or in suspension in turbulently moving water, (suspended load). In addition there will usually be a wash load consisting of very fine particles carried into the channel with no relation to the bed material. For our purposes the bed load and suspended load are of major importance, and the others may be neglected, since they represent a very small percentage of the particle movement.

The rate of bed load was determined using a modification by L.G. Straub of the DuBoys formula presented in "Applied Hydraulics in Engineering," by H. Morris. A second formula presented by Morris and developed by Kalinske was used to confirm the results obtained with the modified DuBoys formula.

Essentially it was reasoned that the amount of material moving into the intake structure would depend on the amount of material moving in the intake canal. A cross-section presented in the PSAR, and representative of the canal after liquefaction was used to determine velocity. Sieve analysis data representative of the top 30 feet of onsite materials was used to establish the particle sizes to be used in calculation. The D_{50} was used in the DuBoys equation while the particle size used in the Kalinske equation was dependent on the velocity in the intake canal.

These equations were applied to a case where only the intake cooling water pumps are operating since the circulating water pumps would clog and automatically trip off immediately after the assumed liquefaction. For such a case it was found that the bed load was negligible; basically no materials are moved at the very low velocities associated with the intake cooling water pump operation.

By assuming the hypothetical condition of sand size in suspension and by formulas used in design of sedimentation basins based on Stoke's Law, the size particle which could be expected to reach the cooling water pumps was determined. It was further assumed that no particles would be placed in suspension after passing the concrete lined forebay which lies before the intake structure.

Once liquefaction has occurred, particles with grain size less than 0.075 mm can be expected to reach the cooling water pumps: This corresponds to the suspended particle-size encountered during normal operation. It must be noted that the pumps can pump particle sizes up to 1/2 inch.

The effect of the particle build up has been taken into account and it has still been determined as shown on Figure No. 2.5-88 that the cooling water pump bells are not covered and sufficient emergency cooling water can be pumped.

The above analysis always using conservative assumptions, indicates that the required flow for emergency shutdown can be supplied. It should be remembered that it is not expected that the SSE will cause this condition to occur at all since the SSE for St. Lucie is less than that which causes extensive slope failures and is of much shorter duration. In any case there are three reasons (37) that if a slide caused by soil liquefaction was assumed it could not be as extensive as postulated above since (1) a liquefied zone would not extend all the way to a free surface at St. Lucie so that sliding occurs partly through liquefied soil and partly through non-liquefied soil and the strength of the non-liquefied soil would be sufficient to prevent or reduce sliding, (2) a medium dense cohesionless soil such as the soil in the slopes at St. Lucie, may exhibit liquefaction over a small deformation range but stiffen rapidly due to a tendency for dilation and an accompanying reduction in pore water pressures if severe deformations develop, and (3) liquefaction can only persist as long as high pore water pressures persist in a soil and at St. Lucie the coarser clean sands in the slopes would permit rapid drainage so that liquefaction would persist for such a short period of time that large displacements are unable to develop.

The preceeding discussion of slope stability is in conformance with design criteria for the St. Lucie site as specified during the construction permit review for St. Lucie Unit 1. On October 21, 1974, the Regulatory Staff requested:

1) A soils exploration and sampling program to define critical soil conditions that could conceivably affect delivery of water to the ICW pumps,
2) a testing program to establish the characteristics of these materials, and
3) appropriate stability and/or deformation analyses to quantitatively predict the behavior of slopes critical to the ultimate heat sink function.

The program requested by the Staff has been completed, and a detailed discussion is provided as Appendix 2G. The results of this study reaffirm the conclusions reached by the earlier studies discussed above and demonstrate the conservatism in the methodology. Specifically, the recent study demonstrates that the stability of slopes is acceptable. The DBE will not cause massive sliding of materials. It could cause local straining or sloughing of one to two feet along the submerged canal slopes and bottom, a condition much less severe than the massive flows resulting in final slope surfaces of 20:1 assumed above.

Class I slopes and other Class I earthwork will be constructed in accordance with the Foundation Excavation and Backfill Specification included as Appendix 2B. The excavation and backfill for Class I material shall be within the confines of Class I material already in place for Unit 1. No new material is involved in the Class I excavation and backfill work.

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TABLE 2.5-1

HAWTHORNE CLAY

X-RAY DIFFRACTION ANALYSES

Total Sample (Per Cent)

Constituents	B-1 271 Feet	B-1 285 Feet	B-1 310 Feet	B-1 365 Feet
Quartz	65	30	45	20
Feldspar	10	15	8	7
Calcite	-	15	24	-
Dolomite	20	20	23	55
Opal	-	5	-	-
Phosphorite	-	-	-	5
Clay	5	15	Trace	13

Clay Fraction (Per Cent)

Constituents	B-1 271 Feet	B-1 285 Feet	B-1 365 Feet
Illite	30	30	25
Montmorillonite	30	35	30
Kaolinite	25	30	25
Chlorite	15	5	-
Attapulgite	-	-	20

TABLE 2.5-1A

ST. LUCIE UNIT NO. 1 SETTLEMENTS

STRUCTURE	CALCULATED	ACTUAL
Reactor Building	0.9 in.	1.1 in.
Reactor Auxiliary Building	0.5 in.	0.5 in.
Turbine Pedestal Mat	1.0 in.	1.32 in.
Intake Structure	0.0 in.	0.0 in.

TABLE 2.5-1B

SUMMARY OF DYNAMIC SETTLEMENT TEST RESULTS

1. Sample was combined and remolded from undisturbed samples obtained from the Intermediate Zone

2. Soil Description
   a) gray silty, fine, gravelly coarse to fine sand (fine gravel and sand sizes are shell fragments and sand)
   b) D_{50} = 0.11 mm
   c) % Passing #200 Sieve = 17

3. Dry Density = 98 lb/ft^3

4. Moisture Content = 26%

5. Relative Density = 55%

6. Confining Pressure = 6760 psf

7. Cyclic Stress Ratio = 0.181

8. Final Axial Strain = 0.1%

TABLE 2.5-2

PENETRATION RESISTANCE AND PERCENT FINES
FOR BORINGS B-4,5,6,15,19,20

Boring No.	Depth (Feet)	Standard Penetration Resistance	% < #200 Sieve
B-4	60	34	13
	65	81	12
	70	42	13
	75	30	8
	80	100+	10
	85	41	7
	90	57	12
	95	95	30
	100	97	25
B-5	5	23	6
	7	22	4
	10	21	1
	15	35	1
	20	19	2
	23	18	2
	30	14	29
	36	11	63
	47	10	3
	70	-	10
	125	-	23
	128	-	10
	130	-	6
	135	50/3"	37
	140	40	33
	145	-	11
B-6	30	4	76
	50	65	27
	55	57	8
	60	37	10
	65	20	16
	70	54	23
	75	59	15
	80	-	7
	90	61	7
	95	105	25
	100	108	23
B-15	35	20	0
	50	64	2
	55	41	4
	60	41	13
	65	60	17

TABLE 2.5-2 (Cont'd)

Boring No.	Depth (Feet)	Standard Penetration Resistance	% < #200 Sieve
	70	64	12
	76	45	18
	80	86	13
	85	100	13
	90	83	23
	95	75	43
	100	81	45
B-19	70	40	17
	75	12	22
	80	19	28
B-20	70	89	18
	75	16	17
	80	62	26

TABLE 2.5-3

CYCLIC SHEAR TEST DATA

	psi $\sigma d\pm$	$x\ 10^{-5}$ IN/IN $\varepsilon\ \pm$	E psi
SERIES I	0.60	1.32	45,000
	1.72	4.06	42,300
	2.76	6.70	41,400
	4.41	11.45	38,400
	5.70	17.40	32,700
	6.30	23.20	27,300
SERIES II	0.38		
	(180)		
	1.26	2.66	47,400
	3.32	7.10	46,800
	4.41	11.55	38,100
	7.71	26.65	28,800
SERIES III	6.79	18.6	36,600
	9.15	32.3	28,500
	(1,700)		
	11.98	40.4	29,700

TABLE 2.5-4

SHEAR STRENGTH SUMMARY

BORING NUMBER	ELEVATION (FEET)	BY TORVANE (WITHOUT CONFINEMENT)	SHEAR STRENGTH (PSF) BY TRIAXIAL SHEAR TEST	BY WEIGHT OF HAMMER AND ROD	MATERIAL DESCRIPTION
B-1	141		2800		Clayey Silty Fine Sand
	230			6940	Silty Clayey Very Fine Sand
	245			7310	Silty Clayey Very Fine Sand
	250	700		3090	Clayey Very Fine Sandy Silt
	255	1800		3140	Clayey Very Fine Sandy Silt
	260	800		3200	Clayey Very Fine Sandy Silt
	265	1400		3250	Clayey Very Fine Sandy Silt
	270	1900		3300	Clayey Very Fine Sandy Silt
	275	1600		3350	Clayey Very Fine Sandy Silt
	280	1900+			Clayey Very Fine Sandy Silt
	285	1800			Very Fine Sandy Clayey Silt
	290	1000			Very Fine Sandy Clayey Silt
	295	1900+			Very Fine Sandy Clayey Silt
	300	1800		5100	Clayey Silty Very Fine Sand
	305			5180	Clayey Silty Very Fine Sand
	325			9310	Fine Micaceous Sandy Clayey Silt
	340			5690	Very Fine Sandy Clayey Silt
	345			9810	Very Fine Sandy Clayey Silt
	350			9930	Very Fine Sandy Clayey Silt
	355			10050	Very Fine Sandy Clayey Silt
	360			5980	Clayey Very Fine Sandy Silt
	365			6060	Clayey Very Fine Sandy Silt
	380			10680	Very Fine Sandy Clayey Silt
	385			10800	Very Fine Sandy Clayey Silt
B-3	158		6500		Slightly Silty Fine Sand
B-5	25				Fine Sand With Shells
	37.5		1300		Slightly Fine Sandy Silty Clay
	218			6560	Slightly Fine Sandy Silty Clay
	223			6680	Slightly Fine Sandy Silty Clay
	228			2830	Slightly Fine Sandy Silty Clay
	233			2890	Slightly Fine Sandy Silty Clay
B-6	30		1400		Slightly Fine Sandy Silt
B-8	175		6000		Slightly Fine Sand

TABLE 2.5-5

REGIONAL EARTHQUAKE SUMMARY
(HISTORIC EARTHQUAKES FELT IN PENINSULAR FLORIDA)

*October 29, 1927: Several secondary sources report a severe earthquake in St. Augustine on this date but the original record has not yet been located. New England suffered a severe shock about 10:40 A.M. on this date, and an earthquake was reported from Martinique on the same day[1].

Distance of epicenter from site: 190 miles
Epicentral Intensity: VI MM estimated

February 8, 1843: Great earthquake centering at Guadeloupe, West Indies, N. Latitude 16, W. Longitude 62, which did considerable damage and which appears to have been felt in the eastern part of the United States especially at Washington, D.C.[2].

Distance of epicenter from site: 1500 miles
Epicentral Intensity: Unknown

*January 12, 1879: A general earthquake felt through north and central Florida from a line drawn from Punta Rassa to Daytona on the south, to one drawn from Tallahassee to Savannah on the north, an area of about 25,000 square miles[1]. Epicentral location at N. Latitude 29.5, 14. Longitude 82.0[2].

Distance of epicenter from site: 200 miles
Epicentral Intensity: VI MM[2]

January 22, 1880: Severe shocks (intensity VIII MM) were felt in Key West reflecting a disastrous earthquake in Vuelto Abajo, west of Havanna. This quake was felt and rumblings heard in Western Cuba and on the Isle of Pines. It covered about 65,000 square miles[1]. Epicentral location at N. Latitude 22.8, W. Longitude 80.8[2].

Distance of epicenter from site: 300 miles
Epicentral Intensity: VIII MM[2]

August 31, 1886: This is the date of the great earthquake at Charleston, S. C., N. Latitude 32.9, W. Longitude 80.0. It was felt all over northern Florida, church bells rang in St. Augustine, and severe shocks were felt along that part of the east coast. Apparently this quake had an intensity of V MM in Florida[1].

Distance of epicenter from site: 380 miles
Epicentral Intensity: IX MM[3]

September 1, 3, 5, 8, and 9, 1886: Jacksonville experienced more tremors of about intensity IV MM from the Charleston earthquake[1].

Distance of epicenter from site: 380 miles

TABLE 2.5-5 (Cont'd)

October 22, 1886: Strong shocks at Charleston; the first in morning felt with intensity VI MM in Charleston, Atlanta, Augusta, and elsewhere, and second with intensity VII MM at Sumerville, S. C. Felt at Washington Richmond, Louisville, Fayetteville, Jacksonville, and elsewhere[2]. Felt at intensity V MM in Jacksonville[1].

Distance of epicenter from site: 380 miles

November 5, 1886: Another shock centered at Charleston, felt over the same area as second shock of October 22[2].

Distance of epicenter from site: 380 miles

*June 20, 1893: At 10:07 P M., Jacksonville experienced a slight shock lasting about 10 seconds[1].

Distance of epicenter from site: 230 miles Epicentral Intensity: iv MM(l)

*October 31, 1900: Jacksonville, Florida, N. Latitude 30.4, W. Longitude 81.7. Felt with intensity V MM. Eight distinct shocks felt. No damage[2]

Distance of epicenter from site: 230 miles
Epicentral Intensity: V MM[2]

January 23, 1903: Felt at Tybee Island, Savannah, Georgia N. Latitude 32.1, W. Longitude 81.1, with intensity VI MM; Charleston, S. C., IV-V MM; Columbia and Augusta, Ga., III-IV MM. Houses strongly shaken[2]

Distance of epicenter from site: 320 miles
Epicentral Intensity: VI MM[2]

June 12, 1912: This shock centered at Summerville, S. C., N. Latitude 32.9, W. Longitude 80.0, with intensity VII MM and was felt at Wilmington, N. C. Chimneys shaken down at Summerville[2]

Distance of epicenter from site: 380 miles
Epicentral Intensity: VII MM[2]

June 20, 1912: A shock felt strongly at Savannah, Georgia, N. Latitude 32.0, W. Longitude 81[2].

Distance of epicenter from site: 320 miles
Epicentral Intensity: V MM[2]

*1930: A shock felt in Everglades, La Belle, and Ft. Myers. Its seismic origin has been questioned and blasting given as the cause. There is also a report of windows and dishes rattling at Marco Island about this same time. The probable intensity at Marco was V MM[1].

Distance of epicenter from site: 130 miles estimated

TABLE 2.5-5 (Cont'd)

*November 13, 1935: Two short tremors were felt at Palatka the first at 10:10 P.M., and the second lasting 15 seconds, at 10:30. The second shock was felt at St. Augustine and on nearby Anastasia Island, but apparently the disturbance did not extend far as other cities on the coast reported that they had no disturbances[1].

Distance of epicenter from site: 185 miles
Epicentral Intensity: IV MM - V MM[1]

*January 19, 1942: Several shocks occurring near Lake Okeechobee. Tremors also felt at Miami, Everglades and Ft. Myers[1].

Distance of epicenter from site: 100 miles estimated
Epicentral Intensity: IV MM estimated

*November 27, 1973: A shock centered at N. Latitude 28.7, W. Longitude 81.0[4].

Distance of epicenter from site: 115 miles
Epicentral Intensity: V MM[4]

*Earthquake epicenter located in peninsular Florida. Epicentral locations for these events are shown on Figure 2.5-18.

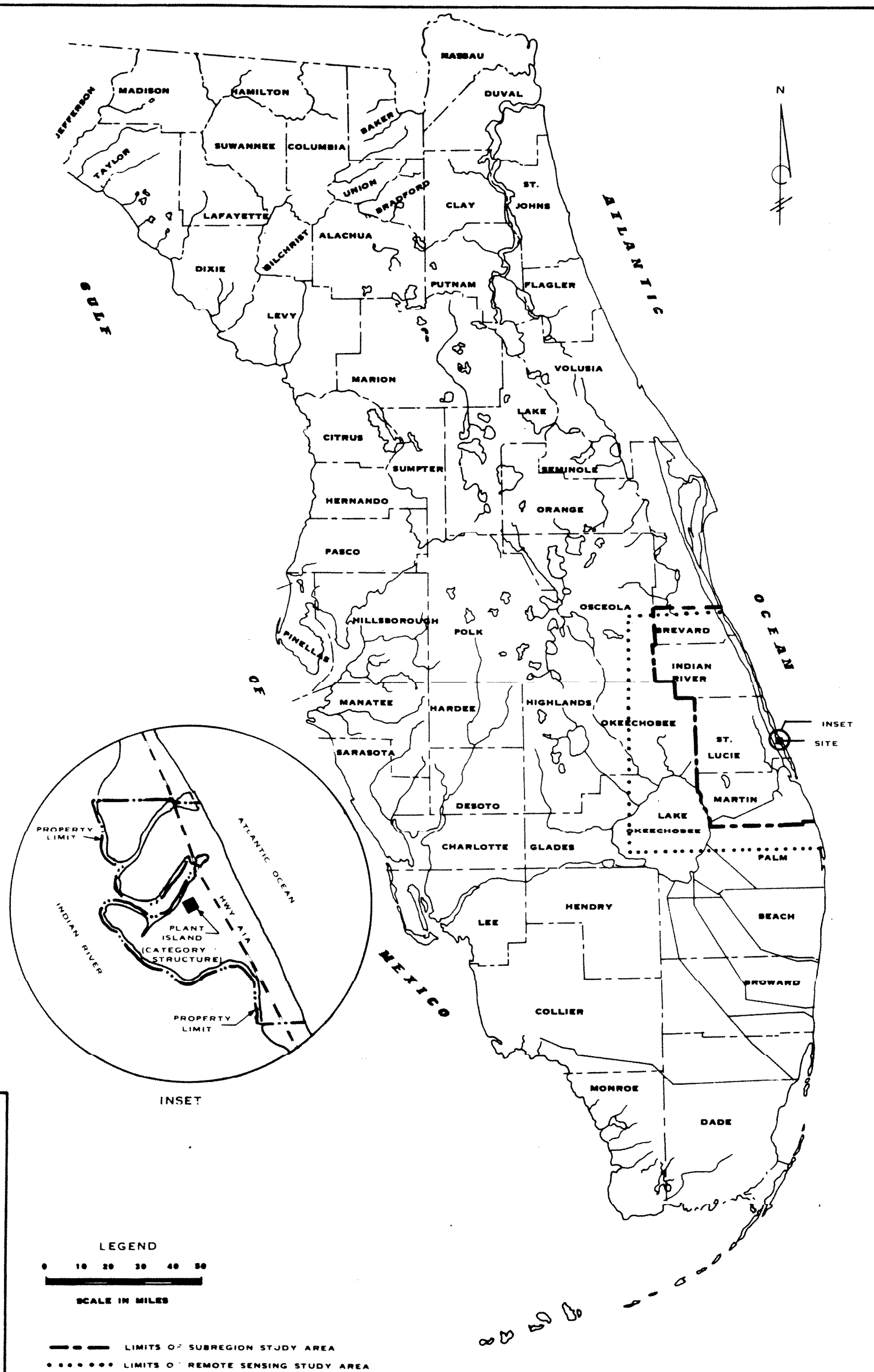
Ref: 1. Campbell, Robert P., "Earthquakes in Florida," Florida Academy of Sciences, Vol. 6, No. 1, 1943.
2. U. S. Coast & Geodetic Survey, "Earthquake History of U.S.," U. S. Government Printing Office, 1951-1958.
3. Murphy, L.; N.O.A.A. Personal telephone conversation with Law Engineering Testing Company, 1973.
4. Preliminary data from the National Earthquake Information Center (1-28-74).

TABLE 2.5-6

SUMMARY OF LIQUEFACTION TEST RESULTS
(DYNAMIC TRIAXIAL TESTS - STRESS CONTROLLED)

Test No.	Boring No.	Depth (Ft.)	Sample Undisturbed or Recompacted	D_{50} (mm)	%Passing No 20G Sieve	Test Dry Density (pcf)	Test Moisture Content (%)	Test D_r (%)	Confining Pressure (psf)	Cyclic Stress Ratio	Cycles to Pore Pressure = Confining Pressure	Cycles to Liquefaction	Applied Back Pressure (psi)	"B" Coefficient
1	B-5	96	U	0.09	15	113.8	33	100	8,000	.468	15	50	42.7	0.98
2	B-19	77-79	R	2.10	6	113.0	16	100	6,800	.573	10	35	33.5	0.95
3	B-19	78	R	2.10	6	113.0	16	100	6,800	.693	6	20	33.5	0.96
4	B-19	114	R	0.09	18	94.7	28	47	8,850	.472	7	15	48.4	0.98
5	B-20	69	R	0.09	12	95.4	27	50	6,160	.546	1	10	29.2	0.96
6	B-20	97	R	-	-	91.7	30	36	7,960	.464	2	4	42.7	0.98
7	B-20	137	U	0.10	20	104.9	21	83	10,350	.486	20	125	59.6	0.92
8	E-20	137	R	0.10	20	100.3	24	68	10,350	.347	10	40	59.6	1.00
9	B-20	137	R	0.10	20	104.1	22	81	8,590	.347	23	28	59.6	0.99
10	B-20	137	U	0.10	20	104.0	20	65	10,350	.257	45	75	60.0	0.96
11	B-19	134	U	0.13	34	100.6	24	58	10,000	.479	8	15	57.3	0.98
12	B-19	134	U	0.13	34	103.6	21	68	10,000	.443	10	18	57.3	0.98
13	B-19	134	U	0.13	34	97.8	24	49	10,000	.429	8	18	57.3	0.99
14	Composite		R	-	-	98.0	26	55	10,000	.429	4	5	70.0	0.98
15	Composite		R	-	-	98.0	26	55	10,000	.350	6	8	70.0	0.98
16	A-1	12-27	R	0.13	2.5	97.5	22	85	3,500	.286	604	608		
17	A-1	12-27	R	0.13	2.5	97.5	22	85	3,500	.543	91	-		
18	A-1	12-27	R	0.13	2.5	97.5	22	85	3,500	.685	56	-		
19	A-1	12-27	R	0.13	2.5	97.5	22	85	3,500	.927	18	25		

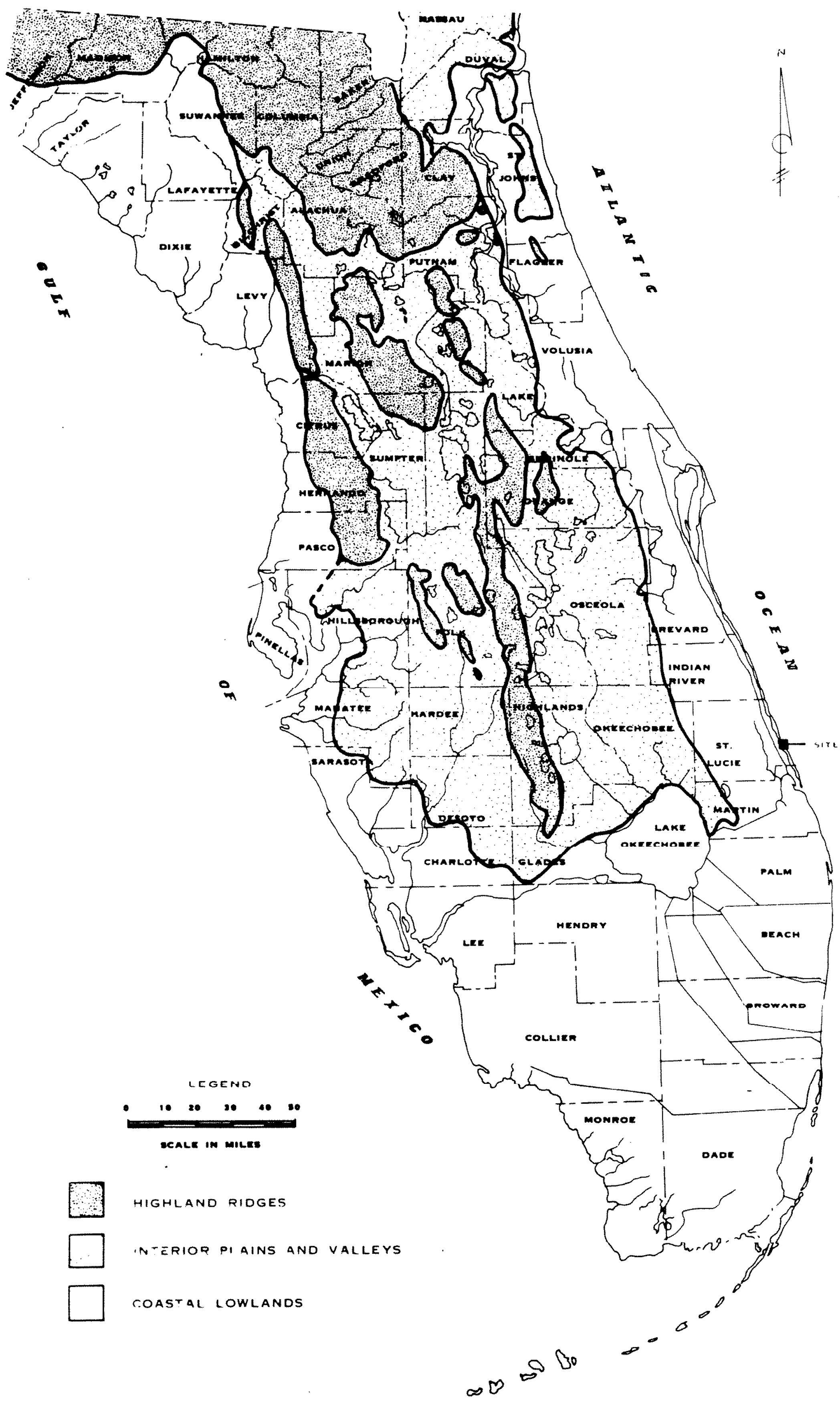
Notes: Test Nos. 1 to 15 presented on Figures 2.5-38 and 2.5-39.
Test Nos. 16 to 19 presented on Figures 2.5-41.



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SITE LOCATION & STUDY AREA

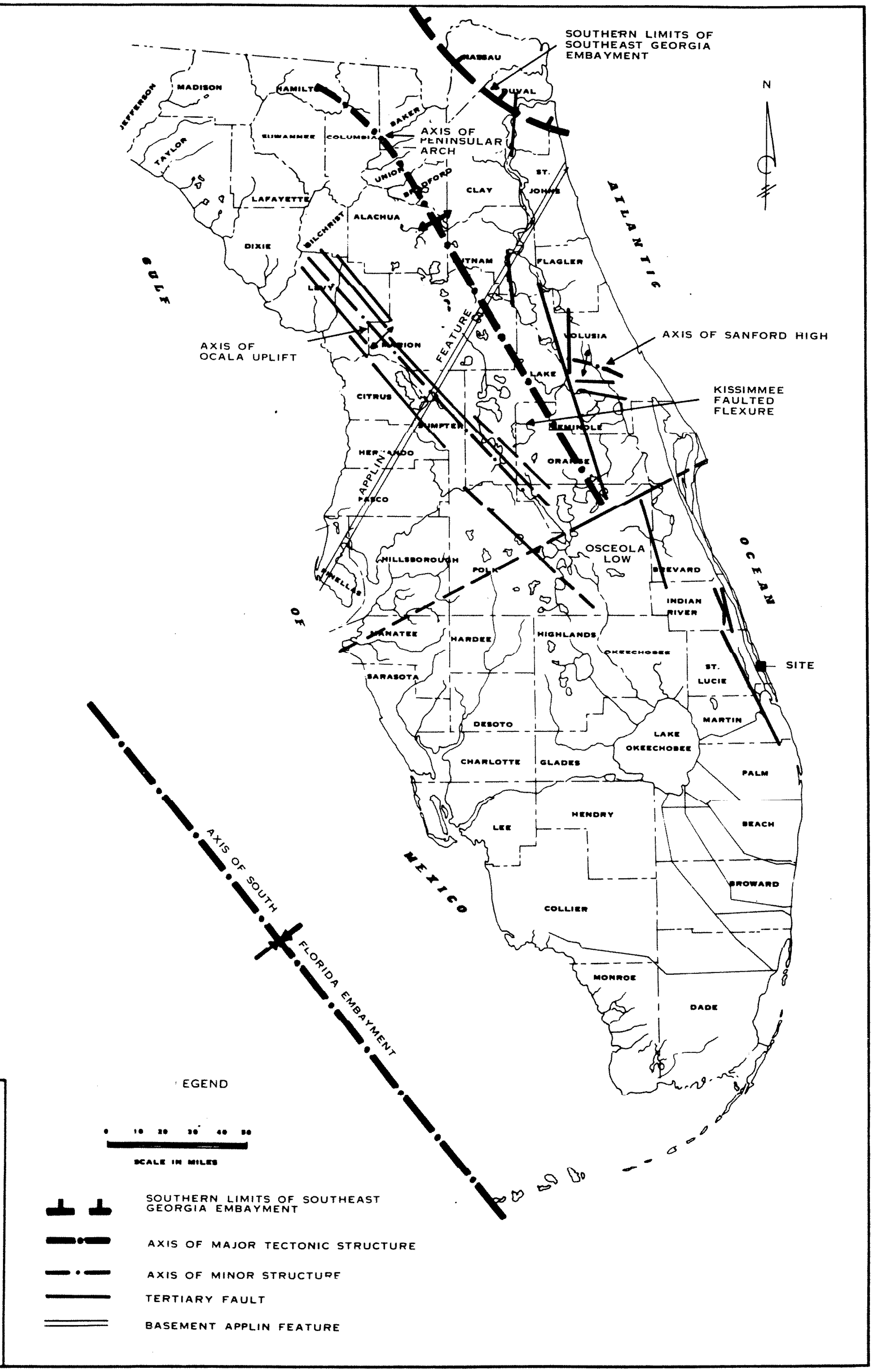
FIGURE 2.5-1



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

REGIONAL PHYSIOGRAPHY

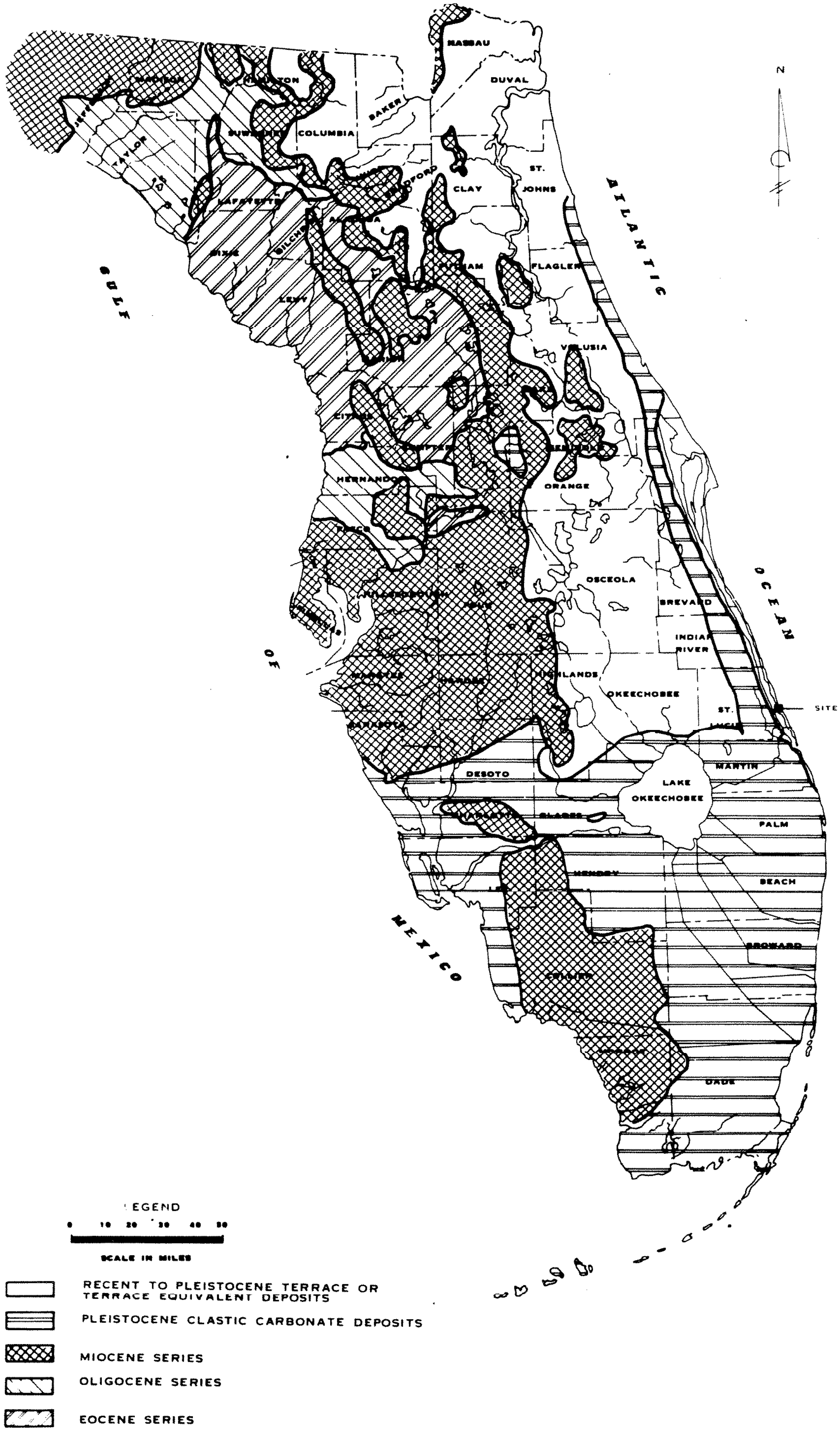
FIGURE 2.5-2



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

REGIONAL STRUCTURE (PUBLISHED)

FIGURE 2.5-3

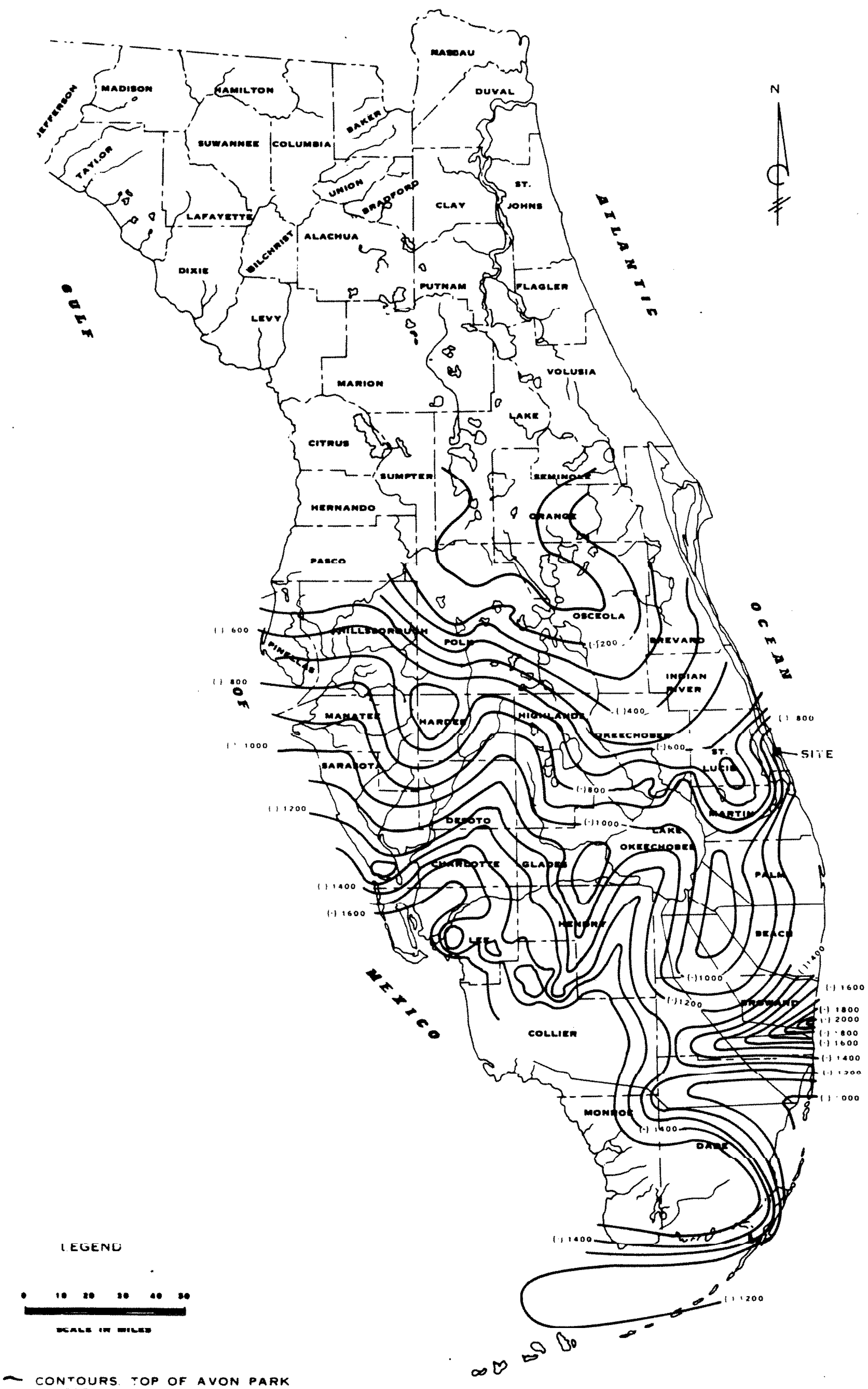


FLORIDA POWER & LIGHT COMPANY

ST. LUCIE PLANT UNIT 1

REGIONAL SURFACE GEOLOGY

FIGURE 2.5-4



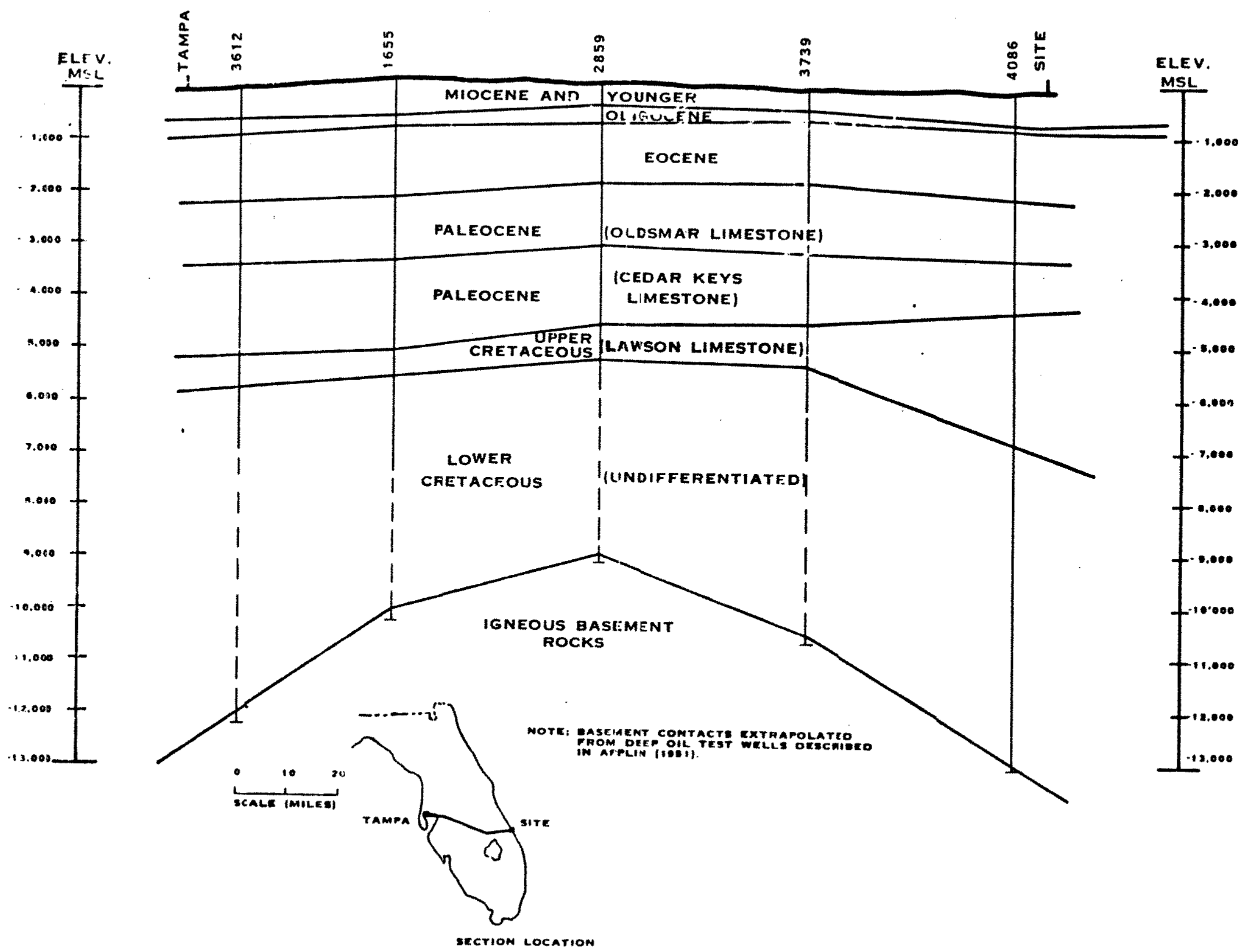
—400— CONTOURS, TOP OF AVON PARK LIMESTONE (CONTOUR INTERVAL 100 FEET)

UNPUBLISHED MAP BY HENDERSON, 1958
DATA POINTS NOT SHOWN

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

TOP OF AVON PARK (REGIONAL)

FIGURE 2.5-5



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

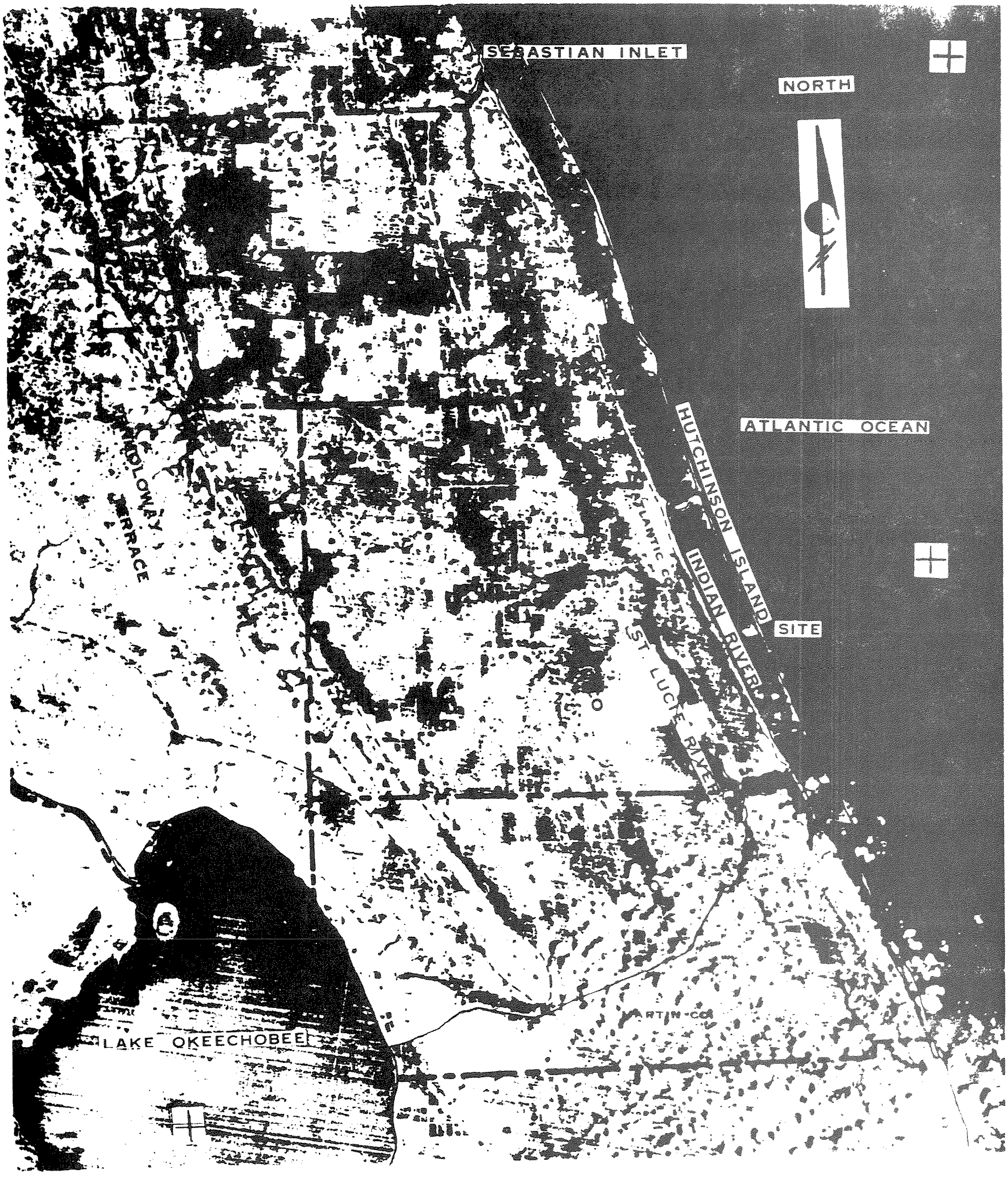
REGIONAL GEOLOGIC PROFILE

FIGURE 2.5-6

ST. LUCIE PLANT

SATELLITE PH

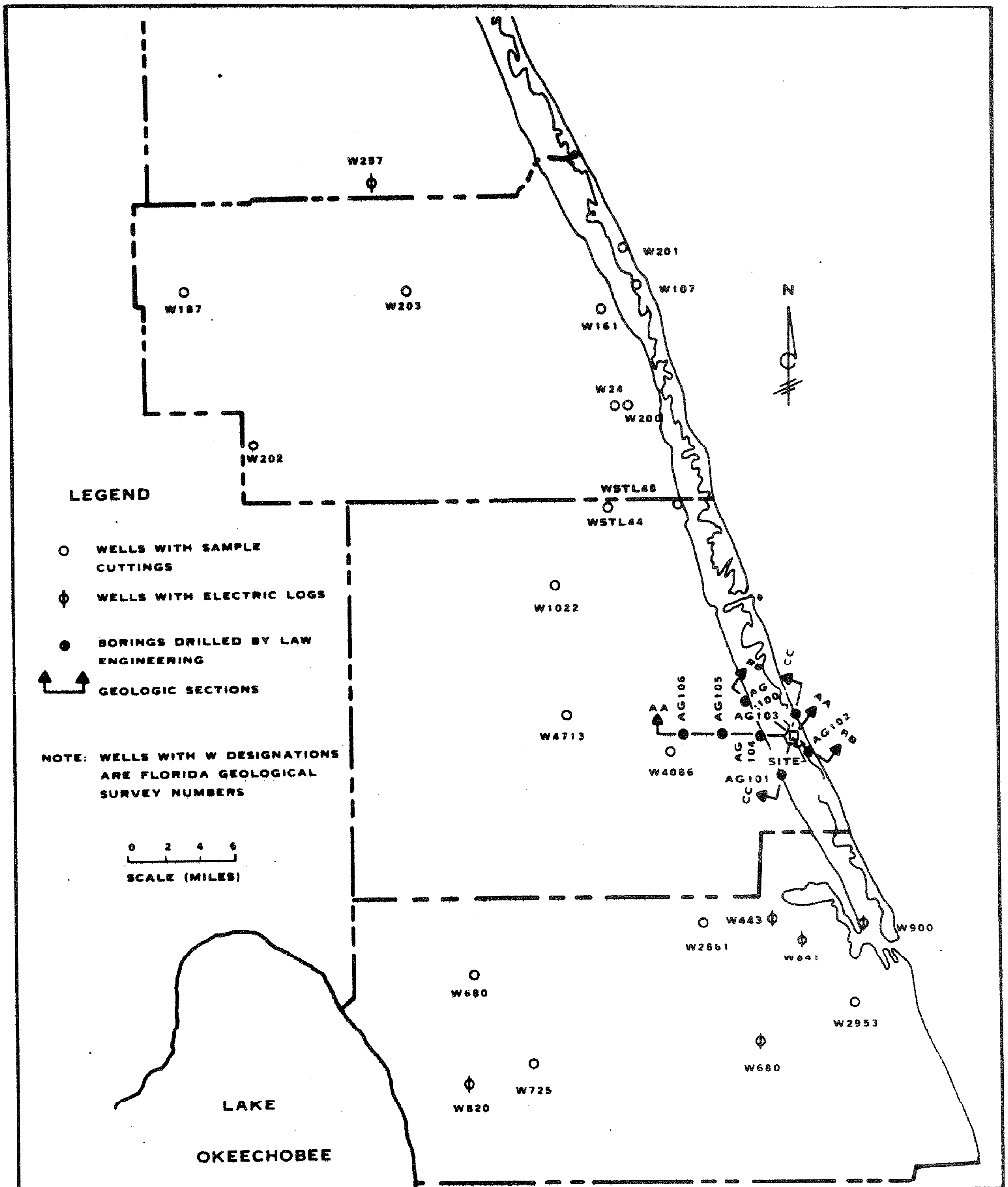
FIGURE 2 3 7



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SATELLITE PHOTOGRAPH OF
SUBREGION AND SUBREGIONAL
PHYSIOGRAPHY

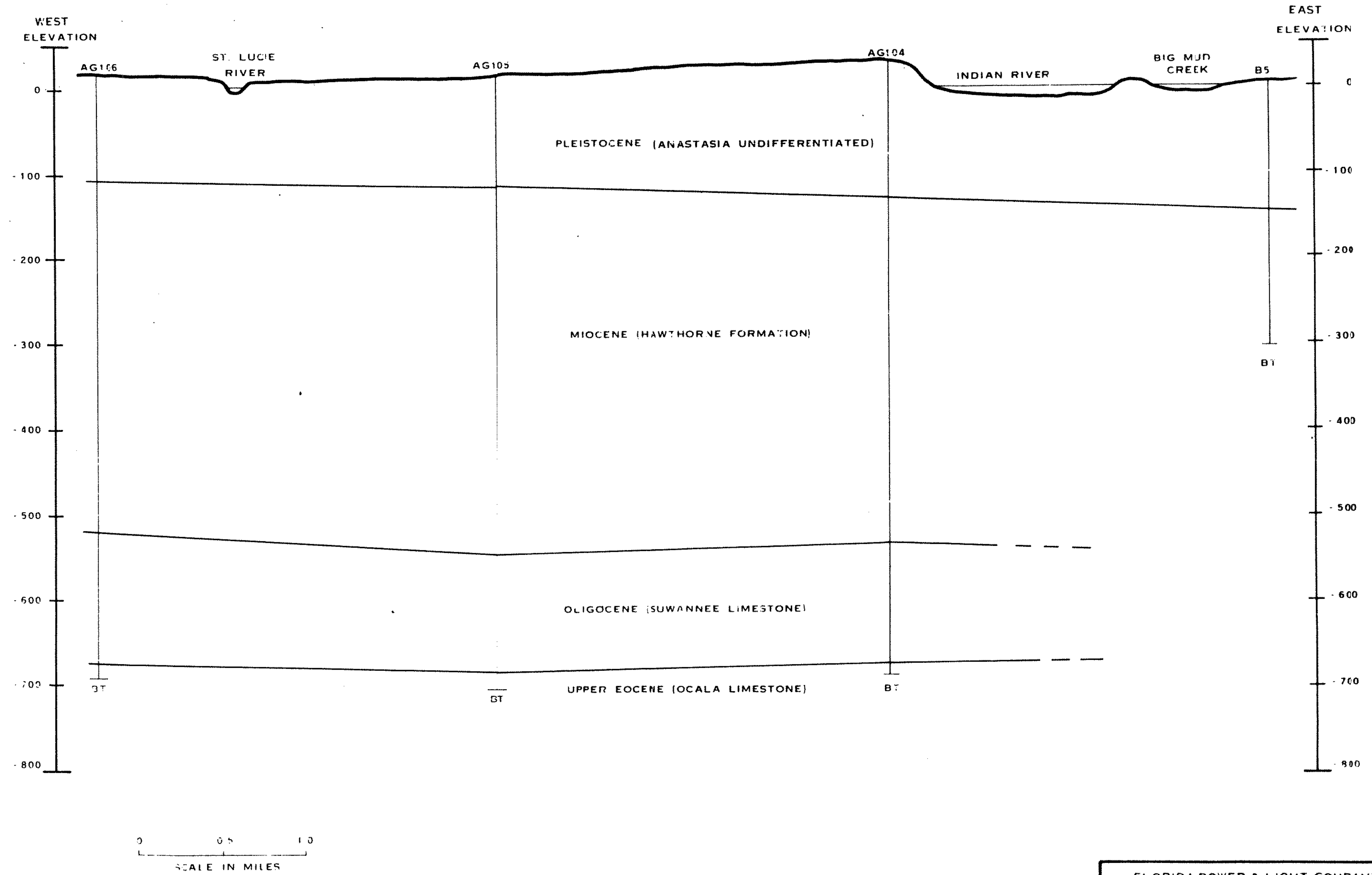
FIGURE 2.5-7a



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

LOCATION OF WELL DATA POINTS

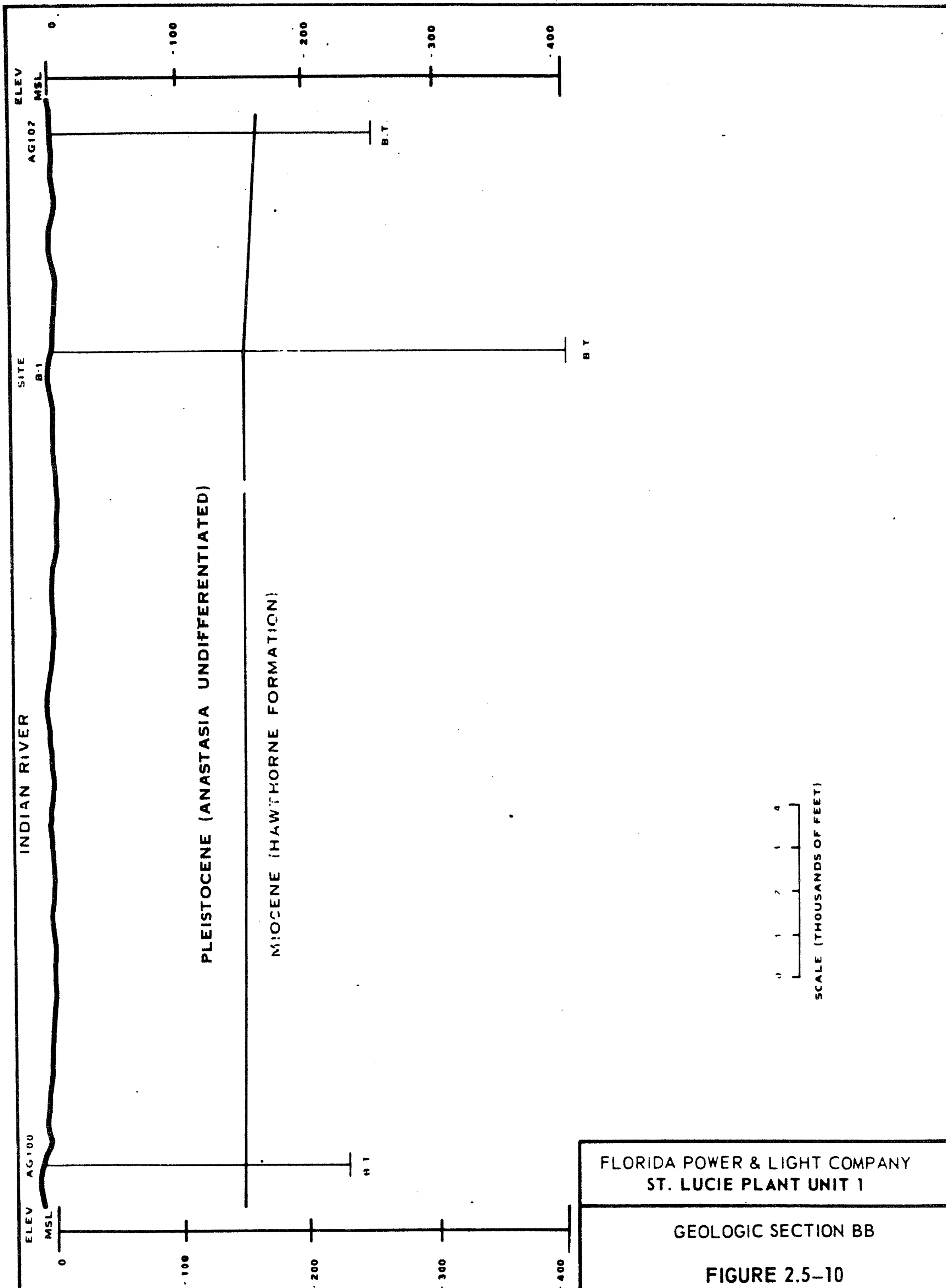
FIGURE 2.5–8



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GEOLOGIC SECTION AA

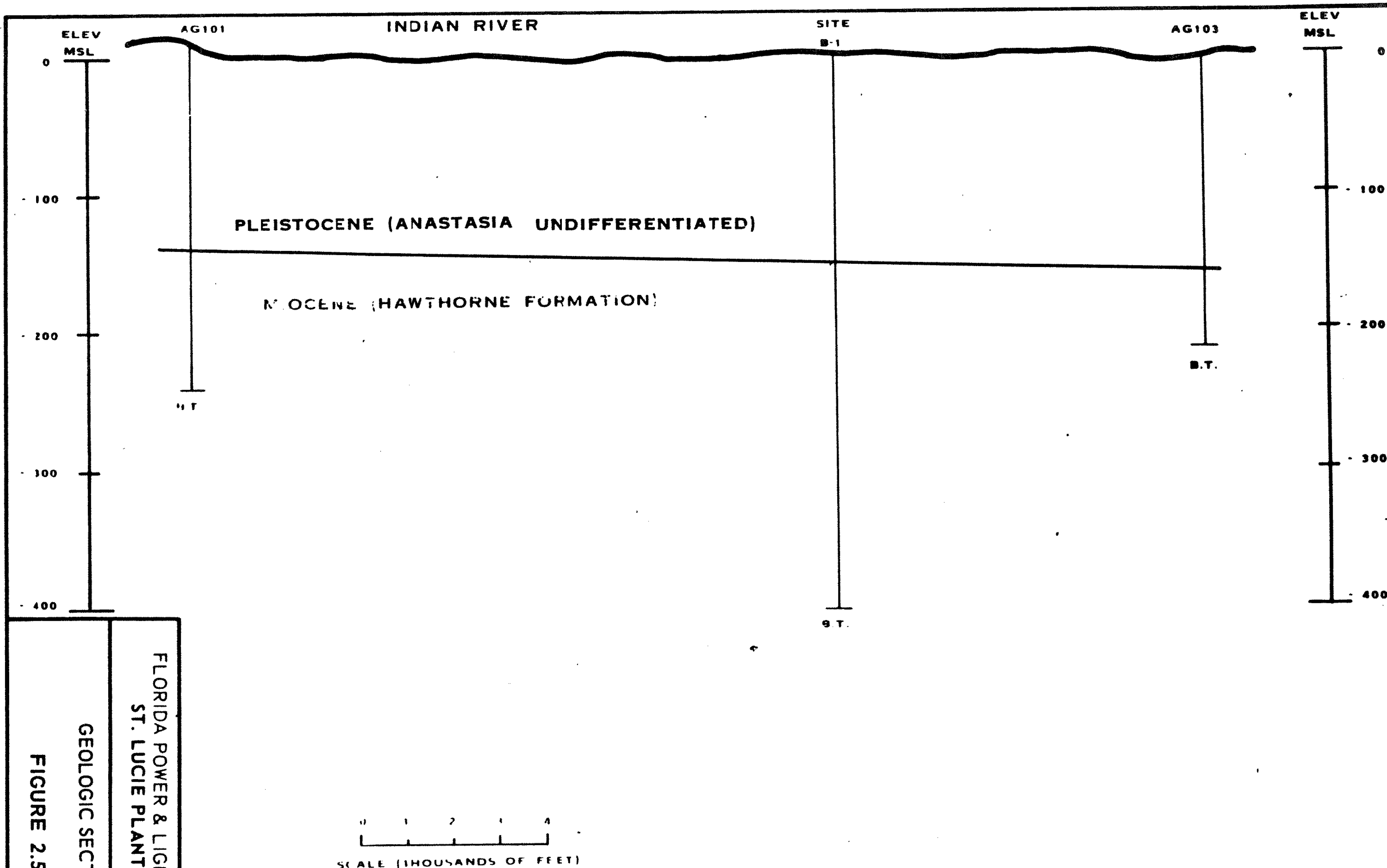
FIGURE 2.5-9



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GEOLOGIC SECTION BB

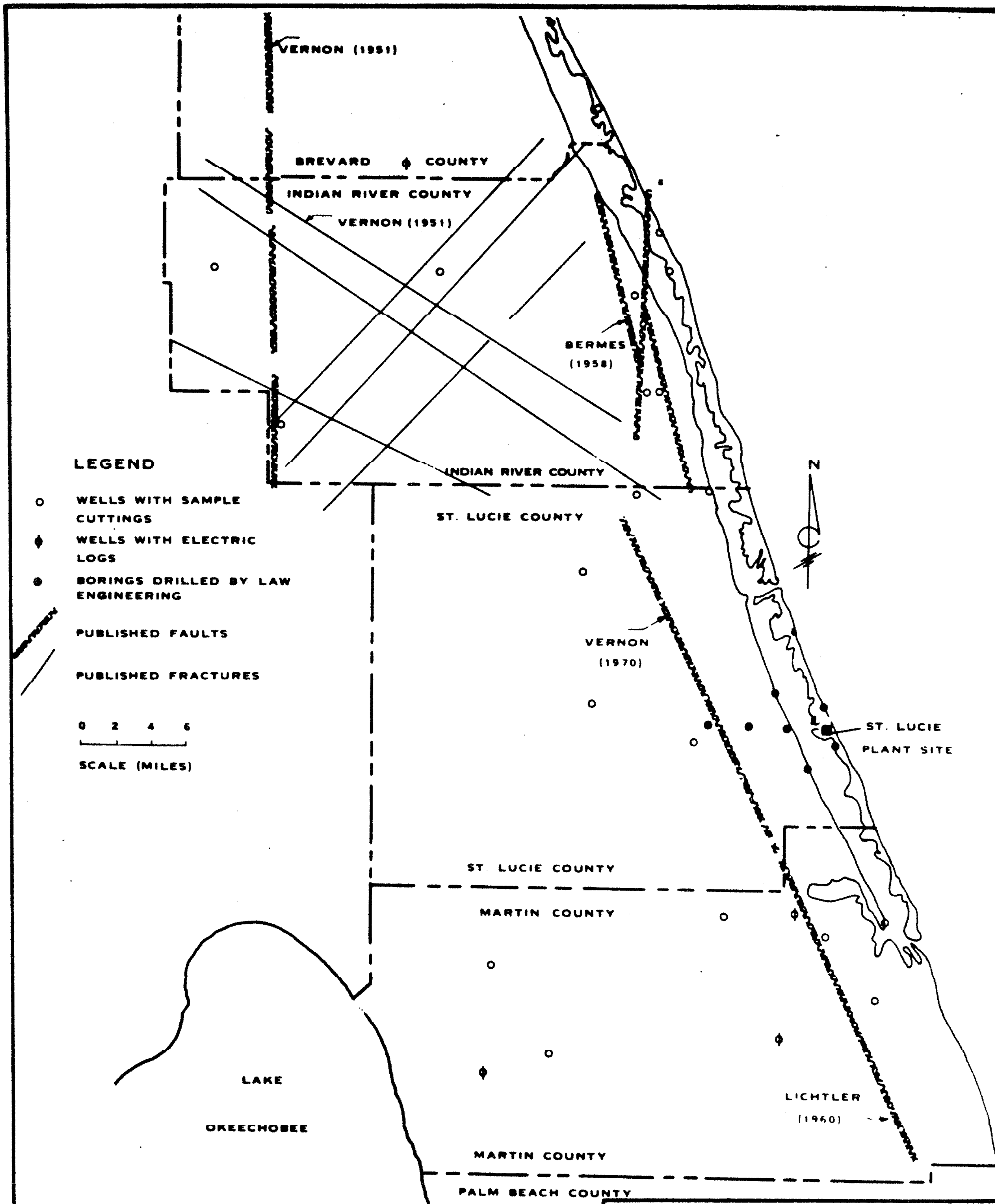
FIGURE 2.5–10



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GEOLOGIC SECTION CC

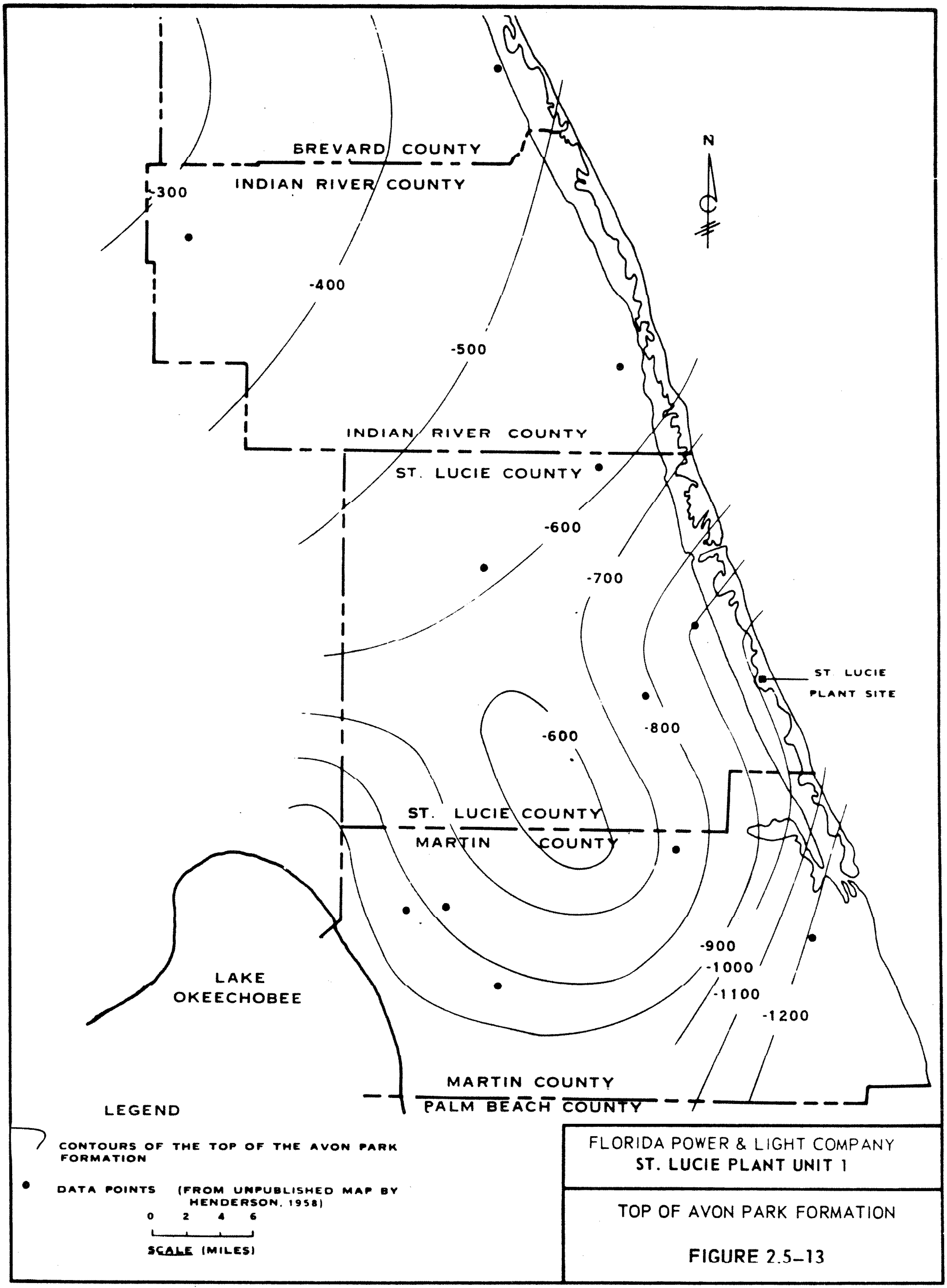
FIGURE 2.5–11



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SUBREGIONAL STRUCTURE (PUBLISHED)

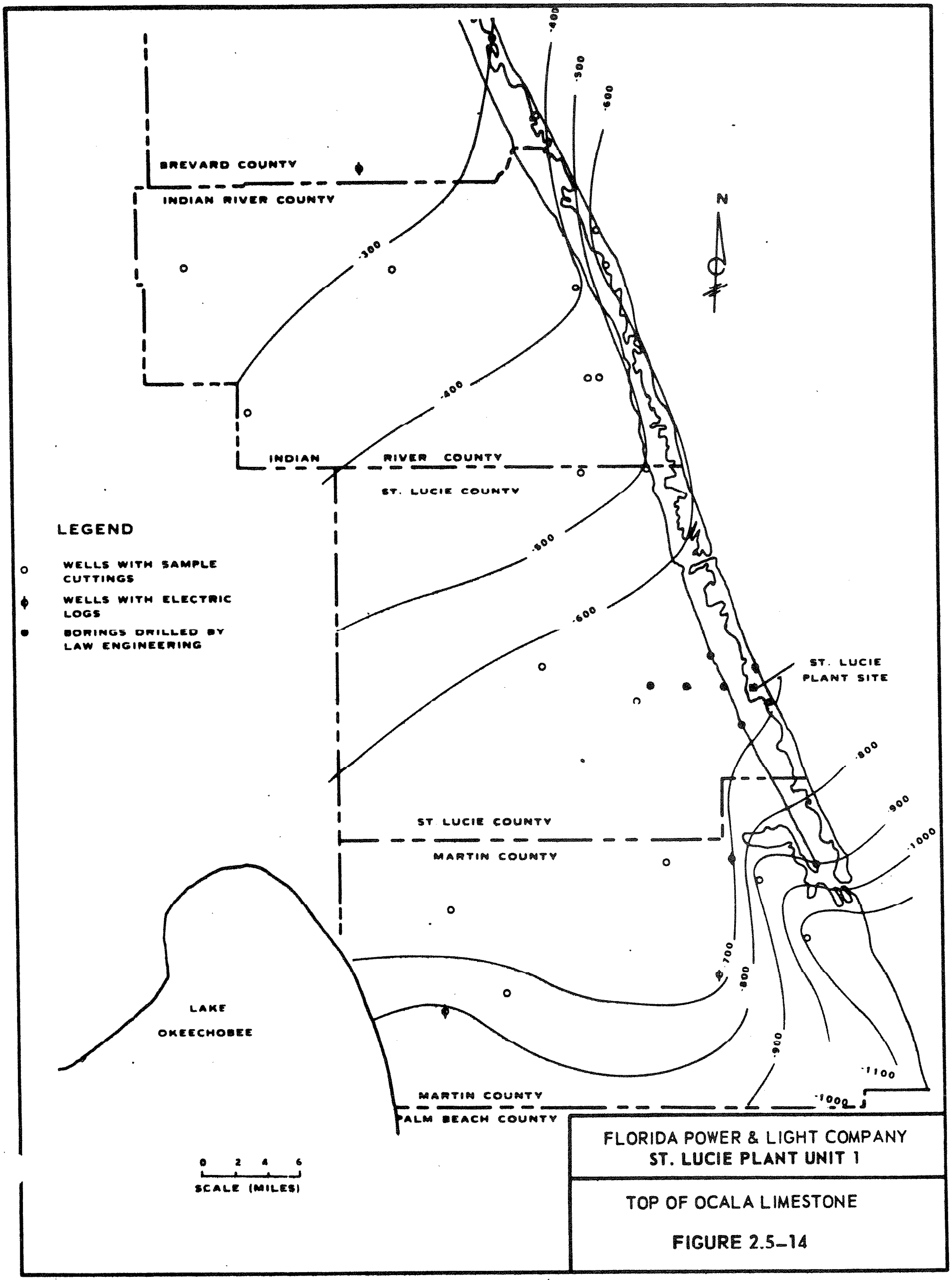
FIGURE 2.5–12



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

TOP OF AVON PARK FORMATION

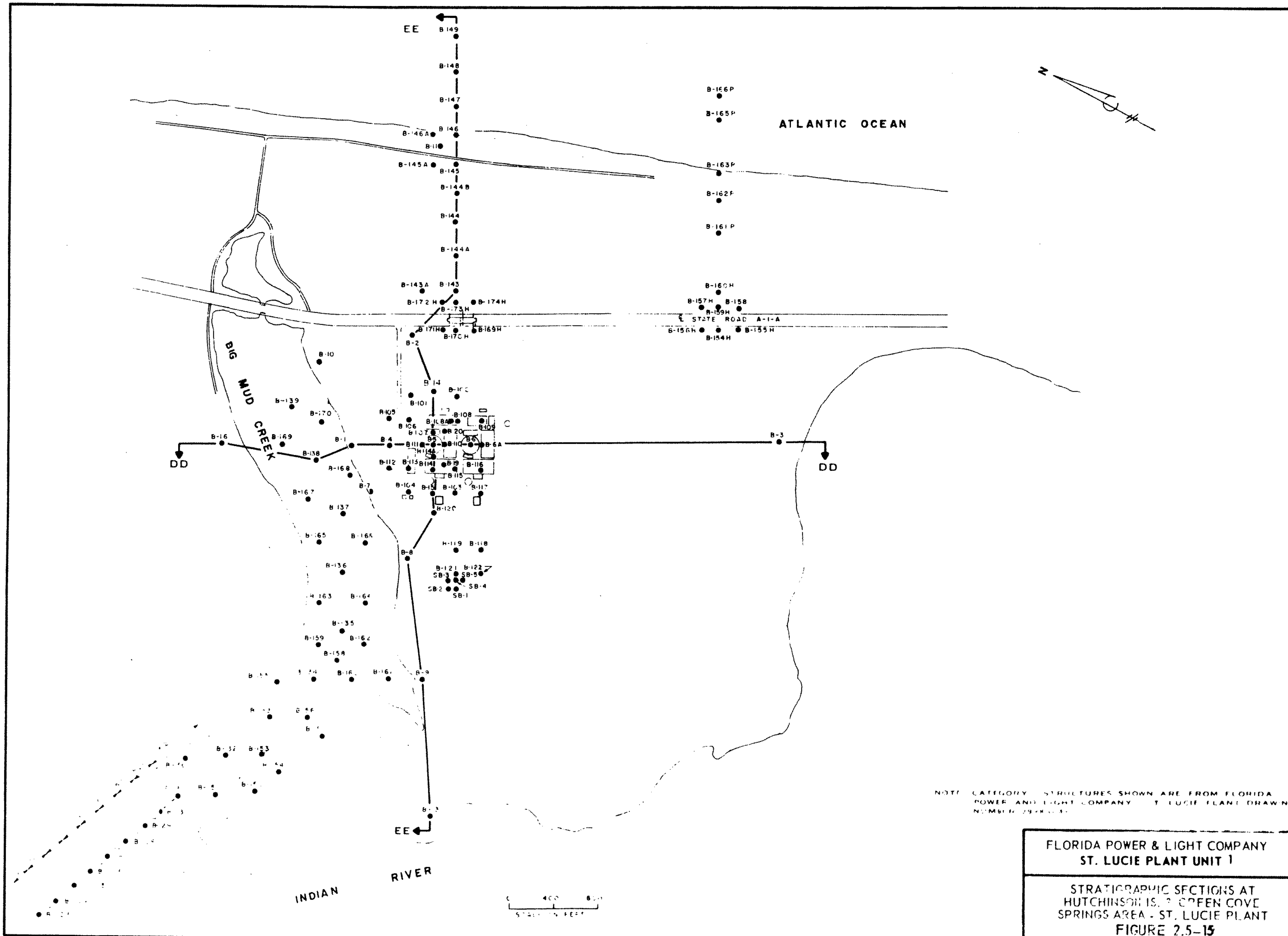
FIGURE 2.5-13



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

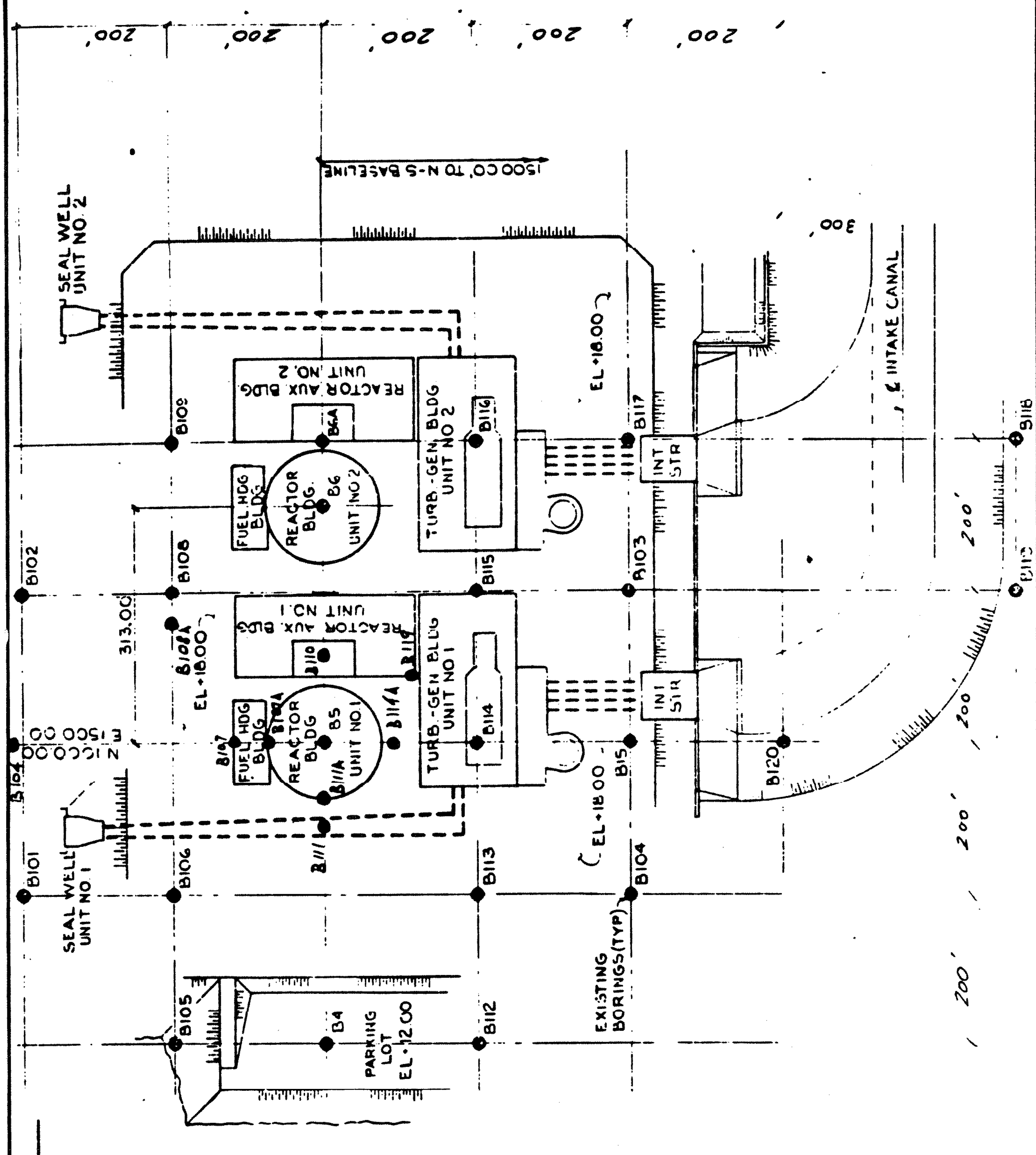
TOP OF OCALA LIMESTONE

FIGURE 2.5–14



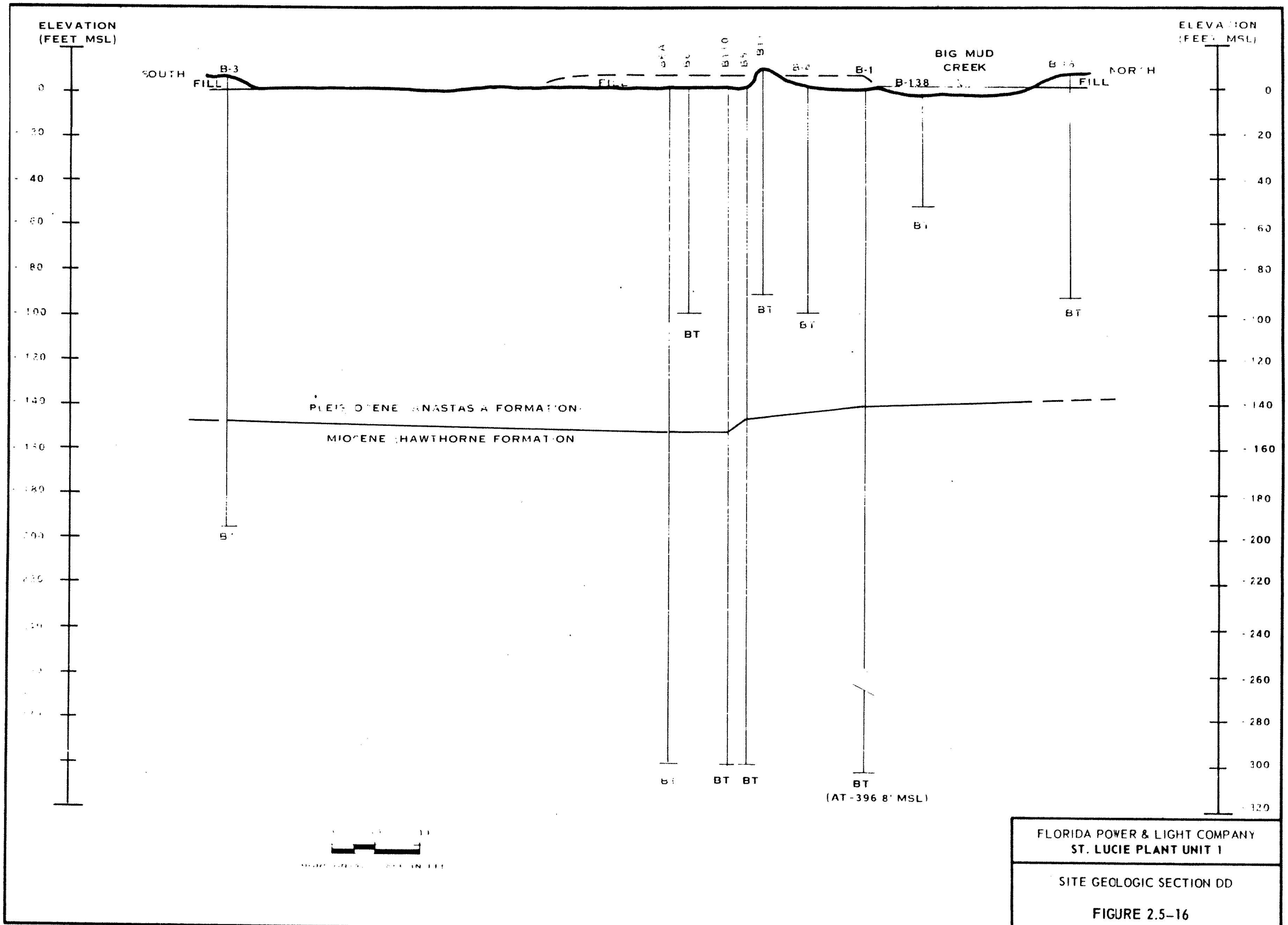
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

STRATIGRAPHIC SECTIONS AT HUTCHINSON IS. & GREEN COVE SPRINGS AREA - ST. LUCIE PLANT
FIGURE 2.5-15



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

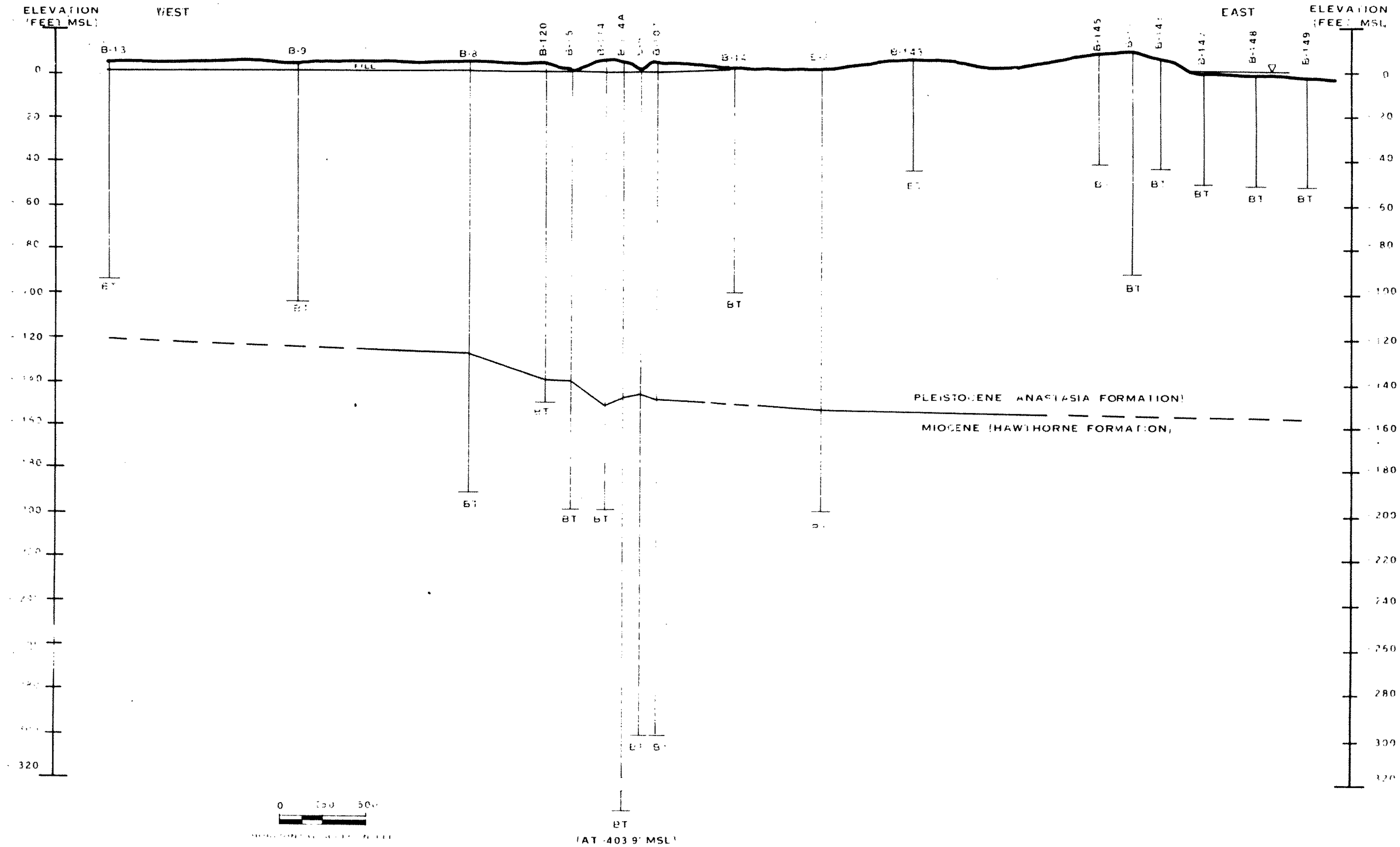
BORING PLAN – PLANT AREA
FIGURE 2.5–15A



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SITE GEOLOGIC SECTION DD

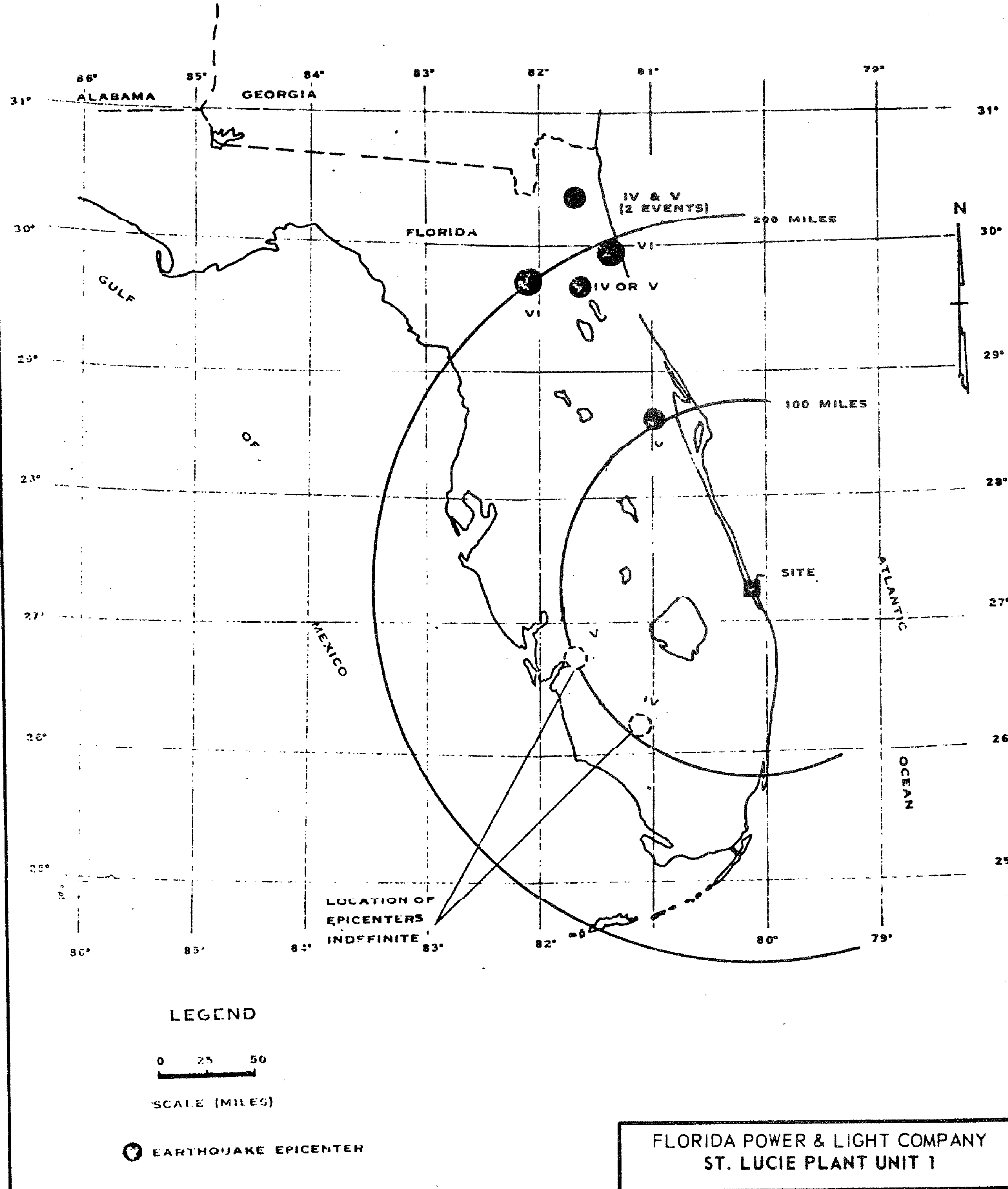
FIGURE 2.5–16



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SITE GEOLOGIC SECTION EE

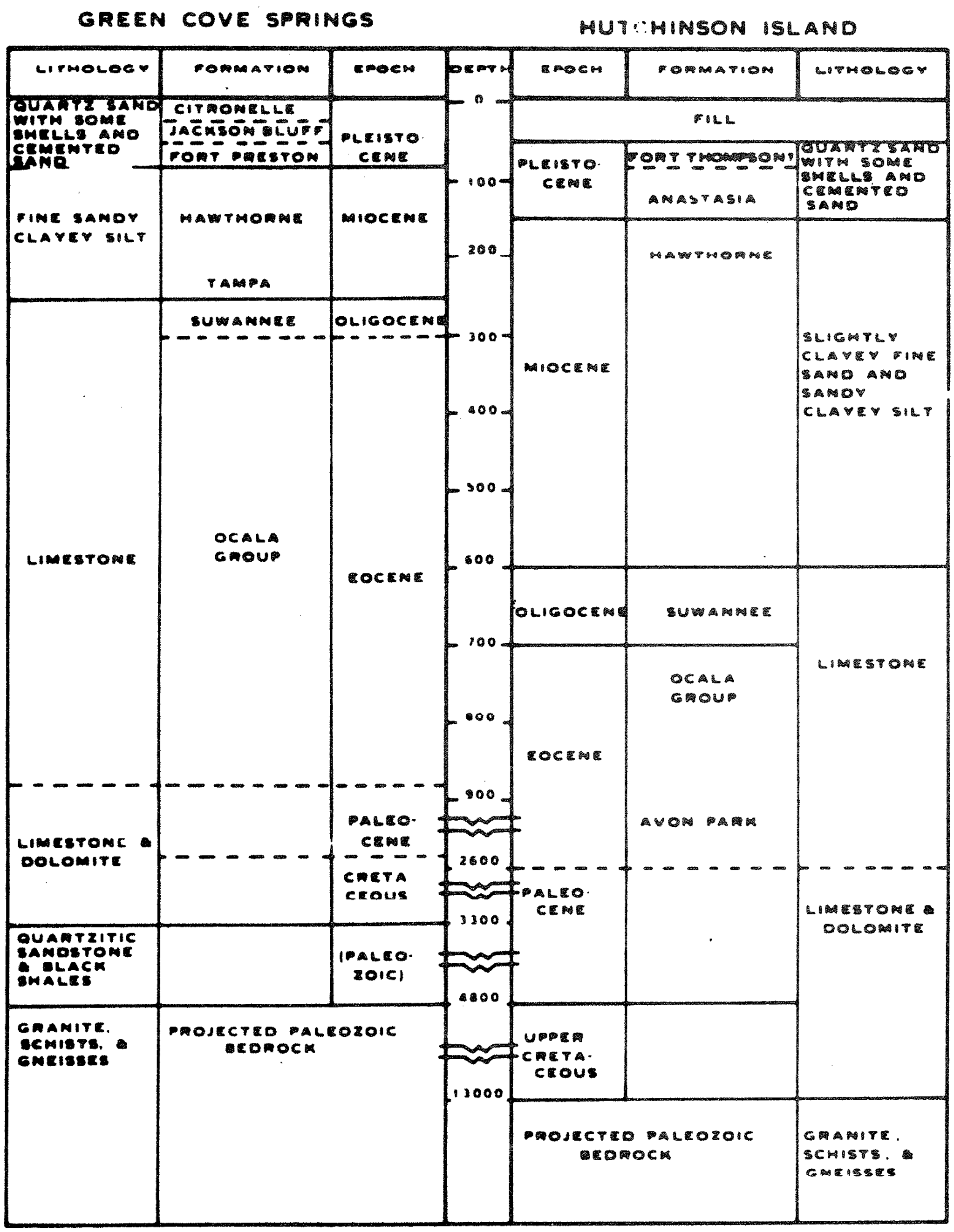
FIGURE 2.5-17



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

LOCATIONS OF EPICENTERS

FIGURE 2.5–18

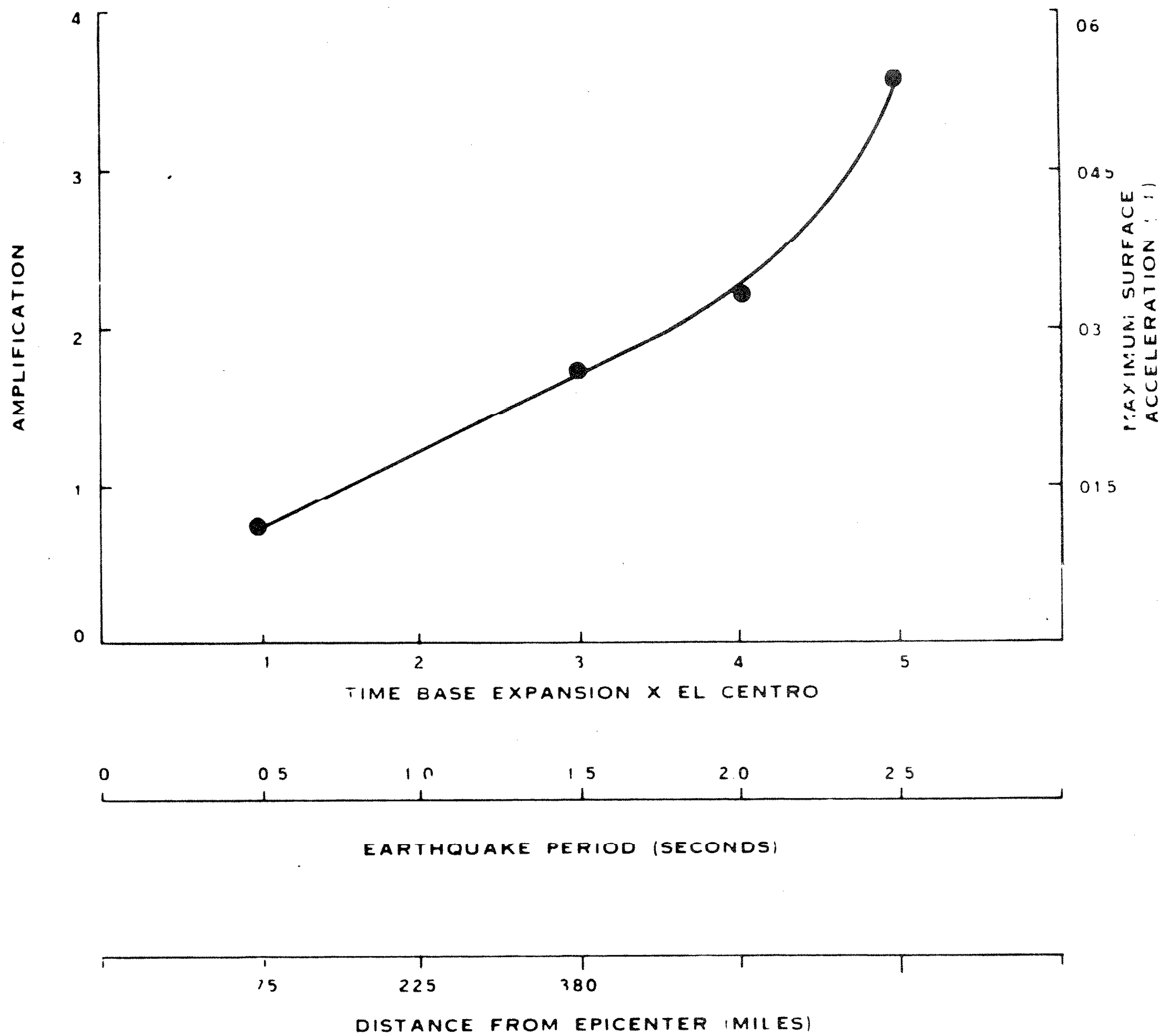


STRATIGRAPHIC SECTIONS AT HUTCHINSON ISLAND AND GREEN COVE SPRINGS AREA

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

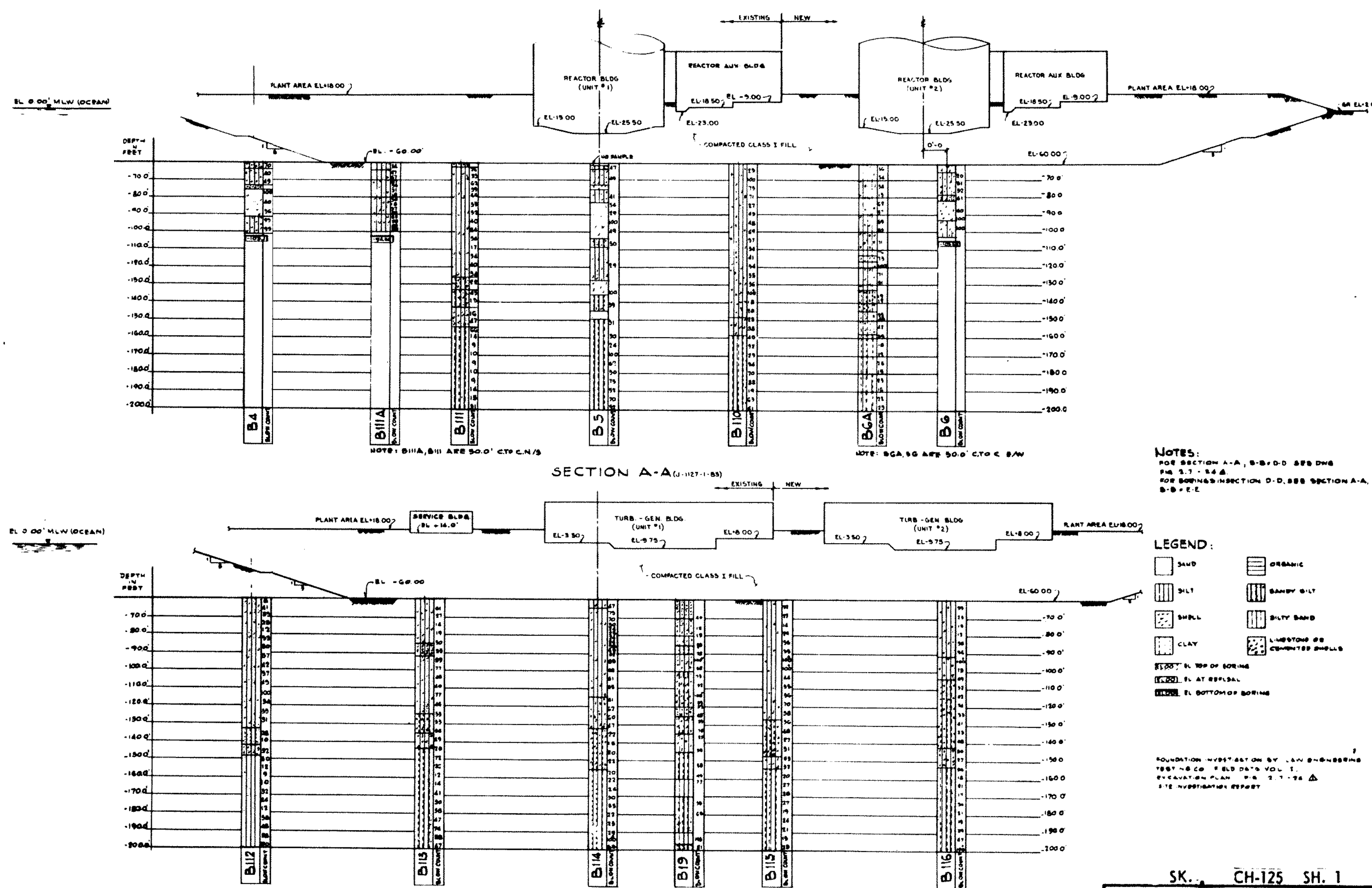
STRATIGRAPHIC SECTIONS AT HUTCHINSON ISLAND & GREEN COVE SPRINGS AREA

FIGURE 2.5-19



NOTE RELATIONSHIPS FOR PERIOD DISTANCE ARE FROM SEED IDRISS & KIEFER, 1968

FLORIDA POWER & LIGHT COMPANY **ST. LUCIE PLANT UNIT 1**
TIME BASE EXPANSION EL CENTRO EARTHQUAKE FIGURE 2.5-20

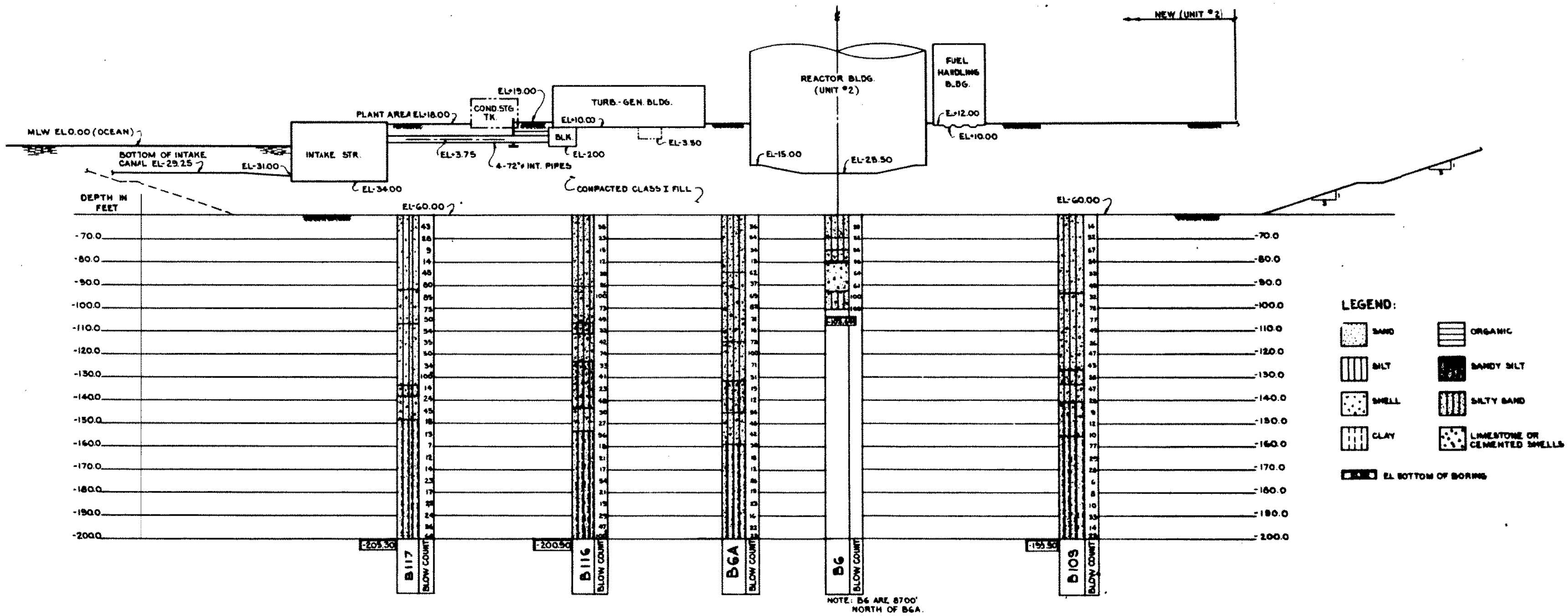


SK. CH-125 SH. 1

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SITE EXPLORATION BORINGS

FIGURE 2.5-21

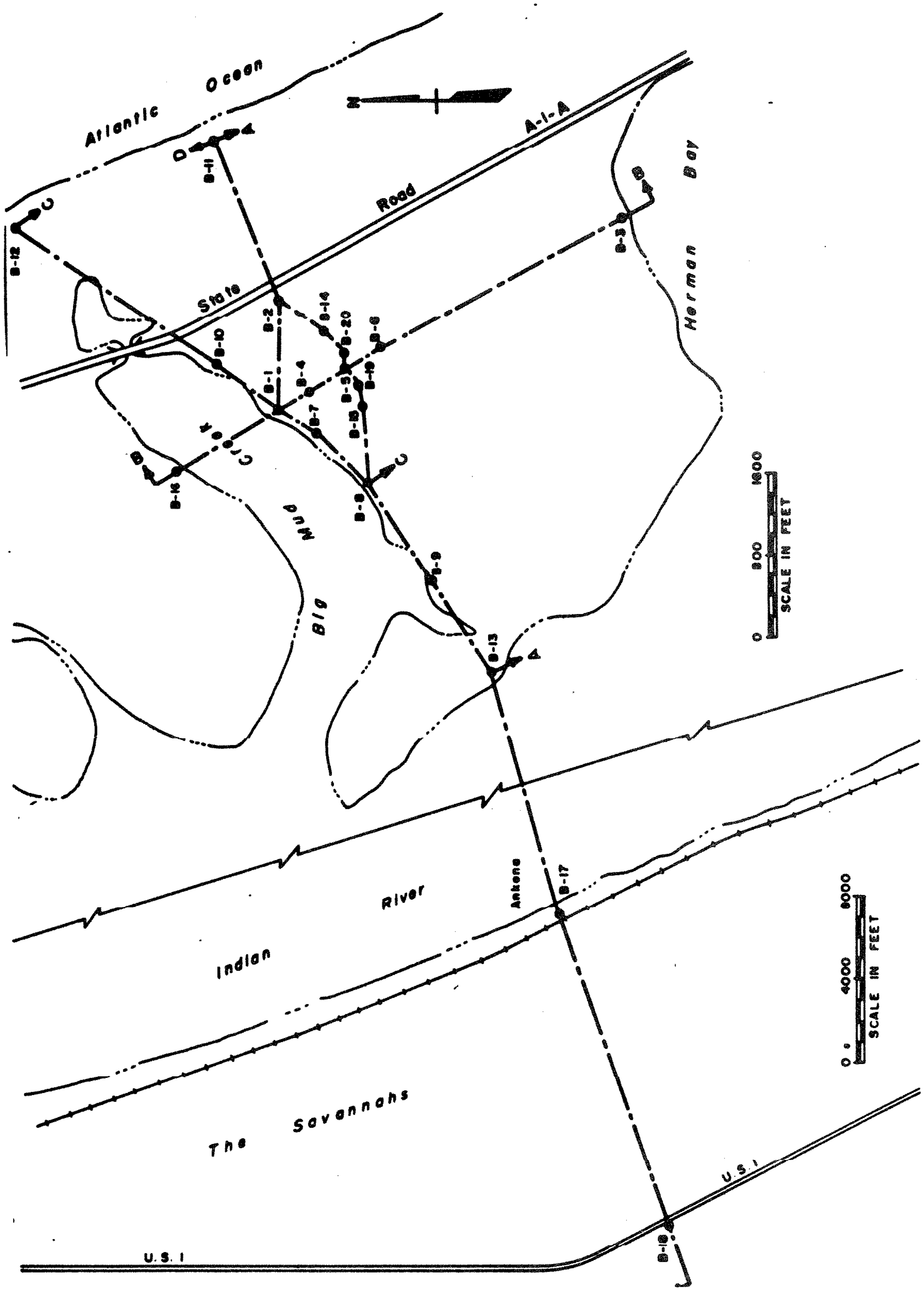


SECTION F-F (J-1127-1-04)

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SITE EXPLORATION BORINGS

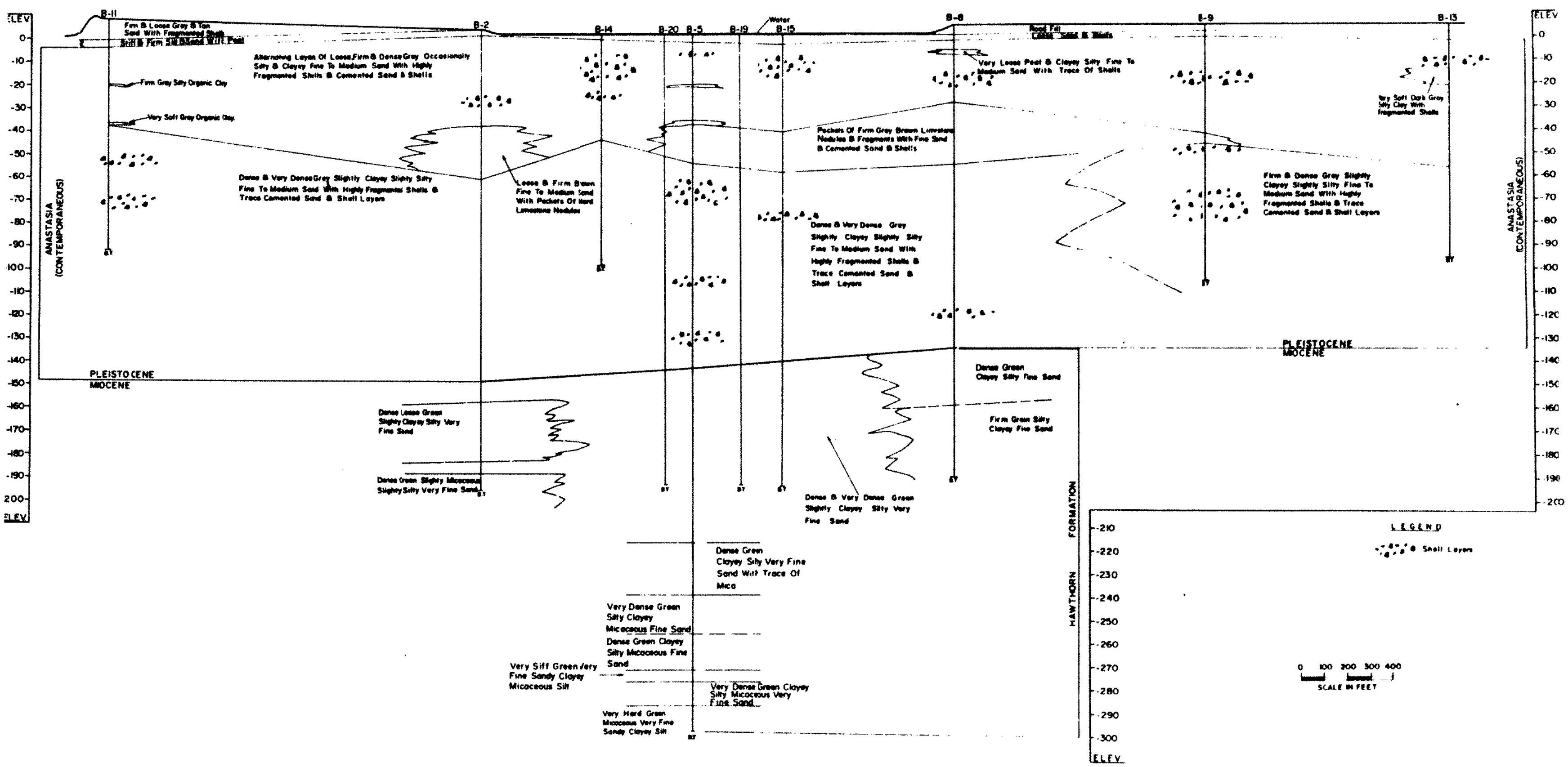
FIGURE 2.5-22



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

BORING PLAN

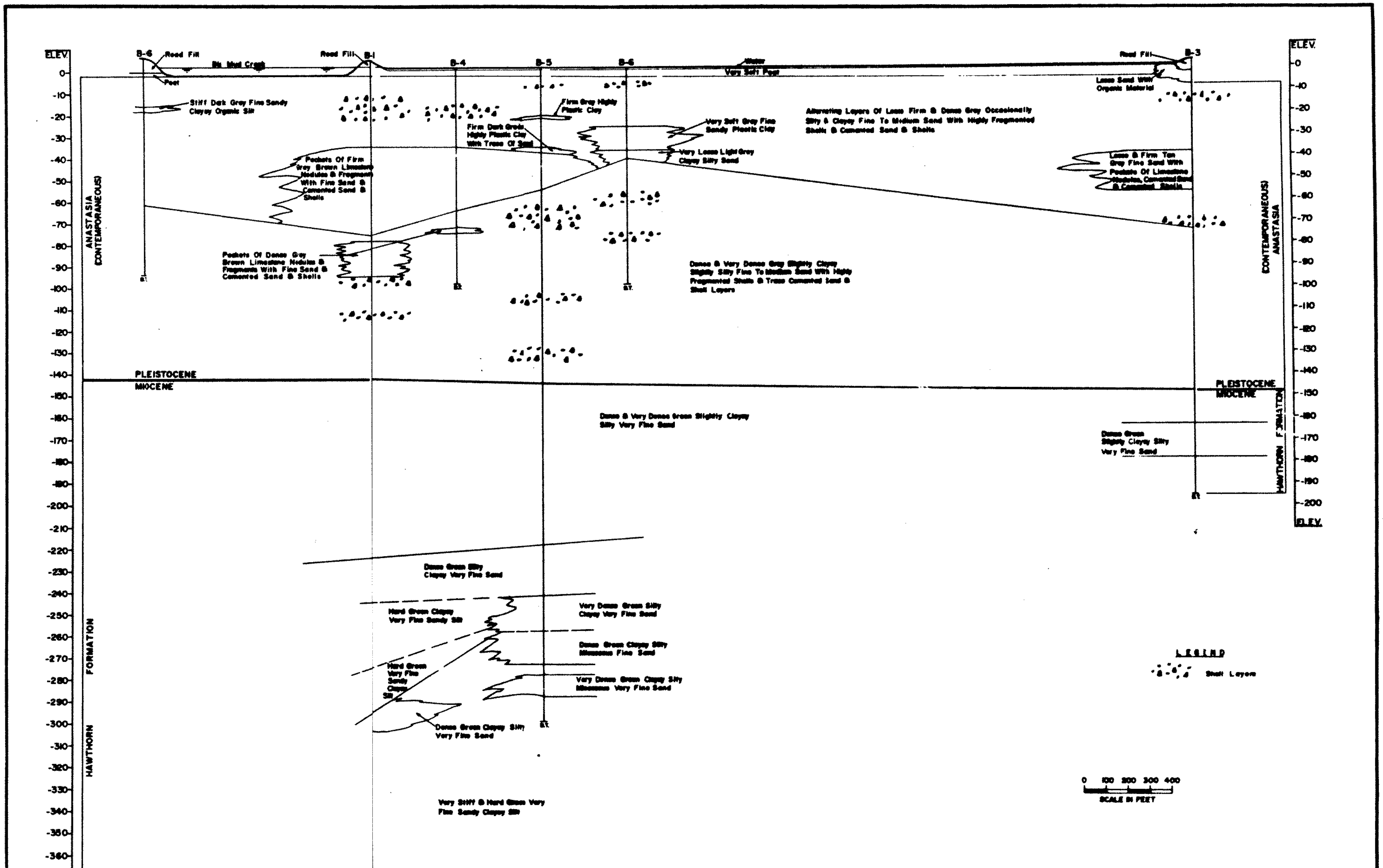
FIGURE 2.5-23



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GEOLOGIC SECTION A-A

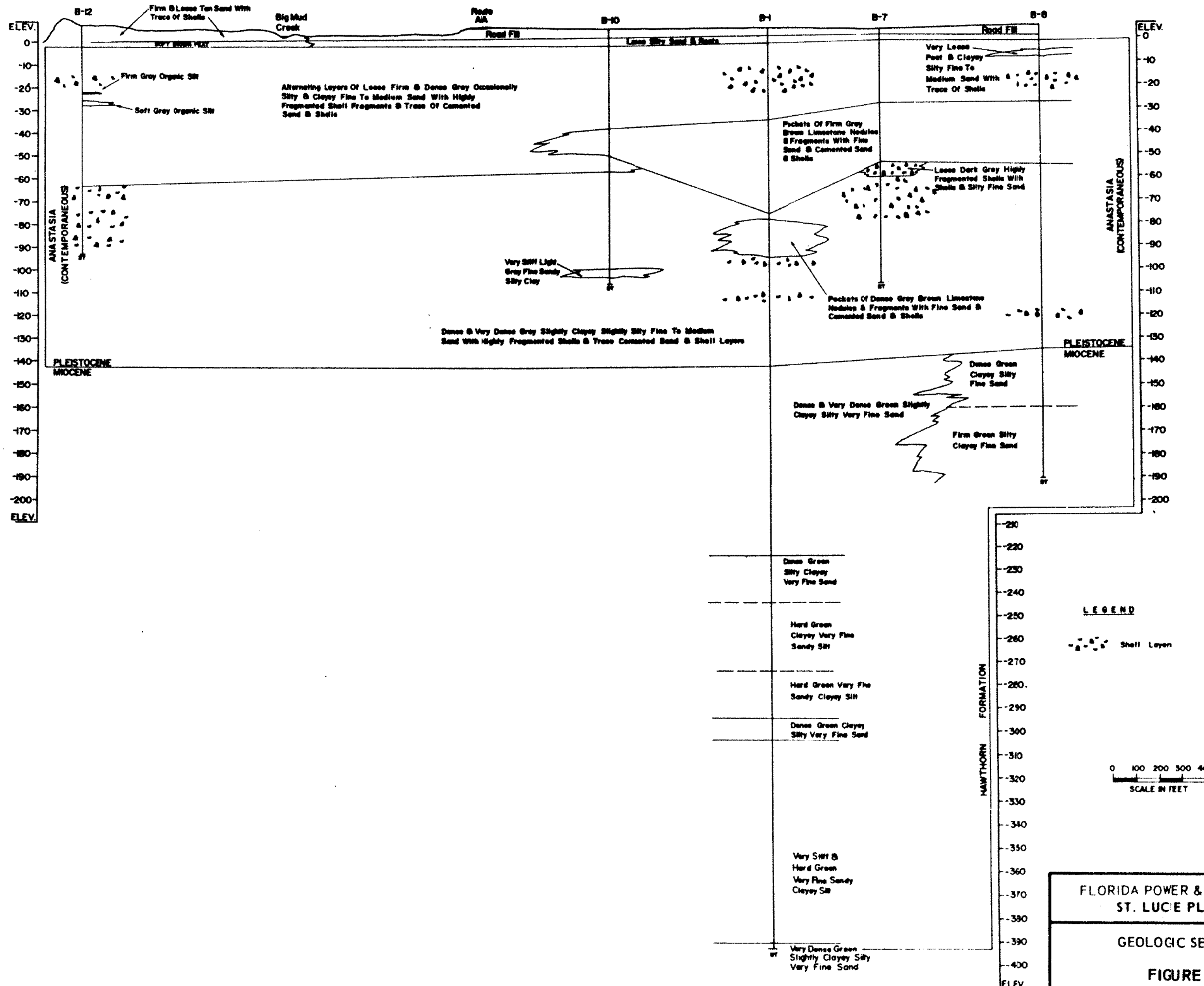
FIGURE 2.5-24



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GEOLOGIC SECTION B-B

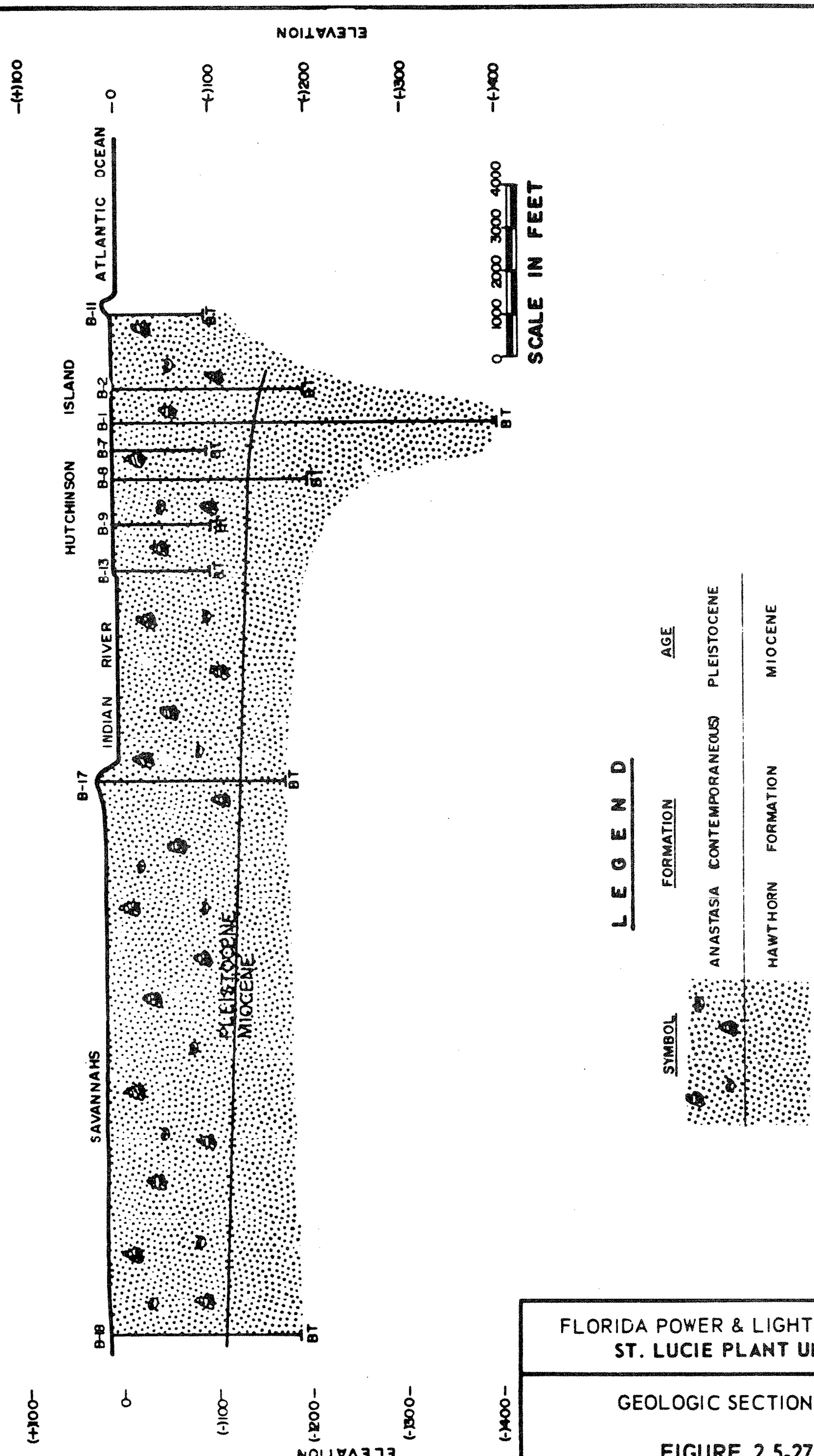
FIGURE 2.5-25



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GEOLOGIC SECTION C–C

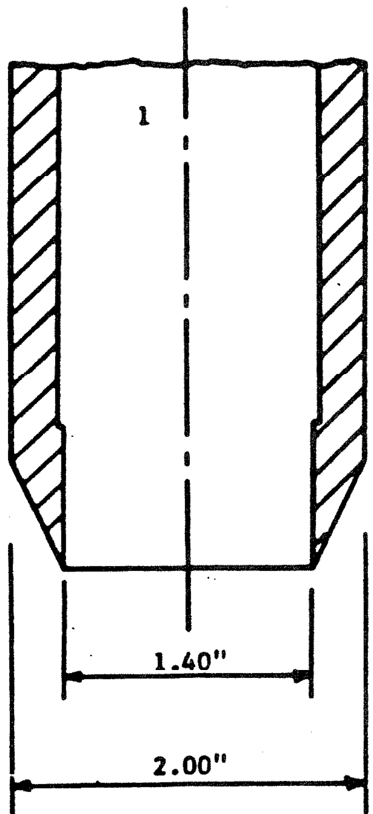
FIGURE 2.5-26



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

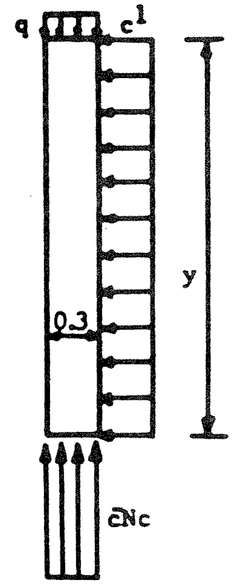
GEOLOGIC SECTION D-D

FIGURE 2.5-27



SPLIT SPOON SAMPLER
CROSS SECTION

FREE BODY DIAGRAM



$\bar{c}$ = Average Undrained Shear Strength
$c^1 = \bar{c}\alpha$
α = adhesion reduction factor (assume = 0.5)
L = depth of embedment when rod stops falling under its own weight (inches)
q = incremental rod weight
N_c = Bearing Capacity factor (assume $N_c = 5$)

SOIL STRENGTH ANALYSIS

A. BEARING AREAS

1. END BEARING: $A = (R^2 - r^2)$: $R=1; r=0.7$

   $A_B = 1.6\ in^2$

2. SKIN FRICTION: $A = 2\pi RL$

   $A_S = 6.28L\ in^2$

B. CAPACITY

$Q = A_B\ \bar{c}\ N_c + A_S\ c' =$ ROD + HAMMER WEIGHT

if $\bar{c}$ & c' in pounds/ft^2 & Q in pounds

then

$$Q = \frac{1.6}{144}\bar{c}\ N_c + \frac{6.28}{144}\ L\ c^1$$

with $N_c = 5$ & $\alpha = 0.5$

$$Q = \bar{c}\ (0.0556 + 0.0218L)$$

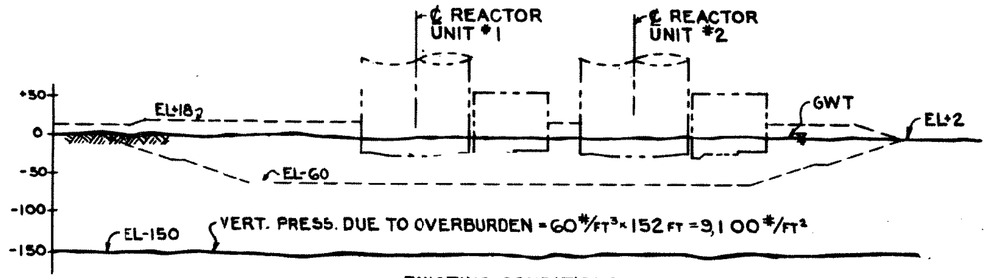
and

$$\bar{c} = \frac{Q}{0.0556 + 0.0218L}$$

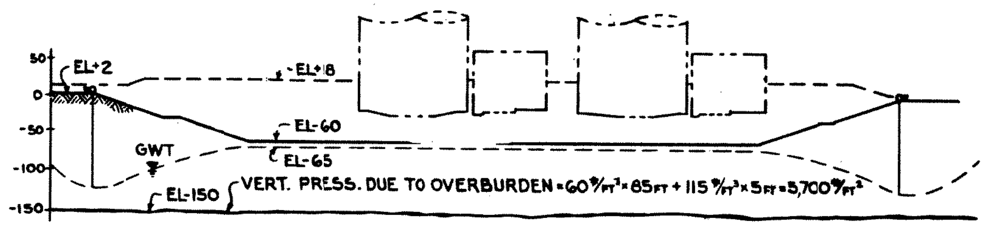
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

APPROXIMATE METHOD OF DETERMINING THE SHEAR STRENGTH OF COHESIVE SOIL

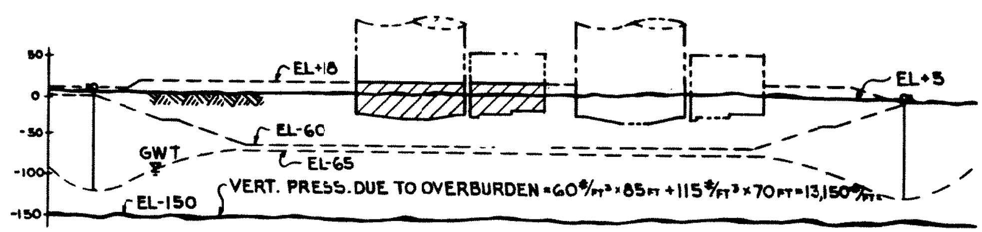
FIGURE 2.5-28



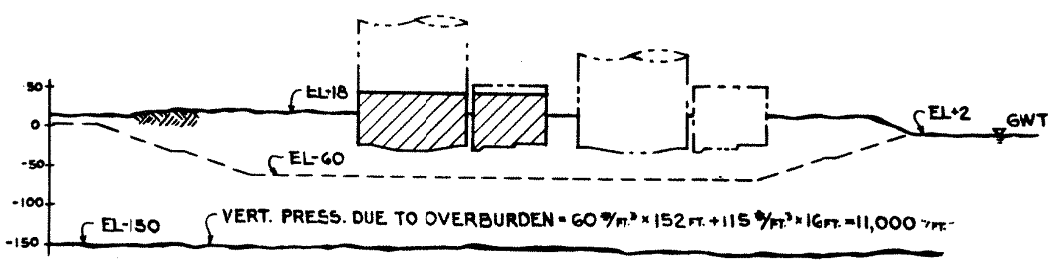
EXISTING CONDITIONS



1ST STAGE - DEWATERING & EXCAVATION TO EL-60



2ND STAGE - BACKFILLING TO ORIGINAL GRADE



3RD STAGE - PLANT BACKFILL TO EL+18

EXISTING STRESS CONDITIONS:

THE STRESSES AT THE MINUS 150 FOOT ELEVATION ARE DUE TO THE SUBMERGED WEIGHT OF THE OVERLYING SANDY SOILS WHICH AMOUNT TO 152 FT. OF 60# SUBMERGED WEIGHT = 9,100#/FT.²

STRESS CONDITIONS AFTER DEWATERING & EXCAVATION:

THE DEWATERING WILL TEND TO INCREASE THE EFFECTIVE OVERBURDEN WEIGHT. HOWEVER, IF THE EXCAVATION CLOSELY FOLLOWS, THE NET EFFECT WILL BE A REDUCTION TO AN EFFECTIVE OVERBURDEN STRESS AT THE -150 ELEVATION OF 5,700 #/FT.².

STRESS CONDITIONS AFTER BACKFILLING TO ELEV. +5 :

THE BACKFILLING WILL BE PERFORMED WITH THE WATER TABLE HELD AT THE -65 FT. ELEVATION. THUS INCREASING THE EFFECTIVE OVERBURDEN WEIGHT & STRESS CONDITIONS AT THE -150 FT. ELEVATION TO 13,150 #/FT.². THIS VALUE IS 3000 #/FT.² HIGHER THAN THESE SOILS HAVE EVER EXPERIENCED.

STRESS CONDITIONS AFTER BACKFILLING PLANT AREA TO ELEV +18:

THE PLANT AREA BACKFILLING WILL BE PERFORMED WITH THE WATER TABLE ALLOWED TO SEEK ITS NORMAL LEVEL. THUS THE EFFECT IS A REDUCTION OF THE OVERBURDEN WEIGHT & THE STRESSES AT THE -150 FT. LEVEL WILL BE 11,000 #/FT.².

LEGEND:

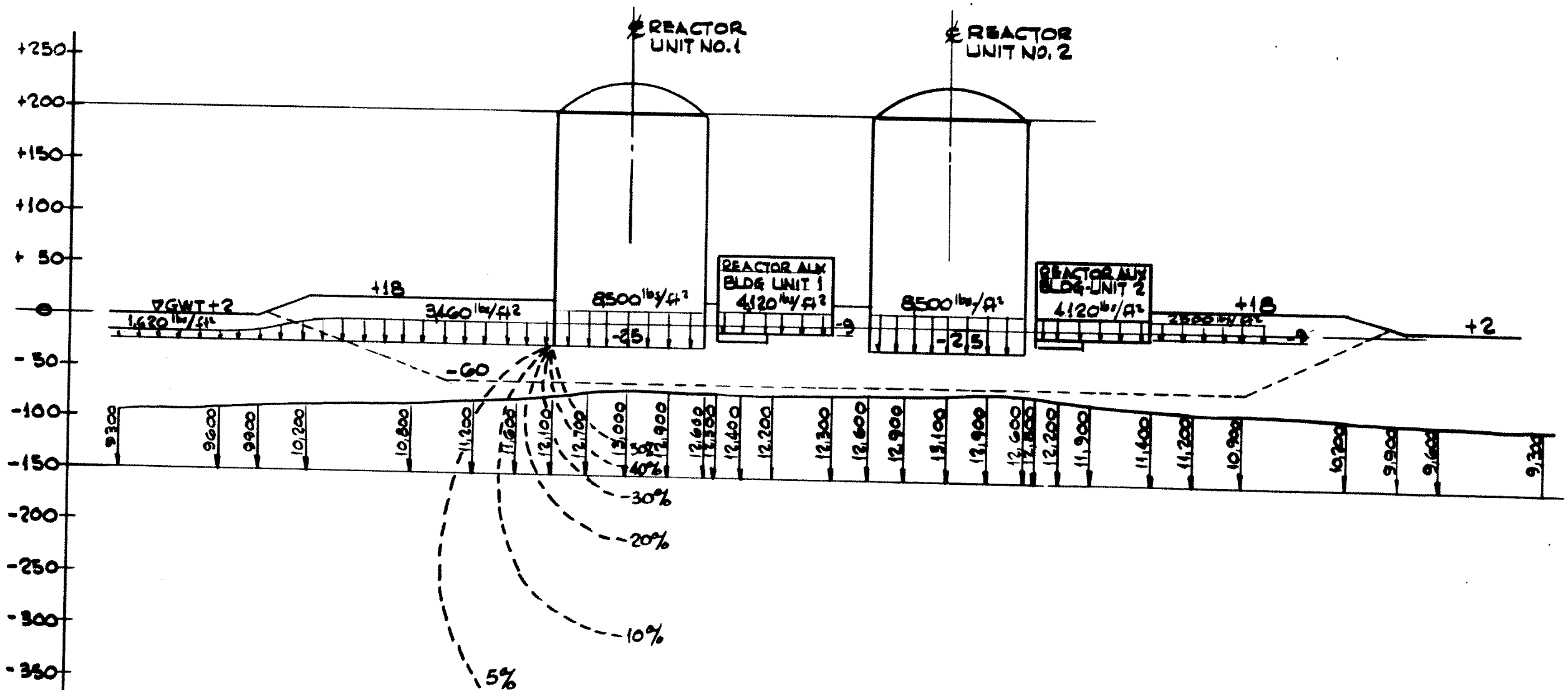
GWT = GROUND WATER TABLE

SK.2998-CP-408

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

EXCAVATION AND BACKFILL PROCEDURES

FIGURE 2.5-29



FINAL LOADING CONDITION

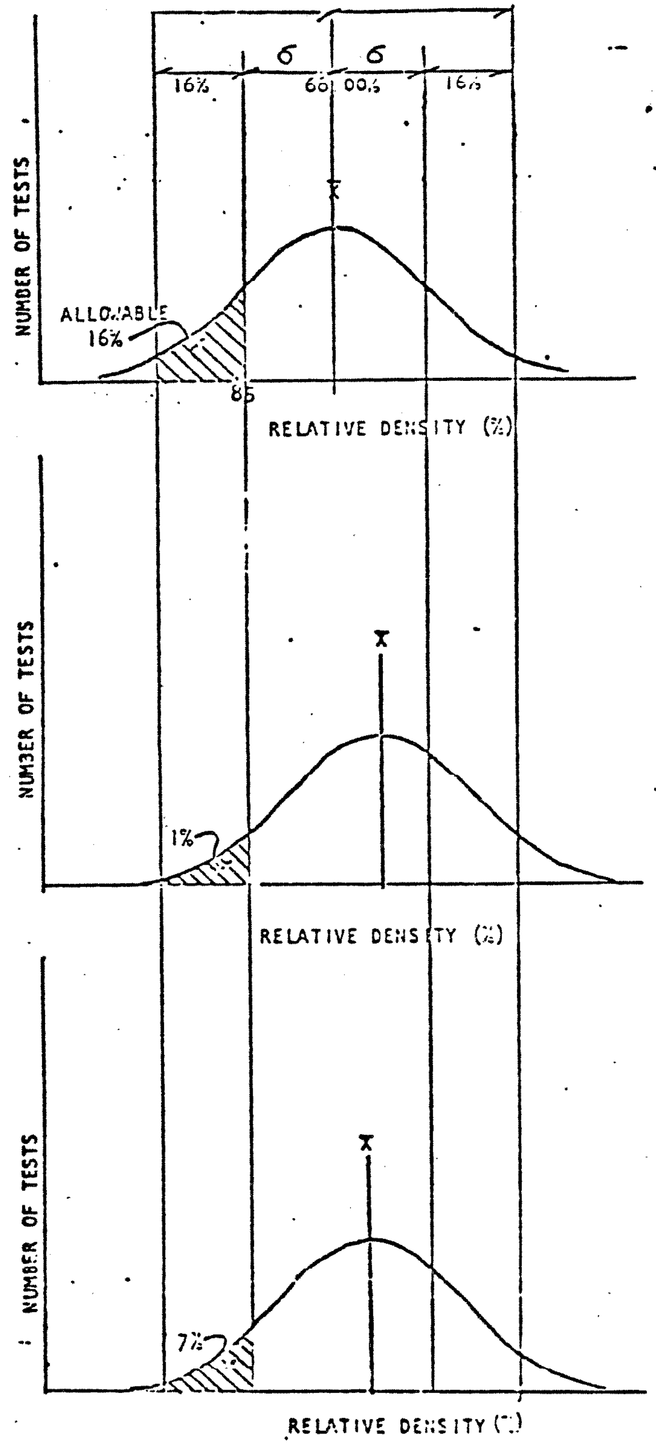
FINAL STRESS CONDITIONS AT THE MINUS 150 FOOT ELEVATION

WITH THE REACTOR FOUNDATION AT MINUS 25 ELEVATION, THE EFFECT IS THAT OF A FLOATING FOUNDATION THUS THE INCREASE IN OVERBURDEN STRESSES AT THE MINUS 150 ELEVATION IS MINIMAL, IN FACT THE ACTUAL LOADINGS DEVELOPED ARE SUCH THAT THE STRESSES ARE WITHIN THE VALUES WHICH HAVE ALREADY PRECONSOLIDATED THESE SOILS TO 13,000 lbs/ft². THIS TYPE OF A STRESS CONDITION SHOULD RESULT IN VERY MINOR SETTLEMENTS AND THESE SHOULD BE LIMITED TO THE DENSE SAND STRATA ABOVE THE MINUS 150 ELEVATION. THESE SETTLEMENTS WILL BE RAPID AND WILL BE COMPLETED DURING CONSTRUCTION.

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

FOUNDATION STUDY

FIGURE 2.5-30



(a)

SPECIFICATION CURVE (STANDARD)
$\bar{X}$ = (85 + 16)%
UNIVERSE = ($\bar{X}$ - 2б)% to ($\bar{X}$ + 2б)%

$\bar{X}$ = MEAN OR AVERAGE RELATIVE DENS
б = STANDARD DIVIATION

(b)

STUDY # 4
$\bar{X}$ = 93.21%
б = 8.95%
UNIVERSE = 80.31% to 1.6.21%

NOTE: DIAGRAMS NOT TO SCALE

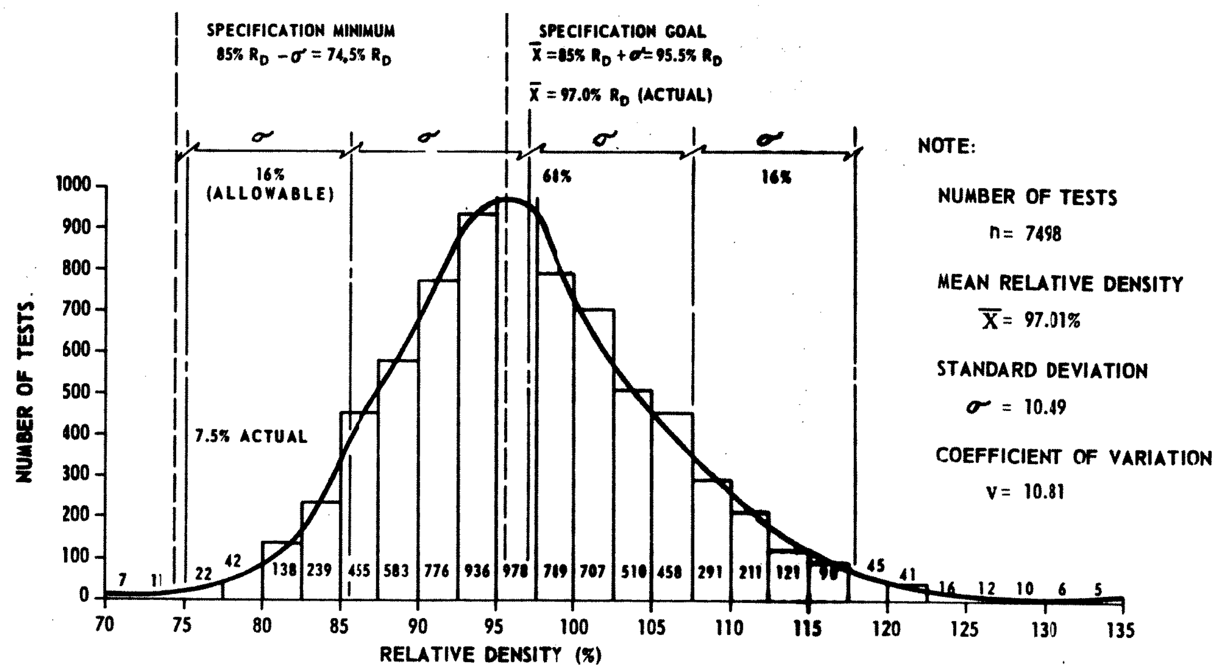
(c)

SUMMARY STUDY
$\bar{X}$ = 98.01%
б = 10.10%
UNIVERSE = 77.81% to 118.21%

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

STATISTICAL ANALYSES

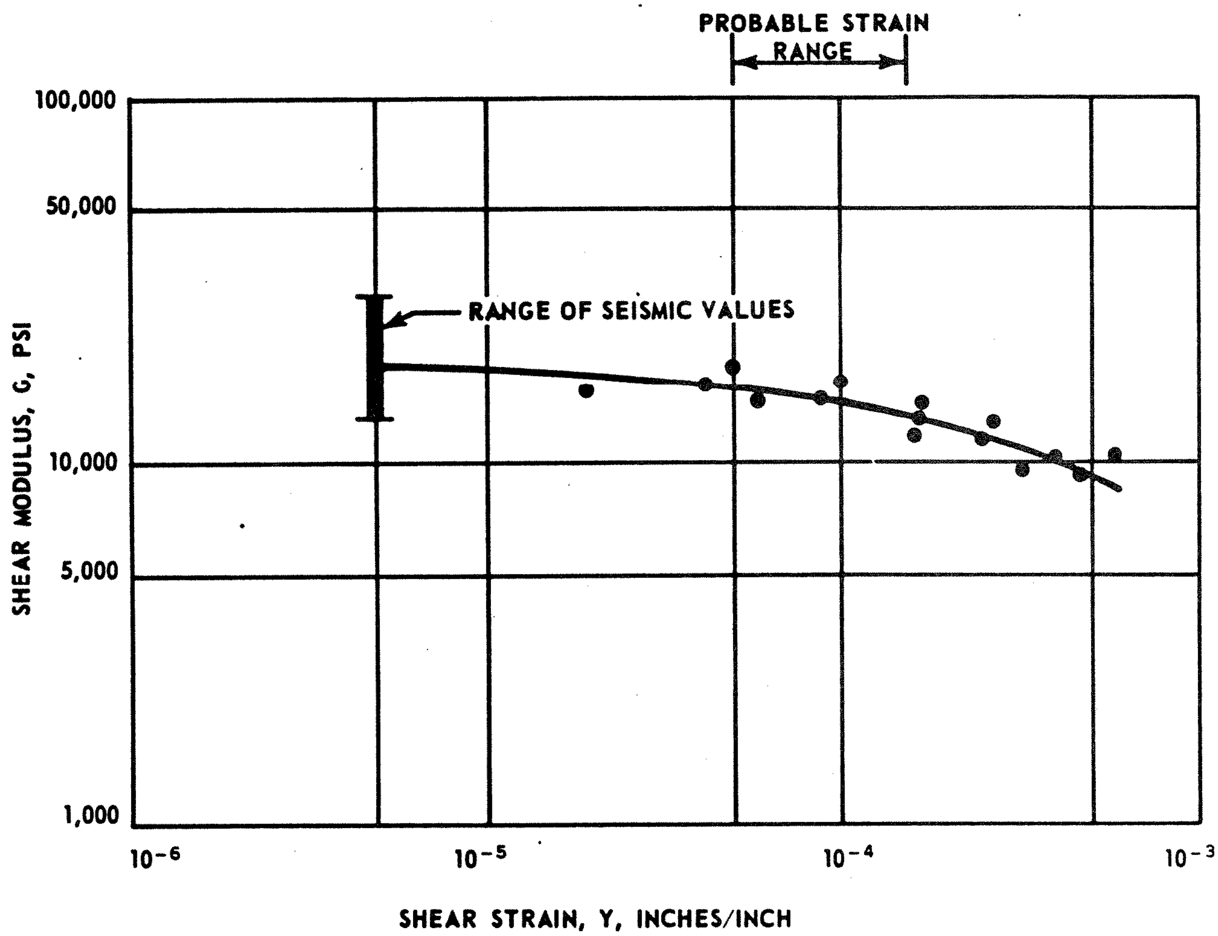
FIGURE 2.5-31



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SUMMARY STATISTICAL ANALYSES
CLASS 1 MATERIAL

FIGURE 2.5-31a



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SHEAR MODULUS VS. SHEAR STRAIN

FIGURE 2.5-32

SAND

RELATIVE DENSITY	0 (VERY LOOSE)	15 (LOOSE)	35 (MEDIUM)	65 (DENSE)	85 (VERY DENSE)	100
ϕ = ANGLE OF INTERNAL FRICTION (IN DEGREES)		29°	30°	36°	41°	
N* = STANDARD PENETRATION	0	4	10	30	50	
G MAX** = (FIELD VALUE PSF) SHEAR MODULUS.		900,000	1,600,000	2,500,000	3,000,000	
FOUNDATION CONDITION	BELOW AVERAGE (0–35)		AVERAGE (35–85)		ABOVE AVERAGE (85–100)	

* AVERAGE N VALUES SINCE N VARIES WITH DEPTH.

** G VALUE FOR SAND FOR DEPTH APPROXIMATELY 50 FT.

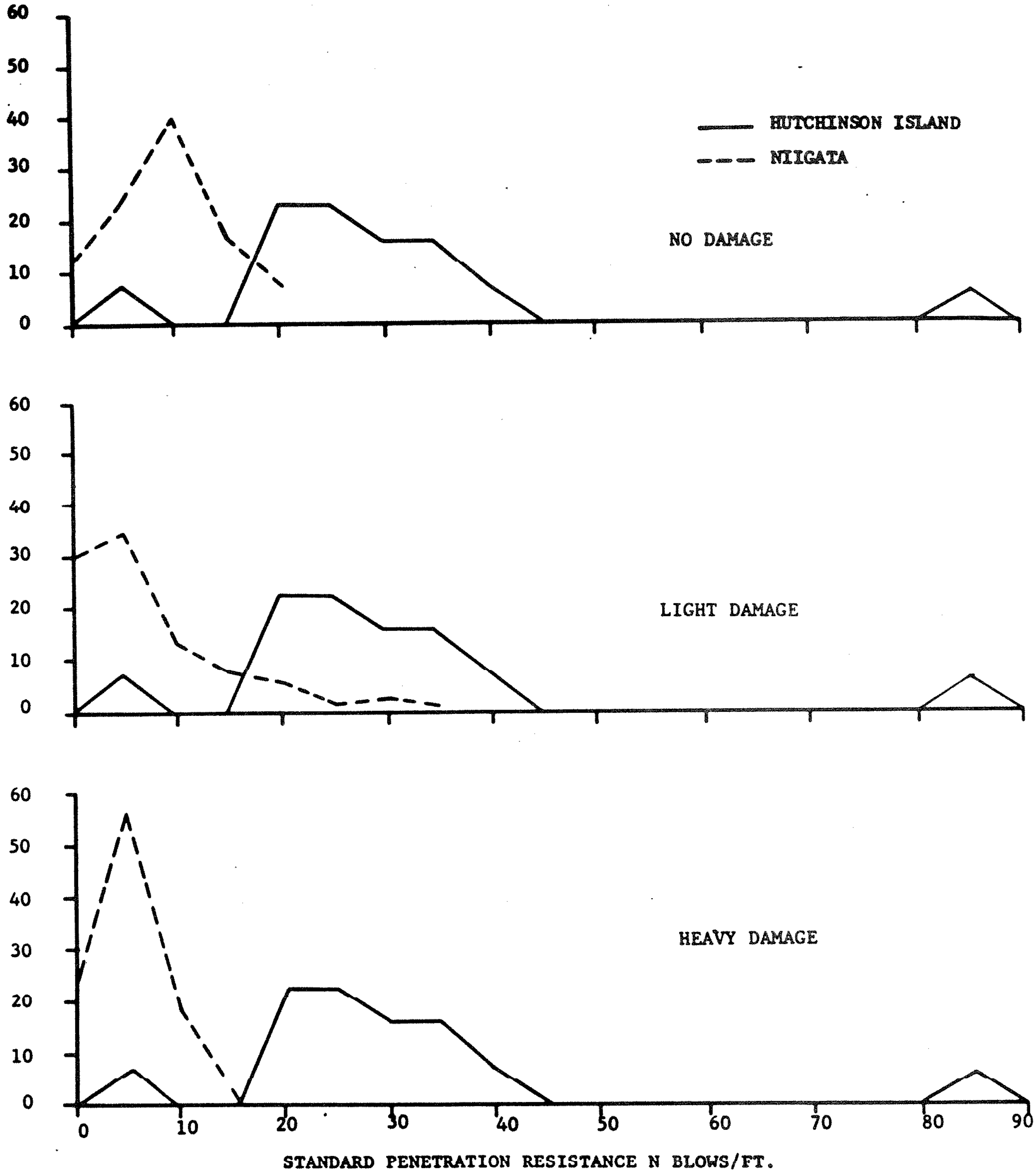
REF:- 1. TERZAGHI, K. AND PECK, R.B., "SOILS MECHANICS IN ENGINEERING PRACTICE." JOHN WILEY & SONS INC., MARCH 1968 PAGE 341.

2. SEED, H.B. AND IDRISS, I.M. "SOIL MODULI AND DAMPING FACTORS FOR DYNAMIC RESPONSE ANALYSES," EERC REPORT 70-10, COLLEGE OF ENGINEERING, UNIVERSITY OF CALIFORNIA, BERKELEY, DEC. 1970 FIG. 5.

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SOIL QUALITY

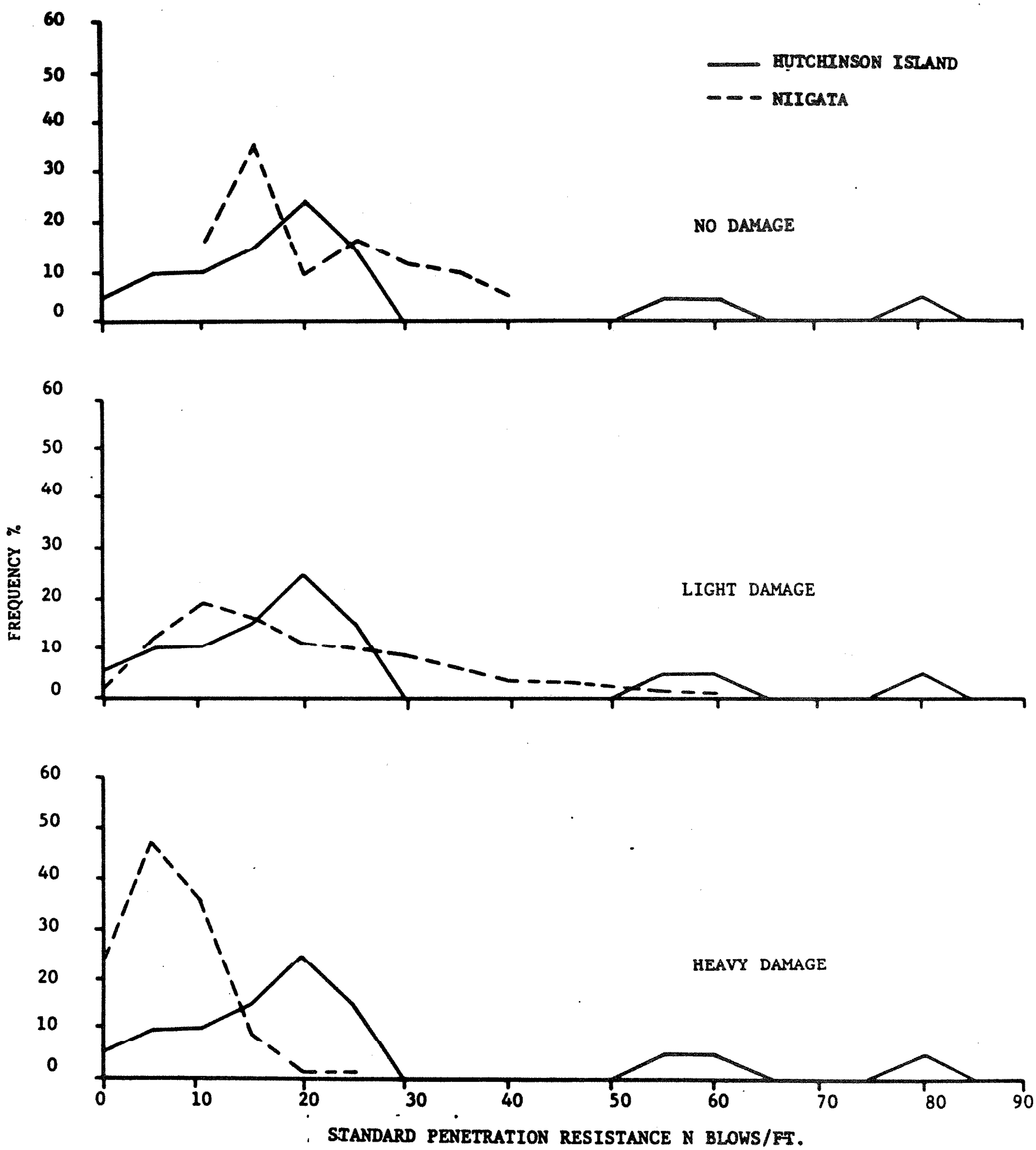
FIGURE 2.5-33



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

FREQUENCY DISTRIBUTION OF PENETRATION RESISTANCE AT NIIGATA AND THE PLANT SITE 2-5 METERS 7-16 FEET

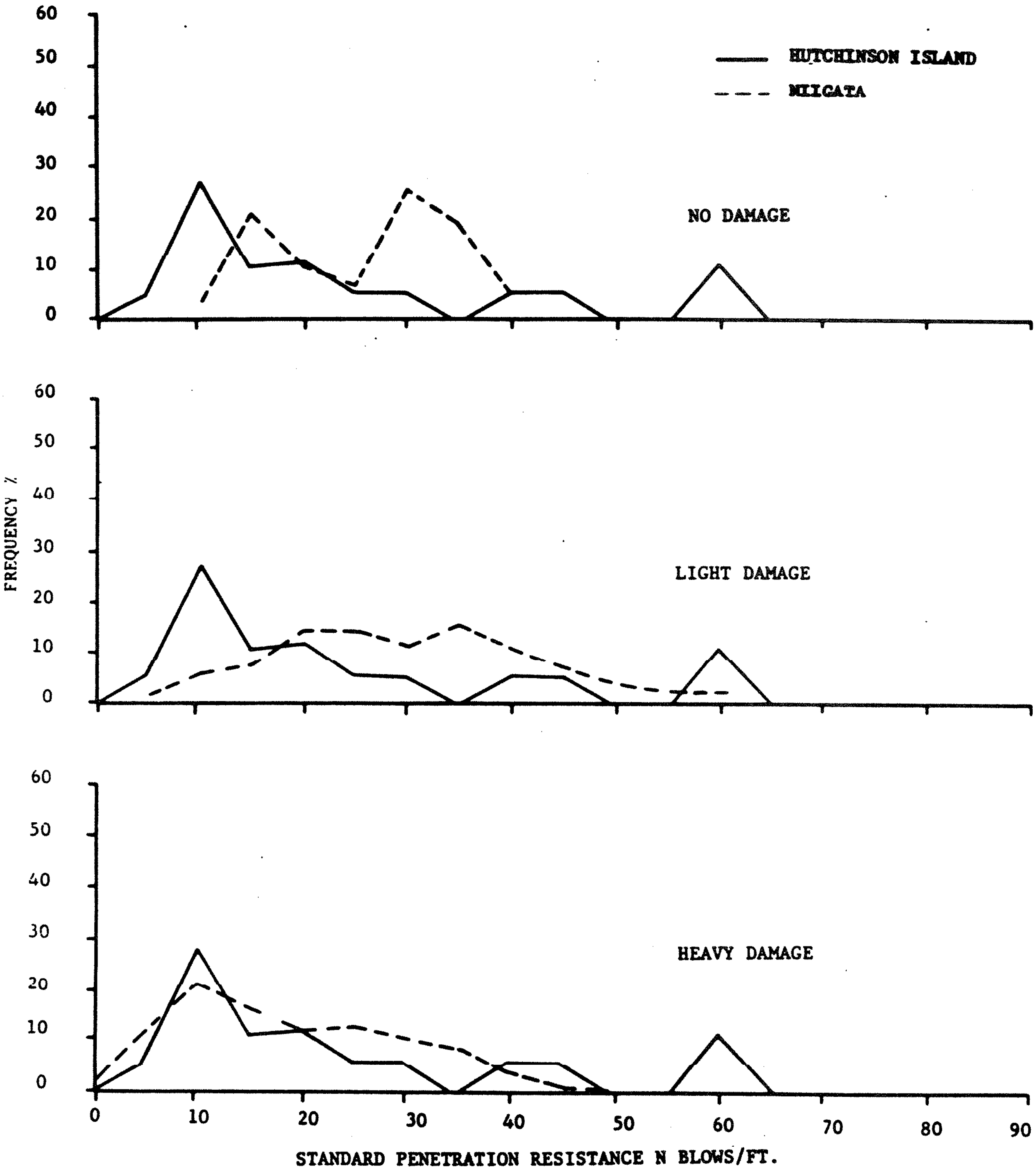
FIGURE 2.5-34



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

FREQUENCY DISTRIBUTION OF PENETRATION RESISTANCE AT NIIGATA AND THE PLANT SITE 5–10 METERS 16–33 FEET

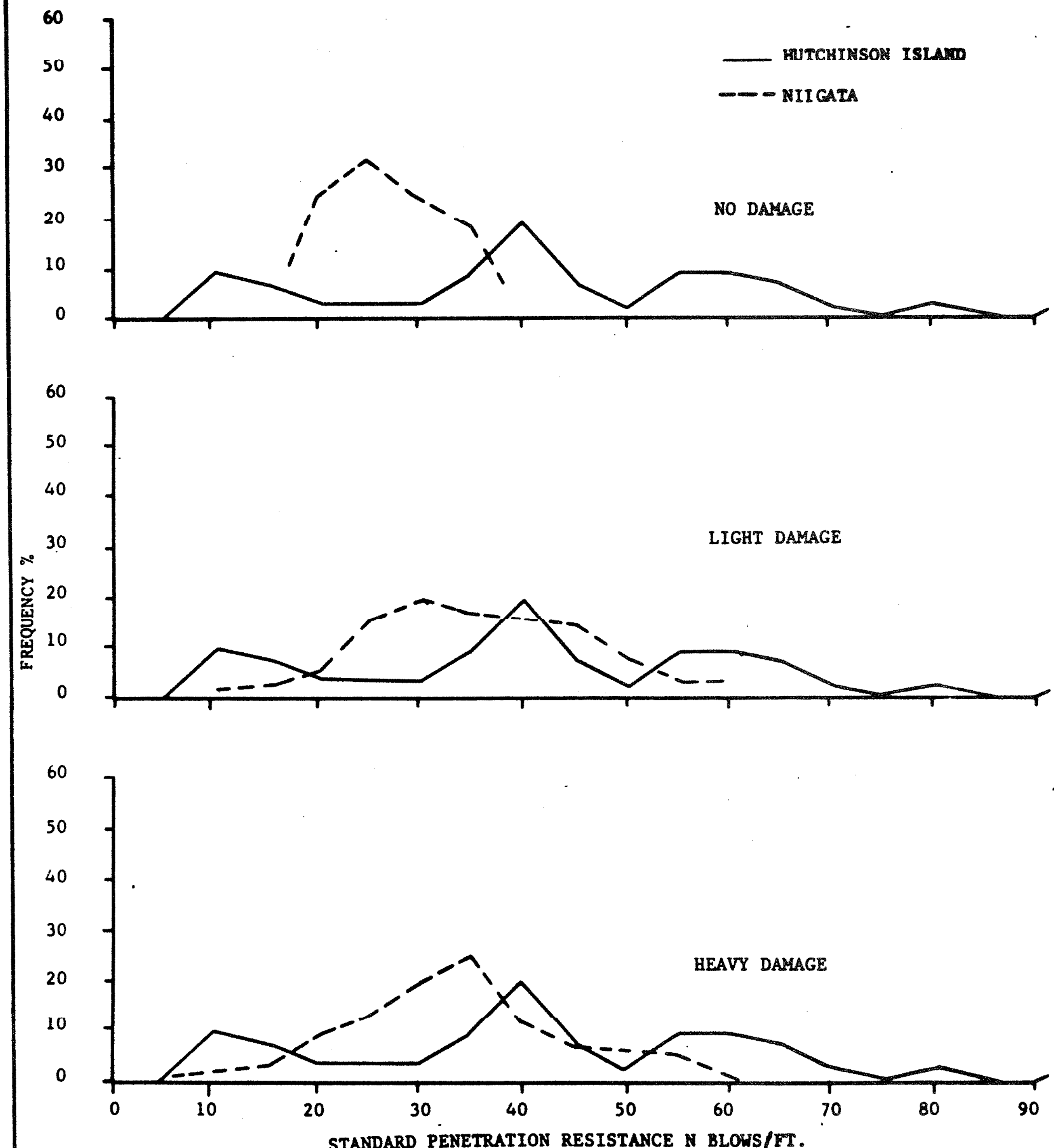
FIGURE 2.5-35



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

FREQUENCY DISTRIBUTION OF PENETRATION RESISTANCE AT NIIGATA AND AT THE PLANT SITE
10–15 METERS 33–48 FEET

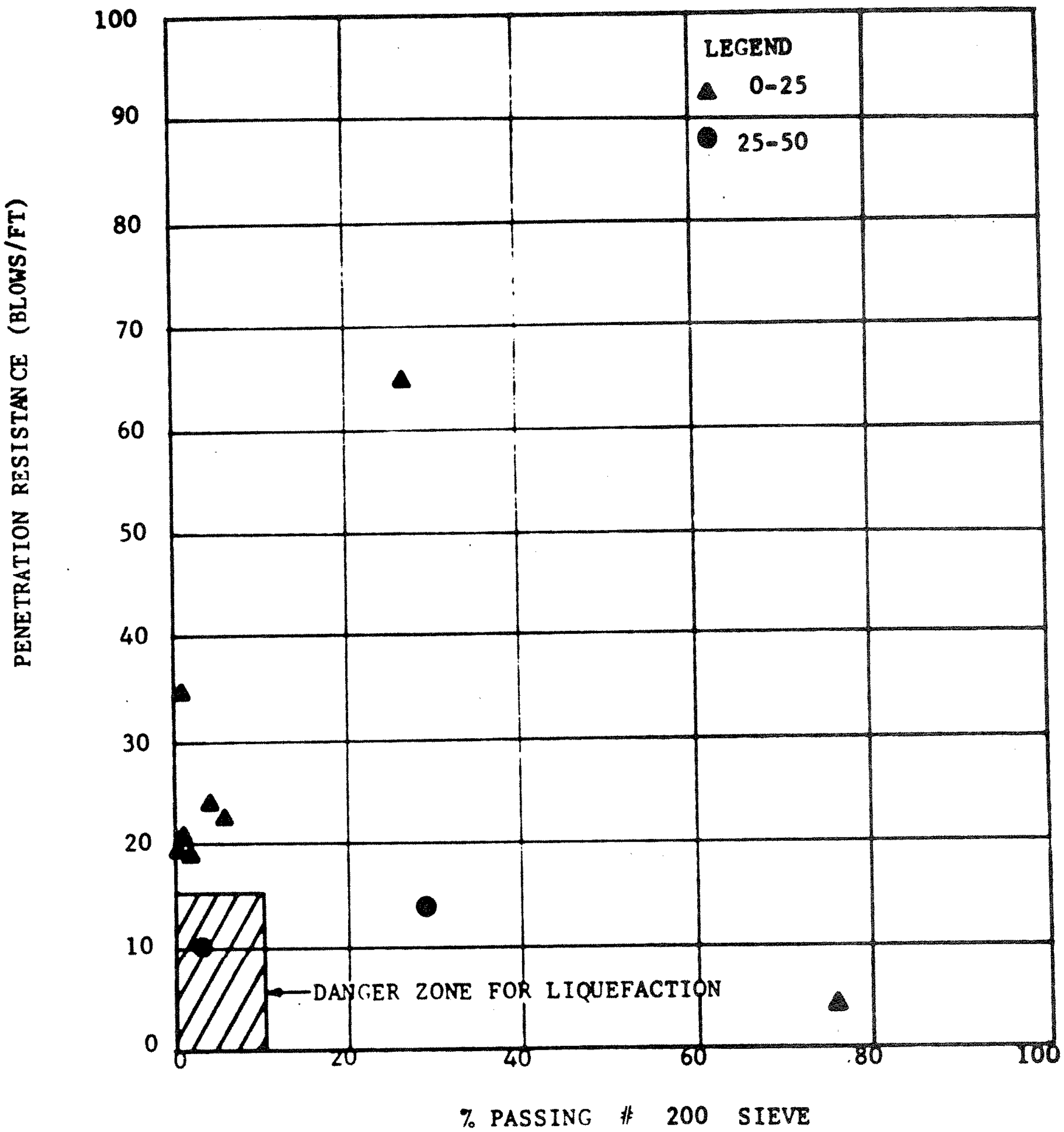
FIGURE 2.5-36



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

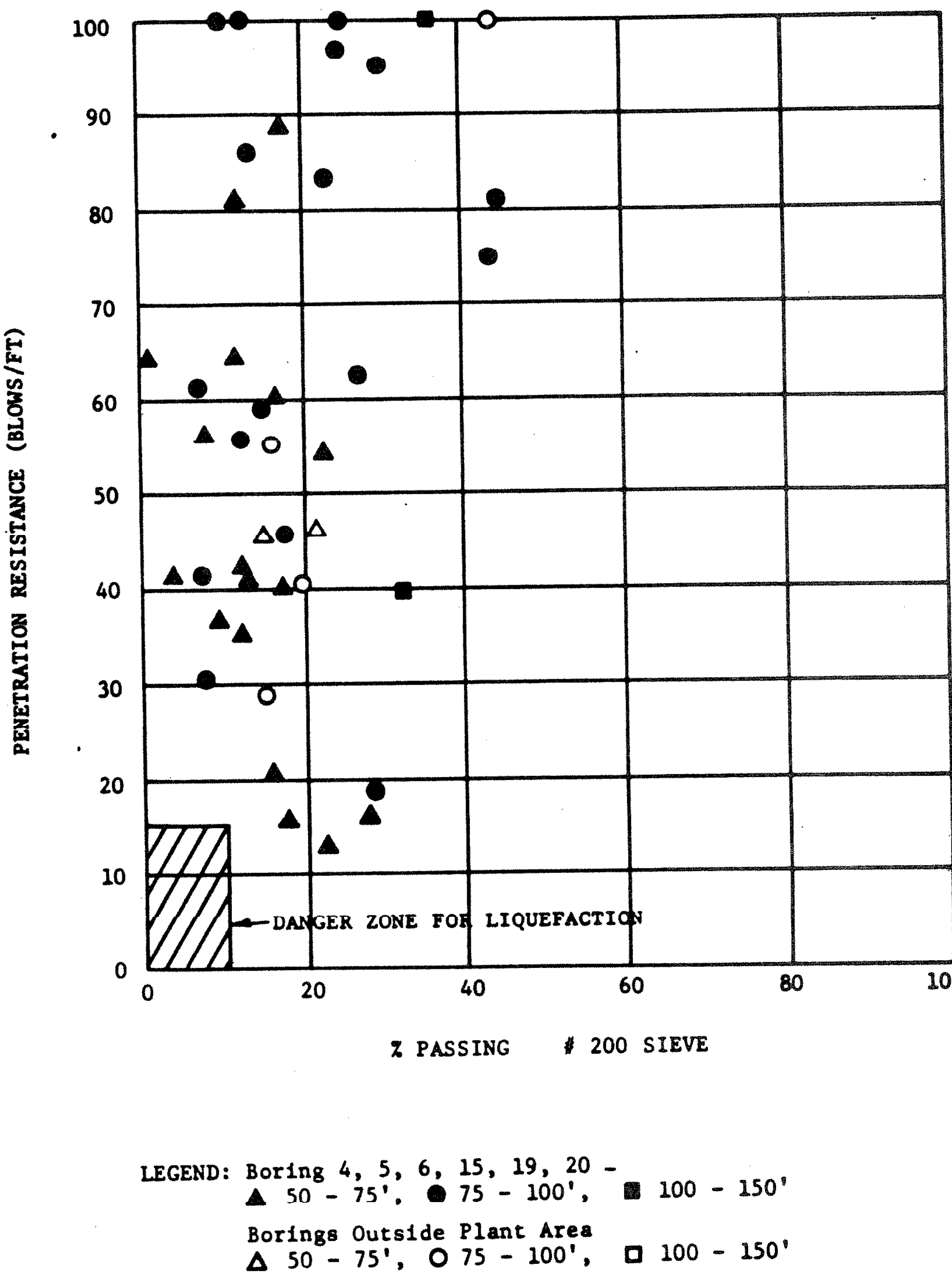
FREQUENCY DISTRIBUTION OF PENETRATION RESISTANCE AT NIIGATA AND AT THE PLANT SITE
15–20 METERS 48–66 FEET

FIGURE 2.5-37



DATA FROM BORINGS B-4,5,6,15

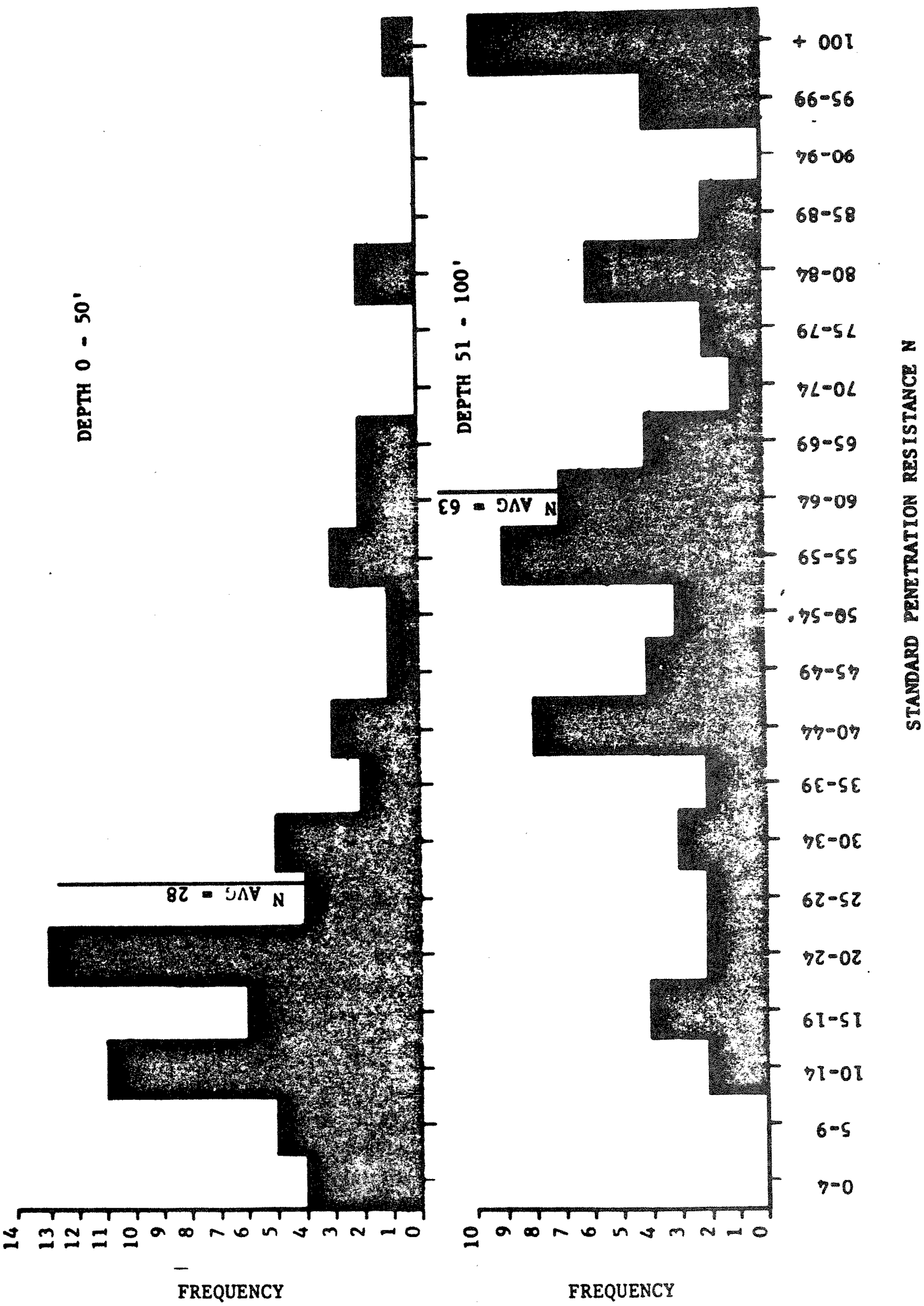
FLORIDA POWER & LIGHT COMPANY ST. LUCIE PLANT UNIT 1
PENETRATION RESISTANCE VS. PERCENT FINES FOR 0-50 FEET **FIGURE 2.5-38**



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

PENETRATION RESISTANCE VS. PERCENT FINES FOR 50-150 FEET

FIGURE 2.5-39



FREQUENCY OF OCCURANCE IN THE STANDARD PENETRATION TEST FOR BORINGS 4,5,6,14,15,19 AND 20

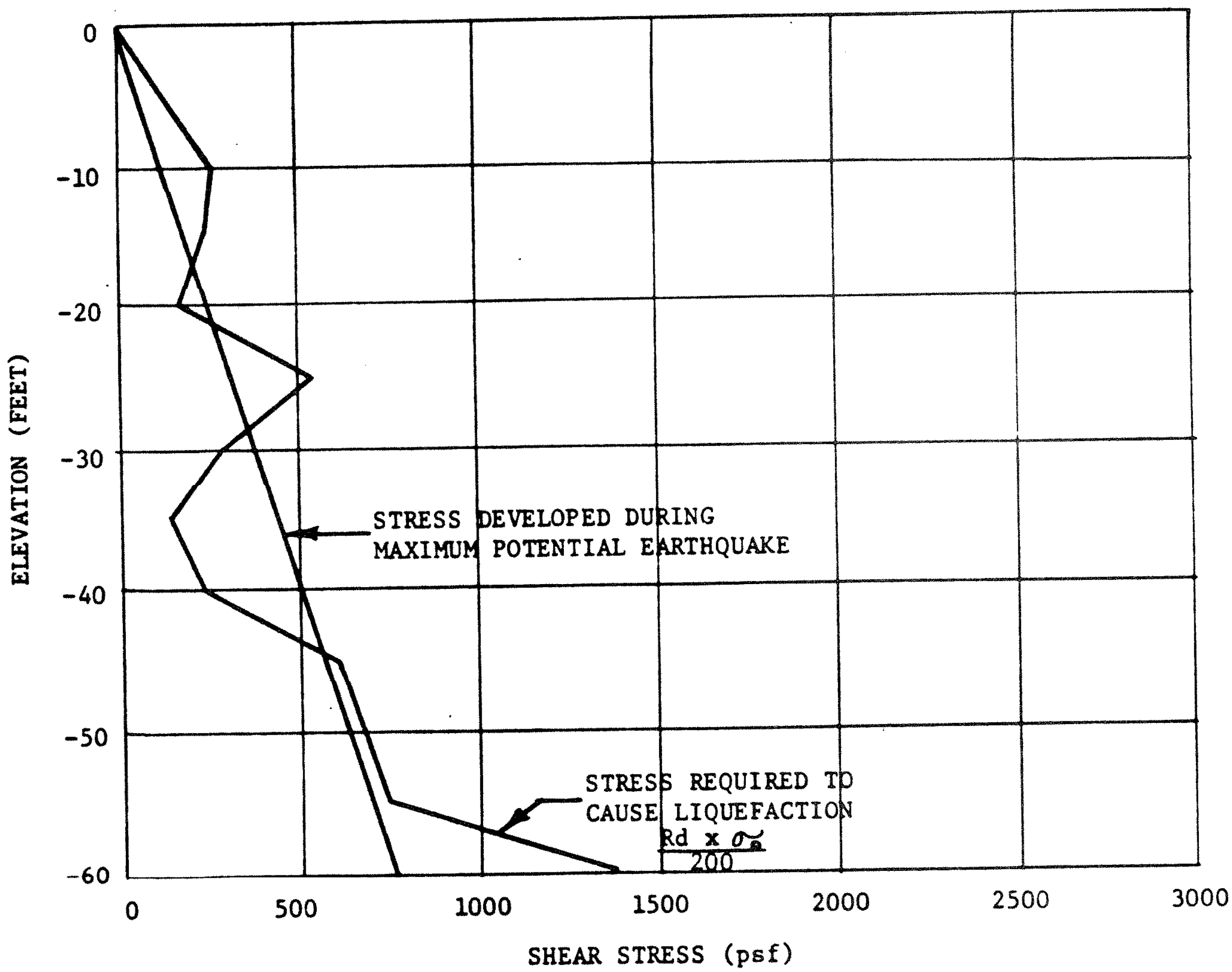
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

HISTOGRAMS OF PENETRATION RESISTANCE

FIGURE 2.5-40

NOTES 1. CRITICAL SHEAR STRESS COMPUTED ON THE BASIS OF THE MINIMUM PENETRATION RESISTANCE FOR THE EXISTING SOIL AT EACH LEVEL

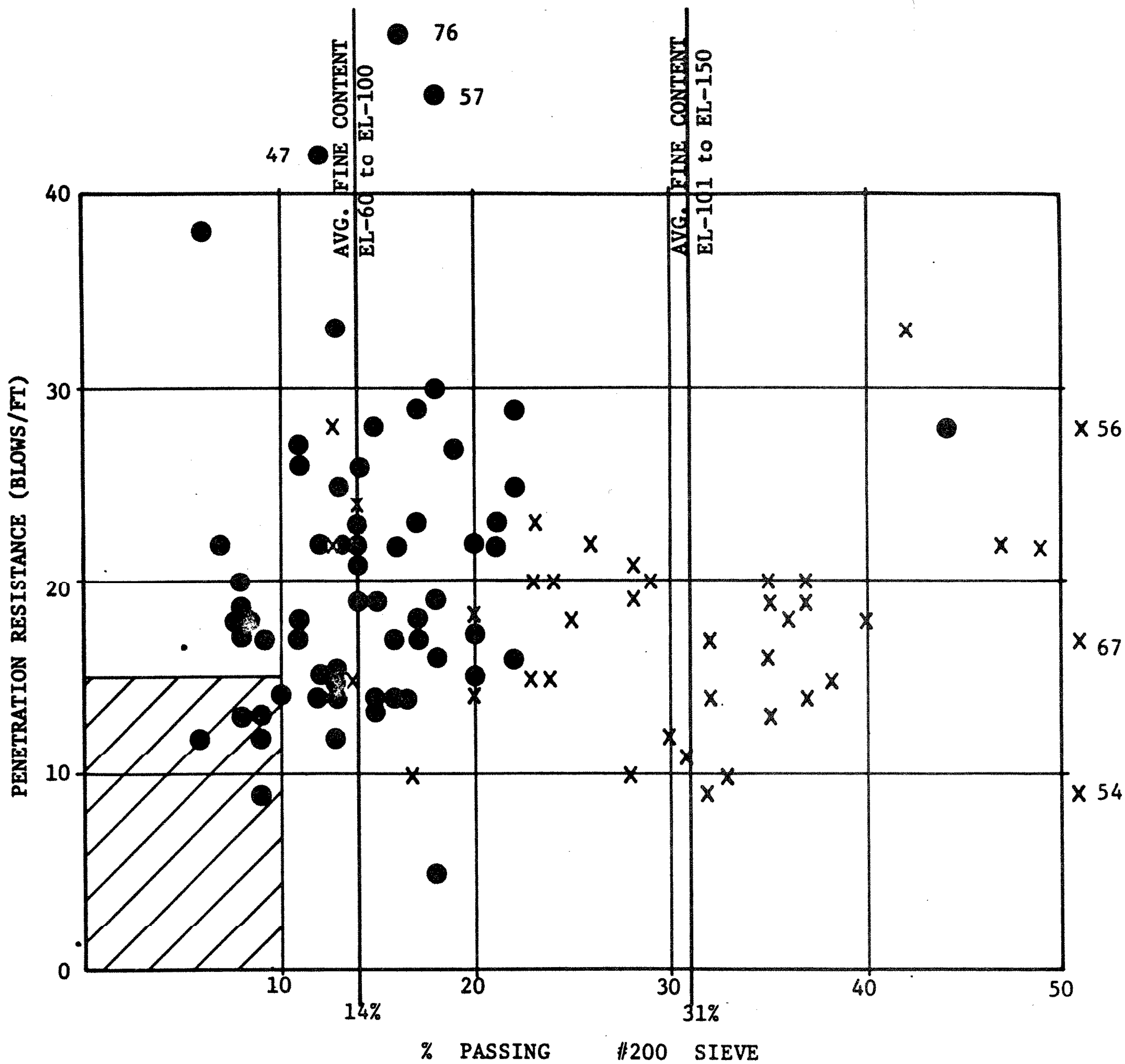
2. DATA FROM BORINGS 4,5,6,14,15,19,20



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

AVAILABLE SHEAR STRENGTH AND SHEAR STRESS CAUSED BY THE MAXIMUM POTENTIAL EARTHQUAKE AS A FUNCTION OF DEPTH

FIGURE 2.5-41

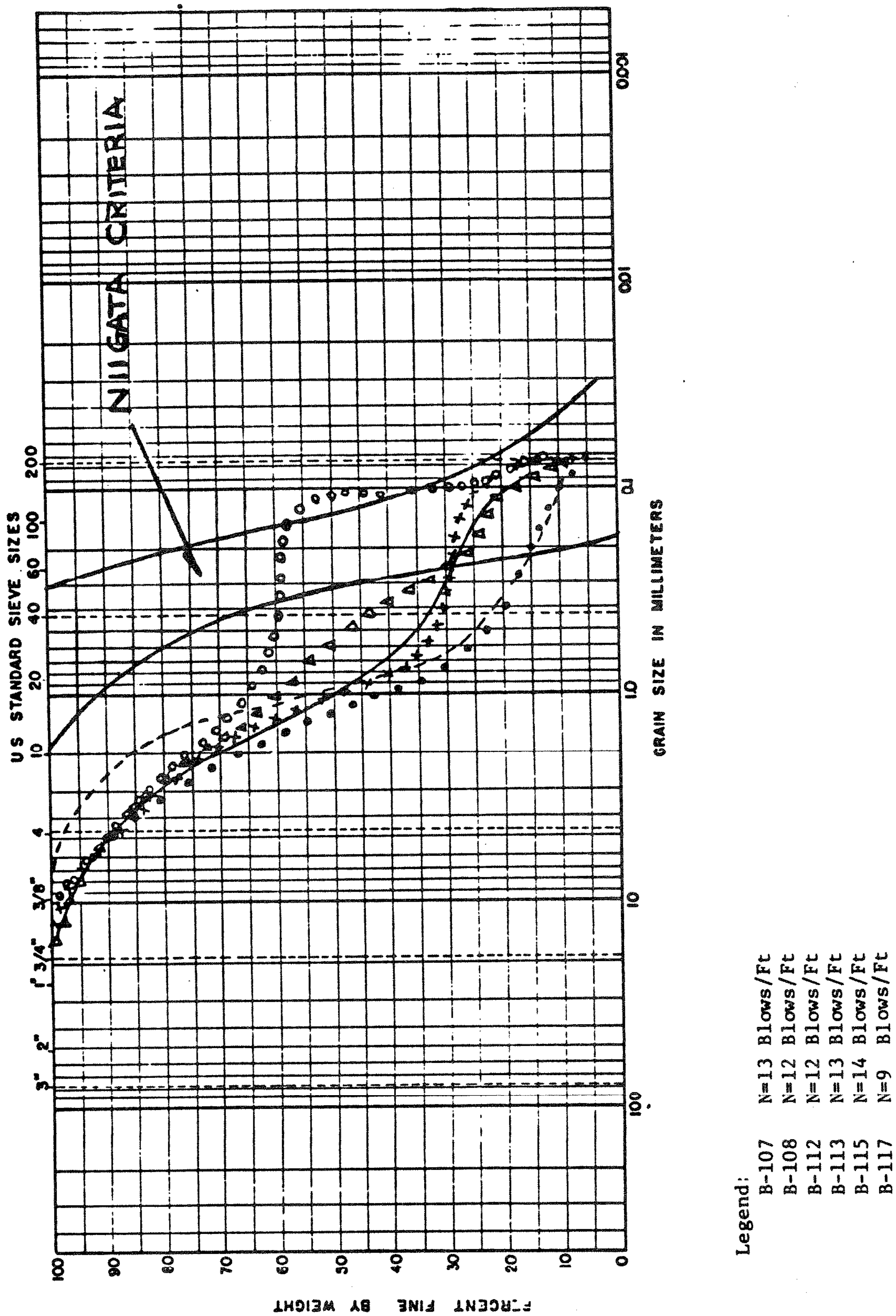


LEGEND:
● Indicates El-60 to El-100
X Indicates El-101 to El-150

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

PENETRATION DISTANCE VS. PERCENT FINES FOR EL-60 TO EL-150

FIGURE 2.5-42

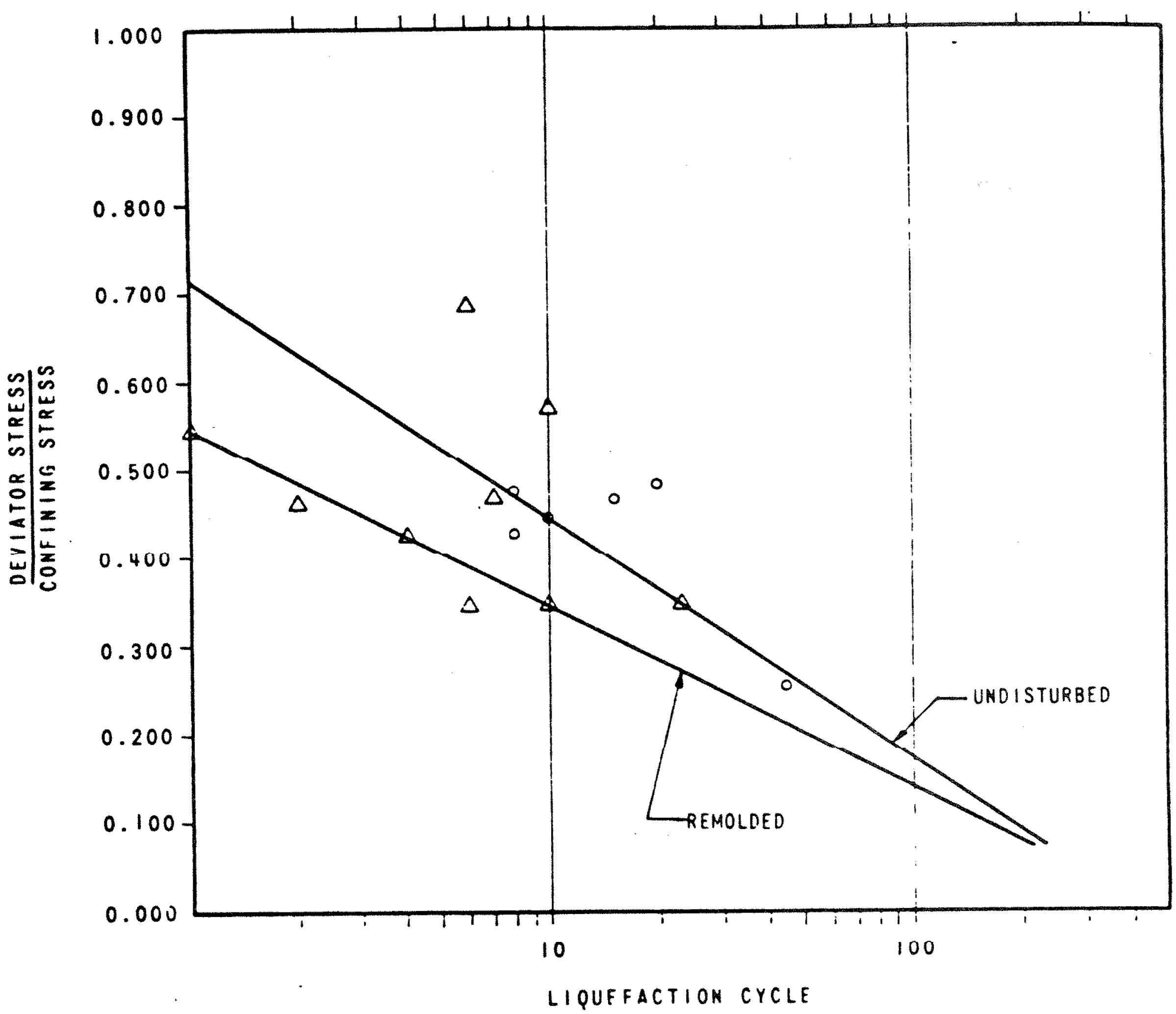


FLORIDA POWER & LIGHT COMPANY

ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTION

FIGURE 2.5-43



CONFINING STRESSES
BETWEEN 42 AND 72 PSI

LEGEND

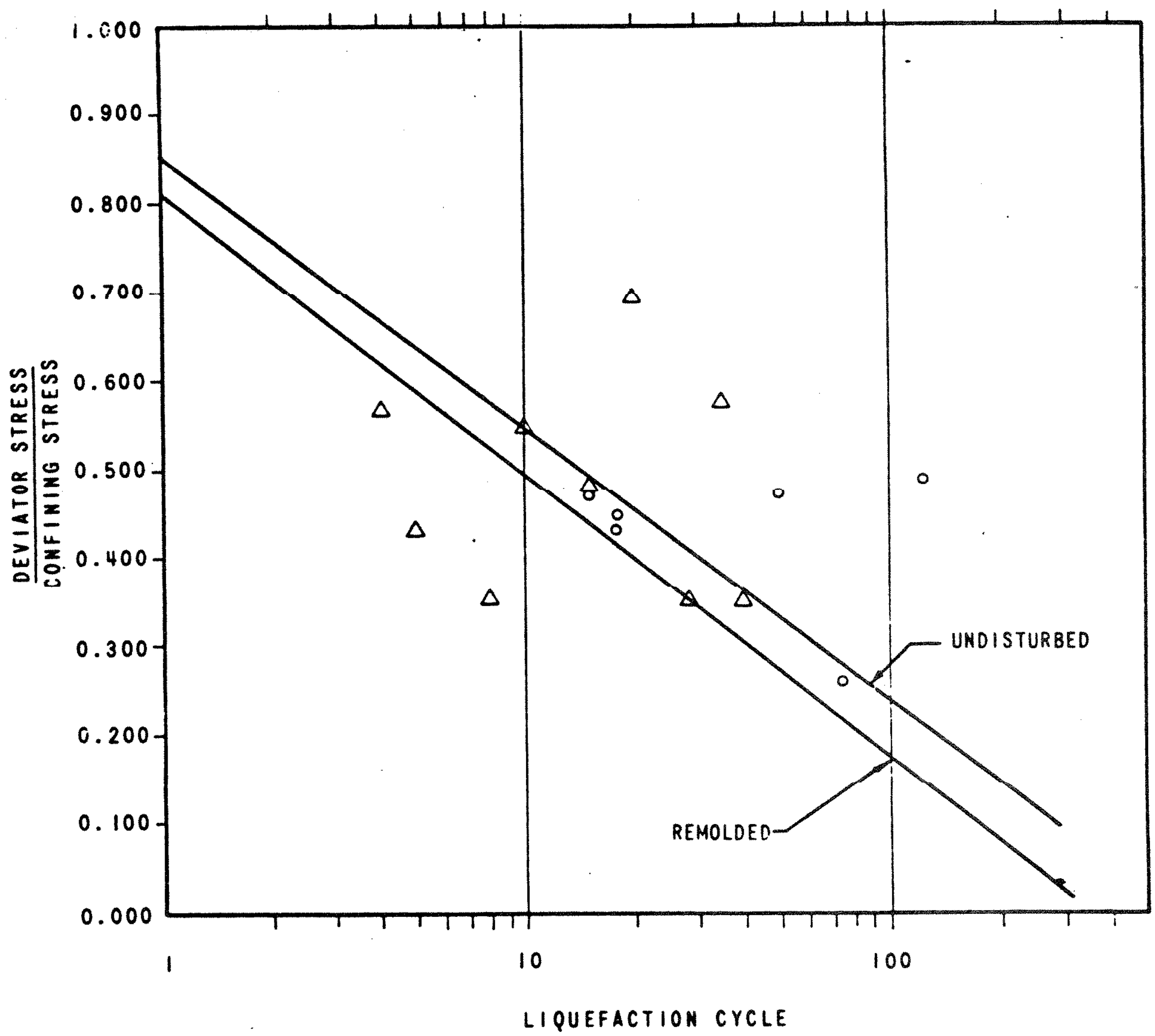
△ REMOLDED SAMPLE

○ UNDISTURBED SAMPLE

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

MOMENTARY LIQUEFACTION

FIGURE 2.5-44



CONFINING STRESSES
BETWEEN 42 AND 72 PSI

LEGEND

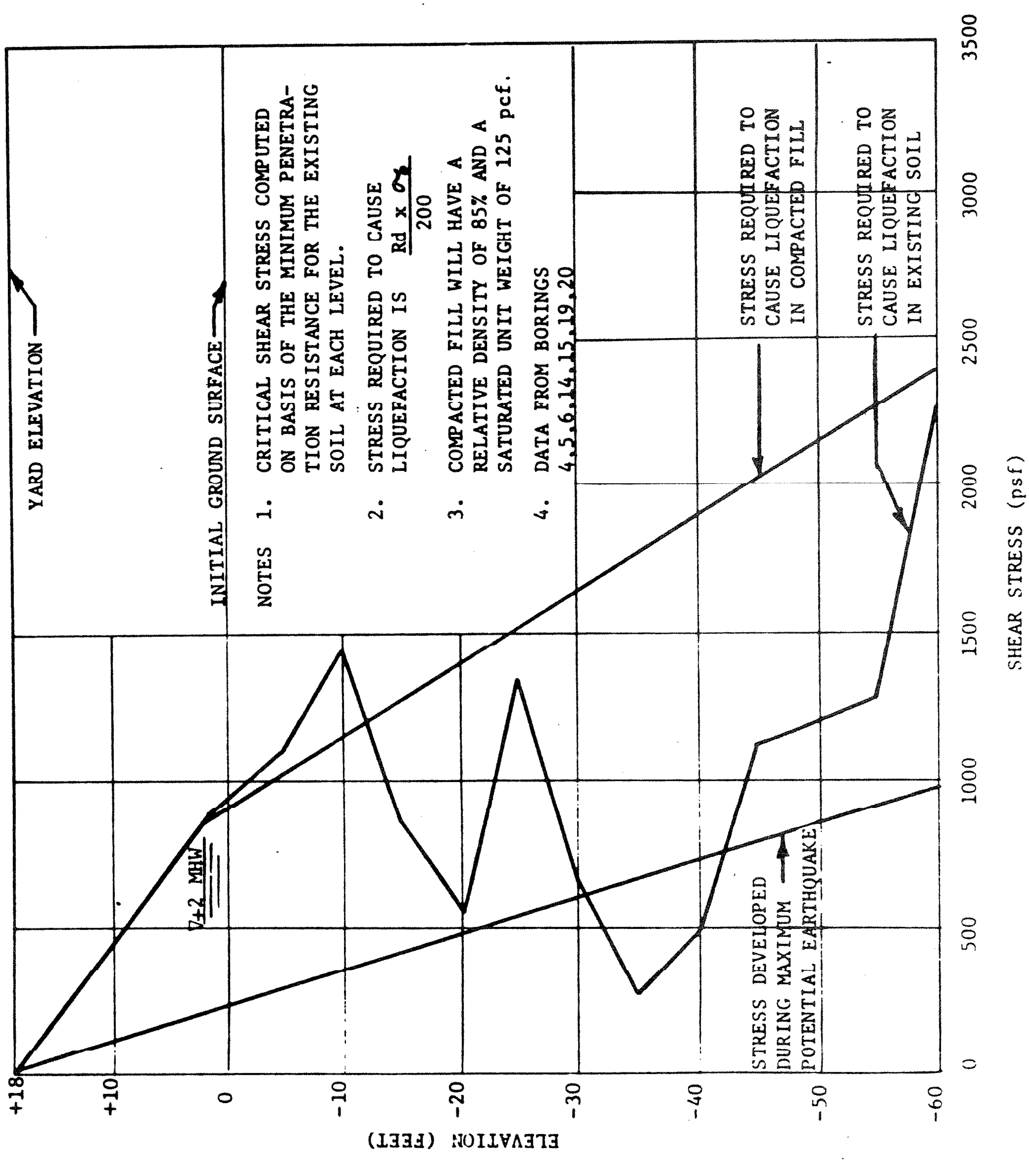
△ REMOLDED SAMPLE

o UNDISTURBED SAMPLE

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

LIQUEFACTION

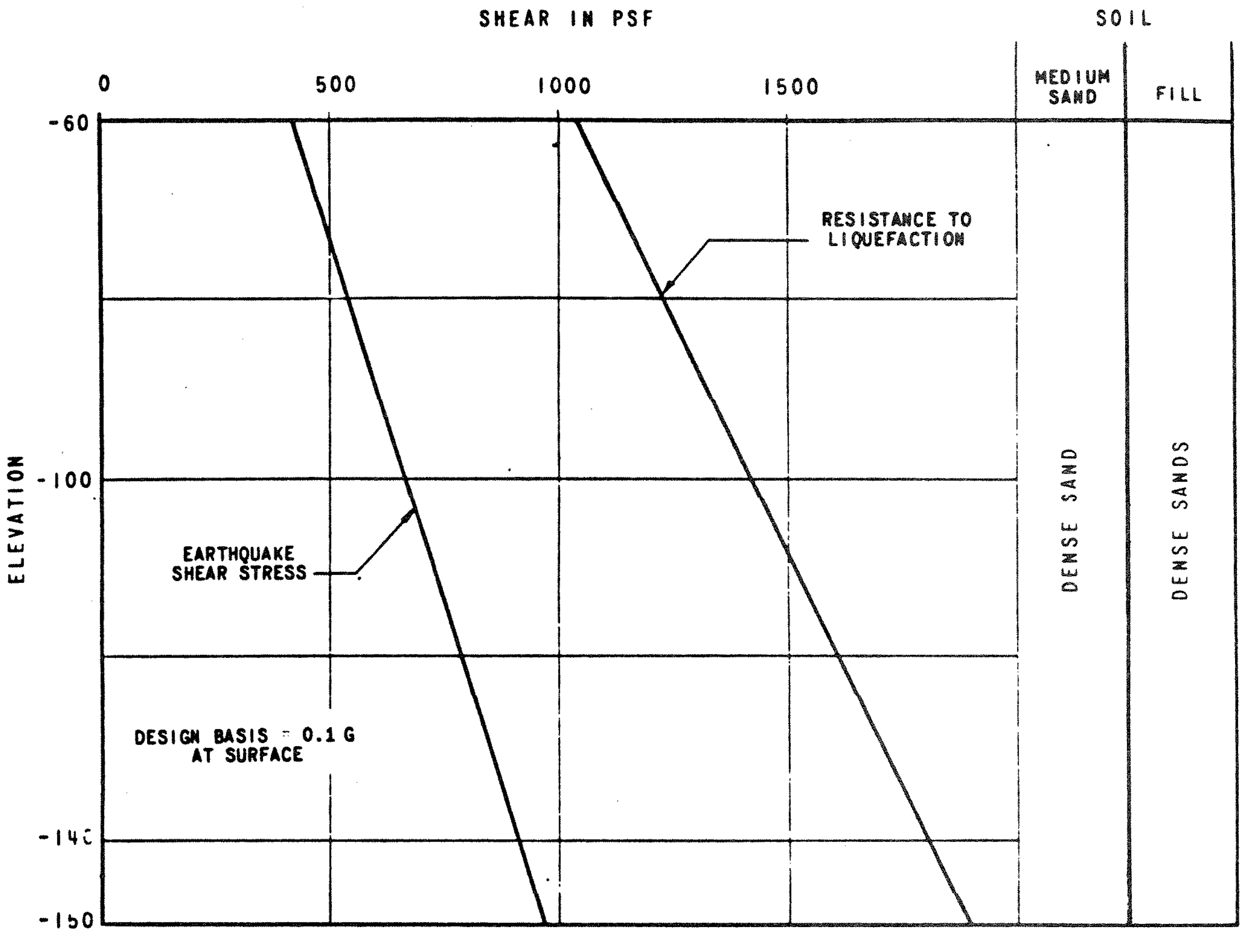
FIGURE 2.5-45



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SHEAR STRESS AND AVAILABLE SHEAR STRENGTH AS A FUNCTION OF DEPTH BEFORE AND AFTER REPLACEMENT

FIGURE 2.5-46



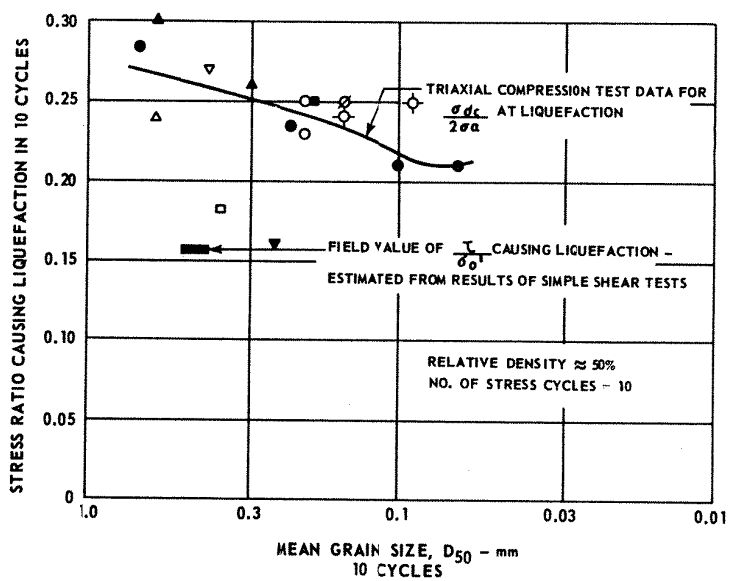
EARTHQUAKE SHEAR STRESS AND DYNAMIC RESISTANCE TO LIQUEFACTION
VIRGIN SOIL BELOW ELEVATION - 60'

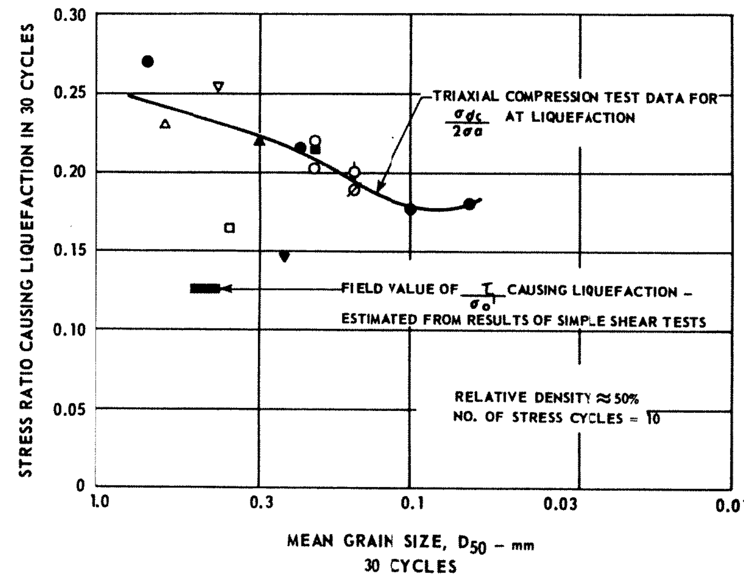
NOTE: BASED ON RESULTS OF UNDISTURBED SAMPLES AND 10 CYCLES OF STRONG MOTION.

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

DYNAMIC RESISTANCE TO LIQUEFACTION – VIRGIN SOIL

FIGURE 2.5-47





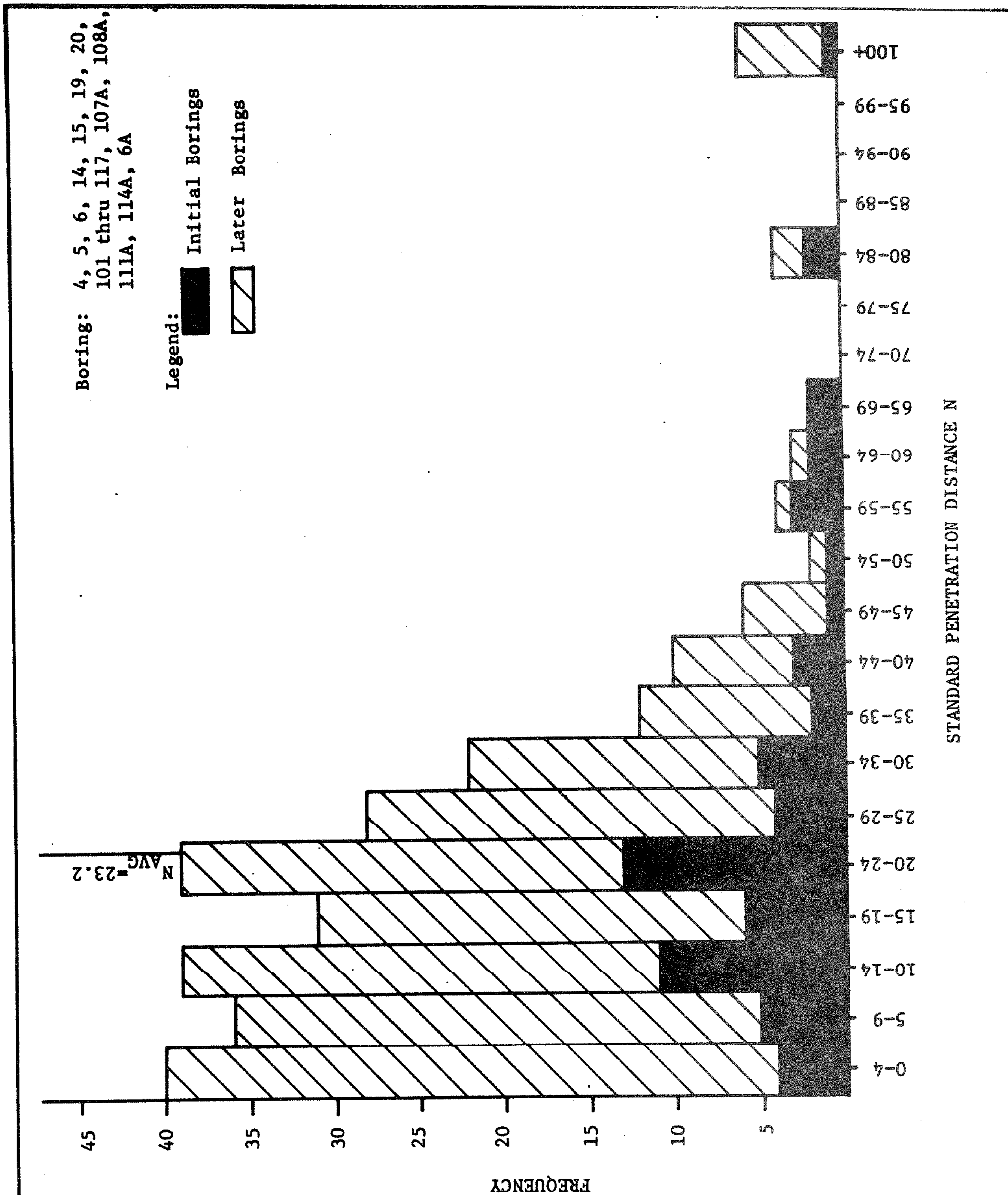
NOTE:

⌖ INDICATES ST. LUCIE TEST DATA USED BY SEED TO DEVELOP THIS RELATIONSHIP

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

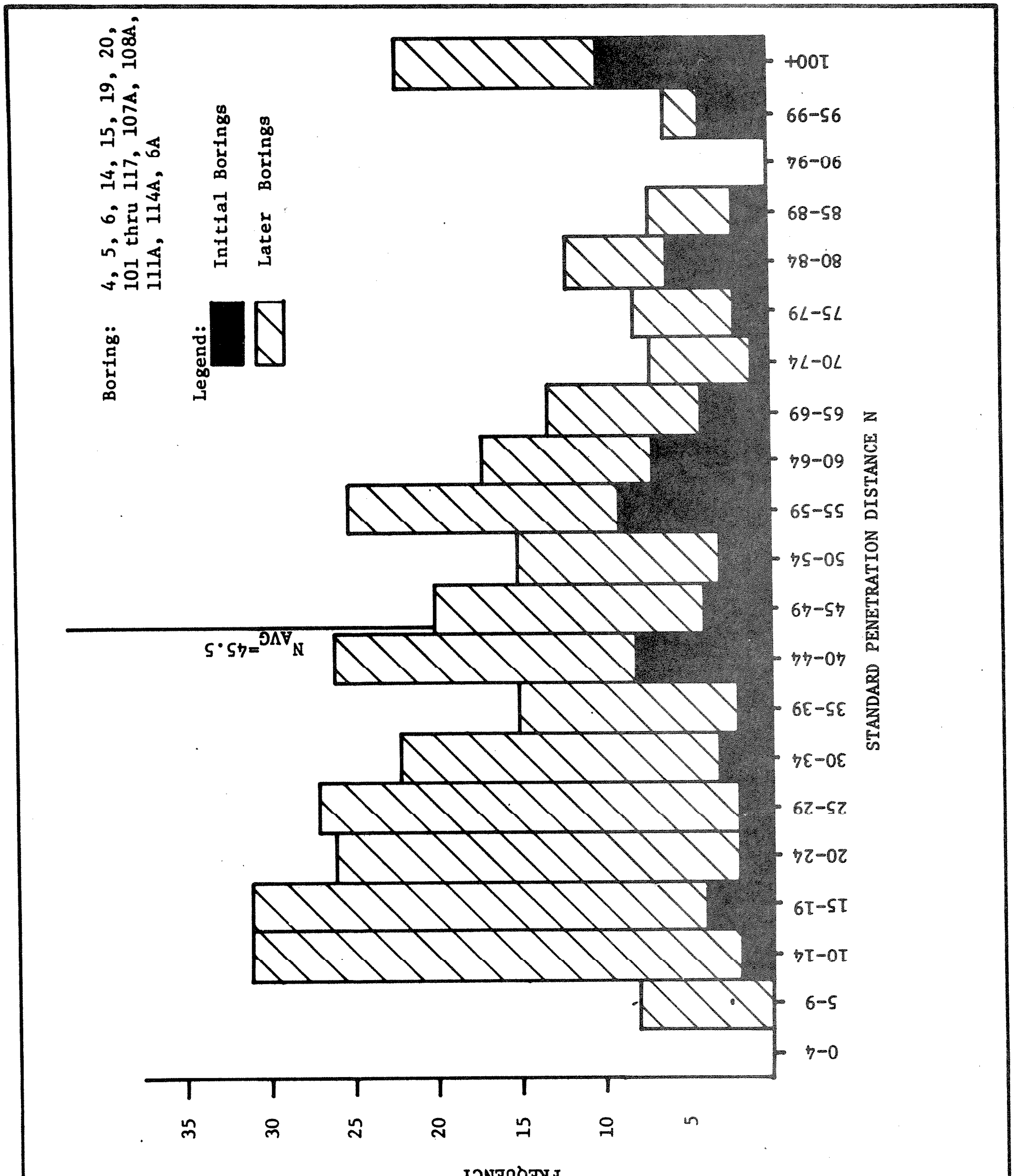
STRESS CONDITIONS CAUSING LIQUEFACTION OF SANDS

FIGURE 2.5-48



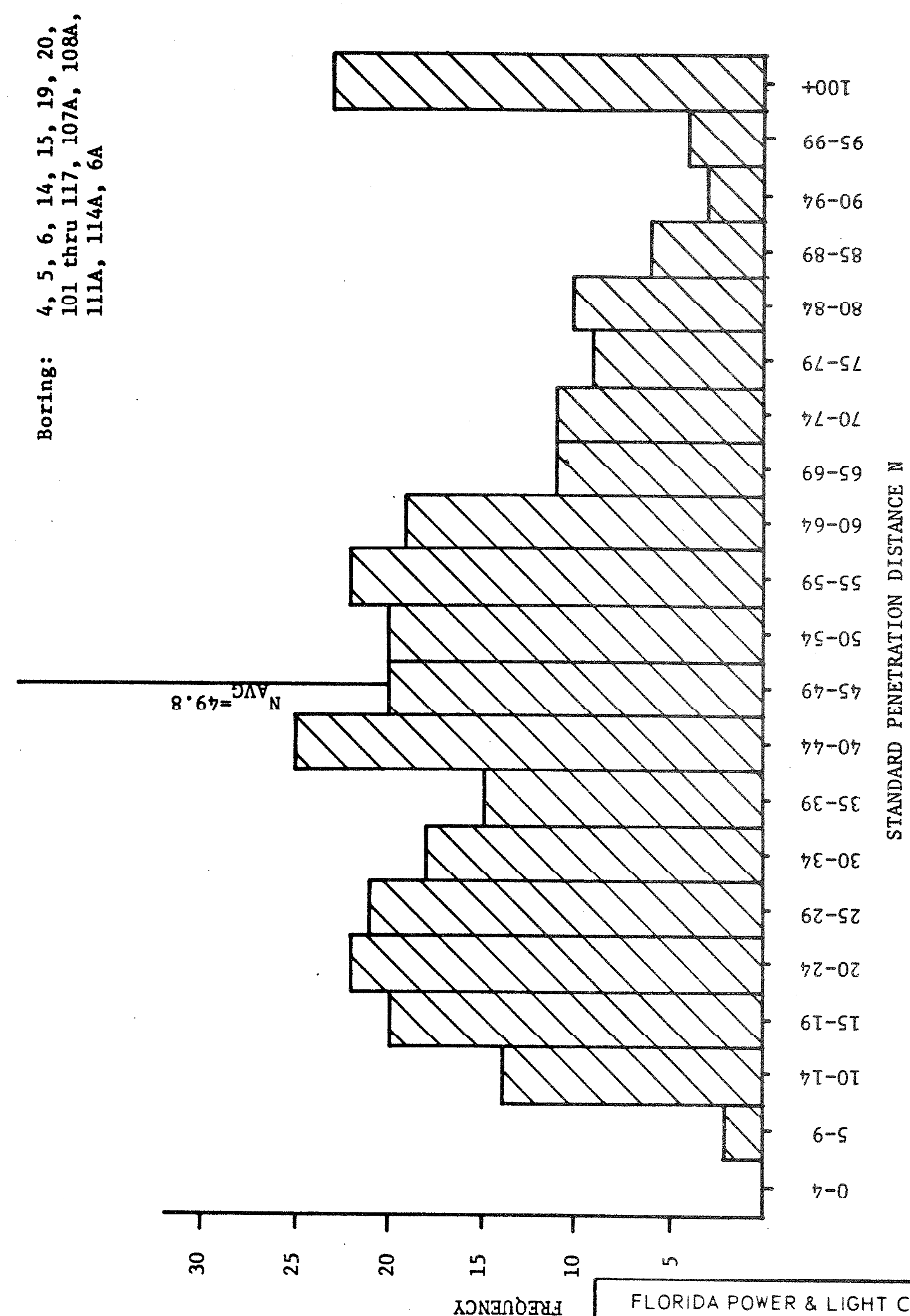
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

HISTOGRAM OF PENETRATION RESISTANCE FOR EL-0 TO EL-50
FIGURE 2.5-49



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

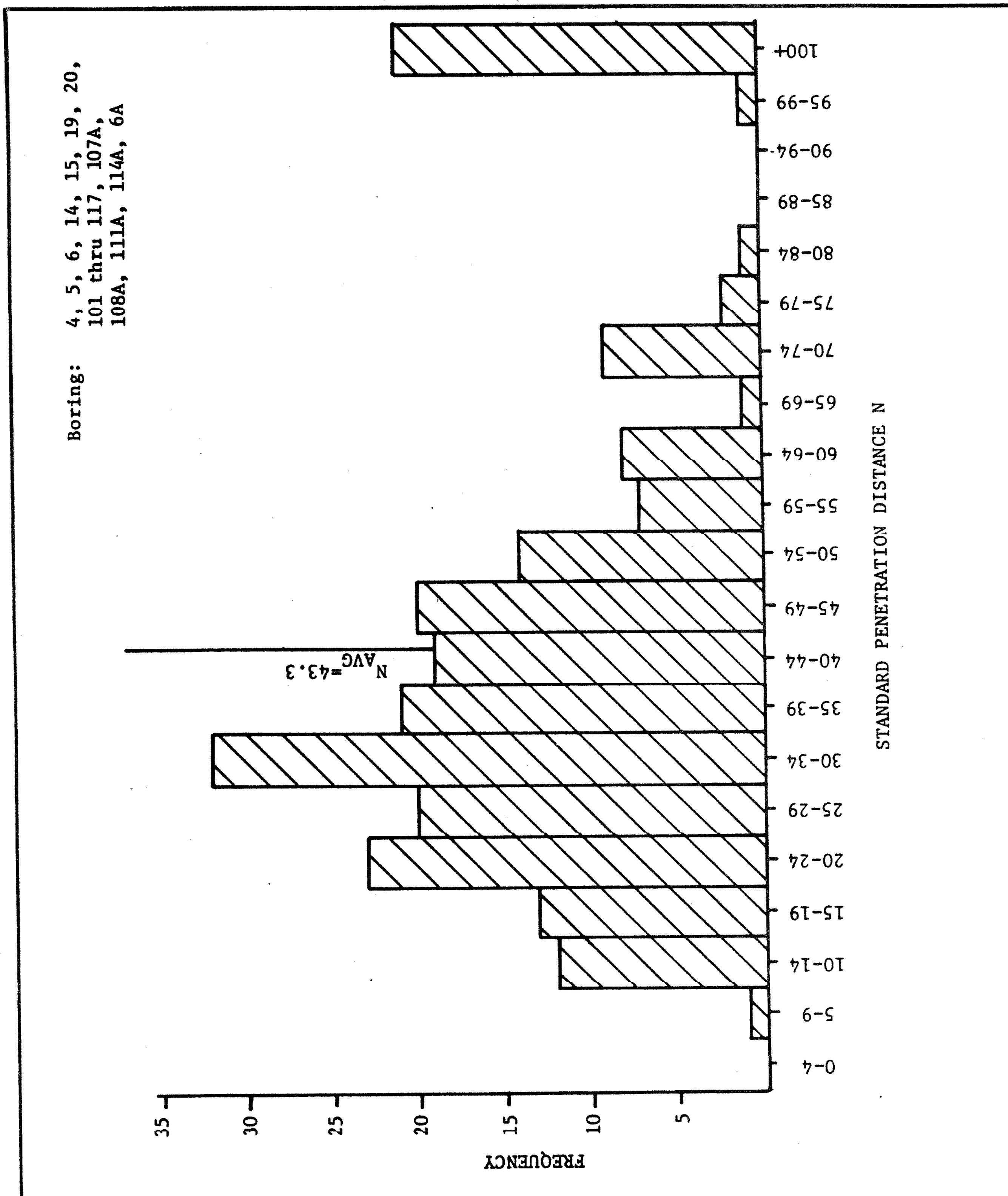
HISTOGRAM OF PENETRATION RESISTANCE FOR EL-51 TO EL-100
FIGURE 2.5-50



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

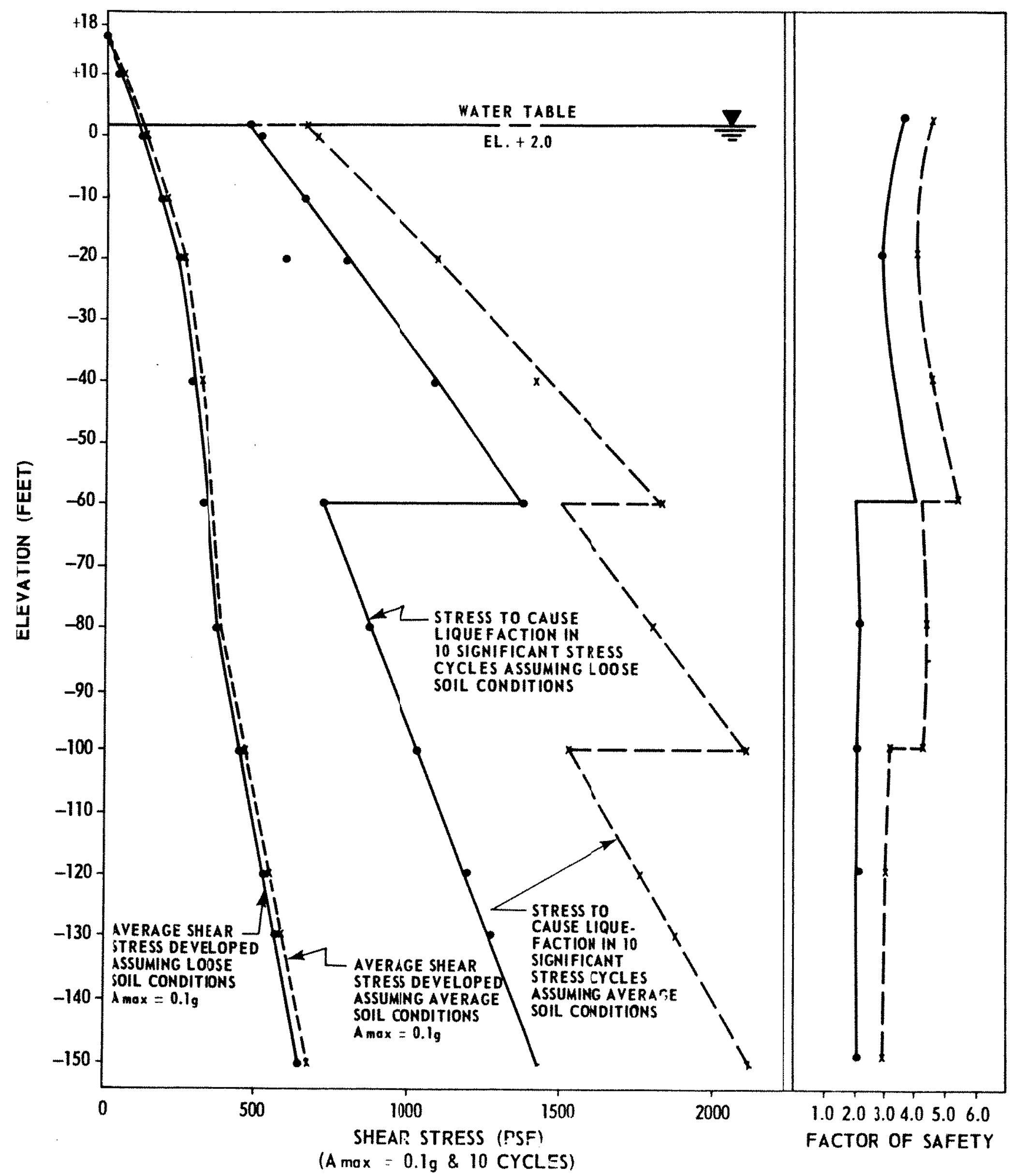
HISTOGRAM OF PENETRATION RESISTANCES FOR EL-60 TO EL-100

FIGURE 2.5-51



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

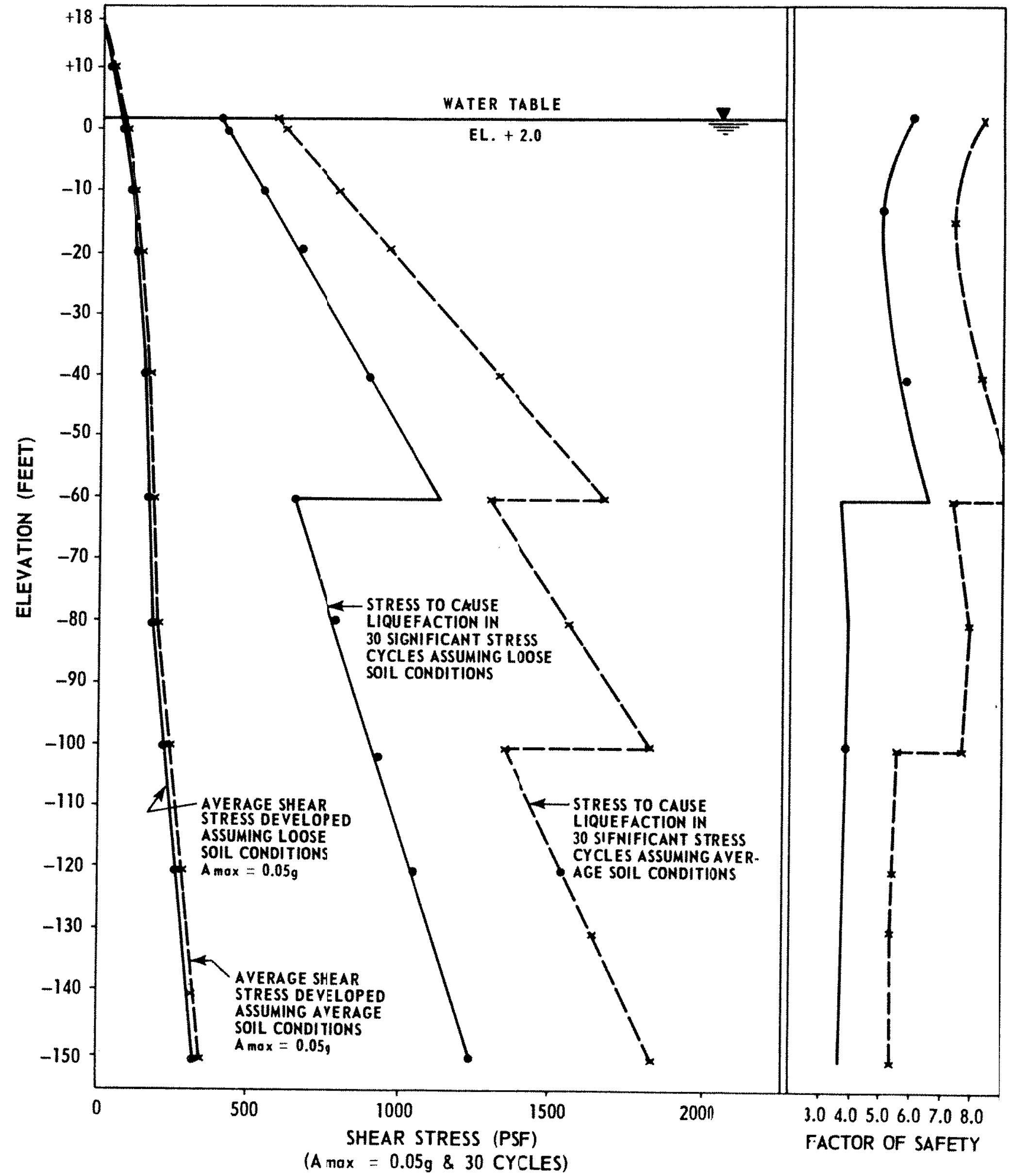
HISTOGRAM OF PENETRATION RESISTANCES FOR EL-101 TO EL-150
FIGURE 2.5-52



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

LIQUEFACTION POTENTIAL

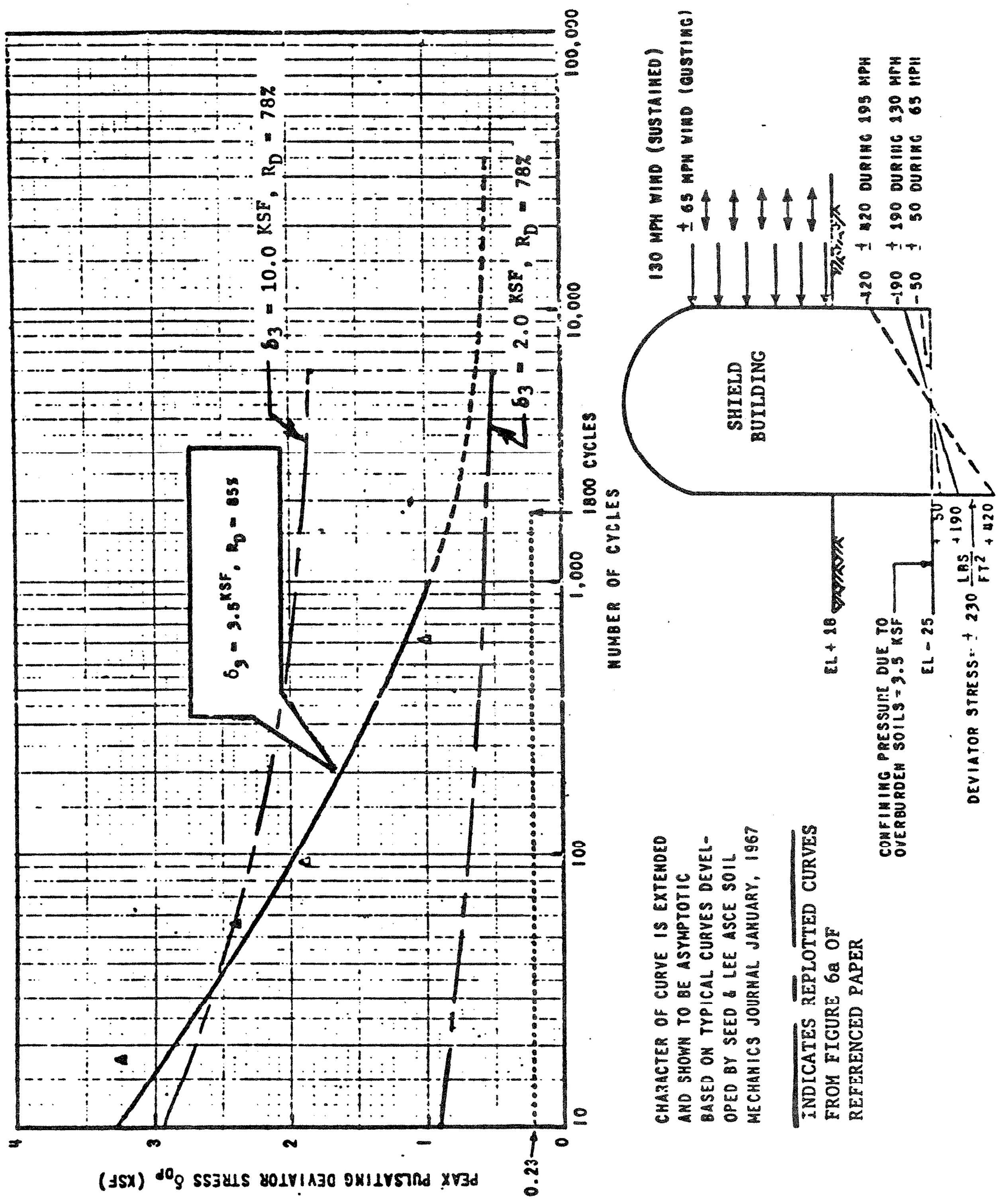
FIGURE 2.5-53



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

LIQUEFACTION POTENTIAL

FIGURE 2.5-54



FLORIDA POWER & LIGHT COMPANY

ST. LUCIE PLANT UNIT 1

LIQUEFACTION EVALUATION OF COMPACTED BACKFILL MATERIAL AT BASE OF REACTOR BUILDING DURING WIND GUSTING

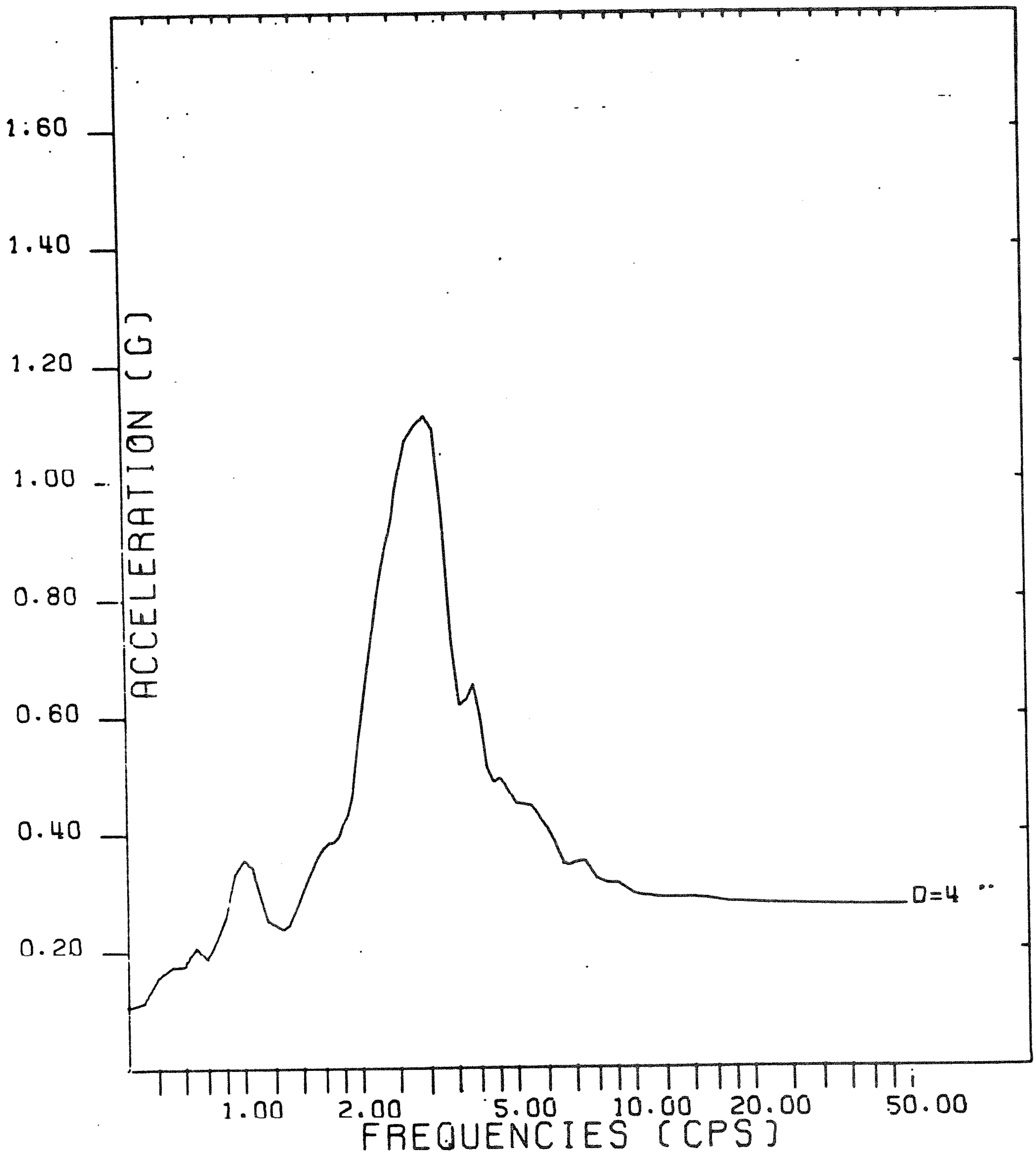
FIGURE 2.5-55

Withheld Under 10 CFR 2.390

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

INTAKE CHANNEL LIQUEFACTION ANALYSIS

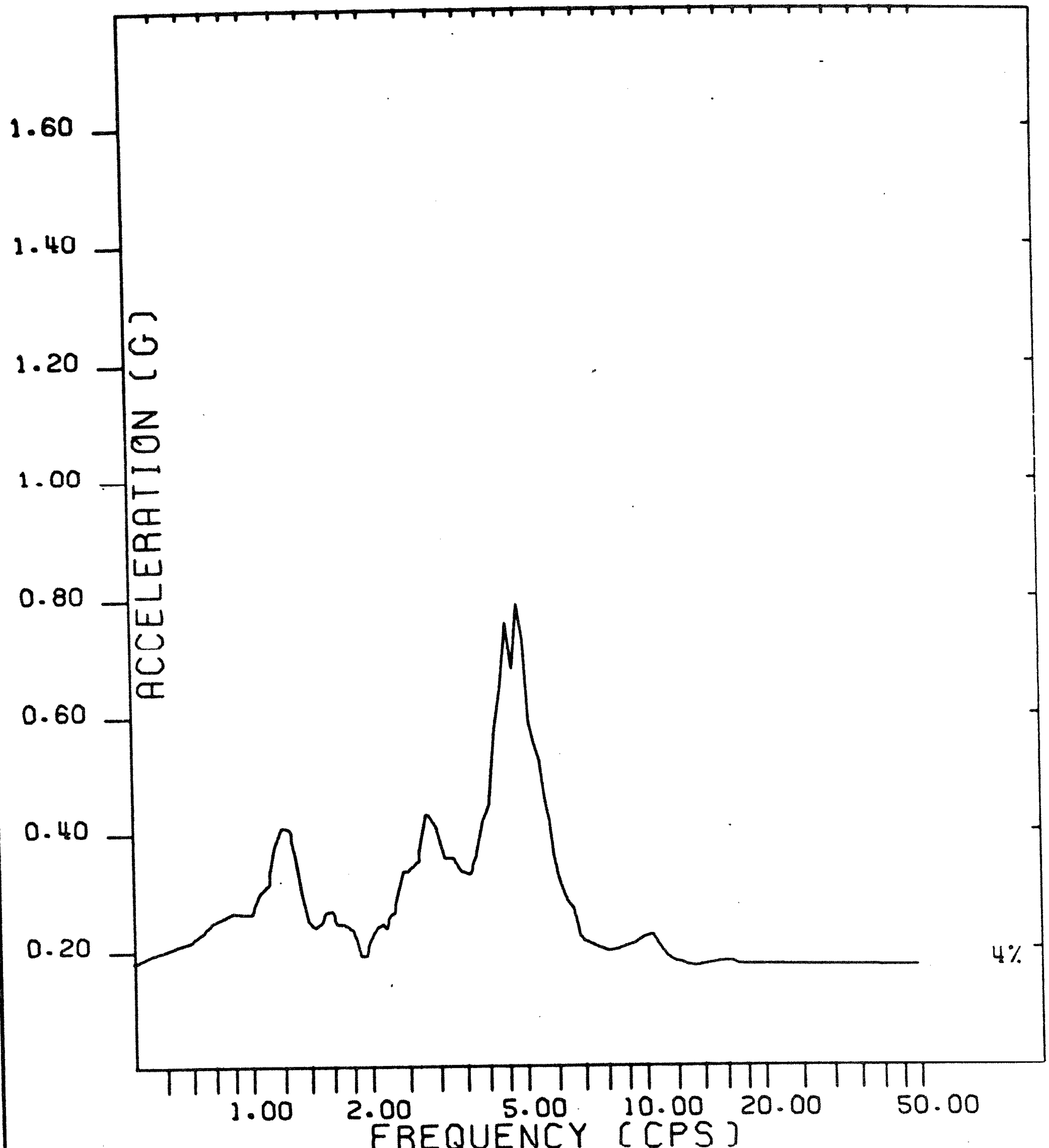
FIGURE 2.5-56



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SHIELD BUILDING MP34
EL. 33.5 FLOOR SPECTRA
OBE N-S

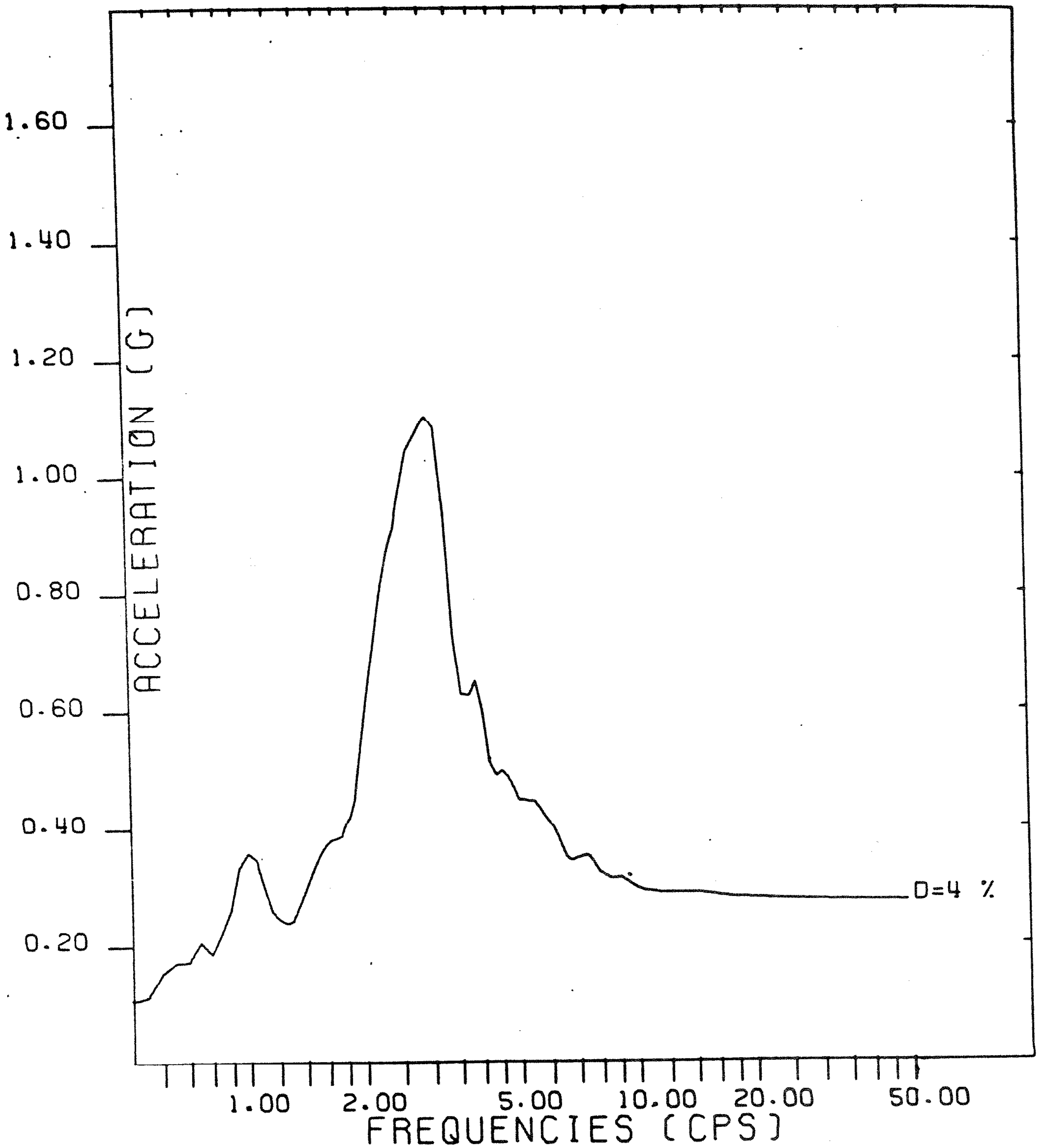
FIGURE 2.5-57



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SHIELD BUILDING – 0.13G
UNAUGMENTED N-S TRANSLATION

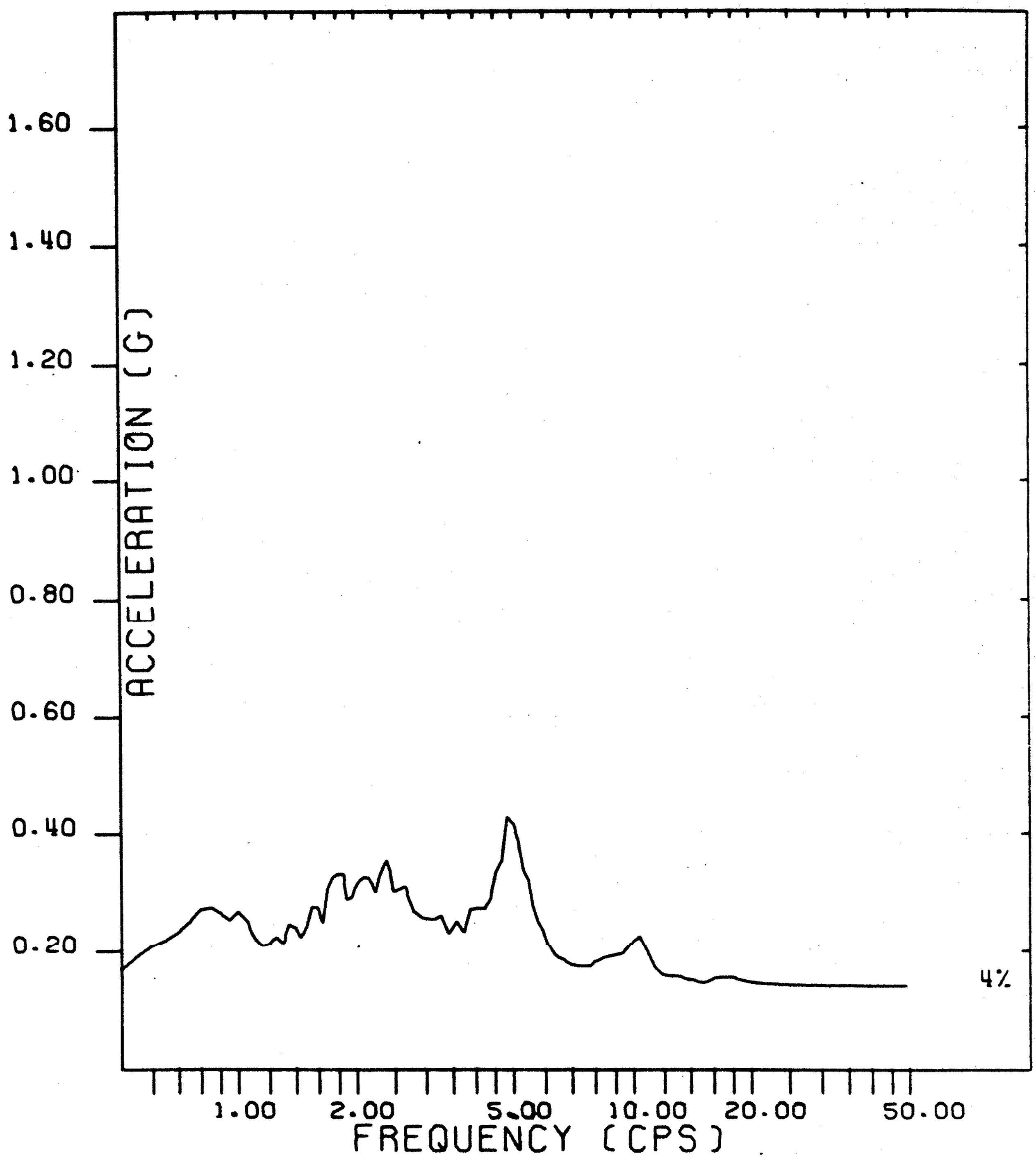
FIGURE 2.5-58



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SHIELD BUILDING MP34
EL. 33.5 FLOOR SPECTRA
OBE E-W

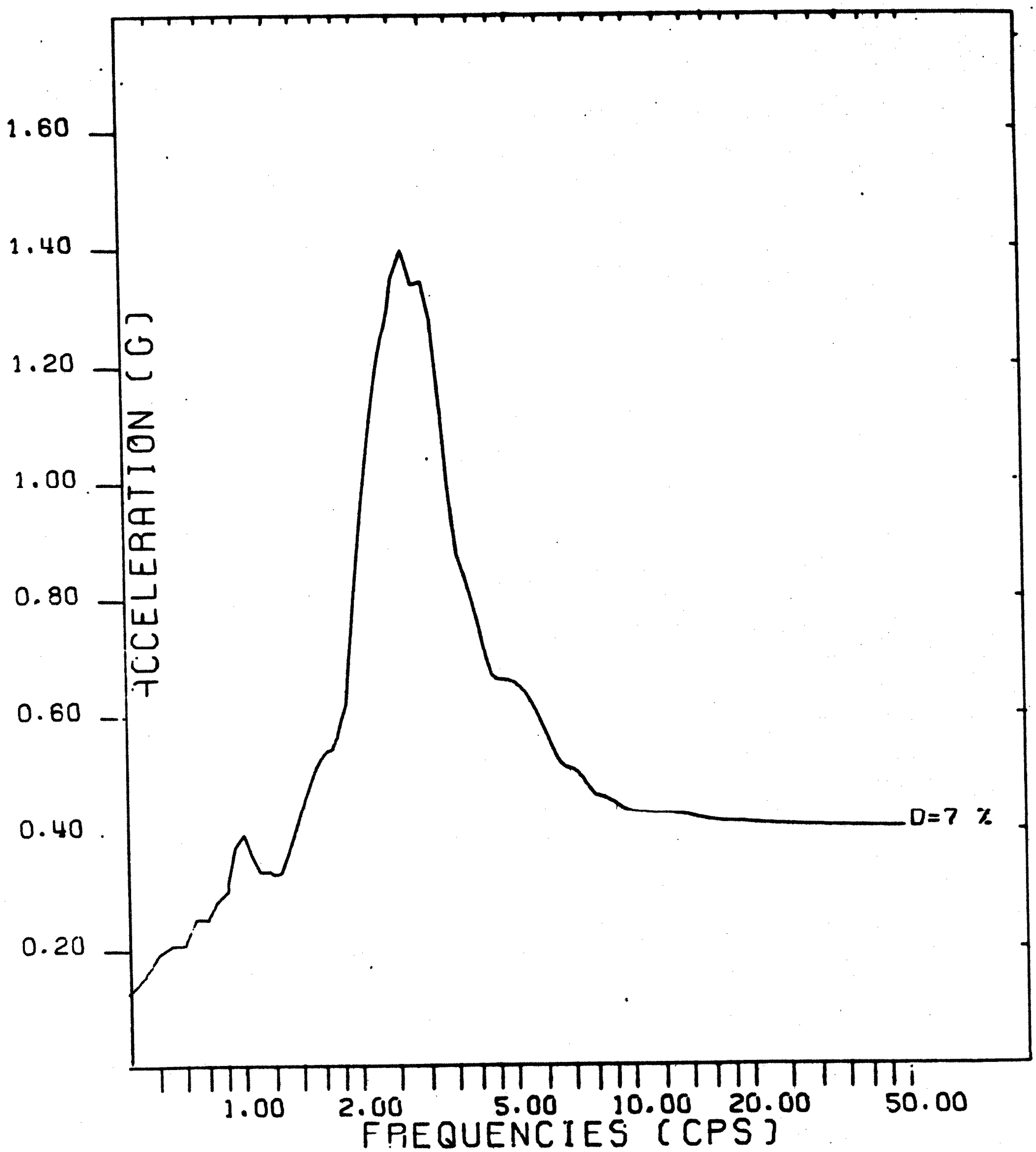
FIGURE 2.5-59



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SHIELD BUILDING – 0.13G
UNAUGMENTED E-W TRANSLATION

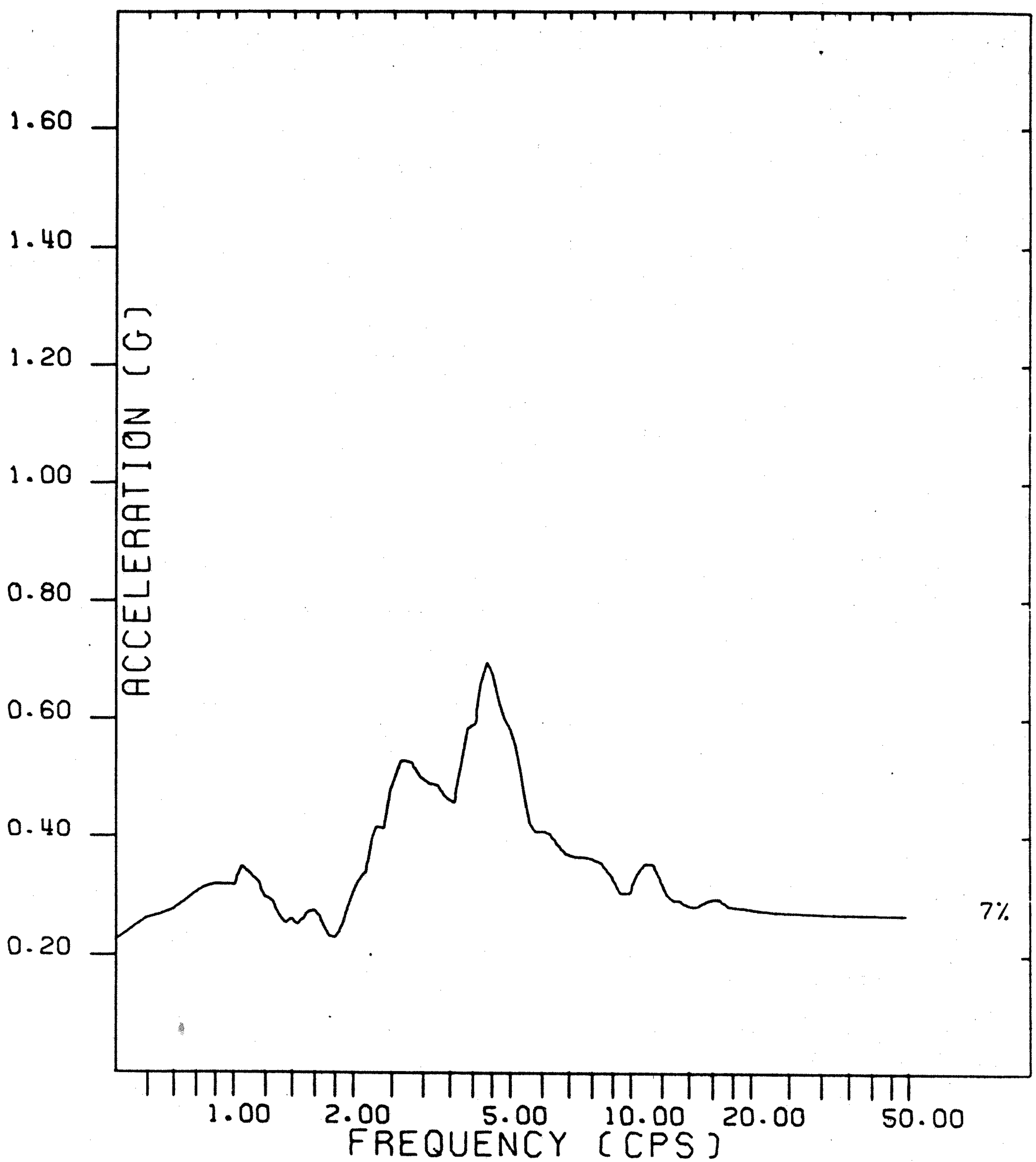
FIGURE 2.5-60



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SHIELD BUILDING MP34
EL. 33.5 FLOOR SPECTRA
DBE N-S

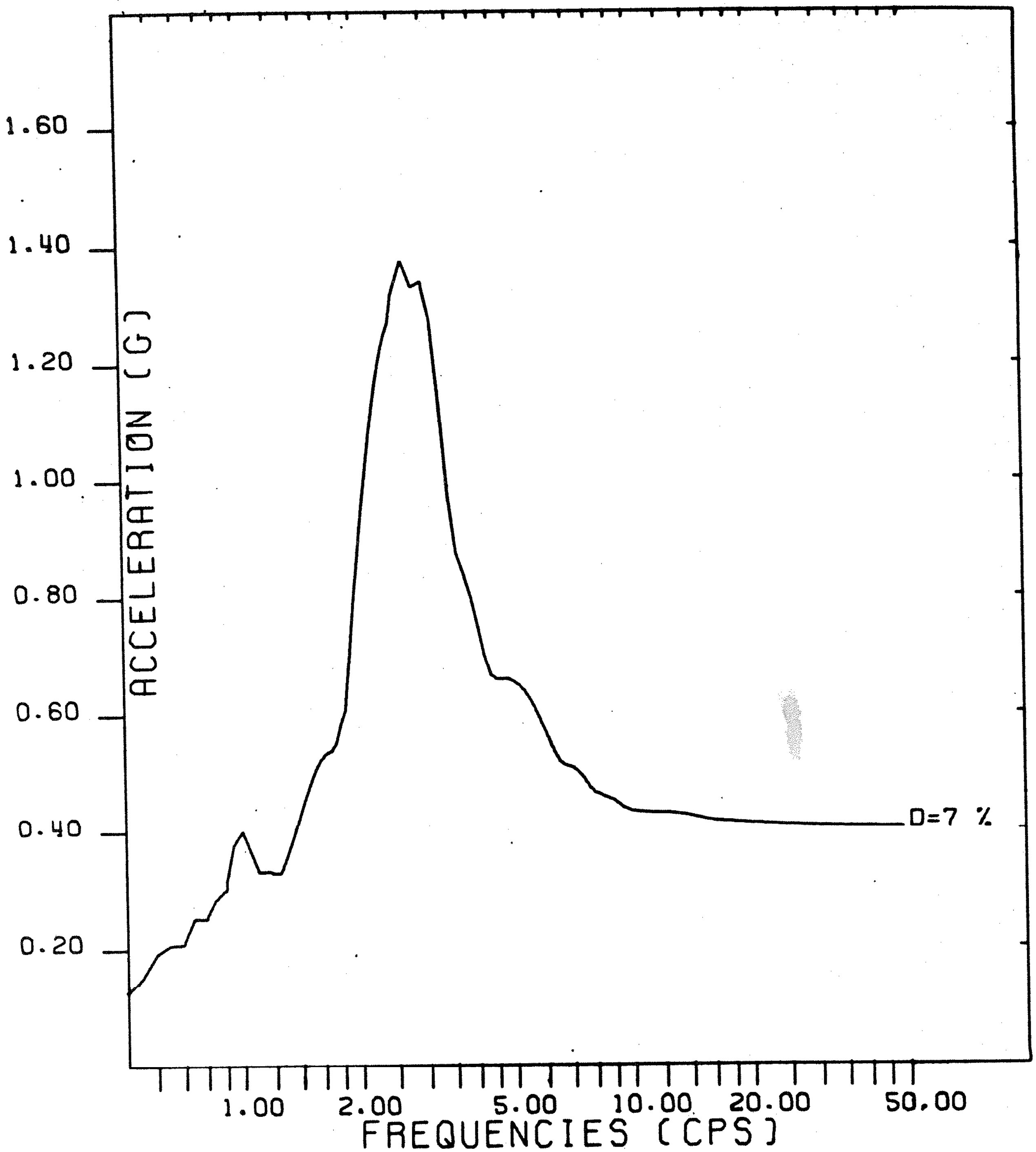
FIGURE 2.5-61



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

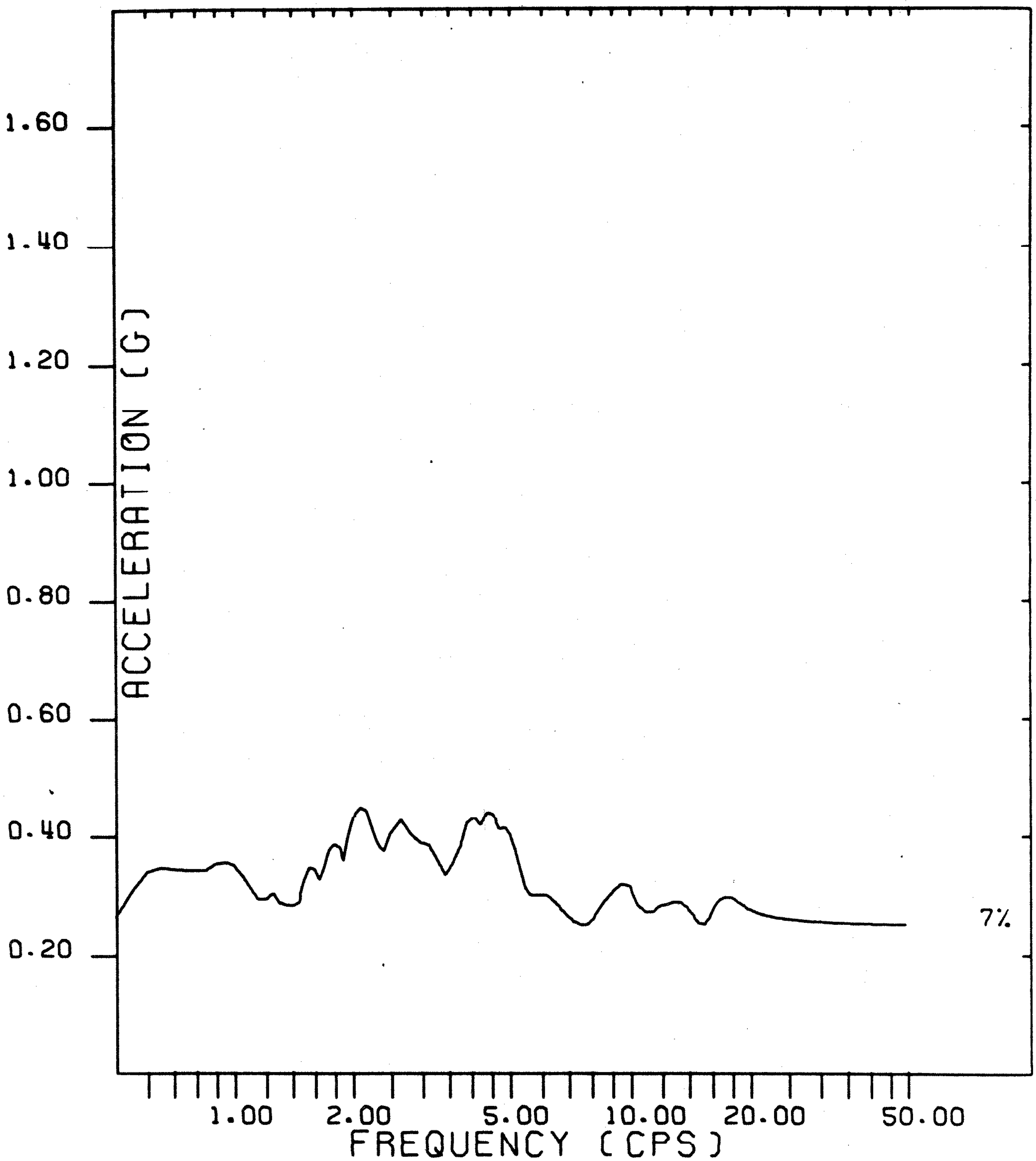
SHIELD BUILDING – 0.2G
UNAUGMENTED N-S TRANSLATION

FIGURE 2.5-62



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

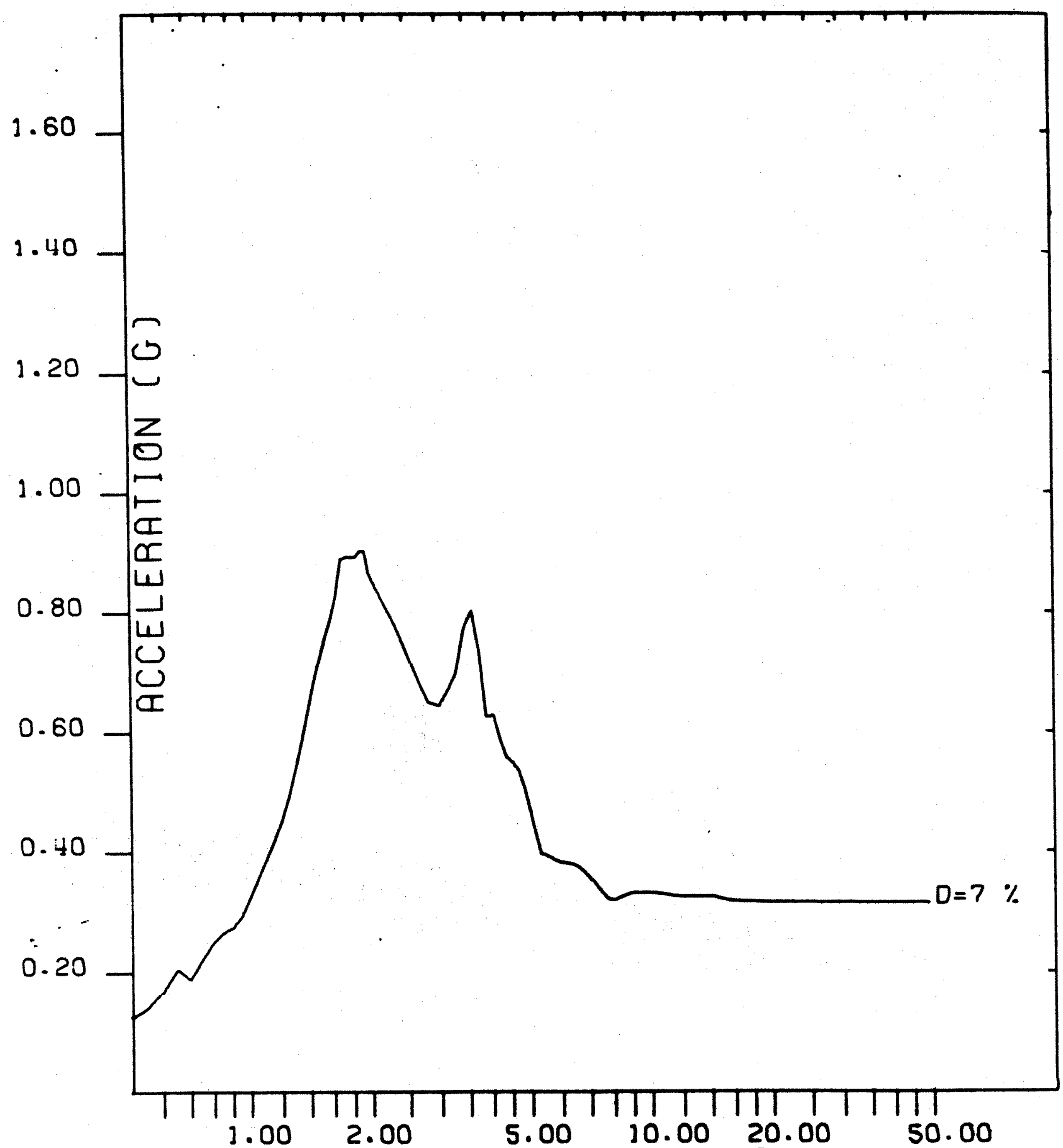
SHIELD BUILDING MP34
EL. 33.5 FLOOR SPECTRA
DBE E-W
FIGURE 2.5-63



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SHIELD BUILDING – 0.2G
UNAUGMENTED E-W TRANSLATION

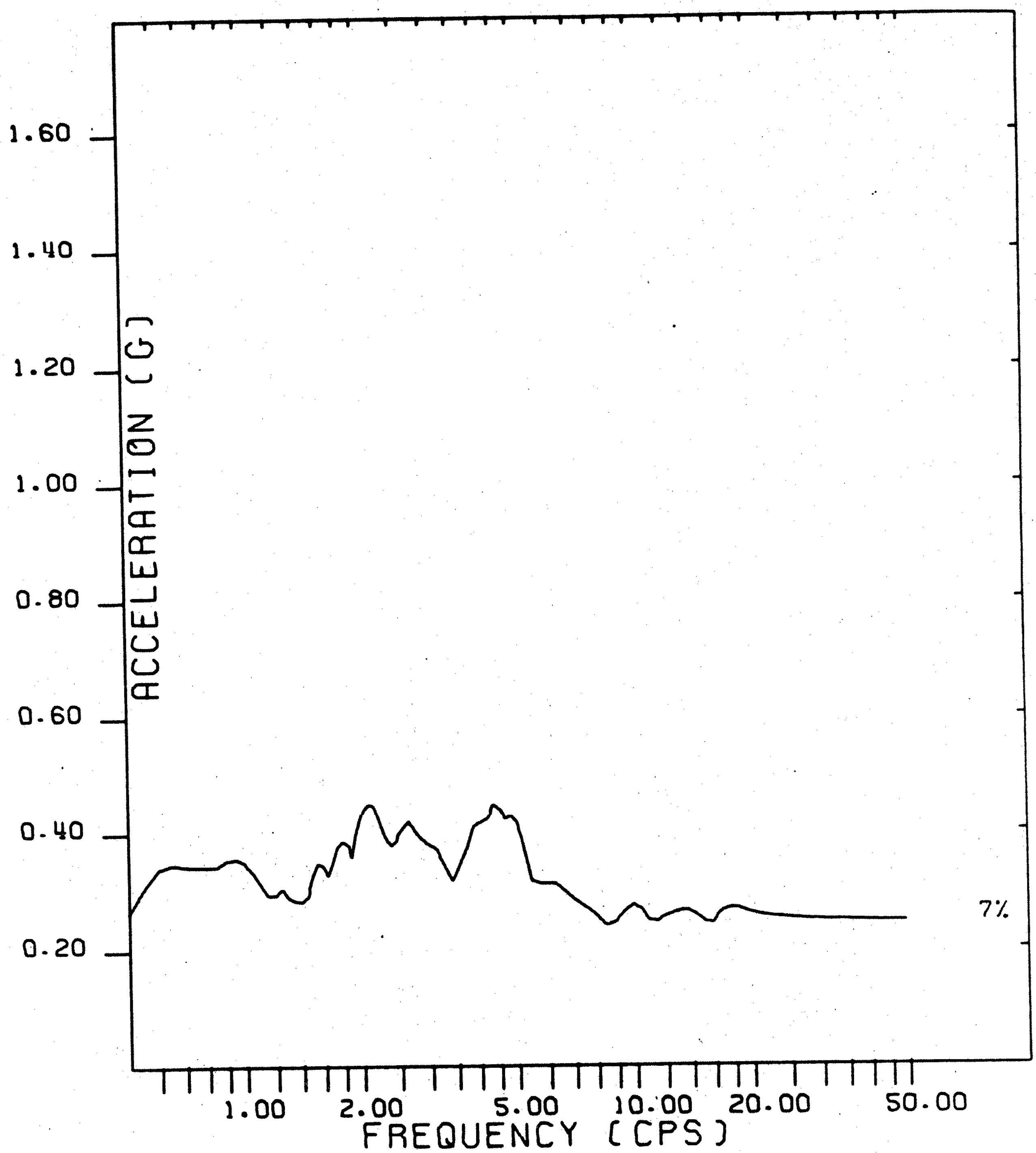
FIGURE 2.5-64



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

REACTOR AUXILIARY BUILDING MP6
EL. 28.5 FLOOR SPECTRA
DBE E-W

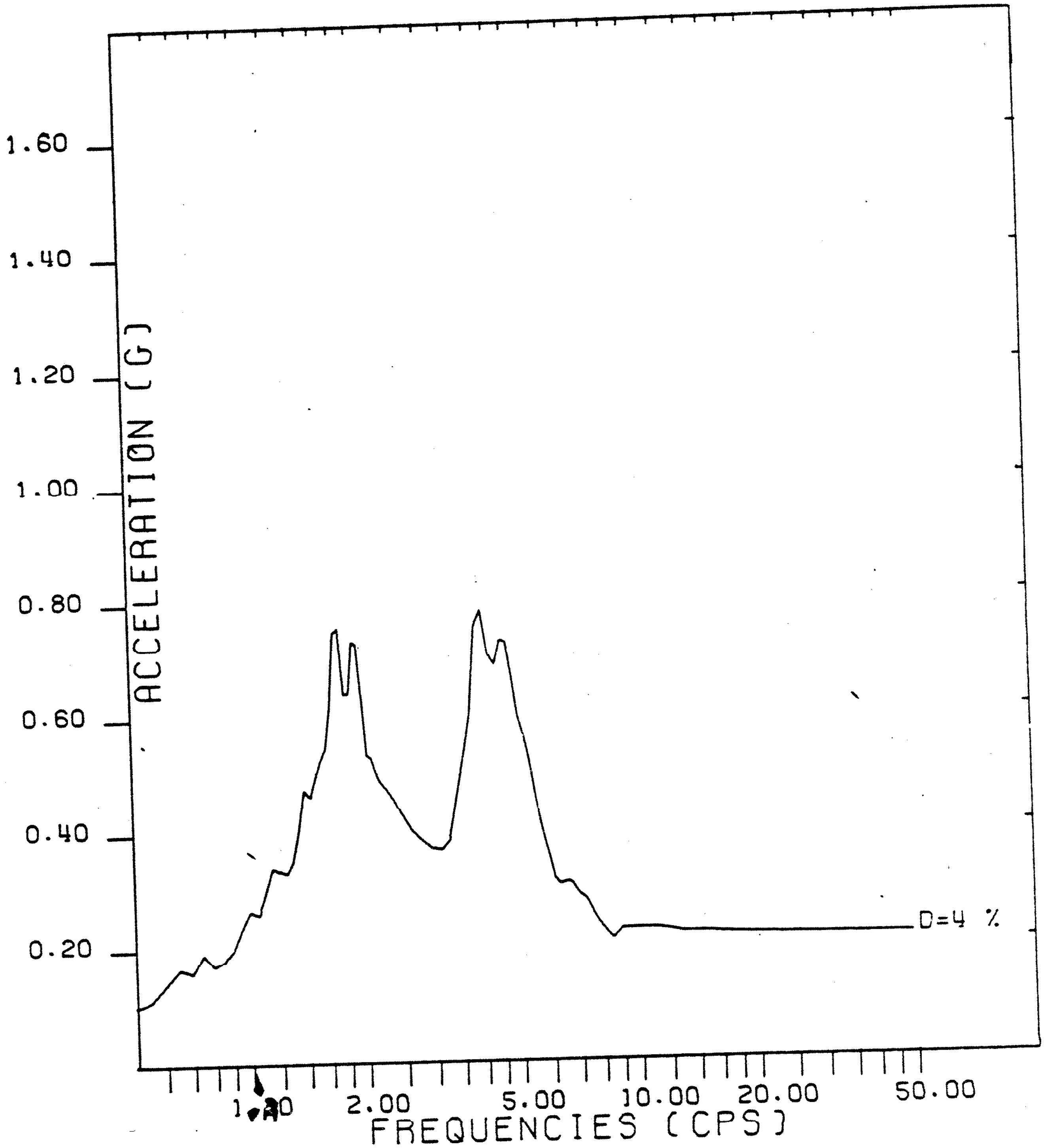
FIGURE 2.5-65



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

REACTOR AUXILIARY BUILDING – 0.2G
UNAUGMENTED E-W TRANSLATION

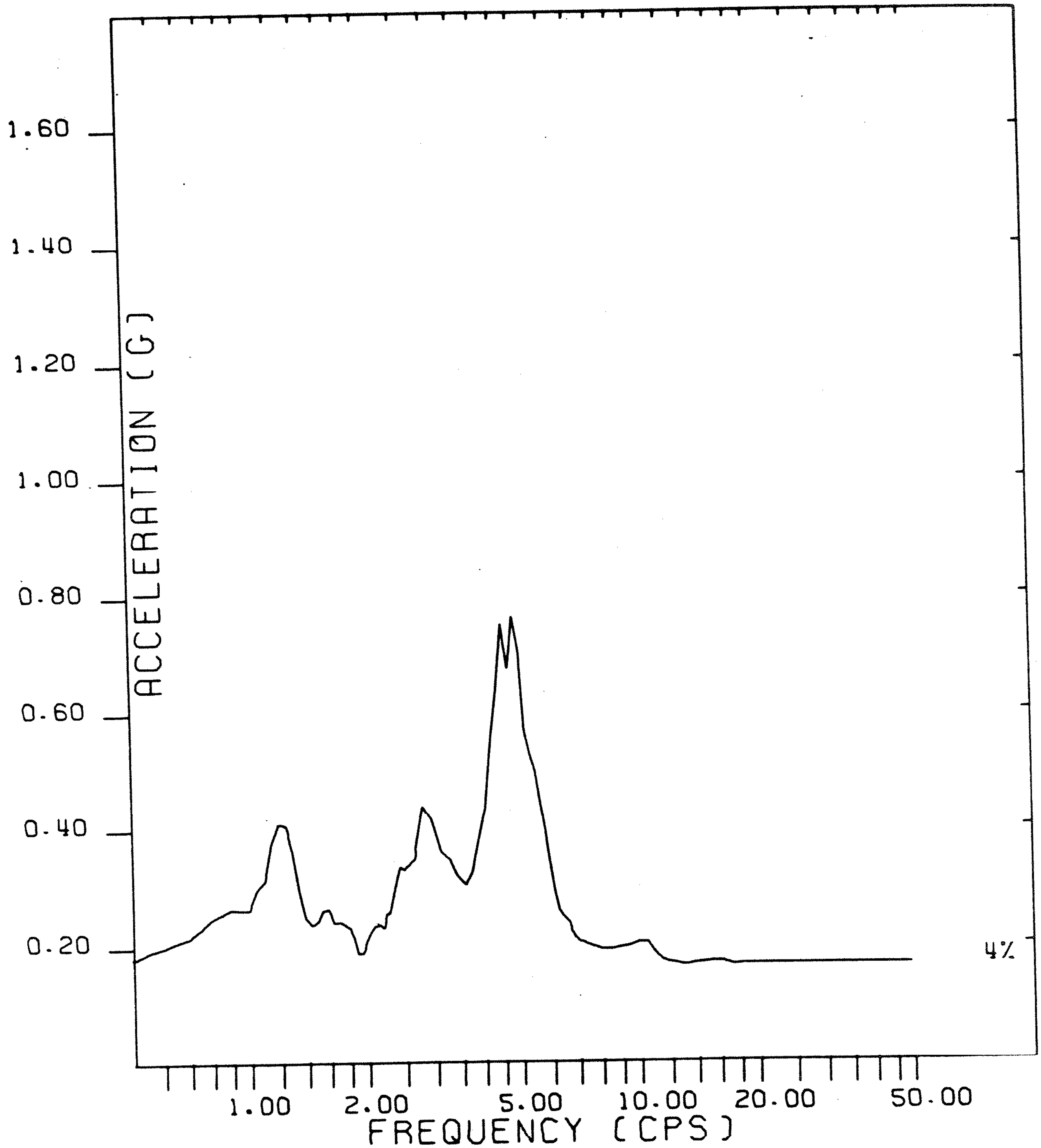
FIGURE 2.5-66



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

REACTOR AUXILIARY BUILDING MP6
EL. 28.5 FLOOR SPECTRA
OBE N-S

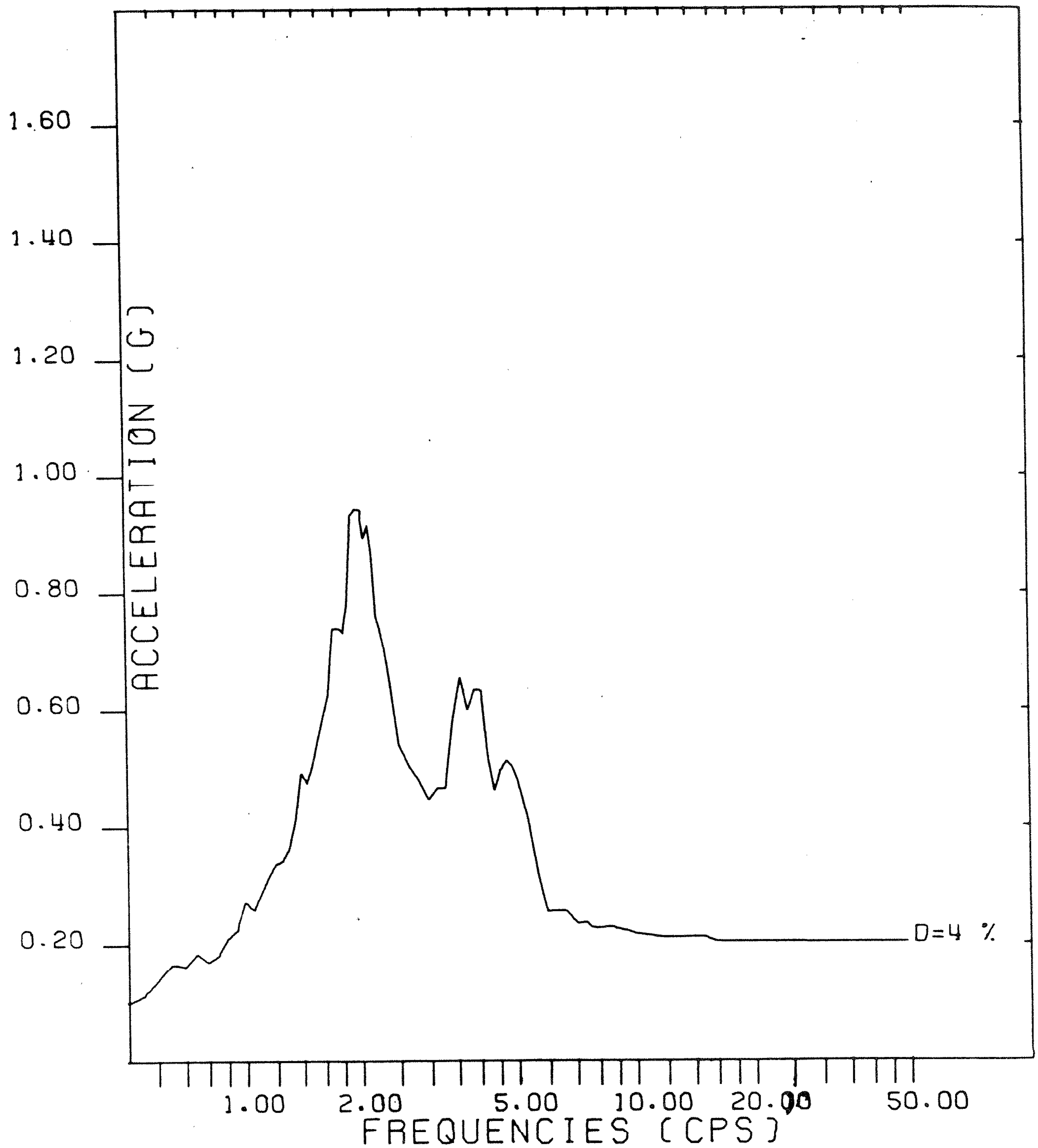
FIGURE 2.5-67



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

REACTOR AUXILIARY BUILDING – 0.13G
UNAUGMENTED N-S TRANSLATION

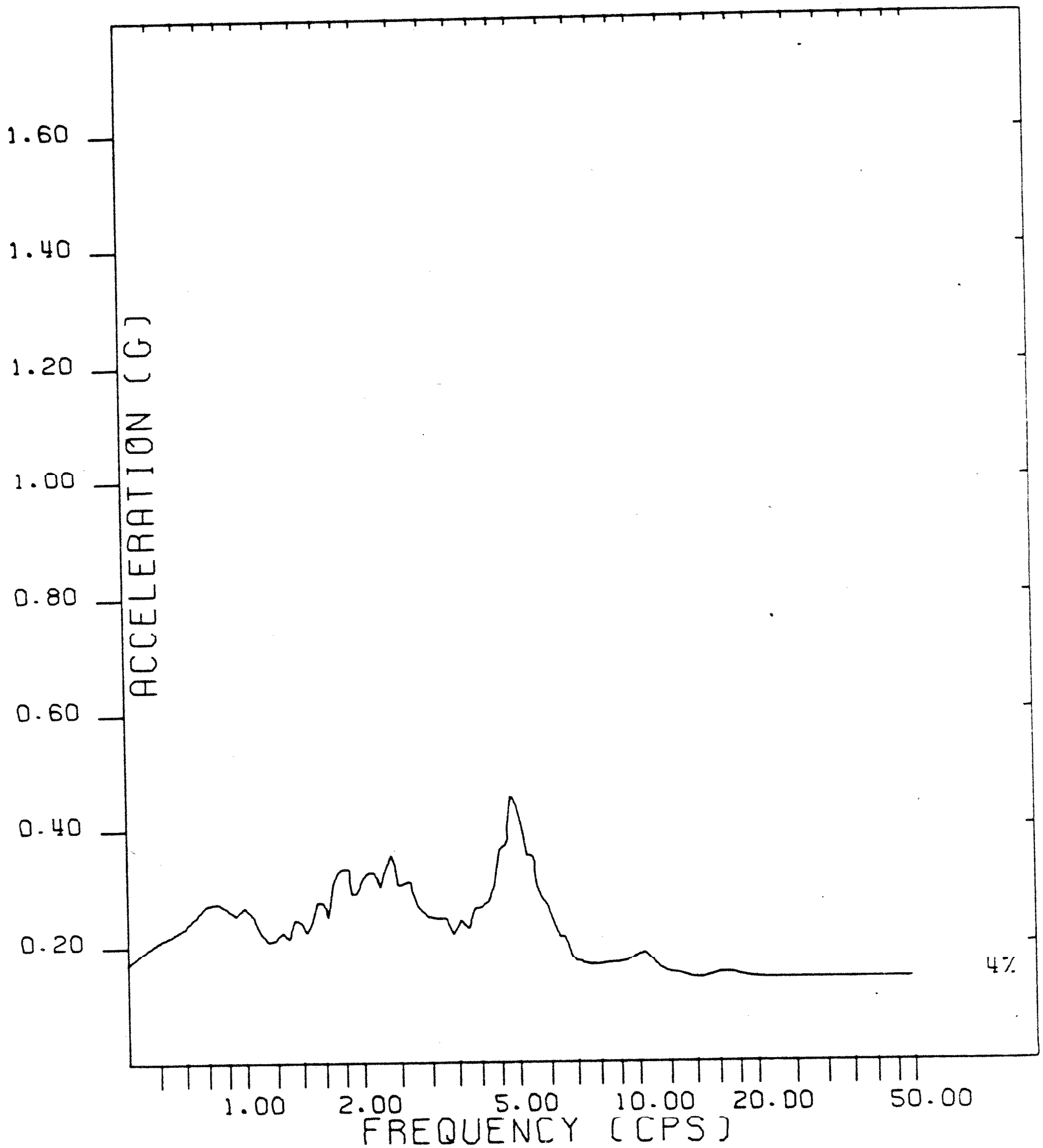
FIGURE 2.5-68



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

REACTOR AUXILIARY BUILDING MP6
EL. 28.5 FLOOR SPECTRA
OBE E-W

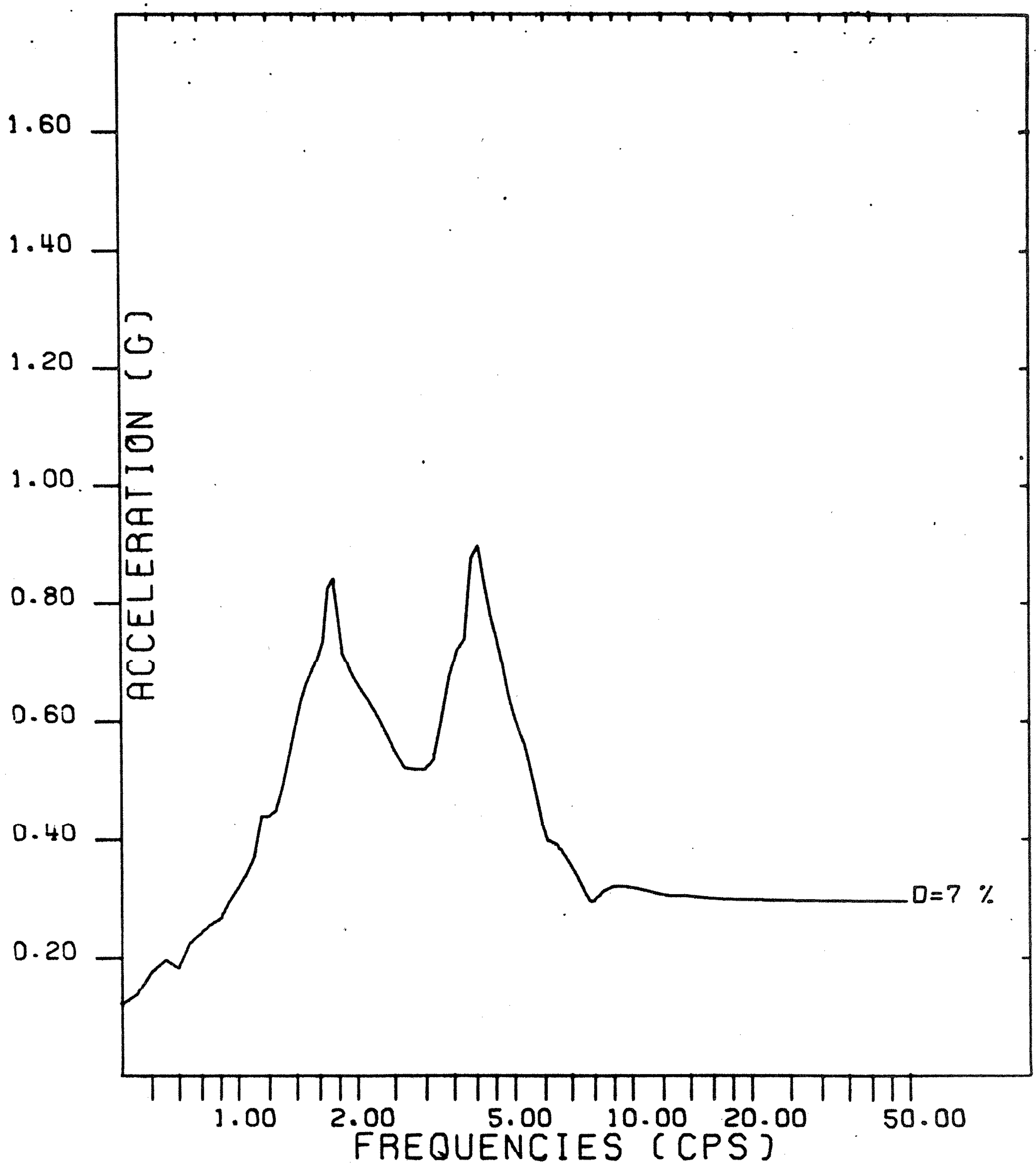
FIGURE 2.5-69



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

REACTOR AUXILIARY BUILDING – 0.13G
UNAUGMENTED E-W TRANSLATION

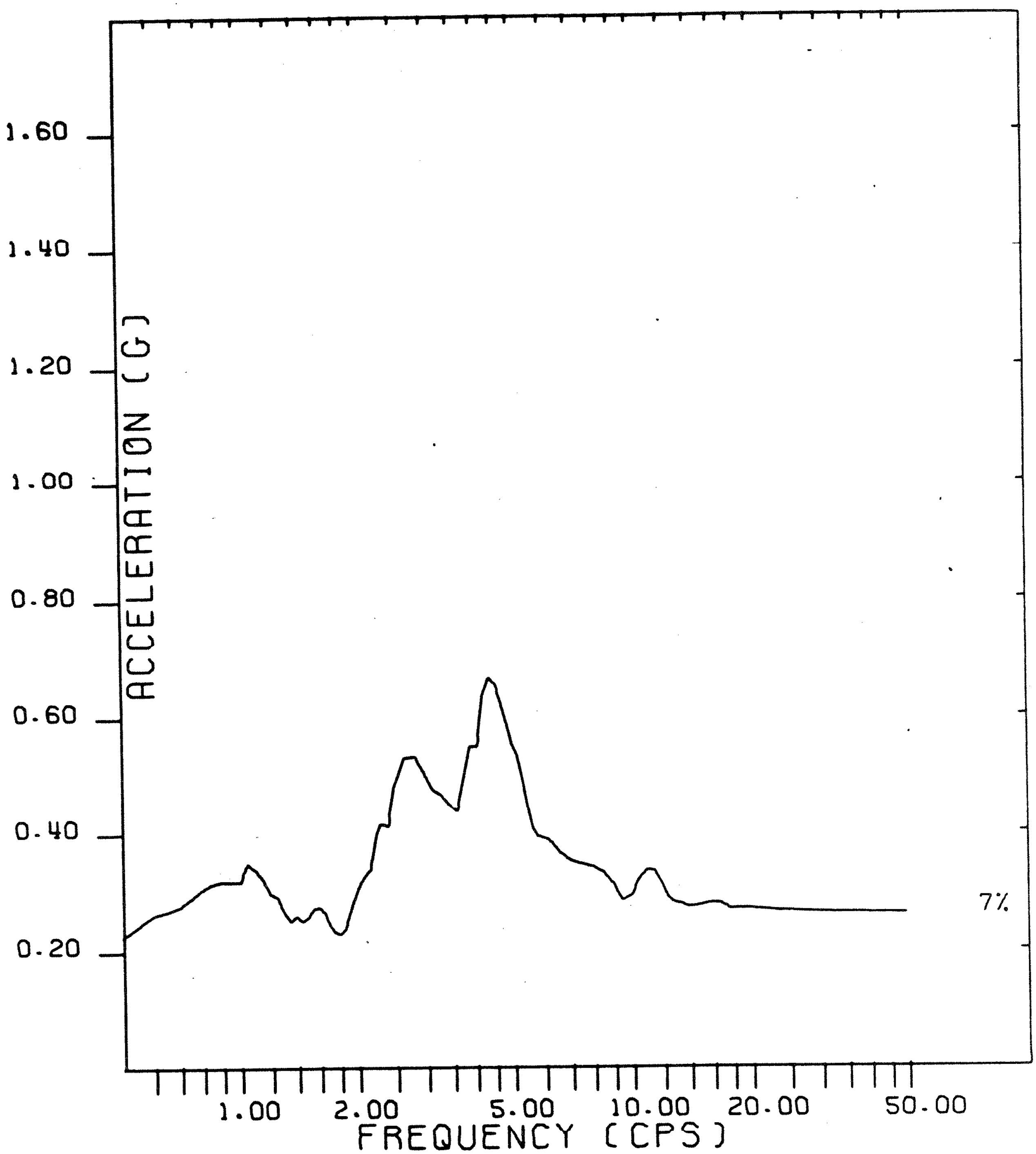
FIGURE 2.5-70



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

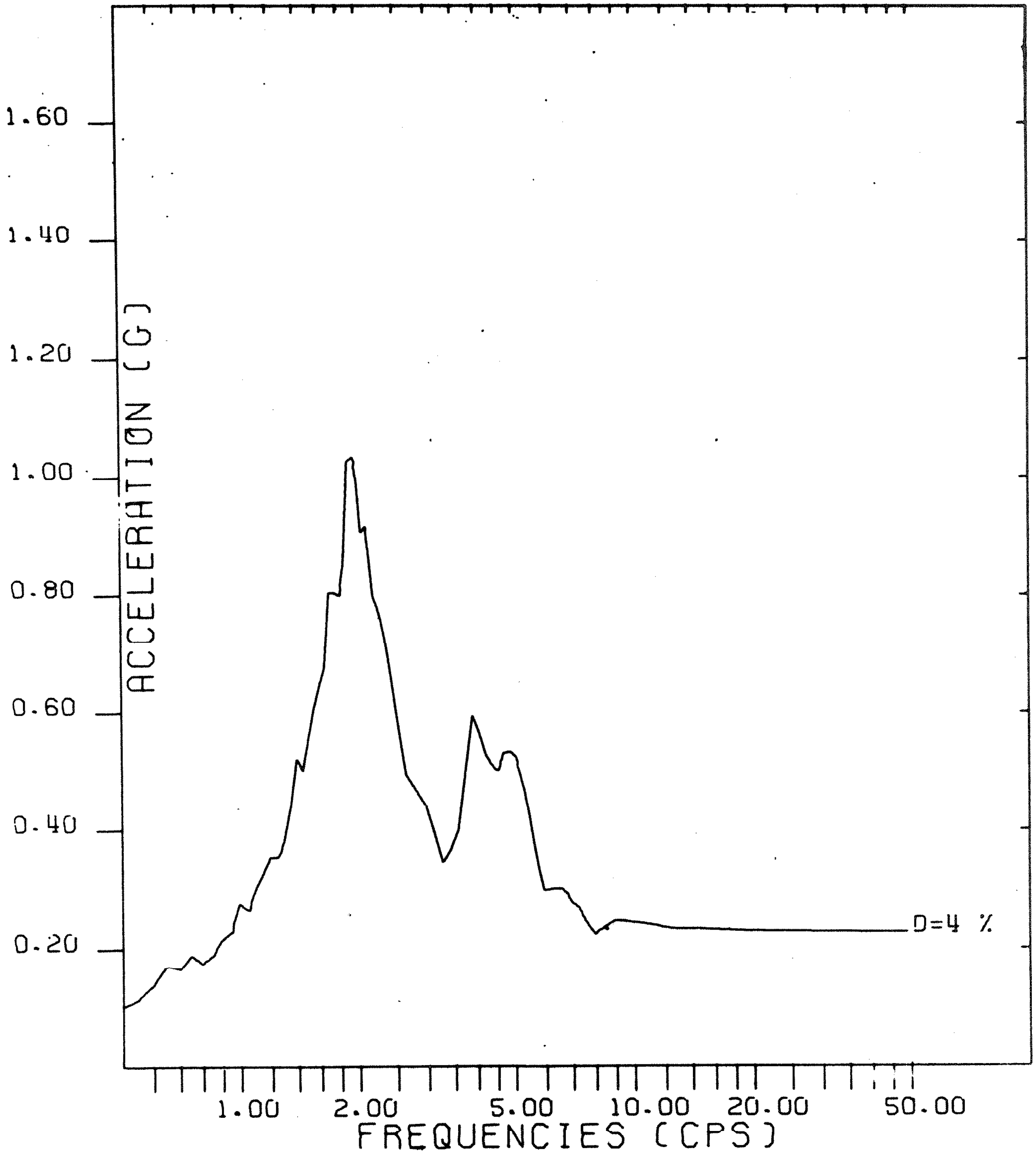
REACTOR AUXILIARY BUILDING MP6
EL. 28.5 FLOOR SPECTRA
DBE N-S

FIGURE 2.5-71



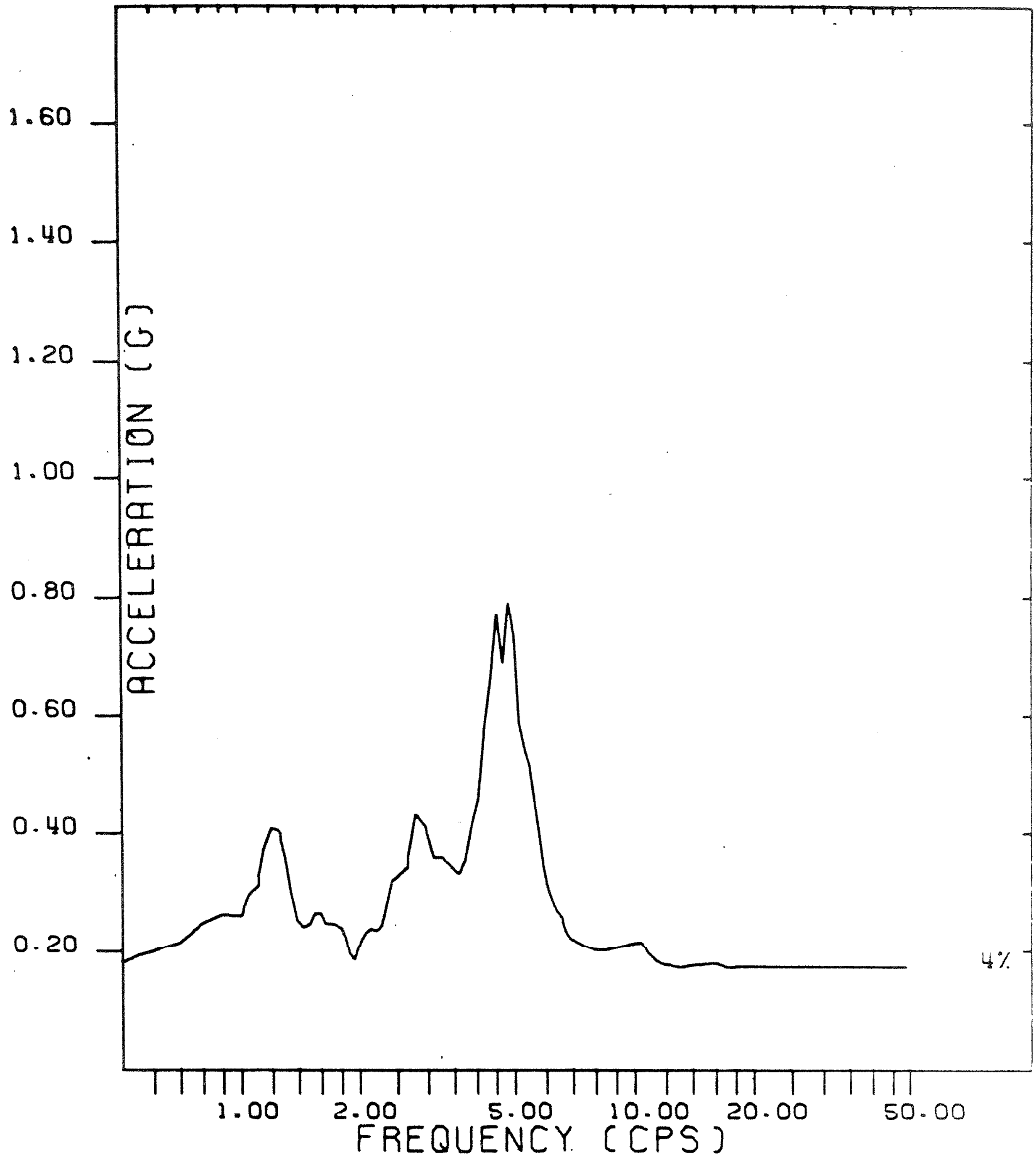
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

REACTOR AUXILIARY BUILDING
UNAUGMENTED N-S TRANSLATION
0.20 G
FIGURE 2.5-72



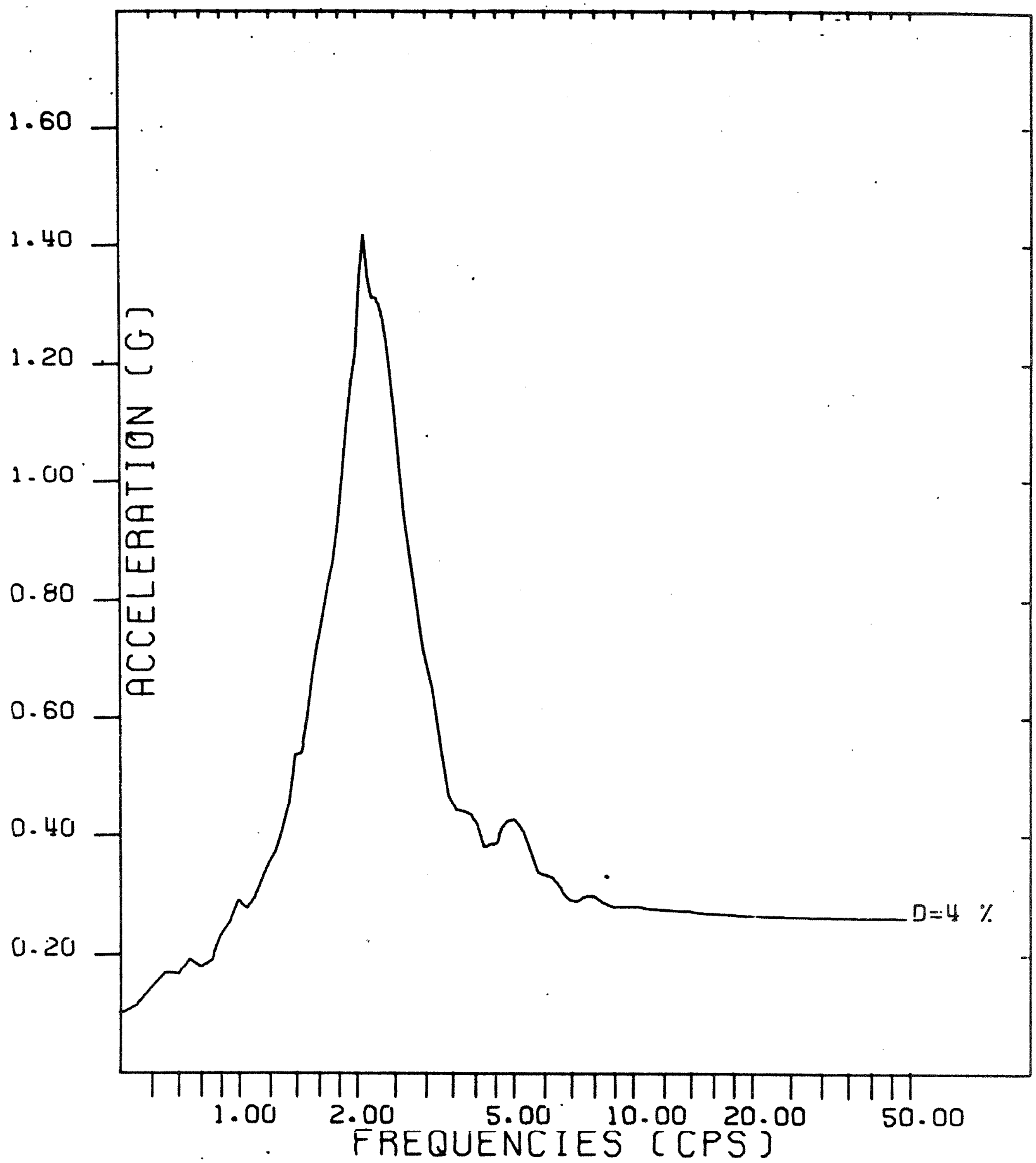
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

FUEL HANDLING BUILDING MP5
EL. 28.25 FLOOR SPECTRA
OBE N-S
FIGURE 2.5-73



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

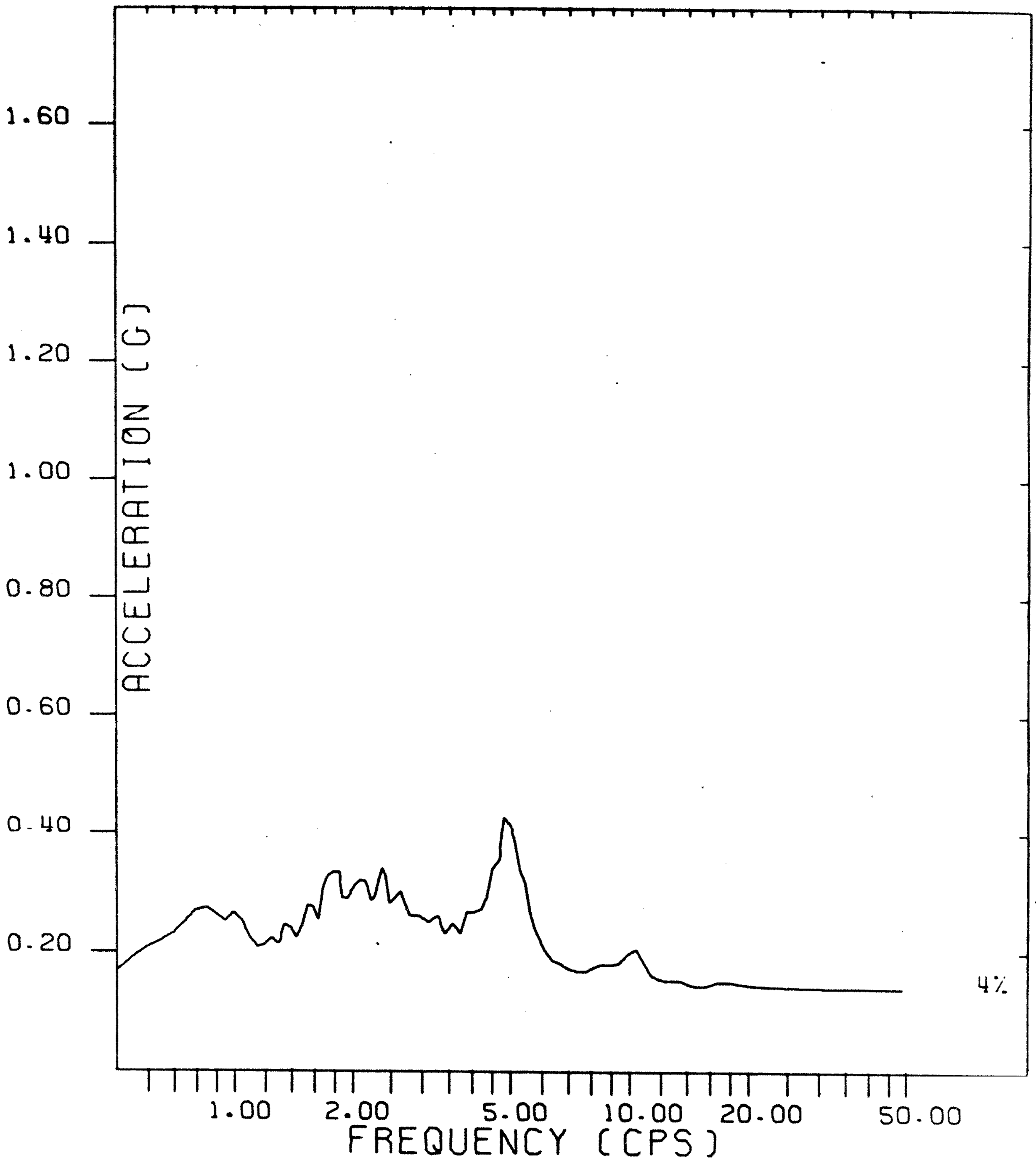
FUEL HANDLING BUILDING
UNAUGMENTED N-S TRANSLATION
0.13G
FIGURE 2.5-74



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

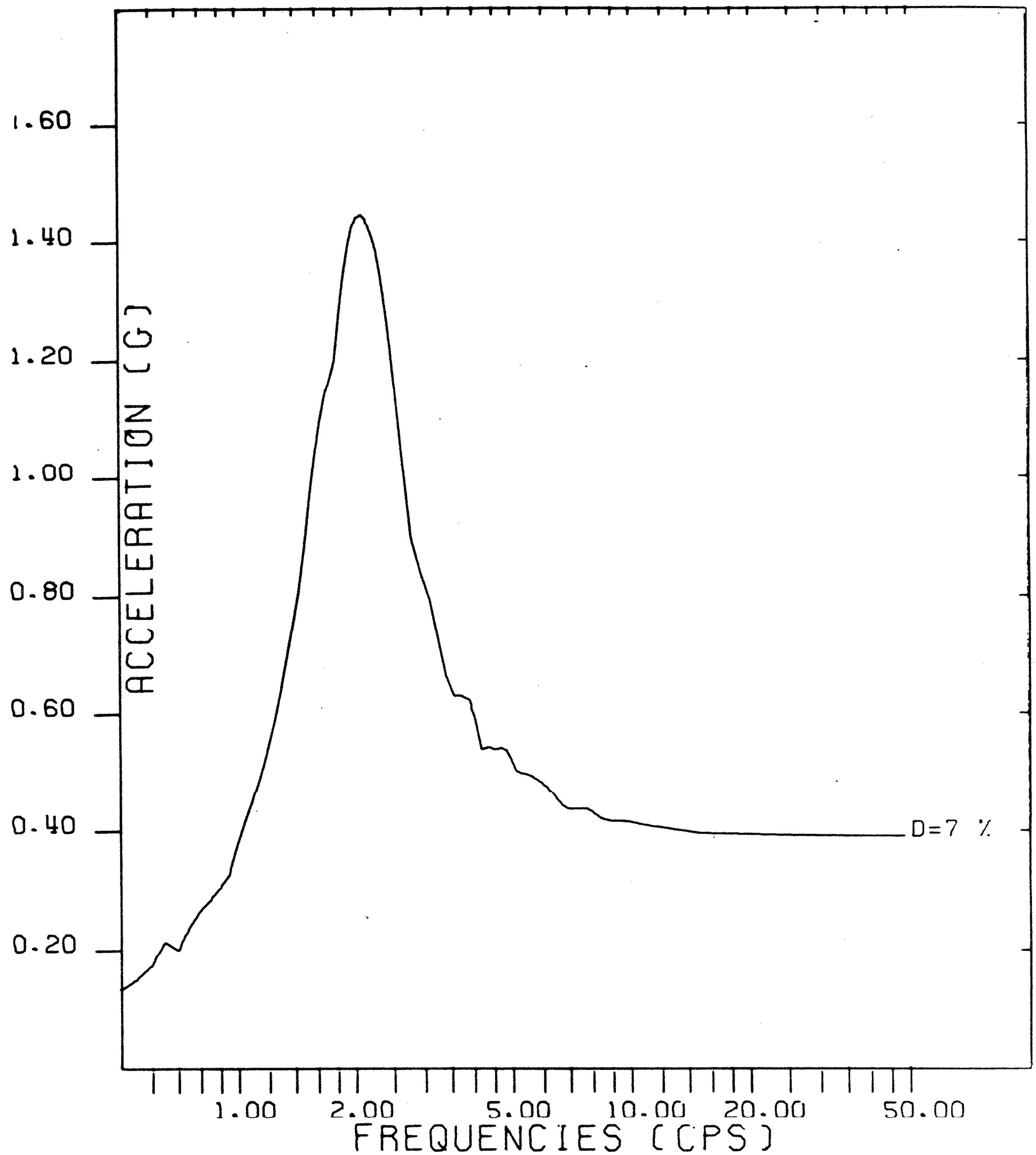
FUEL HANDLING BUILDING MP5
EL. 28.25 FLOOR SPECTRA
OBE E-W

FIGURE 2.5-75



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

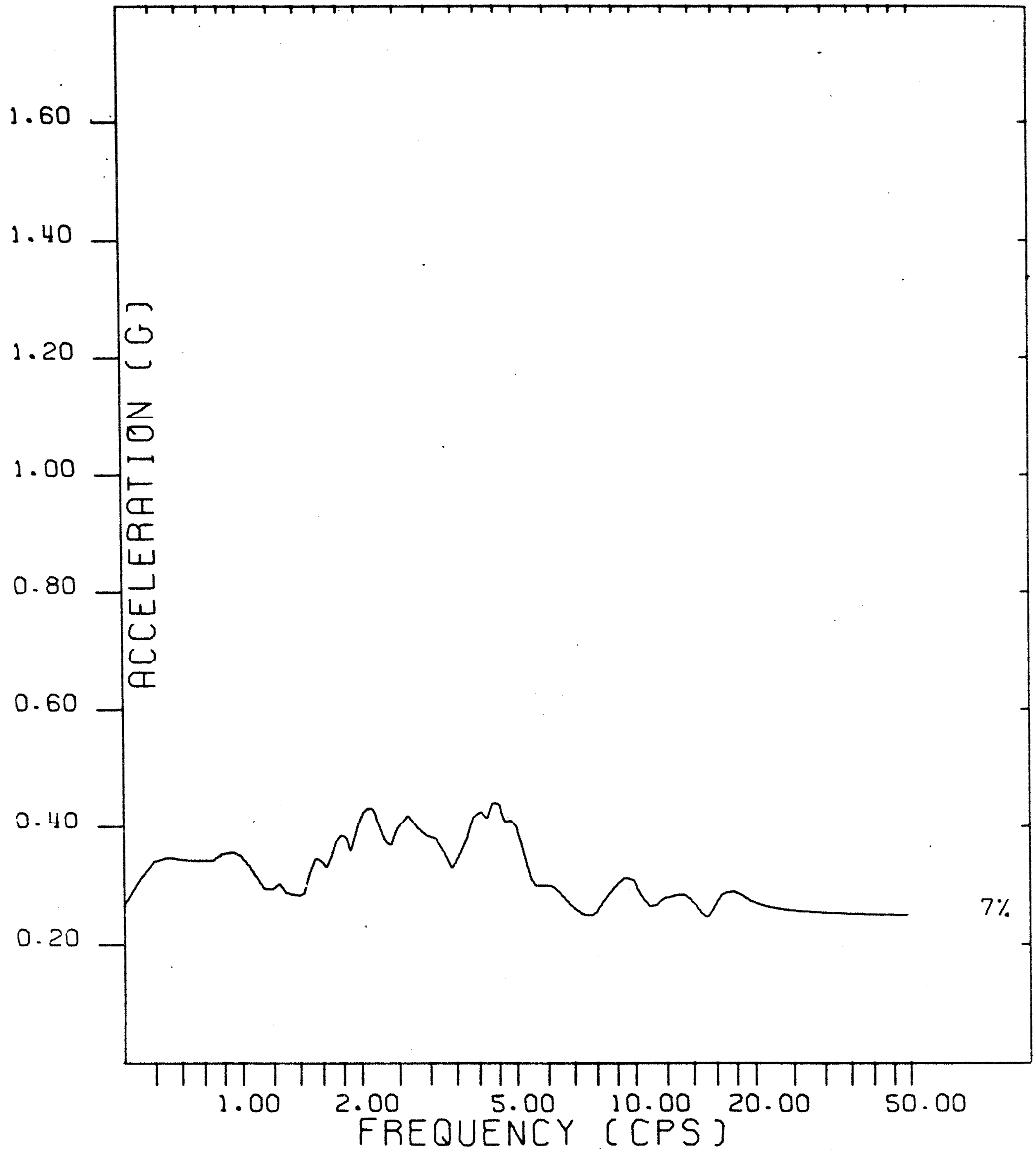
FUEL HANDLING BUILDING
UNAUGMENTED E-W TRANSLATION
0.13G
FIGURE 2.5-76



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

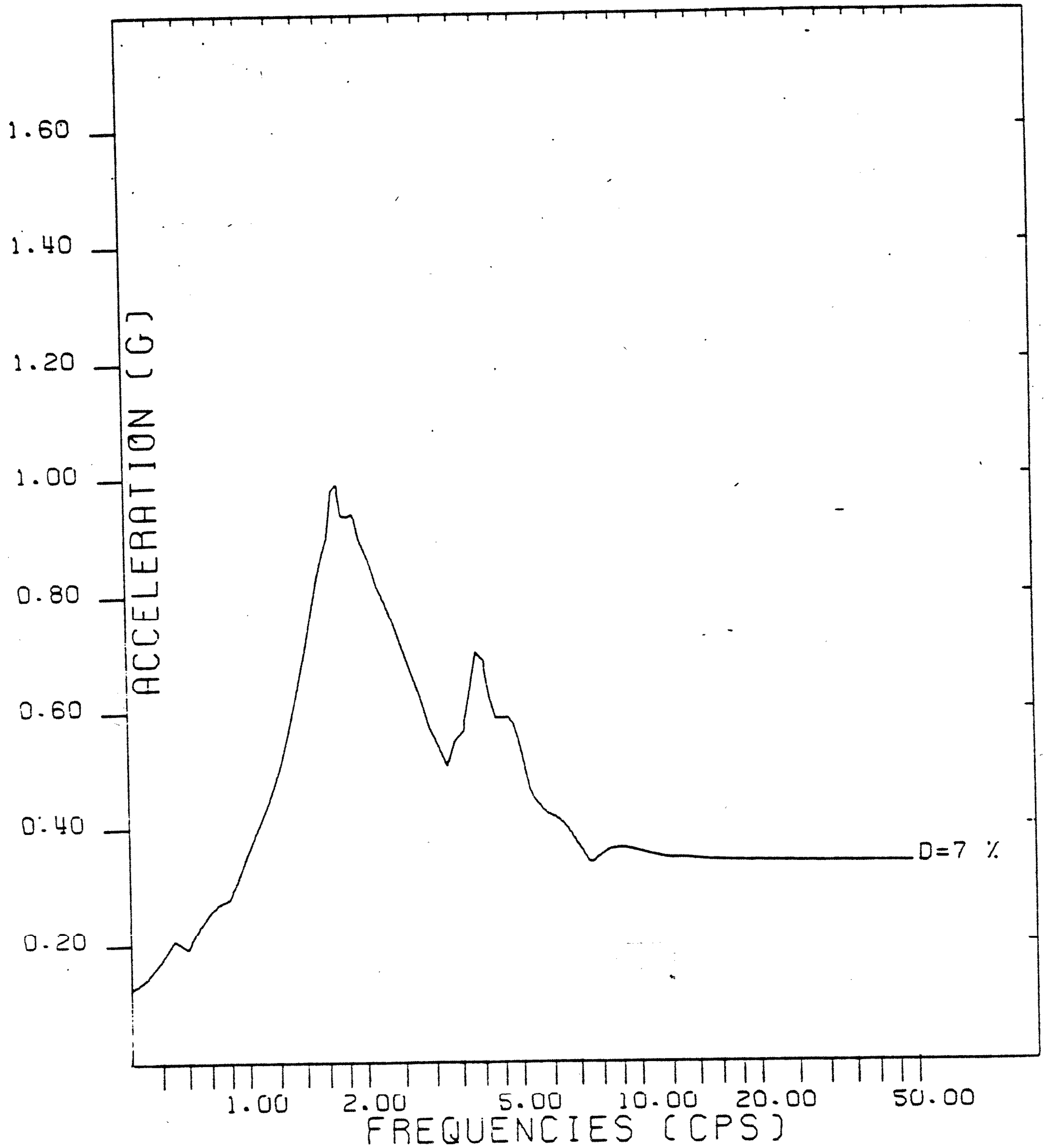
FUEL HANDLING BUILDING MP5
EL. 28.25 FLOOR SPECTRA
DBE E-W

FIGURE 2.5-77



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

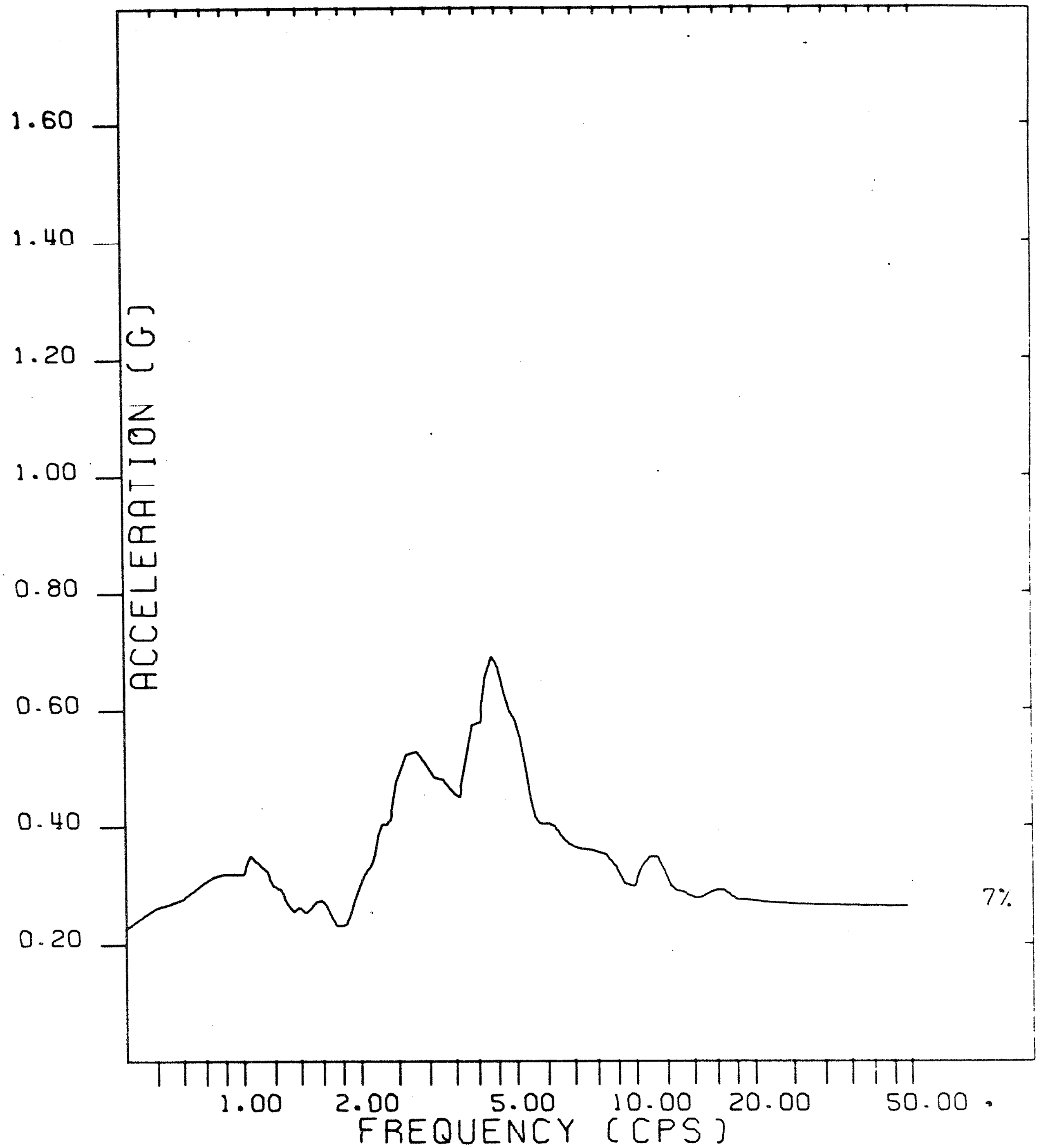
FUEL HANDLING BUILDING
UNAUGMENTED E-W TRANSLATION
0.20G
FIGURE 2.5-78



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

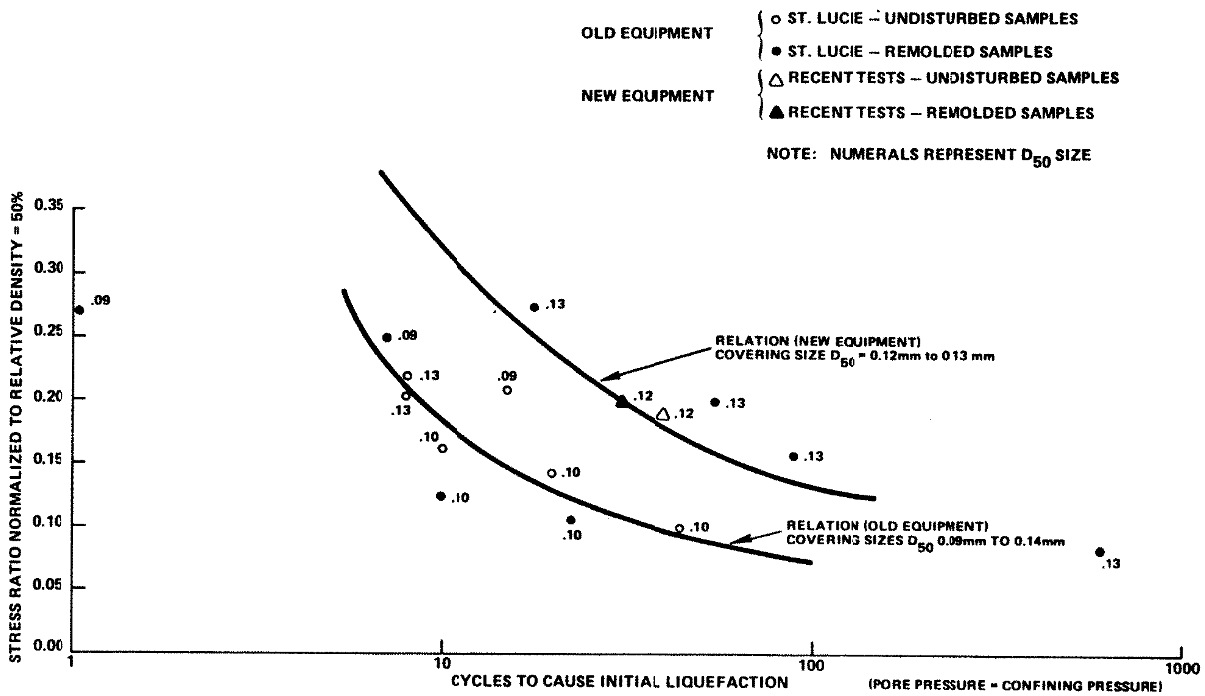
FUEL HANDLING BUILDING MP5
EL. 28.25 FLOOR SPECTRA
DBE N-S

FIGURE 2.5-79



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

FUEL HANDLING BUILDING
UNAUGMENTED N-S TRANSLATION
0.20G
FIGURE 2.5-80



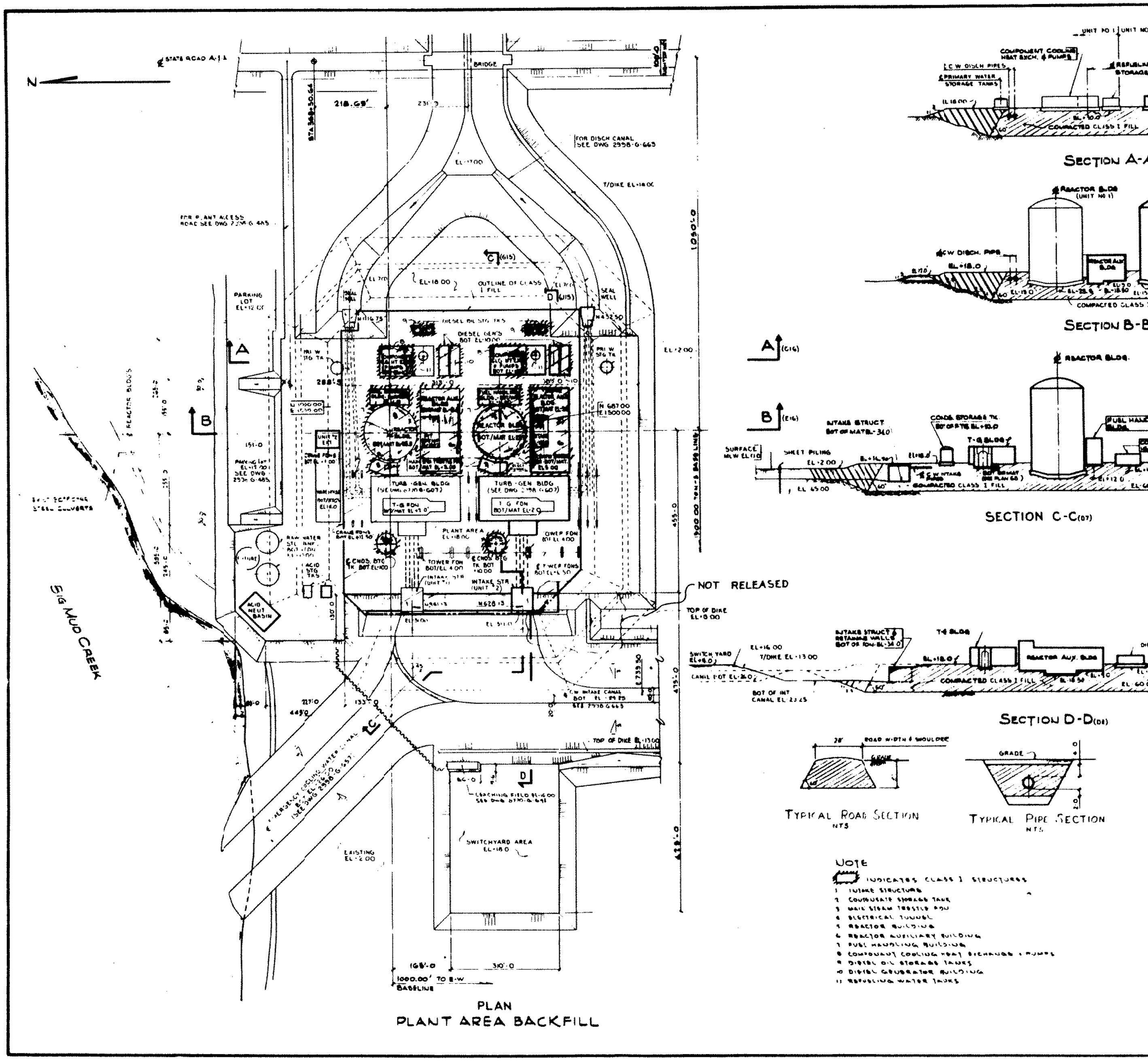
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

EFFECT OF TESTING EQUIPMENT ON CYCLIC STRENGTH CHARACTERISTICS

FIGURE 2.5-81

Refer to drawing
8770-G-481

FLORIDA POWER & LIGHT COMPANY **ST. LUCIE PLANT UNIT 1**
EXCAVATION PLANS & DETAILS-SH. NO 1 FIGURE 2.5-82



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

CLASS I
STRUCTURES AND BACKFILL

FIGURE 2.5-83

DELETED

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

FIGURE 2.5-84

Withheld Under 10 CFR 2.390

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

CLASS I DUCT RUNS

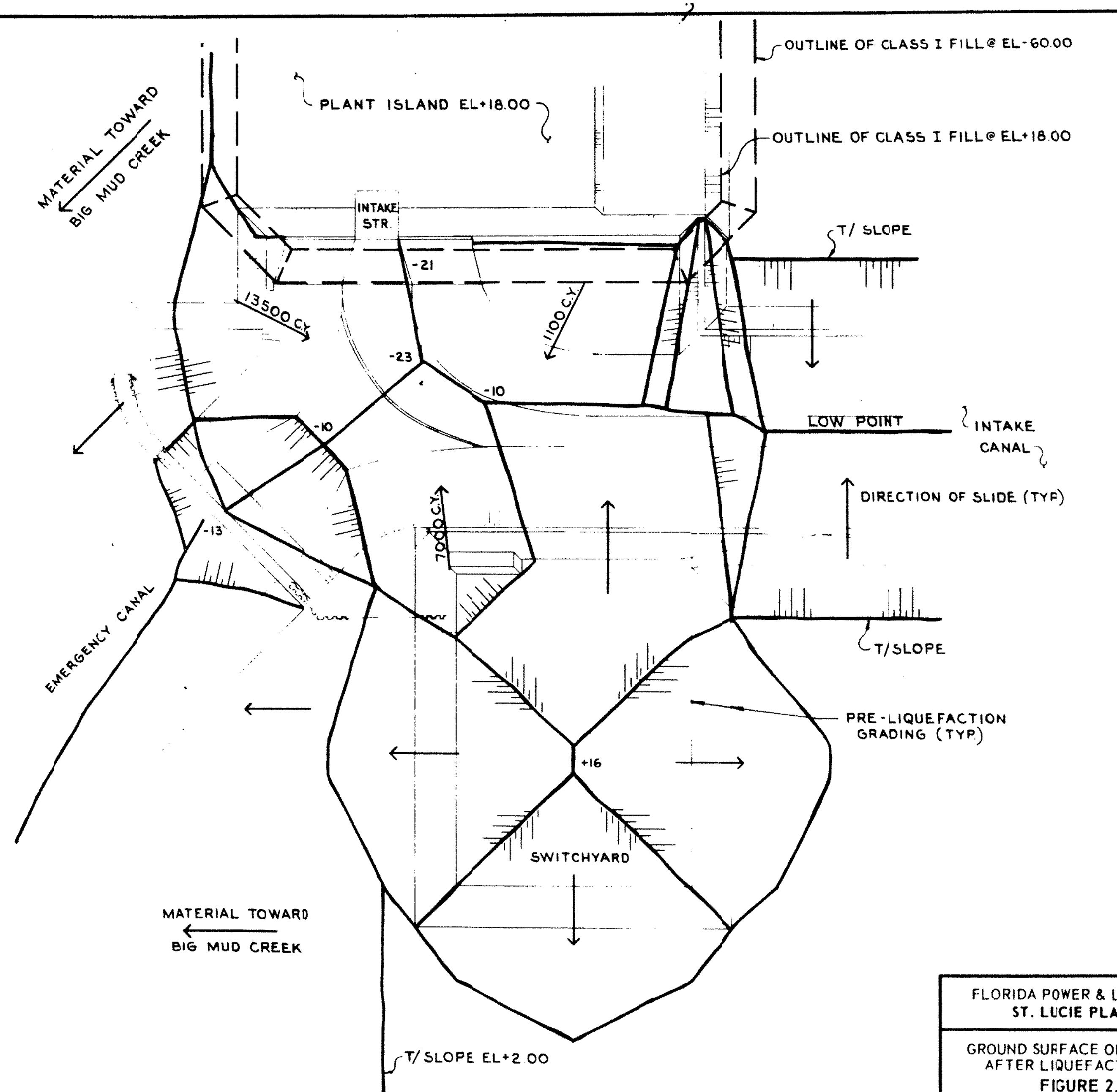
FIGURE 2.5-85

Withheld Under 10 CFR 2.390

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

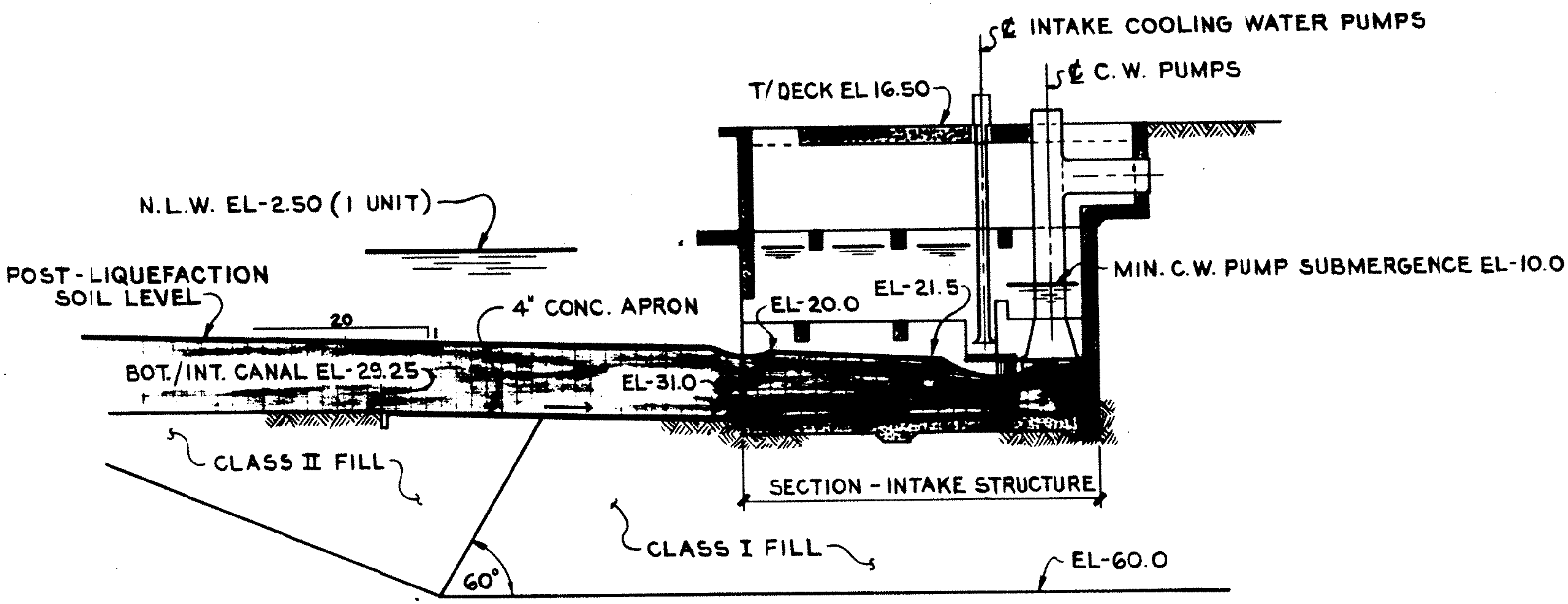
CLASS I BURIED PIPE

FIGURE 2.5-86



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

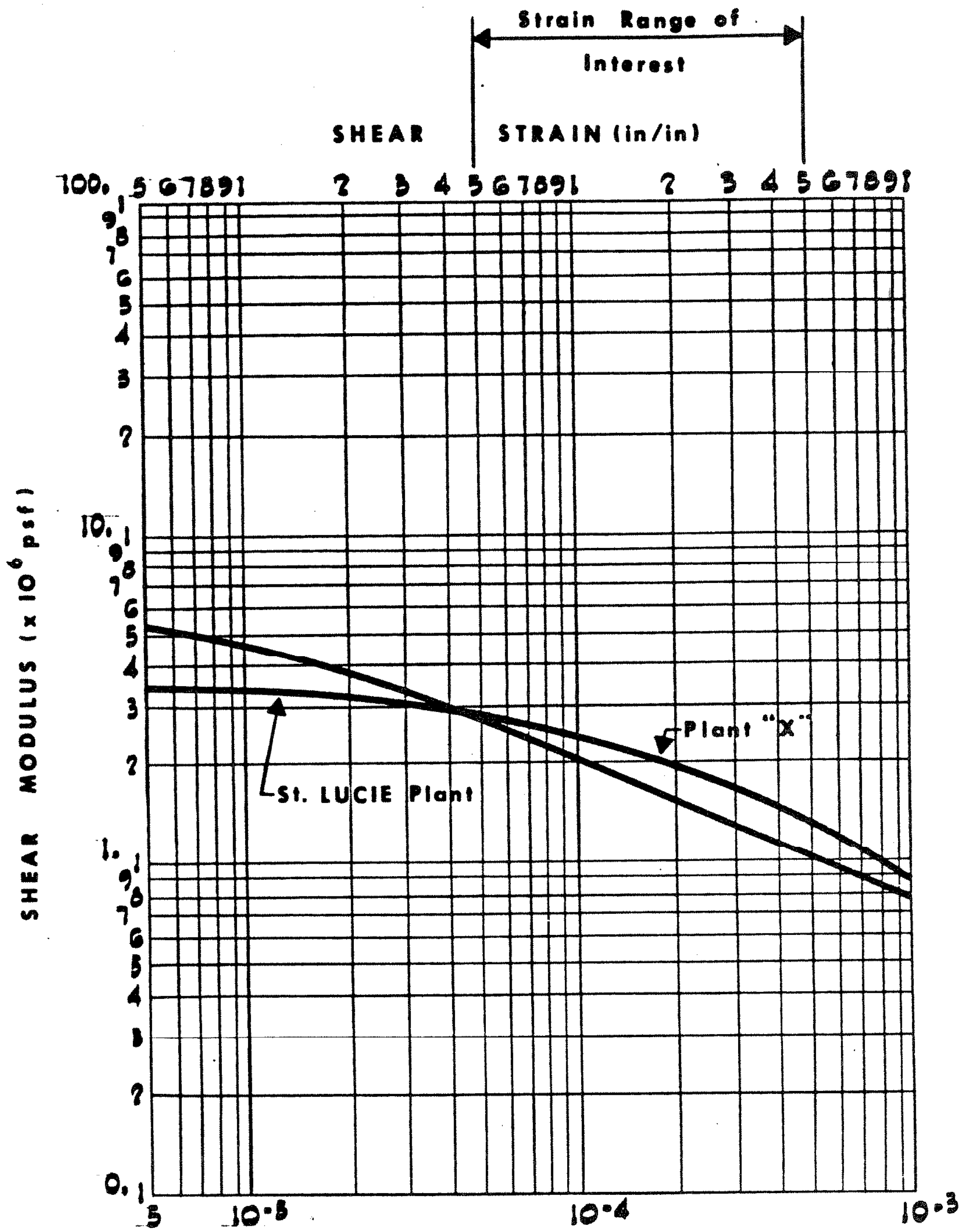
GROUND SURFACE OF INTAKE AREA
AFTER LIQUEFACTION OF SOIL
FIGURE 2.5-87



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

LIQUEFACTION STUDY AT INTAKE STRUCTURE

FIGURE 2.5-88

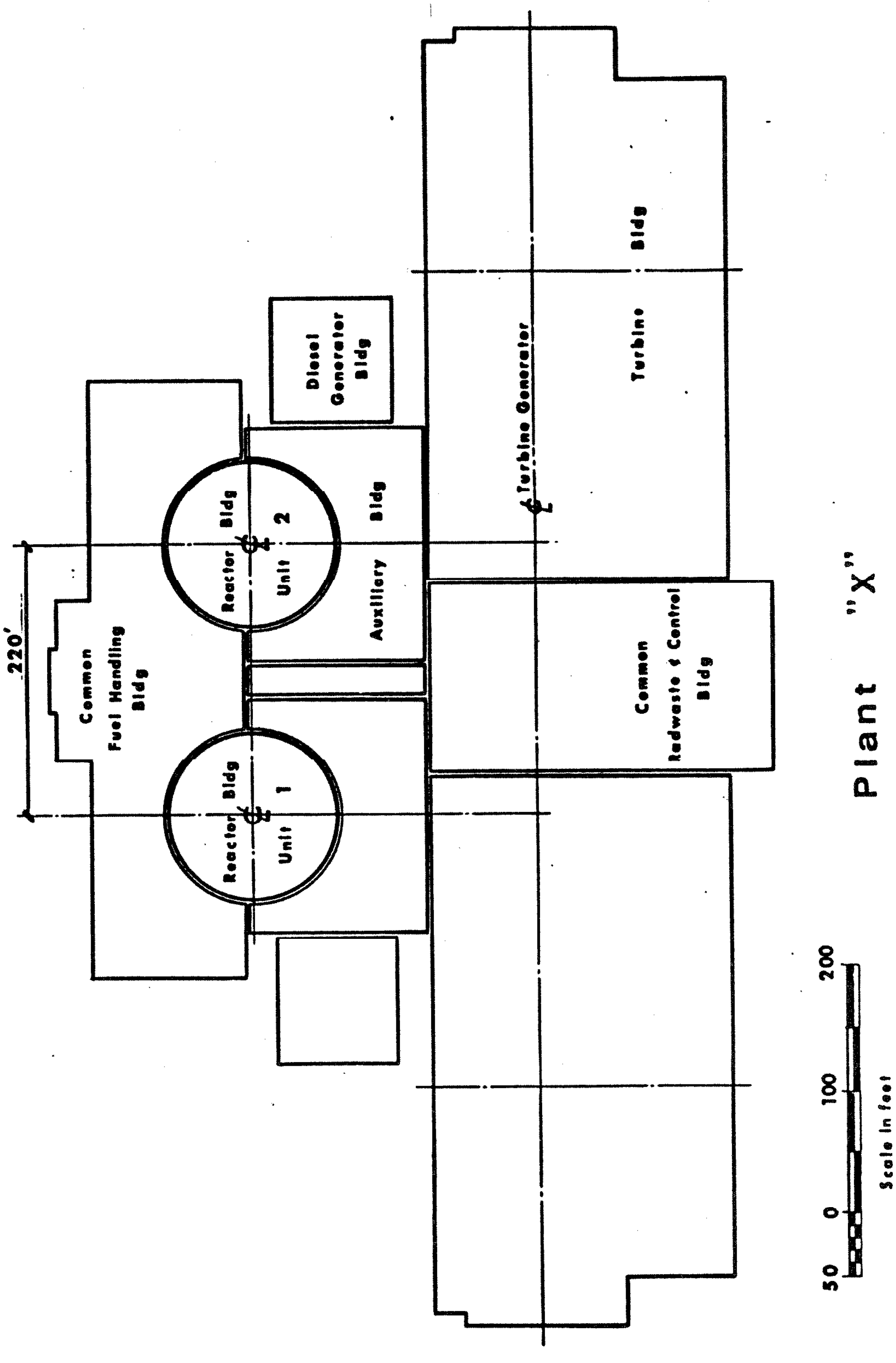


SHEAR MODULUS vs SHEAR STRAIN

St. LUCIE Plant
Plant "X"

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

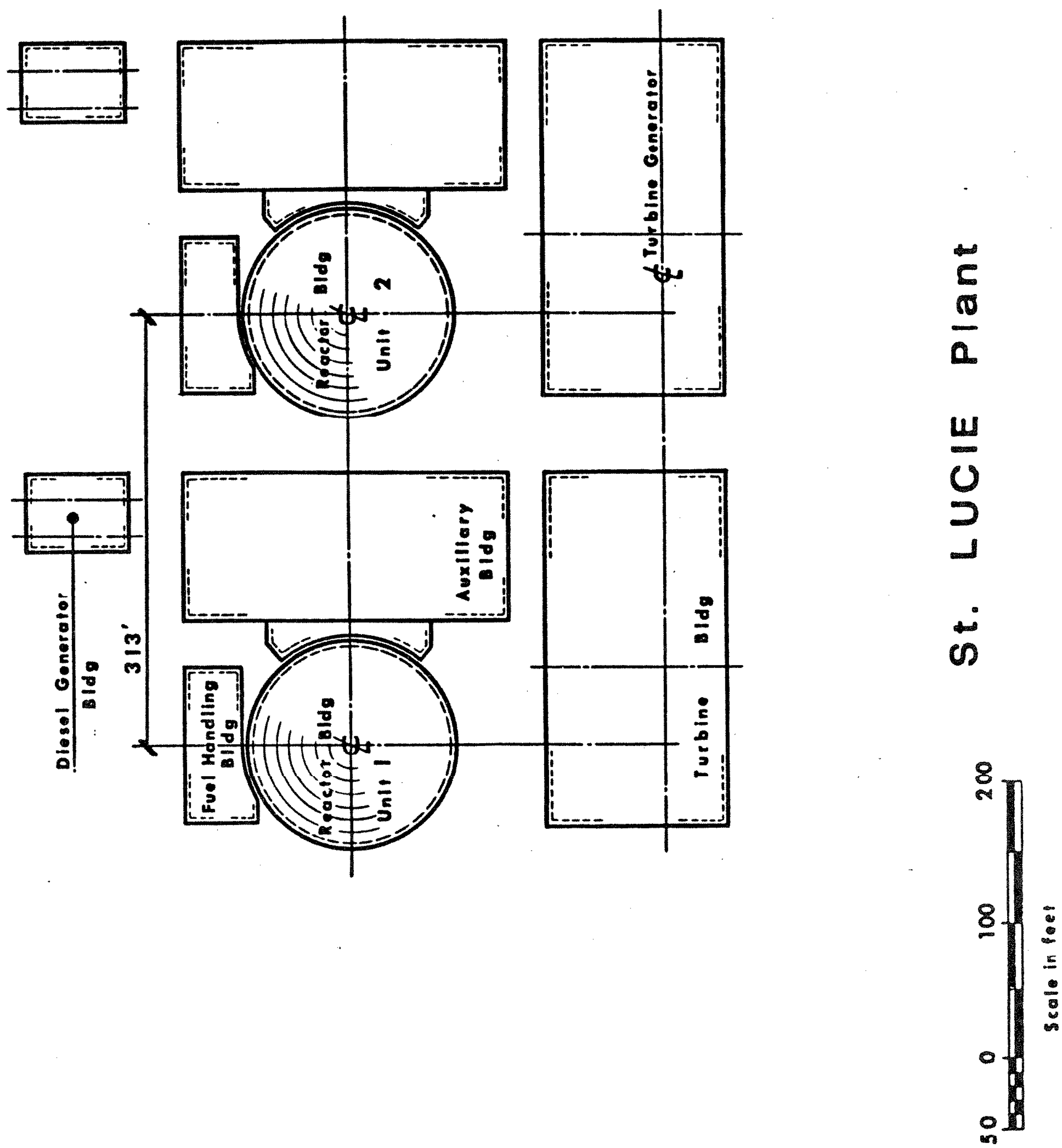
SHEAR MODULUS vs SHEAR STRAIN
ST. LUCIE PLANT
PLANT "X"
FIG 2.5-89



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

TYPICAL 2 UNIT LAYOUT

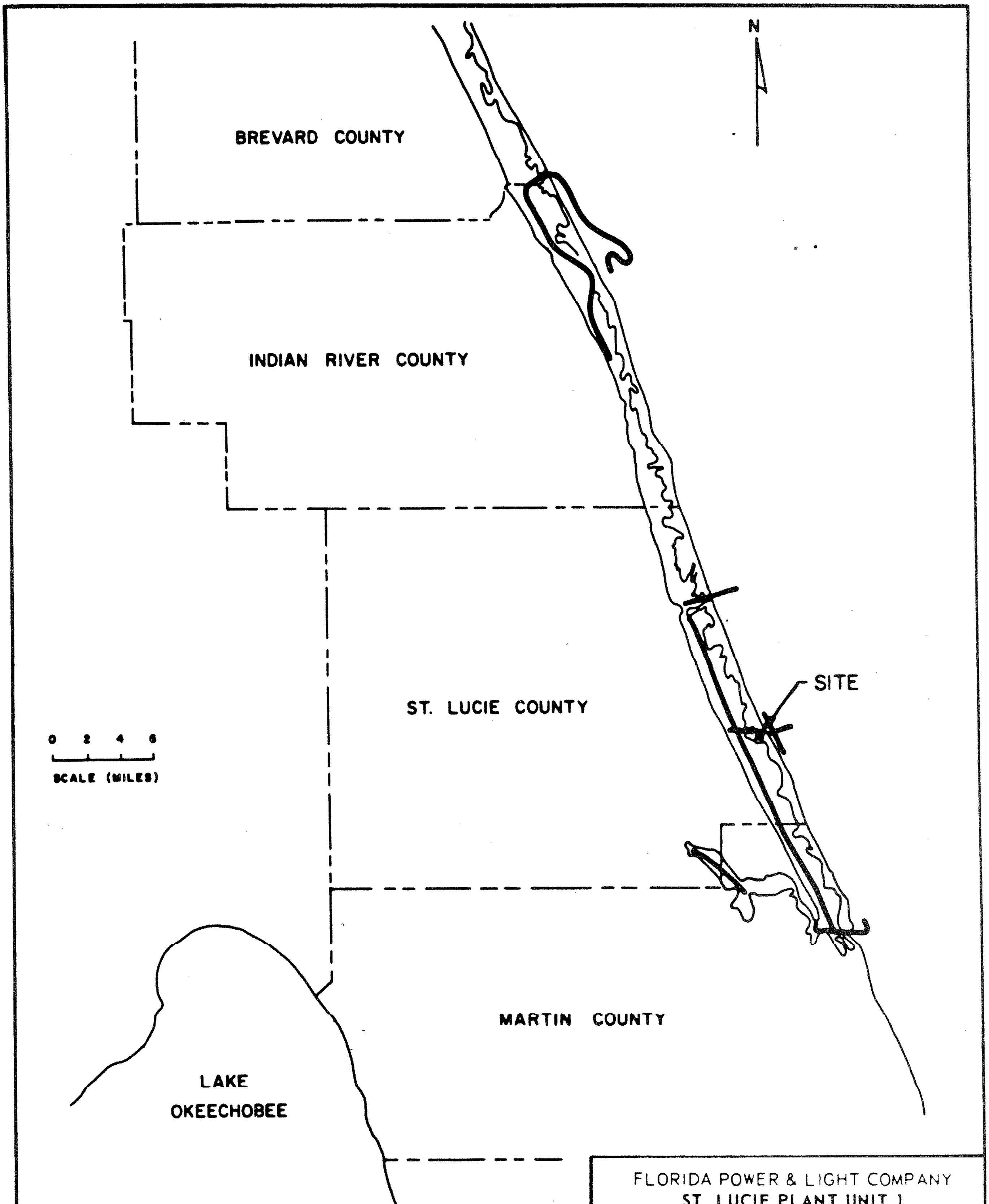
FIGURE 2.5-90



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

ST. LUCIE PLANT LAYOUT - 2 UNITS

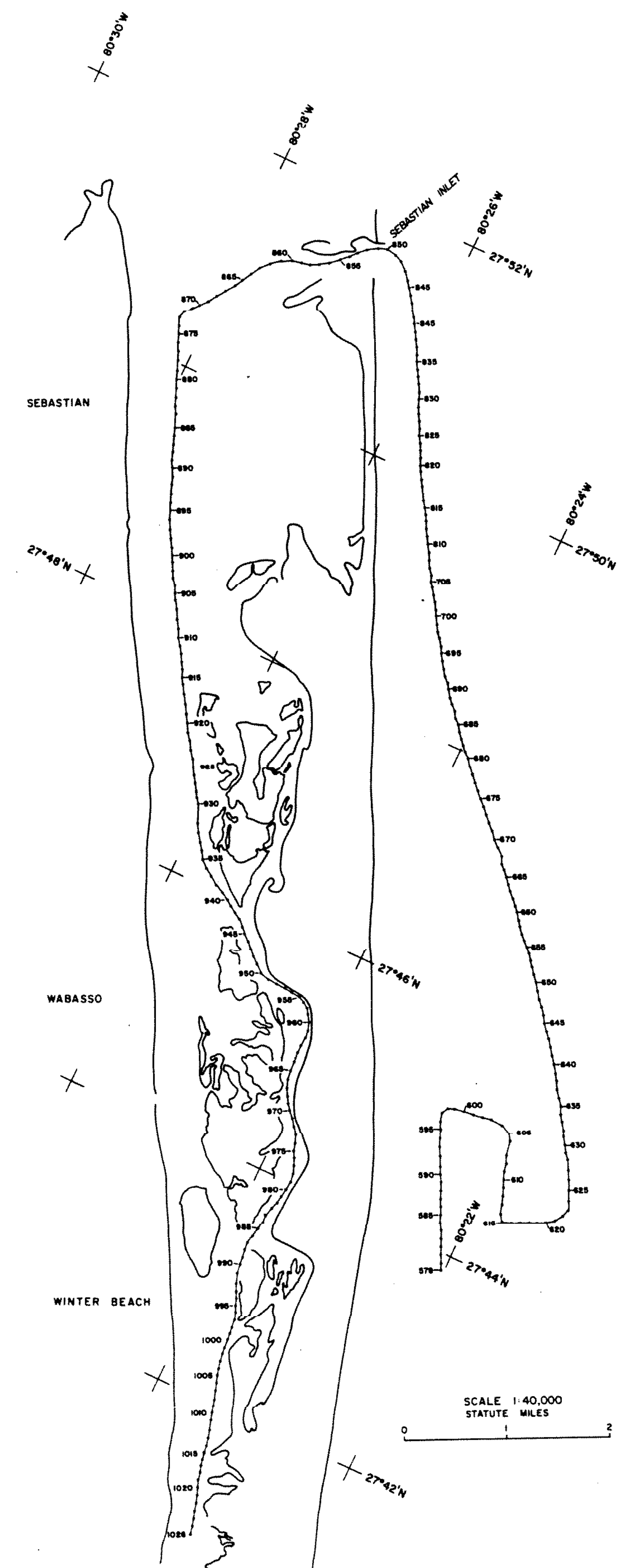
FIGURE 2.5-91



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

AREA OF REFLECTION SURVEY

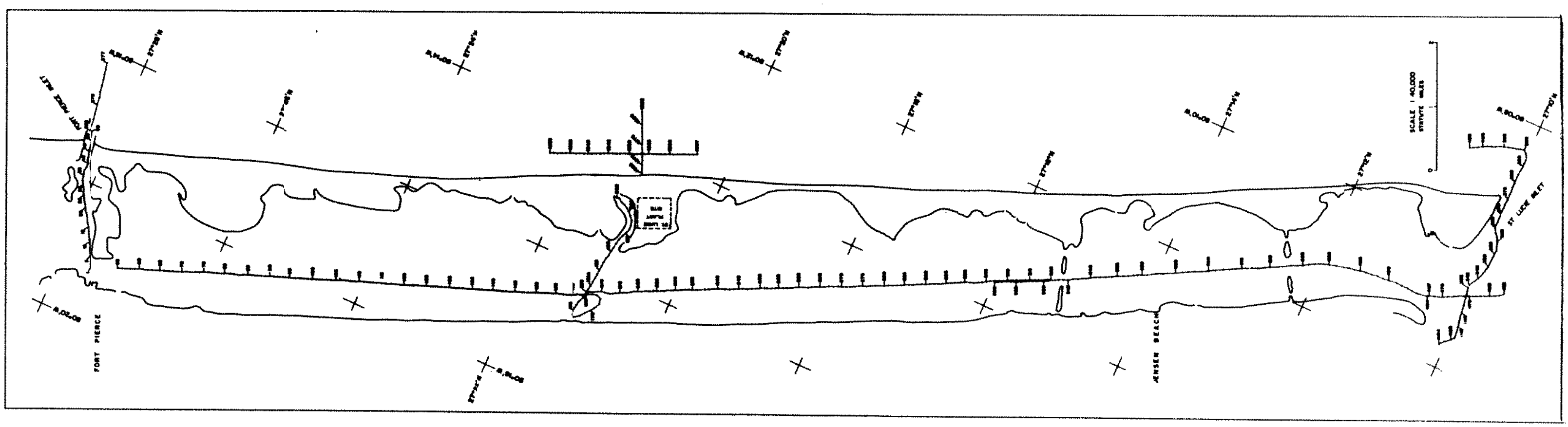
FIGURE 2.5-92



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

NAVIGATION CHART

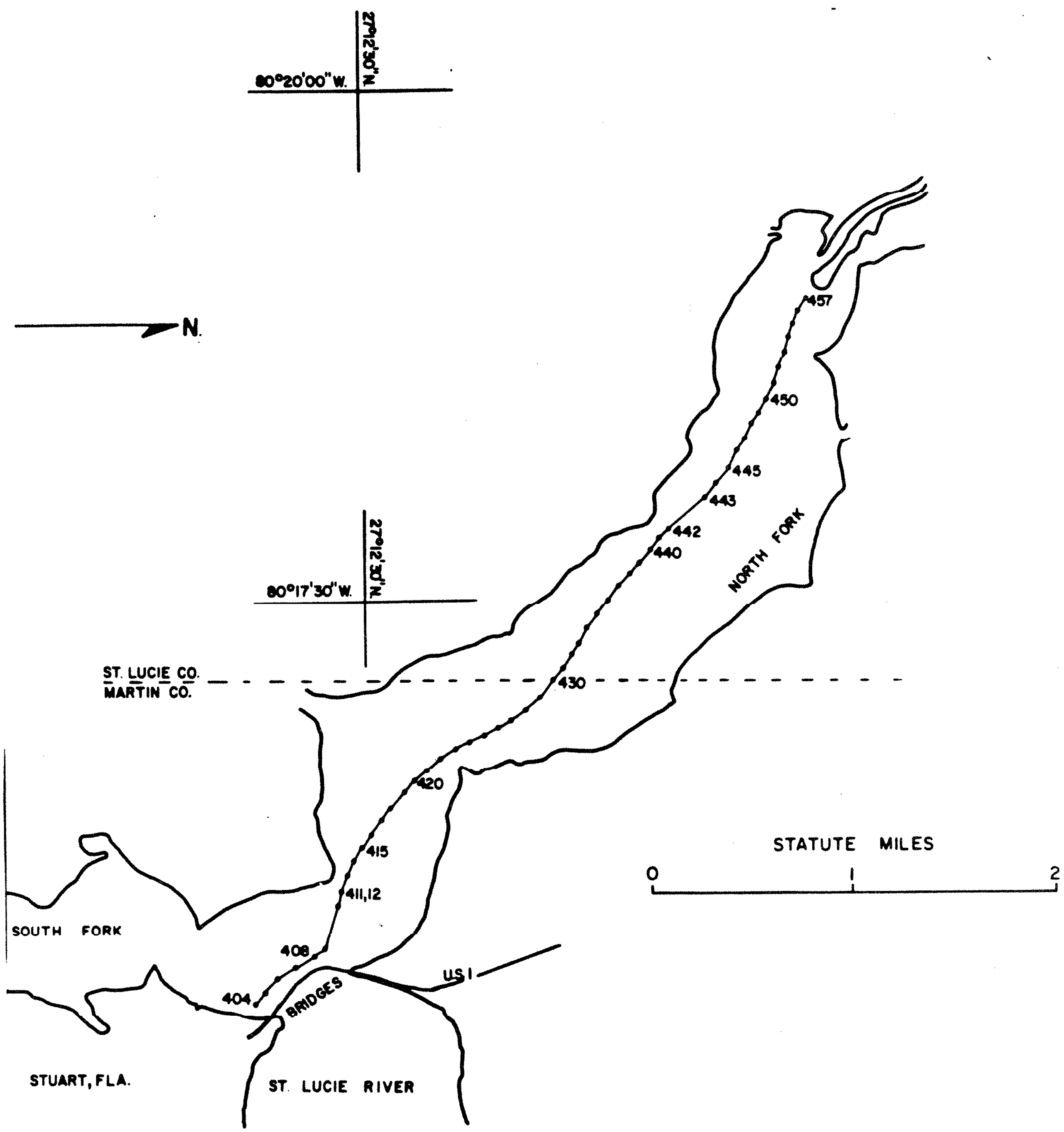
FIGURE 2.5-93



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

NAVIGATION CHART

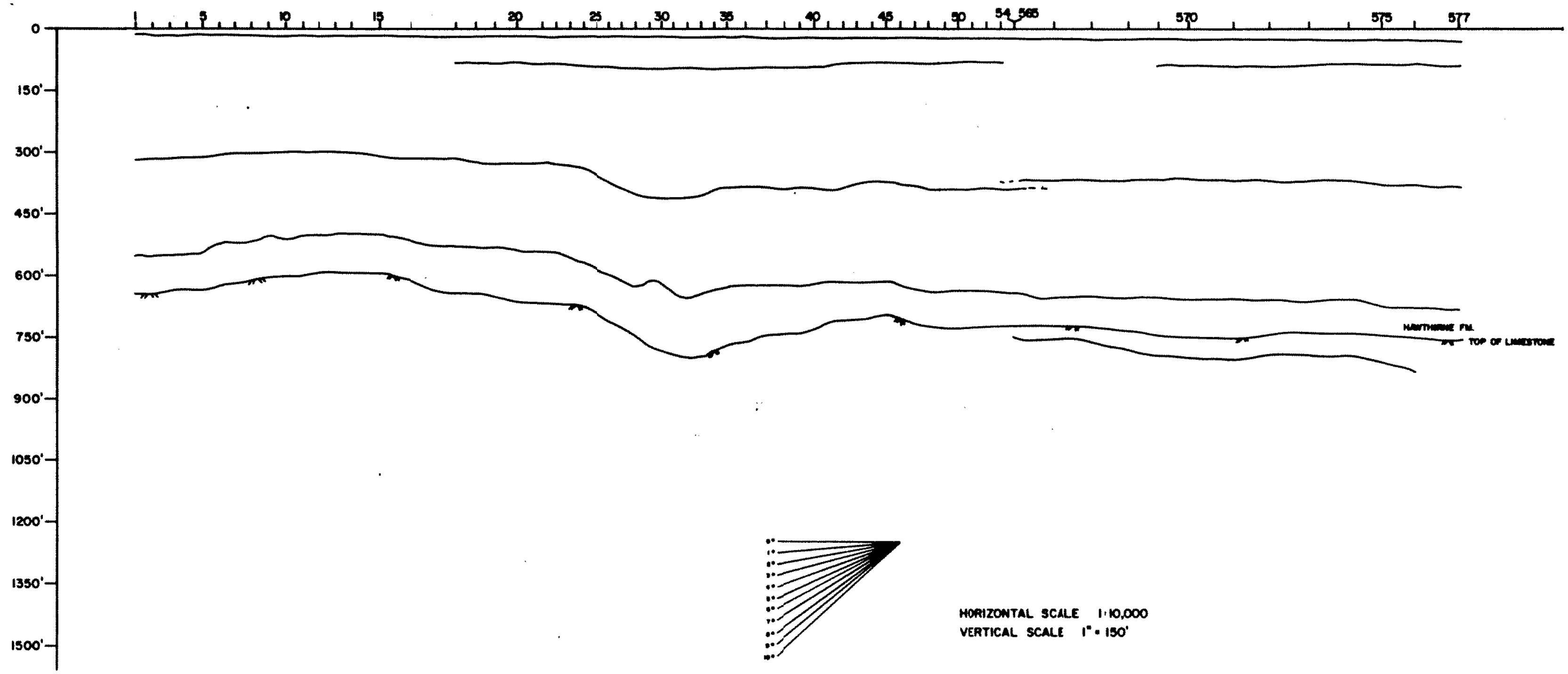
FIGURE 2.5-94



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

NAVIGATION CHART

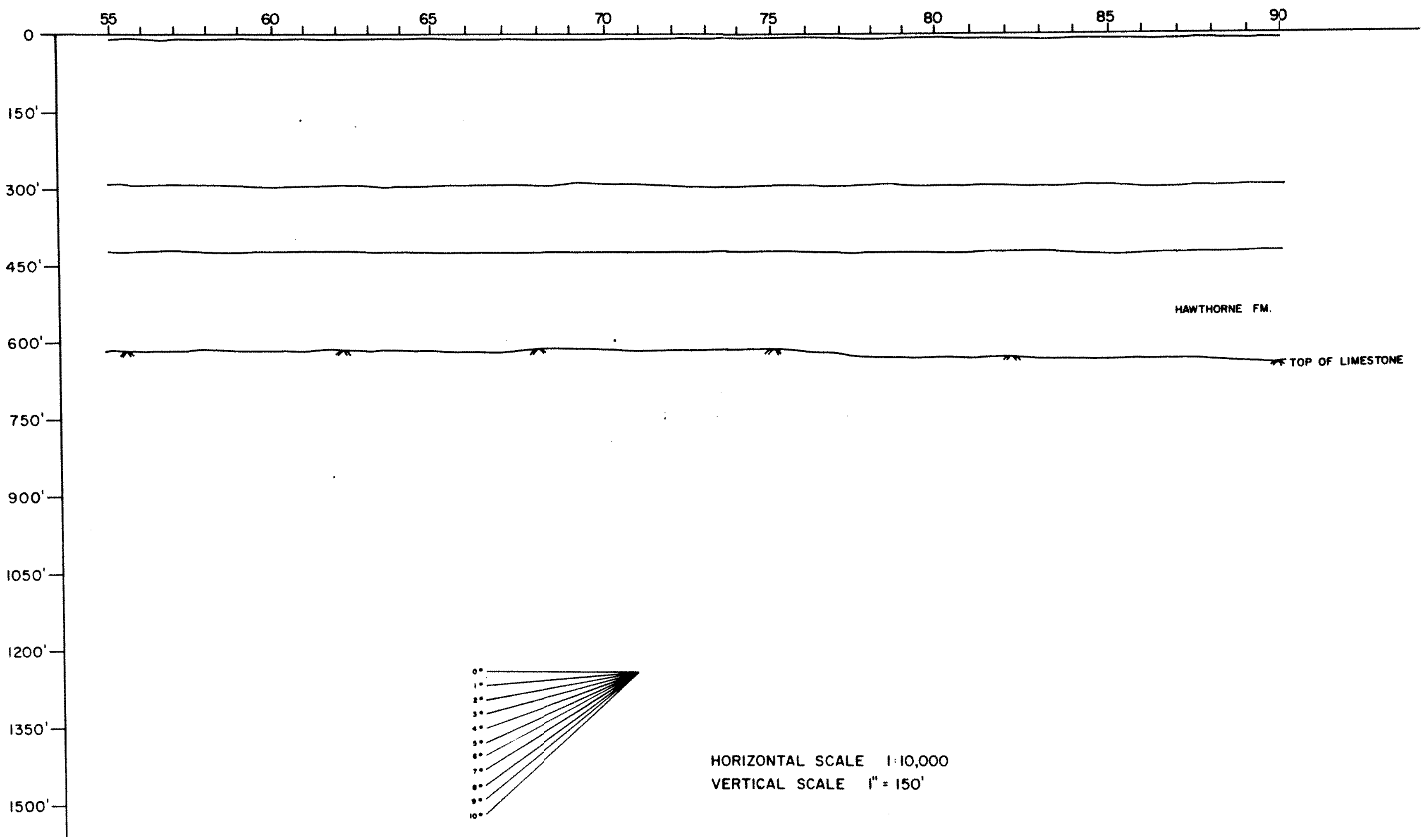
FIGURE 2.5-95



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SEISMIC PROFILE
3000 JOULE SPARKER

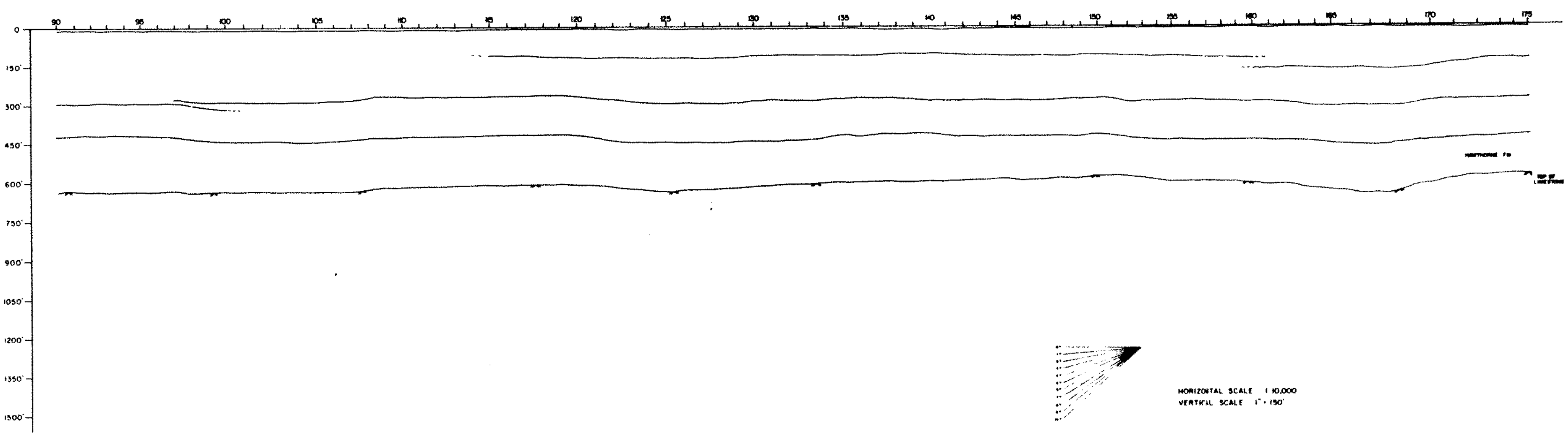
FIGURE 2.5-96



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SEISMIC PROFILE
3000 JOULE SPARKER

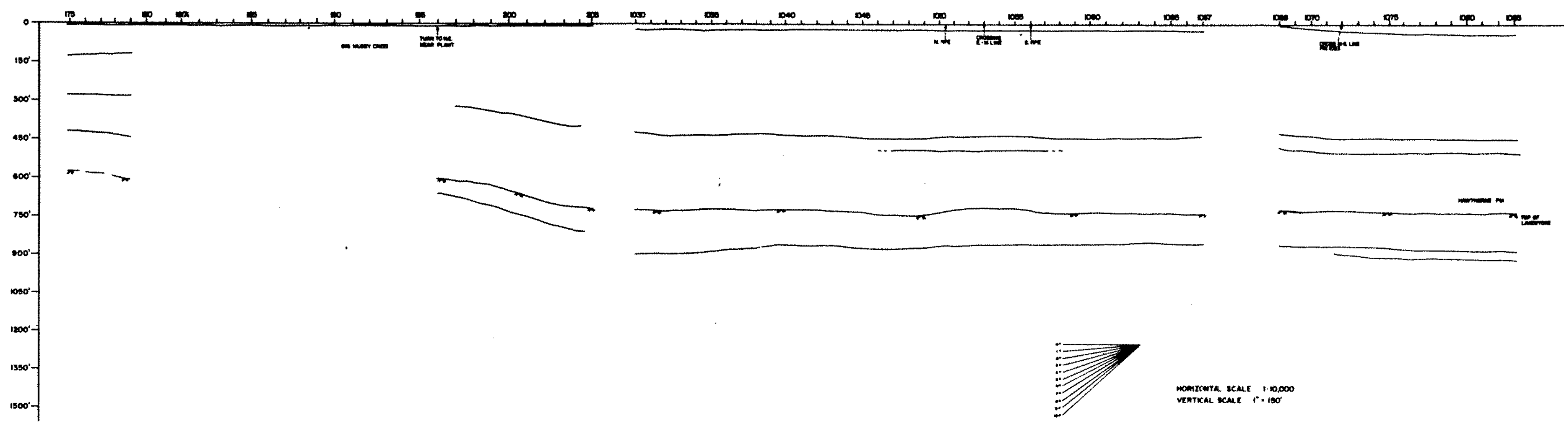
FIGURE 2.5-97



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SEISMIC PROFILE
3000 JOULE SPARKER

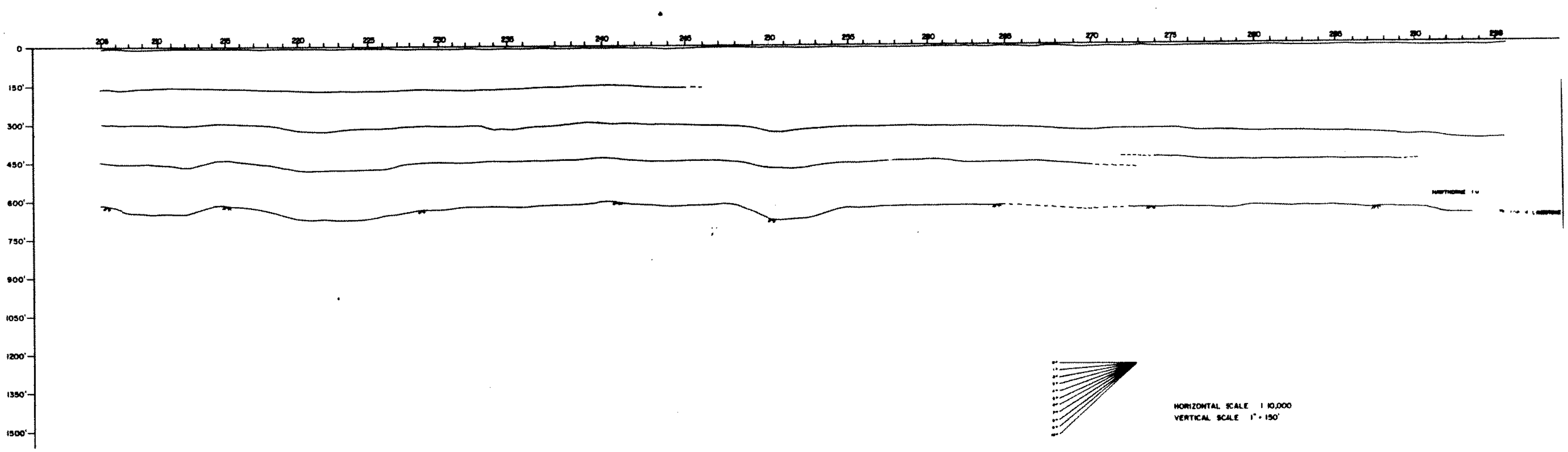
FIGURE 2.5-98



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SEISMIC PROFILE
3000 JOULE SPARKER

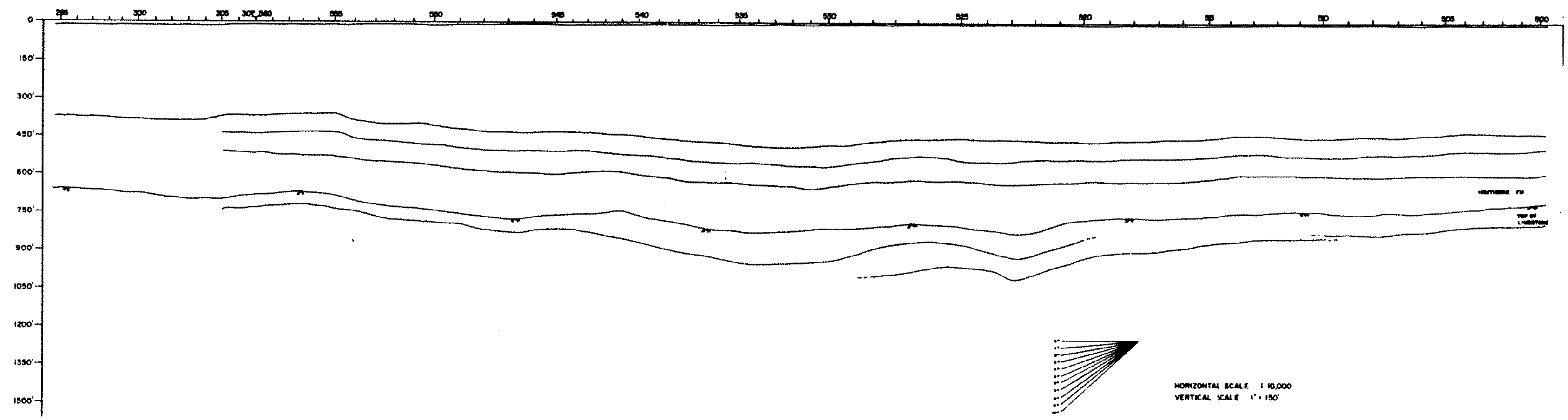
FIGURE 2.5-99



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SEISMIC PROFILE
3000 JOULE SPARKER

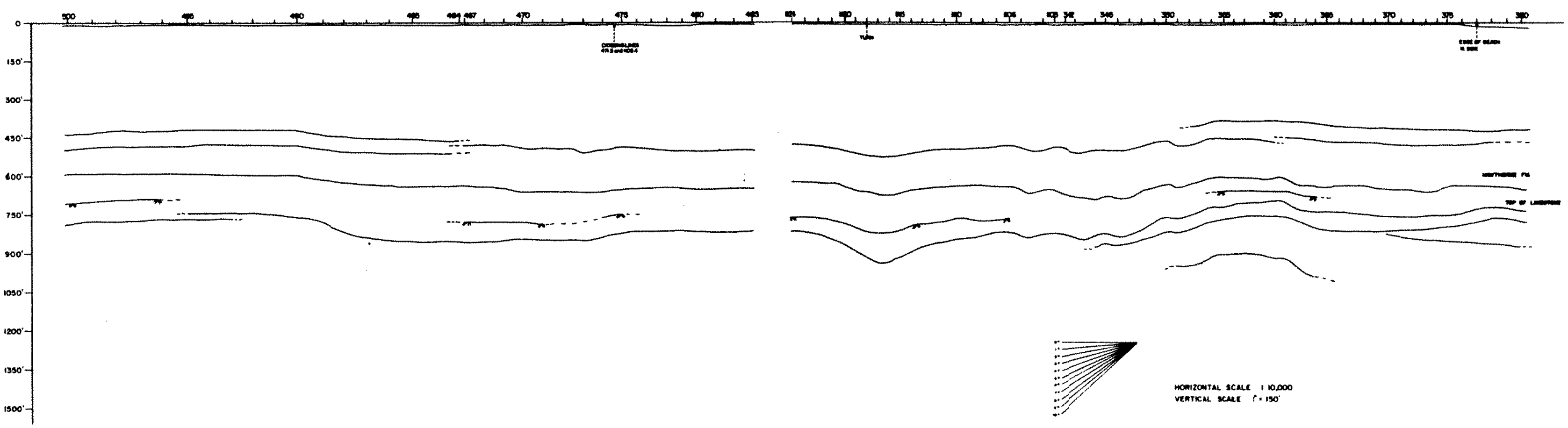
FIGURE 2.5-100



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SEISMIC PROFILE
3000 JOULE SPARKER

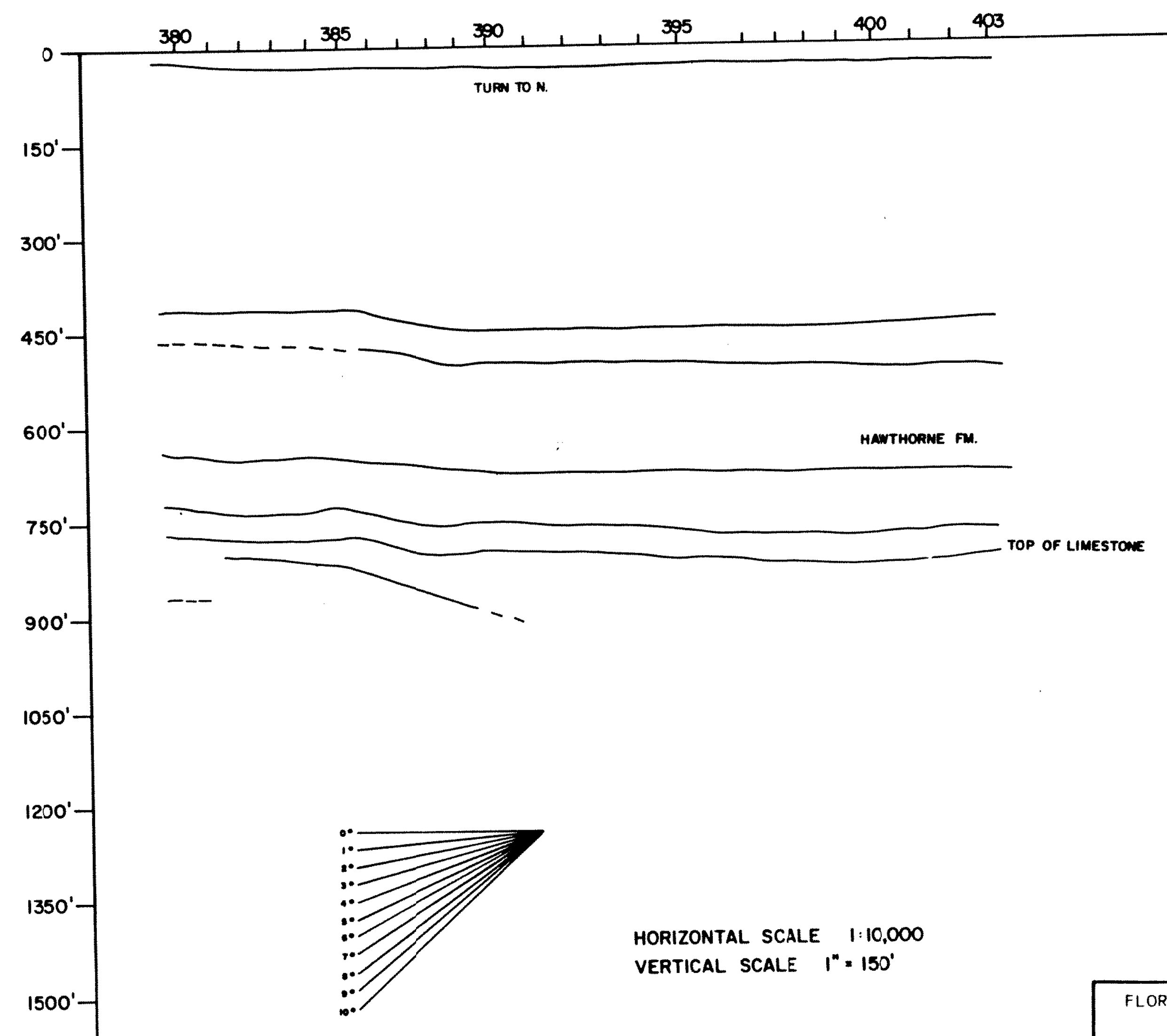
FIGURE 2.5-101



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SEISMIC PROFILE
3000 JOULE SPARKER

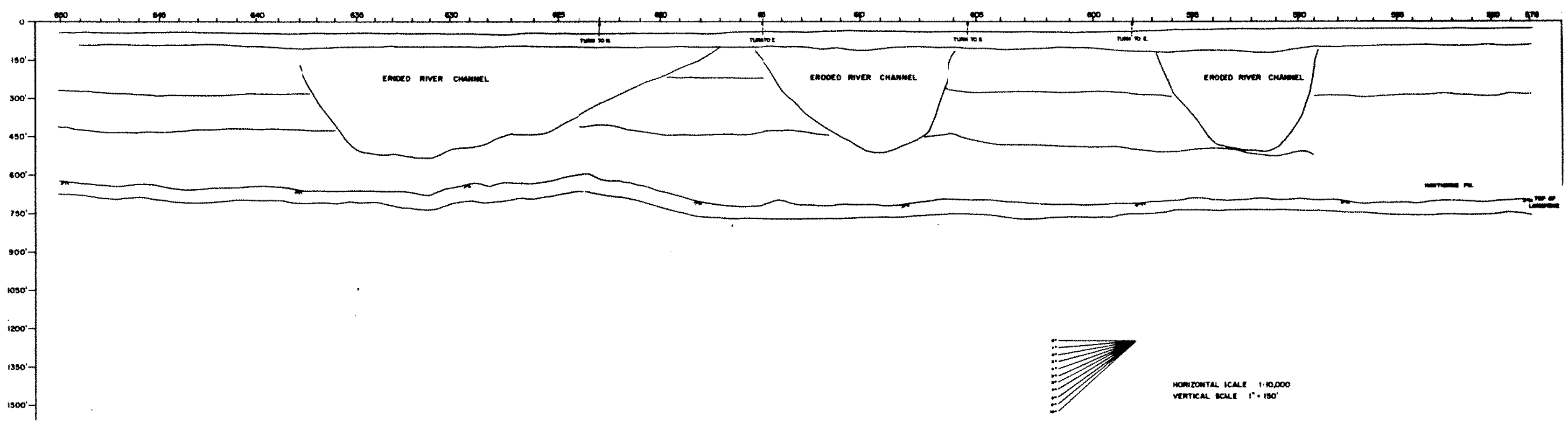
FIGURE 2.5-102



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SEISMIC PROFILE
3000 JOULE SPARKER

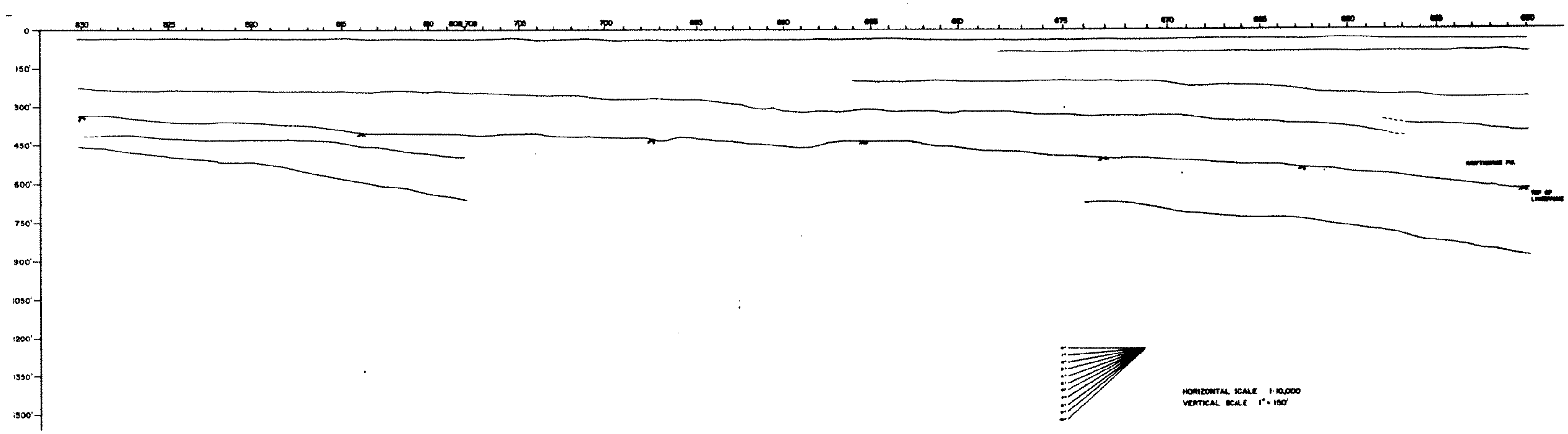
FIGURE 2.5-103



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SEISMIC PROFILE
3000 JOULE SPARKER

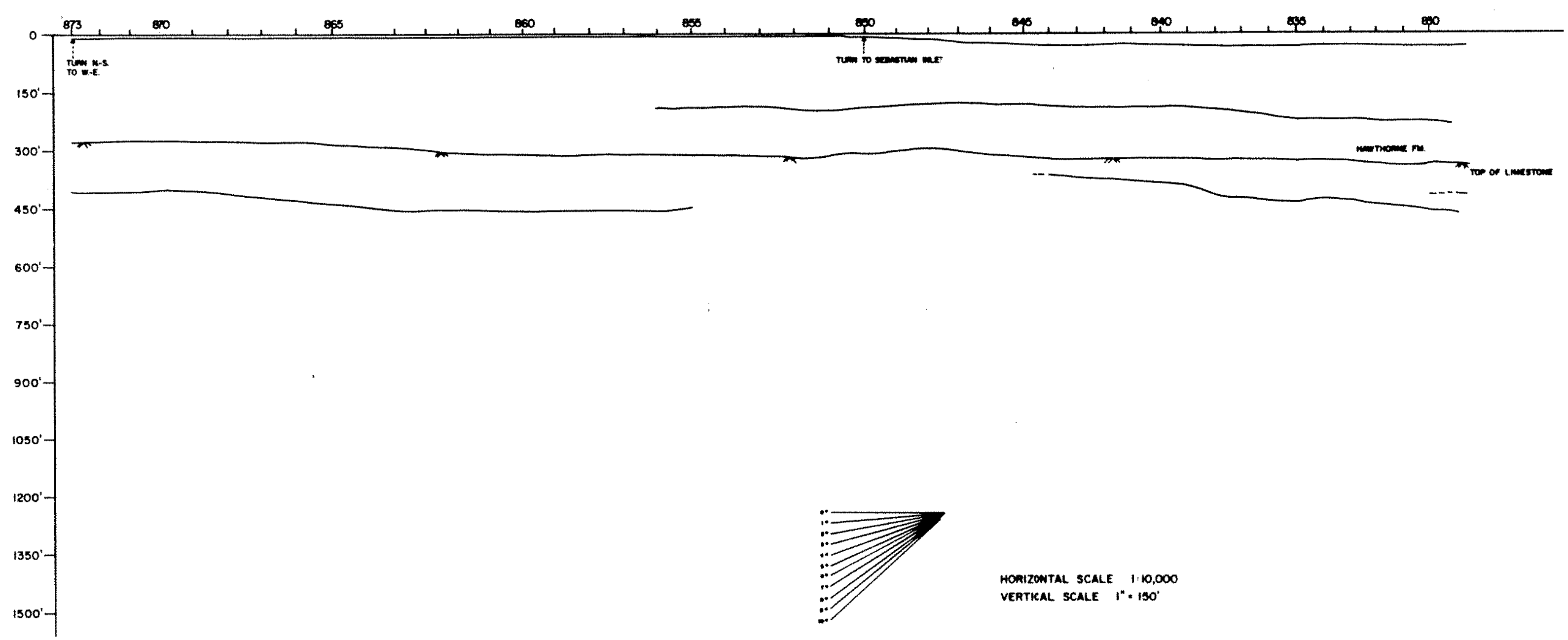
FIGURE 2.5-104



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SEISMIC PROFILE
3000 JOULE SPARKER

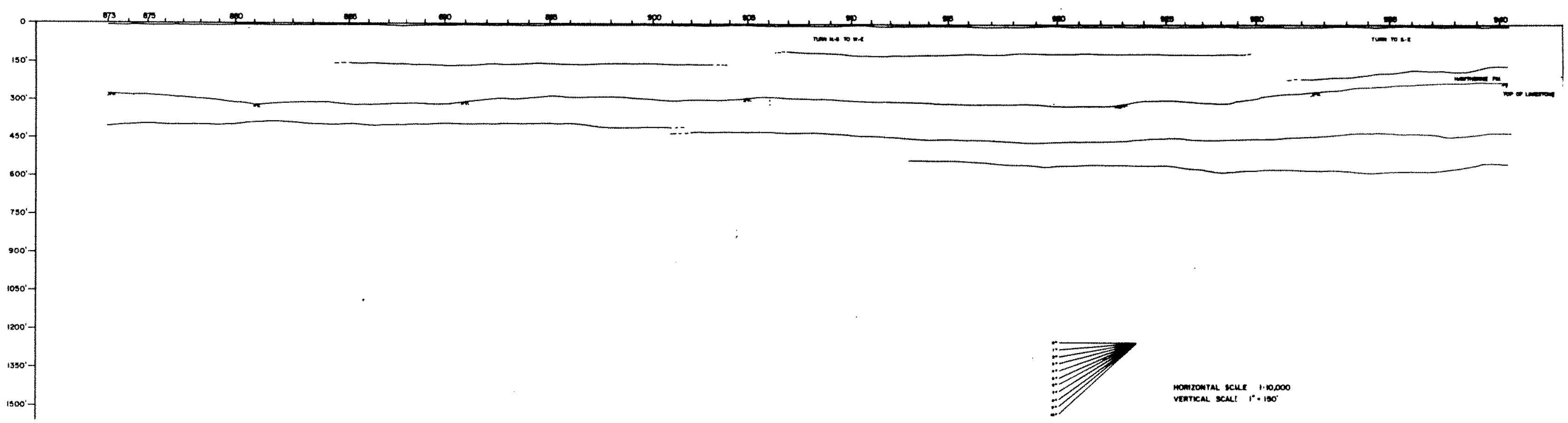
FIGURE 2.5-105



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SEISMIC PROFILE
3000 JOULE SPARKER

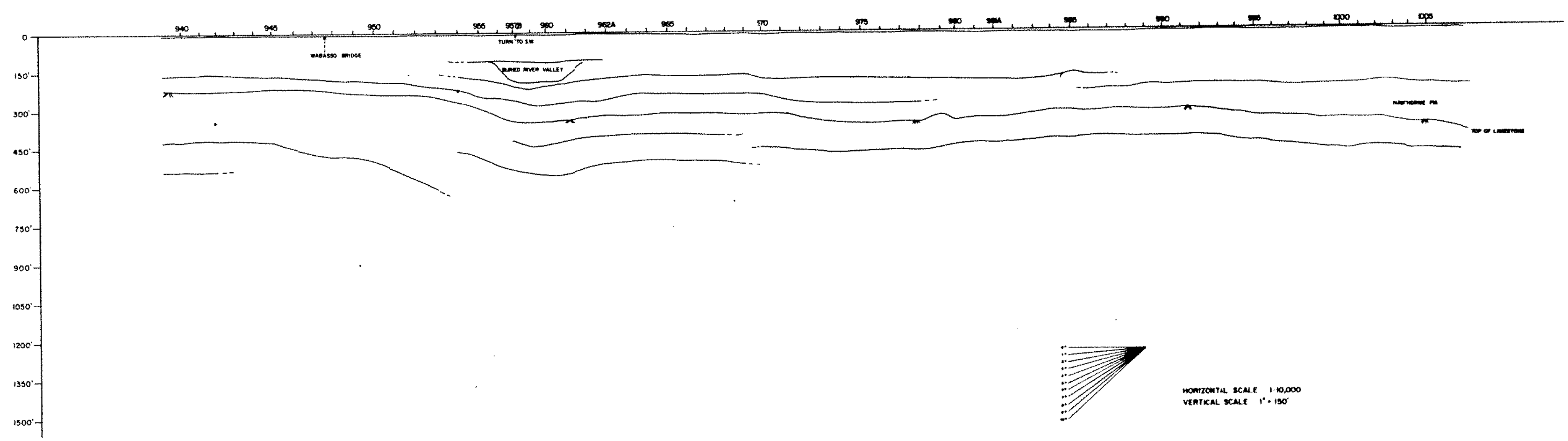
FIGURE 2.5-106



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SEISMIC PROFILE
3000 JOULE SPARKER

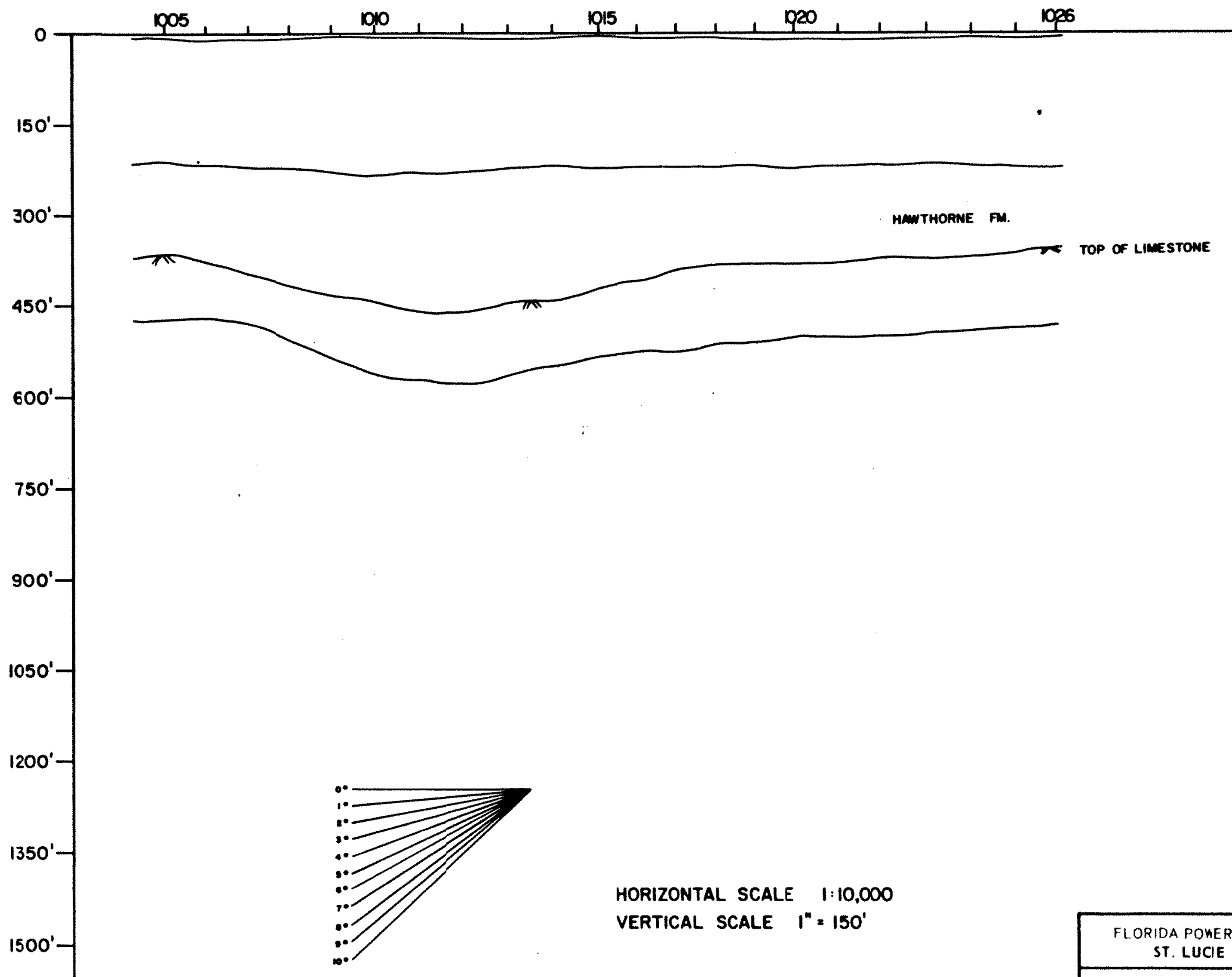
FIGURE 2.5-107



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SEISMIC PROFILE
3000 JOULE SPARKER

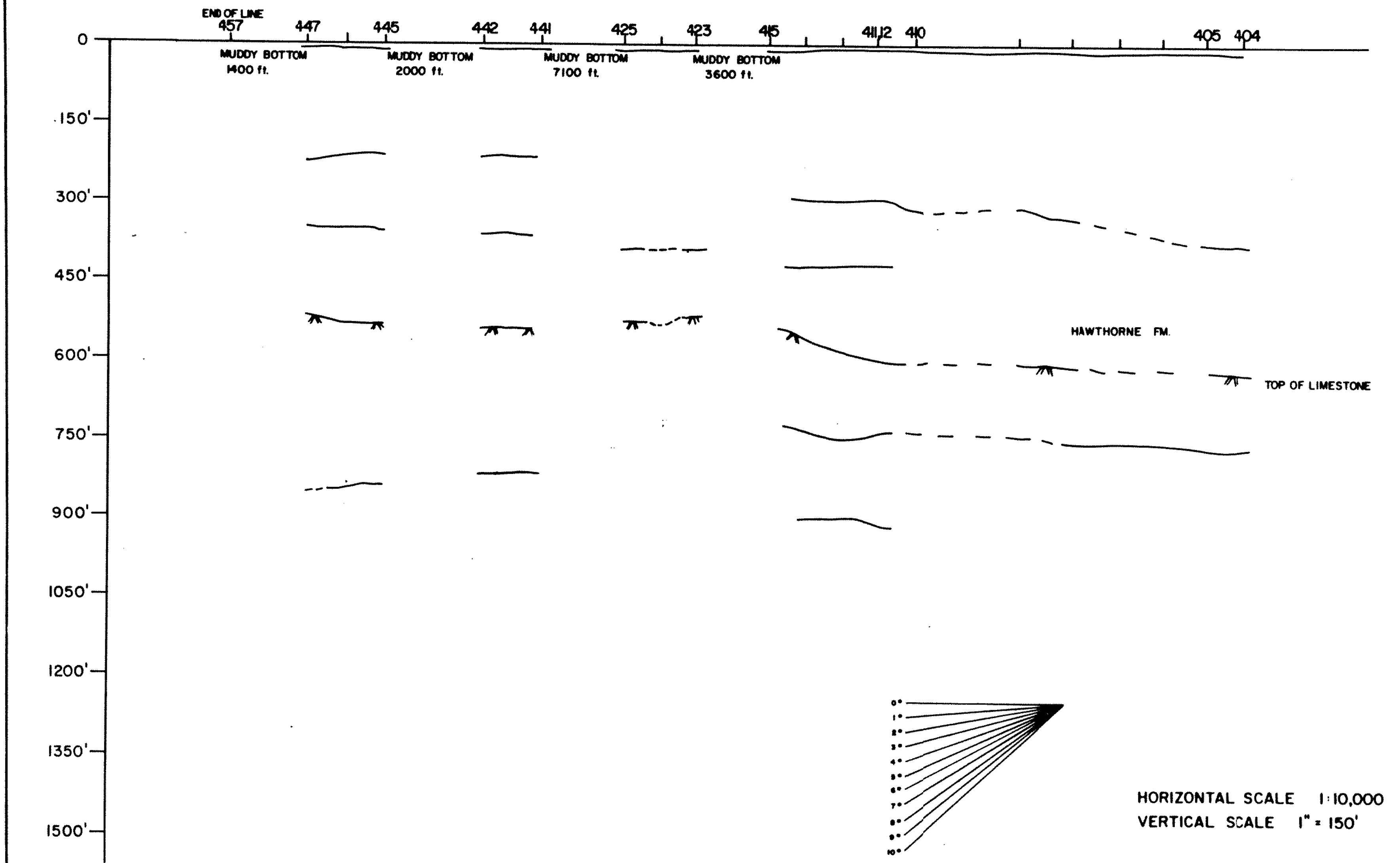
FIGURE 2.5-108



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SEISMIC PROFILE
3000 JOULE SPARKER

FIGURE 2.5-109



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SEISMIC PROFILE
3000 JOULE SPARKER

FIGURE 2.5-110

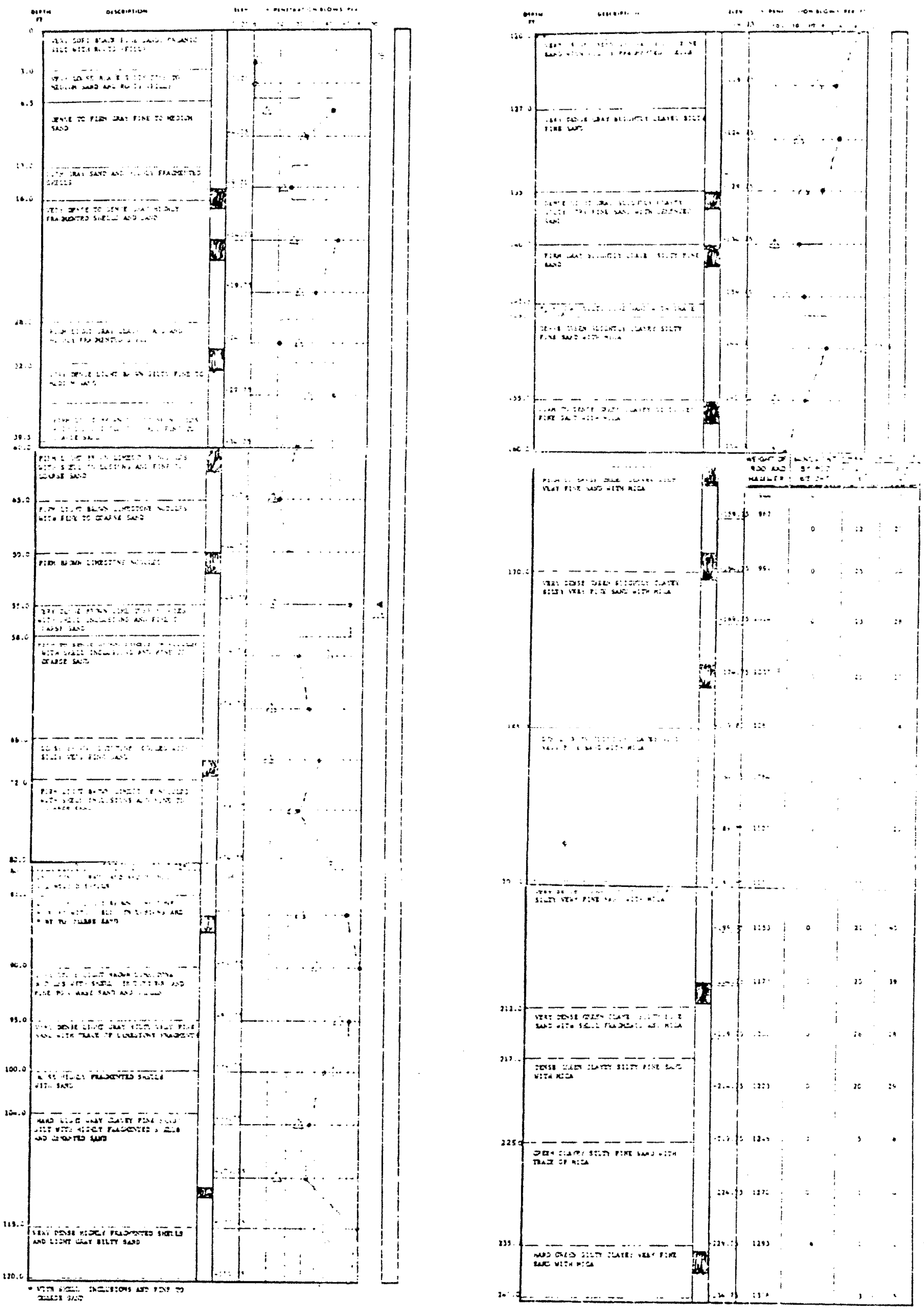
FLORIDA POWER & LIGHT COMPANY

ST. LUCIE PLANT - UNIT NO. 1

FINAL SAFETY ANALYSIS REPORT

APPENDIX 2A

BORING LOGS AND DATA SUMMARIES



△ PENETRATION BLOWS PER FOOT USING "A" ROD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D 2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE — WATER TABLE 24 HR — WATER TABLE 1 HR

% ROCK CORE RECOVERY — LOSS OF DRILLING WATER

"N" ROD USED IN PENETRATION TEST

TEST BORING RECORD

BORING NO ______

DATE DRILLED ______

JOB NO ______

LAW ENGINEERING TESTING CO

DEPTH FT.	DESCRIPTION	ELEV. -234.75	WEIGHT OF ROD AND HAMMER	MOVEMENT BY ROD WEIGHT	HAMMER BLOWS 2nd 6"	HAMMER BLOWS 3rd 6"
240.0	HARD GREEN SILTY CLAYEY VERY FINE SAND WITH MICA AND A TRACE OF SHELLS		1316	0	3	5
		-239.75	1330	0	4	5
		-244.75	1363	6"	1	6
250.0	HARD GREEN VERY FINE SANDY CLAYEY SILT WITH MICA					
		-249.75	1386	18"	0	0
255.0	HARD GREEN CLAYEY VERY FINE SANDY SILT WITH MICA					
		-254.75	1409	18"	0	0
		-259.75	1432	18"	0	0
		-264.75	1456	18"	0	0
		-269.75	1479	18"	0	0
280.0	HARD GREEN VERY FINE SANDY CLAYEY SILT WITH MICA AND A TRACE OF SHELLS	-274.75	1502	18"	0	0
285.0	HARD GREEN VERY FINE SANDY CLAYEY SILT WITH MICA	-279.75	1525	0	0	0
		-284.75	1549	0	0	0
		-289.75	1572	0	0	0
		-294.75	1595	0	0	0
301.5	DENSE GREEN CLAYEY SILTY VERY FINE SAND WITH TRACE MICA					
		-299.75	1618	12"	0	43
310.0	VERY STIFF GREEN VERY FINE MICACEOUS SANDY CLAYEY SILT	-304.75	1642	12"	0	21
315.0	HARD GREEN VERY FINE MICACEOUS SANDY CLAYEY SILT	-309.75	1665	0	21	47
320.0	HARD GREEN VERY FINE MICACEOUS SANDY CLAYEY SILT	-314.75	1688	0	29	50
		-319.75	1711	0	15	22
		-324.75	1735	6"	3	33
		-329.75	1758	0	18	36
340.0	VERY STIFF GREEN VERY FINE SANDY CLAYEY SILT WITH TRACE MICA	-334.75	1781	0	6	13
		-339.75	1804	12"	0	28
344.5	VERY STIFF GREEN VERY FINE SANDY CLAYEY SILT					
		-344.75	1828	6"	2	22
335.0	HARD GREEN VERY FINE SANDY CLAYEY SILT	-349.75	1851	6"	4	33
360.0		-354.75	1874	6"	7	23

DEPTH FT.	DESCRIPTION	ELEV. -354.75	WEIGHT OF ROD AND HAMMER	MOVEMENT BY ROD WEIGHT	HAMMER BLOWS 2nd 6"	HAMMER BLOWS 3rd 6"
360.0	VERY STIFF GREEN CLAYEY VERY FINE SANDY SILT WITH TRACE MICA		1874	6"	7	23
		-359.75	1897	12"	0	22
		-364.75	1921	12"	0	16
370.0	VERY STIFF GREEN MICACEOUS VERY FINE SANDY CLAYEY SILT					
		-369.75	1943	0	19	29
375.0	HARD GREEN MICACEOUS VERY FINE SANDY CLAYEY SILT					
		-374.75	1967	0	0	19
380.0	VERY STIFF GREEN VERY FINE SANDY CLAYEY SILT WITH TRACE MICA					
		-379.75	1990	6"	7	28
385.0	HARD GREEN VERY FINE SANDY CLAYEY SILT WITH TRACE MICA					
		-384.75	2014	6"	9	29
		-389.75	2037	0	33	50
397.0	HARD GREEN SLIGHTLY CLAYEY SILTY VERY FINE SAND WITH TRACE MICA					
400.0	HARD GREEN [illegible] CLAYEY SILTY VERY FINE SAND WITH TRACE MICA	-394.75	[illegible]	0	19	[illegible]
401.3	BORING TERMINATED	-399.75				

△ Penetration blows per foot using "A" rod.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

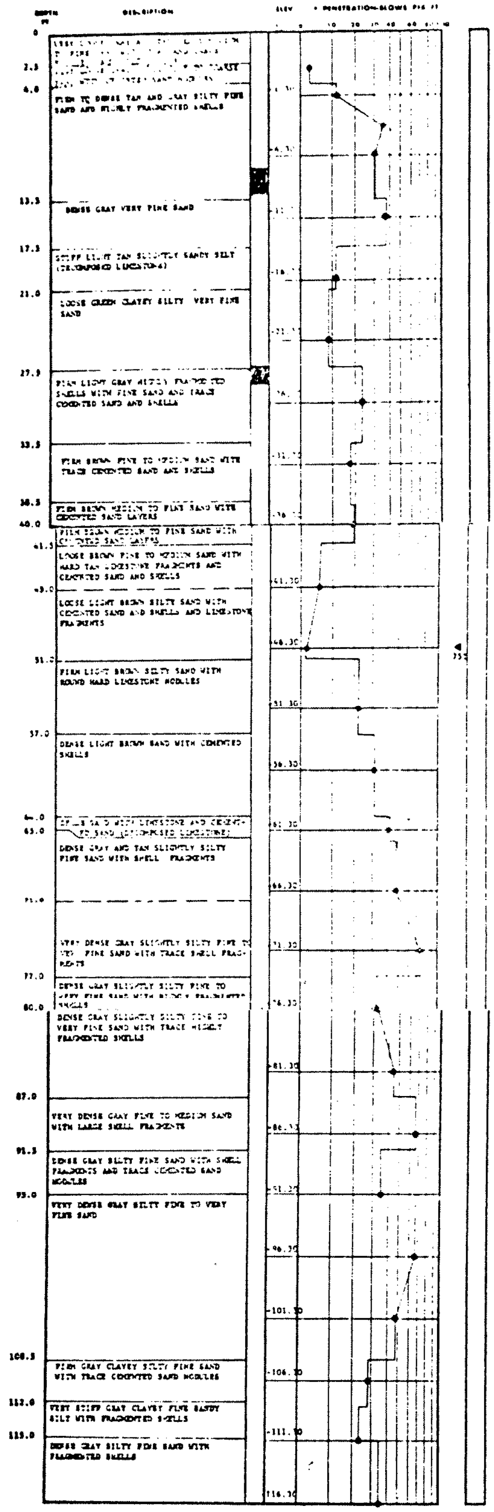
UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE 24 HR.
WATER TABLE 1 HR.
LOSS OF DRILLING WATER

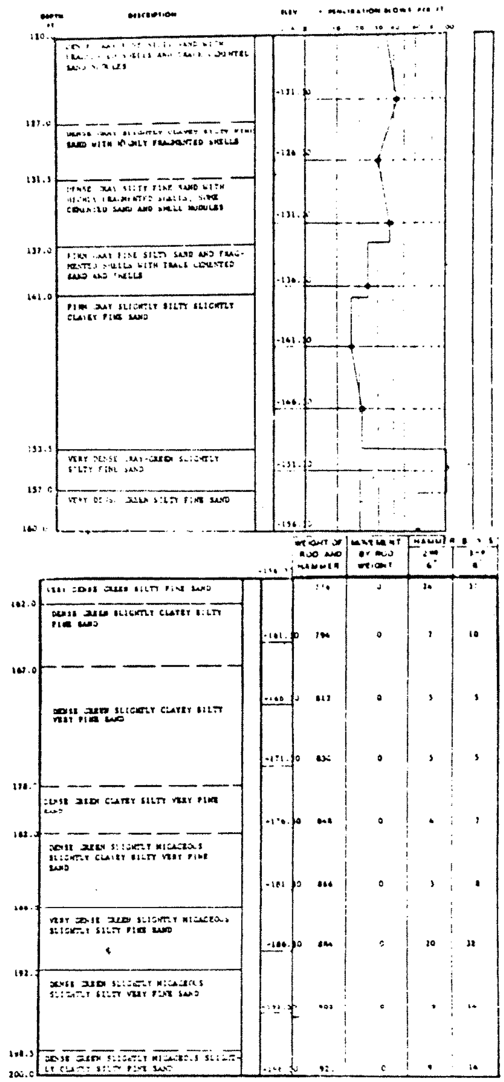
"N" ROD USED IN PENETRATION TEST

TEST BORING RECORD

BORING NO. B-1
DATE DRILLED 3/12/68
JOB NO. EC-163

LAW ENGINEERING TESTING CO





BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
% ROCK CORE RECOVERY
LOSS OF DRILLING WATER

"A" ROD USED IN PENETRATION TEST

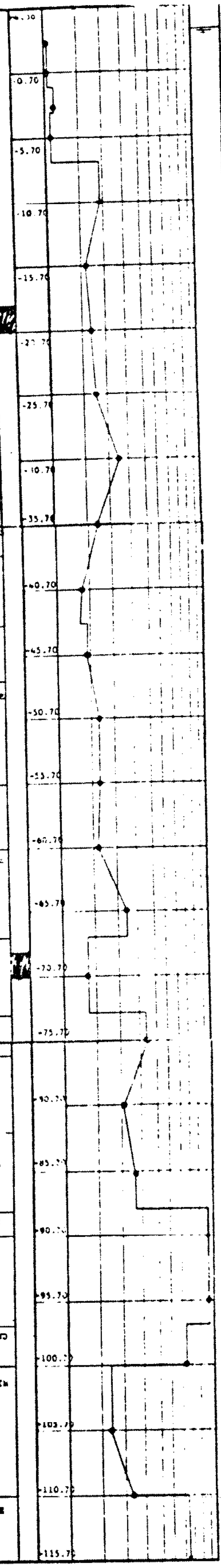
TEST BORING RECORD

BORING NO. B-2
DATE DRILLED 3/18/68
JOB NO. EC-163

LAW ENGINEERING TESTING CO.

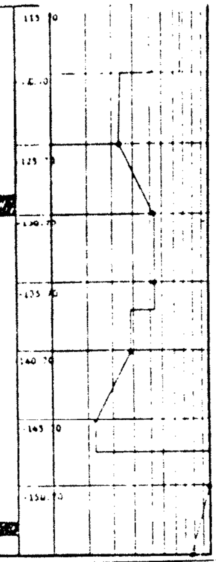
Depth	Description
0	
0.3	DARK GRAY FINE SANDY CLAY *
	VERY SOFT PEAT (FILL)
2.0	VERY LOOSE TAN AND GRAY SILTY FINE SAND
3.0	VERY LOOSE GRAY SILTY FINE SAND WITH THIN LAYERS OF PEAT, TRACE OF SHELLS (FILL)
6.0	SOFT ORGANIC SILT WITH ROOTS AND SILTY FINE SAND WITH SHELLS
8.0	VERY LOOSE DARK GRAY SILTY FINE SAND WITH ORGANIC SILT AND SHELLS
12.0	FIRM DARK GRAY SLIGHTLY SILTY FINE SAND WITH TRACE OF SHELLS
16.0	FIRM DARK GRAY HIGHLY FRAGMENTED SHELLS AND SAND
20.0	FIRM DARK GRAY SAND AND SHELLS
25.0	FIRM DARK GRAY SILTY SAND AND SHELLS
32.0	FIRM DARK GRAY FINE SAND, TRACE SHELLS, WITH CEMENTED SAND AND SHELL SEAMS
40.0	FIRM DARK GRAY FINE SAND, TRACE SHELLS WITH CEMENTED SAND AND SHELL SEAMS
42.0	LOOSE TAN GRAY FINE SAND WITH TRACE CEMENTED SAND AND SHELLS
47.5	LOOSE TAN GRAY FINE SAND AND LIMESTONE NODULES WITH SHELL INCLUSIONS
53.0	FIRM TAN GRAY FINE SAND AND LIMESTONE NODULES WITH SHELL INCLUSIONS
60.0	FIRM GRAY SILTY FINE TO MEDIUM SAND AND HIGHLY FRAGMENTED SHELLS
65.0	FIRM GRAY SLIGHTLY CLAYEY SILTY FINE TO MEDIUM SAND WITH HIGHLY FRAGMENTED SHELLS
72.0	LOOSE GRAY SHELLS AND SILTY FINE SAND, TRACE LIMESTONE PEBBLES WITH SHELL INCLUSIONS
78.0	DENSE GRAY SILTY FINE SAND AND FRAGMENTED SHELLS
80.0	DENSE GRAY SILTY FINE SAND AND *
81.0	FIRM WHITE AND GRAY FINE TO COARSE SANDY CEMENTED SHELLS (DECOMPOSED LIMESTONE)
87.0	FIRM WHITE AND DARK GRAY CEMENTED SHELLS AND HIGHLY FRAGMENTED SHELLS WITH FINE SAND AND SILT (DECOMPOSED LIMESTONE)
93.0	VERY DENSE DARK GRAY SLIGHTLY SILTY FINE SAND WITH FRAGMENTS OF SHELLS
95.0	DENSE DARK GRAY SILTY FINE SAND WITH TRACE OF CLAY
102.0	VERY DENSE DARK GRAY SILTY FINE SAND WITH FRAGMENTS OF CEMENTED SHELLS
105.0	FIRM LIGHT GRAY SILTY FINE SAND WITH CEMENTED SAND
115.0	VERY DENSE GRAY SLIGHTLY SILTY FINE SAND WITH HIGHLY FRAGMENTED SHELLS
120.0	

* FRAGMENTED SHELLS



Depth	Description
120	VERY DENSE GRAY SLIGHTLY SILTY FINE*
121	VERY DENSE GREENISH GRAY SLIGHTLY SILTY FINE SAND WITH TRACE HIGHLY FRAGMENTED SHELLS WITH CEMENTED SAND
125	FIRM TO DENSE GREENISH GRAY SILTY FINE SAND WITH FRAGMENTED SHELLS AND CEMENTED SAND
137.5	DENSE GRAY SILTY FINE SAND
142.0	FIRM LIGHT GRAY SLIGHTLY CLAYEY SILTY FINE SAND WITH CEMENTED SAND
152.5	VERY DENSE GREEN SLIGHTLY SILTY VERY FINE SAND WITH CEMENTED SAND
155.0	VERY DENSE GREEN SLIGHTLY SILTY FINE SAND WITH TRACE MICA
160.0	

* SAND WITH HIGHLY FRAGMENTED SHELLS



Depth	Description	ELEV	WEIGHT OF ROD AND HAMMER	MOVEMENT BY ROD WEIGHT	HAMMER BLOWS 2nd 6"	HAMMER BLOWS 3rd 6"
160.0	FIRM GREEN SLIGHTLY CLAYEY SILTY VERY FINE SAND WITH TRACE MICA	-155.7	776	0	32	[illegible]
		-160.7	794	0	9	12
167.0	DENSE GREEN SLIGHTLY CLAYEY SILTY VERY FINE SAND WITH MICA	-165.7	812	0	5	7
		-170.7	830	0	5	7
		-175.7	848	0	2	4
182.0	FIRM GREEN SLIGHTLY CLAYEY SILTY VERY FINE SAND WITH MICA	-180.7	866	0	71	14
187.0	VERY DENSE GREEN SILTY FINE SAND WITH TRACE OF MICA	-185.7	884	0	26	29
		-190.7	902	0	26	43
196.5	STIFF GREEN CLAYEY MICACEOUS VERY FINE SANDY SILT	-195.7	920	0	[illegible]	[illegible]
200.0	BORING TERMINATED					

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE — WATER TABLE, 24 HR.

% ROCK CORE RECOVERY — WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

"A" ROD USED IN PENETRATION TEST

TEST BORING RECORD

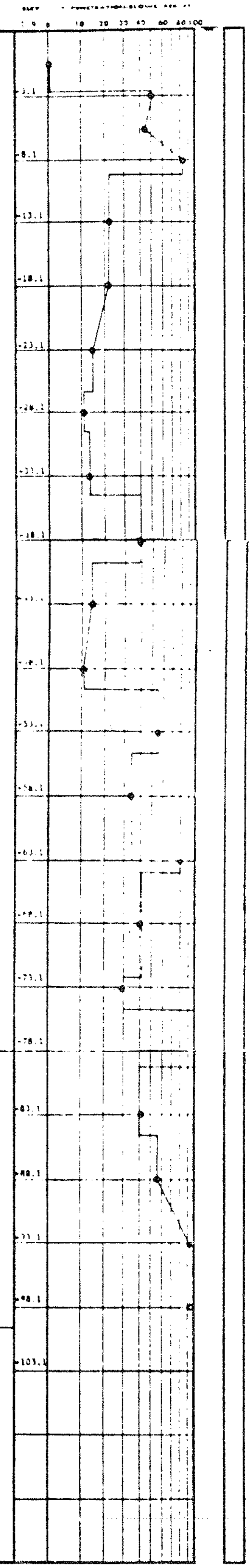
BORING NO. B-3

DATE DRILLED 3/14/68

JOB NO. EC-163

LAW ENGINEERING TESTING CO.

DEPTH FT	DESCRIPTION
0	WATER
1.5	VERY SOFT PEAT
4.5	DENSE GRAY FINE SAND AND SHELLS
6.5	VERY DENSE GRAY COARSE TO FINE SAND AND SHELLS
11.0	FIRM DENSE GRAY COARSE TO FINE SAND AND SHELLS WITH TRACE OF CLAY
16.5	FIRM HIGHLY FRAGMENTED SHELLS WITH GRAY FINE SAND
23.5	FIRM GRAY SLIGHTLY CLAYEY SAND WITH SHELLS
28.5	FIRM SLIGHTLY CLAYEY GREENISH BROWN CEMENTED SAND NODULES WITH SILTY CLAYEY FINE SAND
31.5	FIRM GREENISH BROWN SAND WITH CEMENTED SAND NODULES AND A TRACE OF SHELLS
36.5	DENSE GREENISH BROWN SLIGHTLY SILTY LIMESTONE NODULES
40.0	[illegible] SILTY [illegible]
41.5	FIRM GREENISH BROWN [illegible] NODULES WITH TRACE OF SANDY SILTY CLAY
51.5	VERY DENSE GREENISH BROWN LIMESTONE NODULES WITH TRACE SANDY SILTY CLAY
56.5	DENSE GREENISH BROWN LIMESTONE NODULES WITH TRACE SANDY SILTY CLAY
65.0	VERY DENSE GRAY SLIGHTLY SILTY SAND WITH TRACE OF SHELL FRAGMENTS
66.5	DENSE GRAY SILTY FINE SAND WITH SMALL SHELL FRAGMENTS
74.0	FIRM HIGHLY FRAGMENTED SHELLS WITH SAND AND TRACE OF SMALL LIMESTONE NODULES
76.5	VERY DENSE DARK GRAY SAND WITH HIGHLY FRAGMENTED SHELLS AND FEW NODULES OF CEMENTED SAND AND TRACE OF SHELLS
80.0	[illegible] FRAGMENTED SHELLS AND FEW NODULES [illegible]
81.5	DENSE FRAGMENTED SHELLS WITH TRACE OF SAND AND CEMENTED SHELLS AND SAND
86.5	VERY DENSE DARK GRAY SAND WITH HIGHLY FRAGMENTED SHELLS
91.5	VERY DENSE LIGHT GRAY SILTY FINE SAND WITH TRACE SHELLS
101.5	BORING TERMINATED



* CALCAREOUS CEMENTED SAND AND TRACE OF SHELLS

TEST BORING RECORD

BORING NO. B-4

DATE DRILLED 3/15/68

JOB NO. EC-163

LAW ENGINEERING TESTING CO.

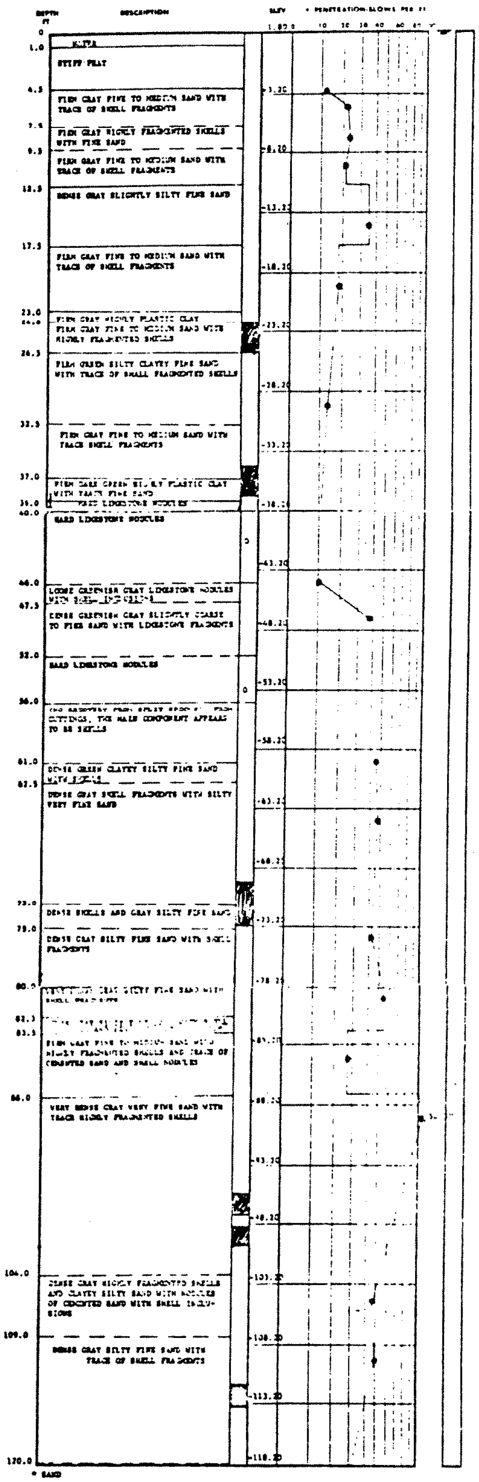
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

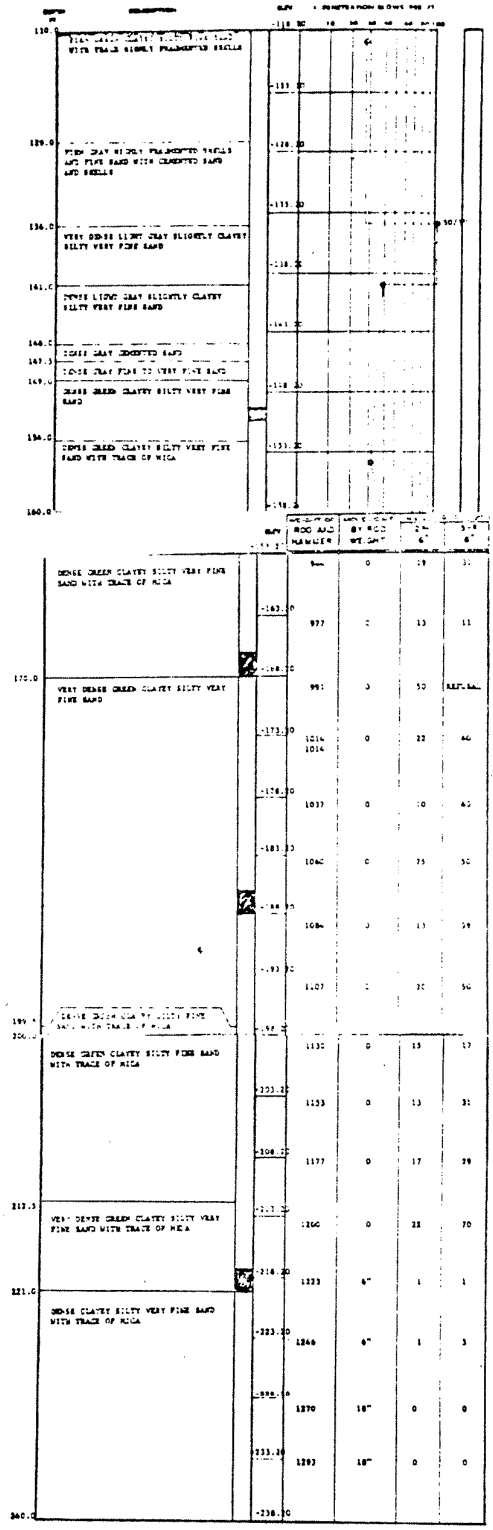
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE — WATER TABLE, 24 HR. — WATER TABLE, 1 HR.

% ROCK CORE RECOVERY — LOSS OF DRILLING WATER

"N" ROD USED IN PENETRATION TEST





TEST BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

% ROCK CORE RECOVERY

LOSS OF DRILLING WATER

"N" ROD USED IN PENETRATION TEST

BORING NO. B-5

DATE DRILLED 3/19/68

JOB NO. EC-163

LAW ENGINEERING TESTING CO.

DEPTH FT	DESCRIPTION	ELEV.	WEIGHT OF ROD AND HAMMER	MOVEMENT BY ROD WEIGHT	HAMMER BLOWS 2nd 6"	3rd 6"
240.0	VERY DENSE GREEN CLAYEY SILTY VERY FINE SAND WITH TRACE OF MICA	-218.2	1318			
243.3	VERY DENSE GREEN SILTY CLAYEY VERY FINE SAND WITH TRACE OF MICA	-241.20	1339	0	29	40
		-248.20	1363	0	23	36
251.0	VERY DENSE GREEN CLAYEY SILTY MICACEOUS FINE SAND	-253.20	1386	0	25	41
		-258.20	1409	0	8	12
261.0	DENSE GREEN CLAYEY SILTY MICACEOUS FINE SAND	-263.20	1433	0	7	14
		-268.20	1456	0	8	12
271.0	FIRM GREEN SILTY CLAYEY MICACEOUS VERY FINE SAND	-273.20	1479	0	10	15
276.0	VERY STIFF GREEN VERY FINE SANDY CLAYEY MICACEOUS SILT	-278.20	1502	0	30	39
[illegible]	[illegible] FINE SANDY CLAYEY MICACEOUS SILT					
[illegible]1.0	VERY DENSE GREEN CLAYEY SILTY MICACEOUS VERY FINE SAND	[illegible]	1525	0	28	36
		[illegible]	1549	0	29	35
[illegible]	VERY DENSE GREEN MICACEOUS VERY FINE SANDY CLAYEY SILT	[illegible]	[illegible]	0	[illegible]	31
		[illegible]	1595	0	31	39
[illegible]	BORING TERMINATED	[illegible]				

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

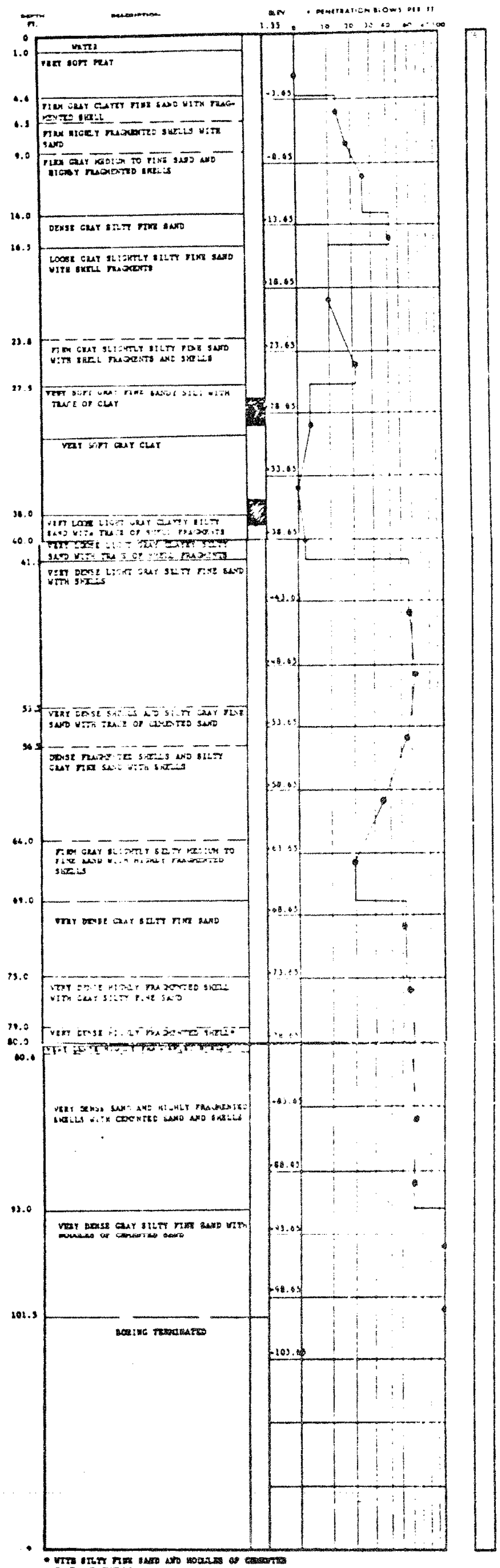
WATER TABLE, 24 HR.
UNDISTURBED SAMPLE
WATER TABLE, 1 HR.
% ROCK CORE RECOVERY
LOSS OF DRILLING WATER

"N" ROD USED IN PENETRATION TEST

TEST BORING RECORD

BORING NO. B-5
DATE DRILLED 3/19/68
JOB NO. EC-163

LAW ENGINEERING TESTING CO.



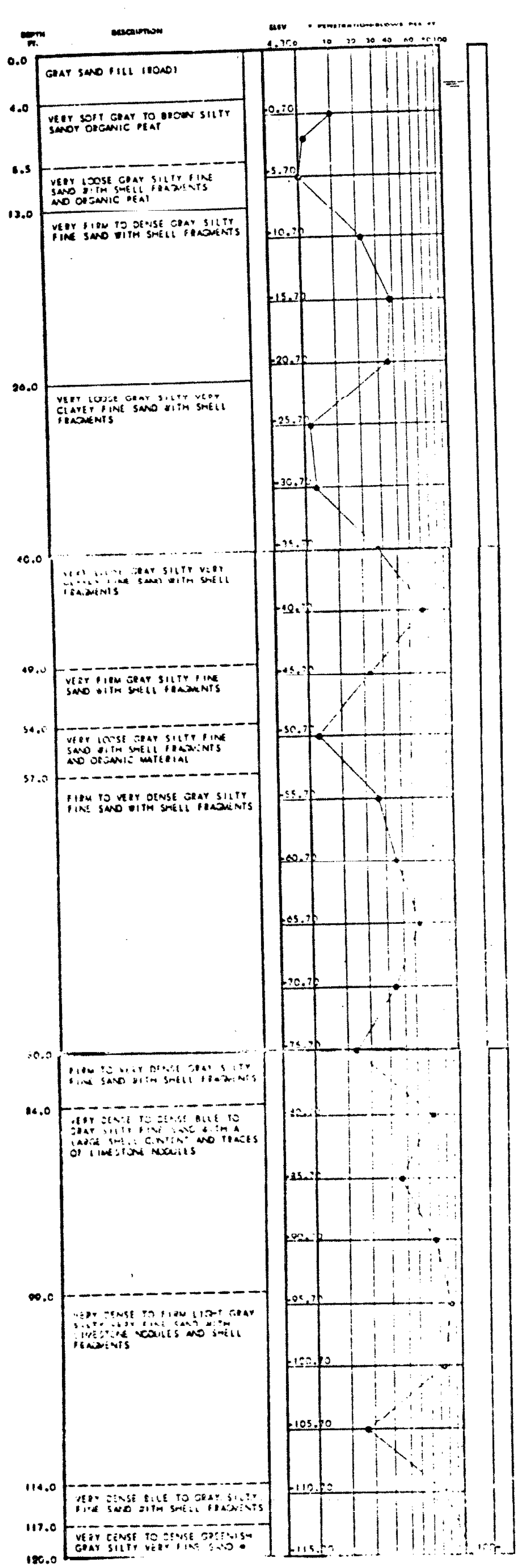
TEST BORING RECORD

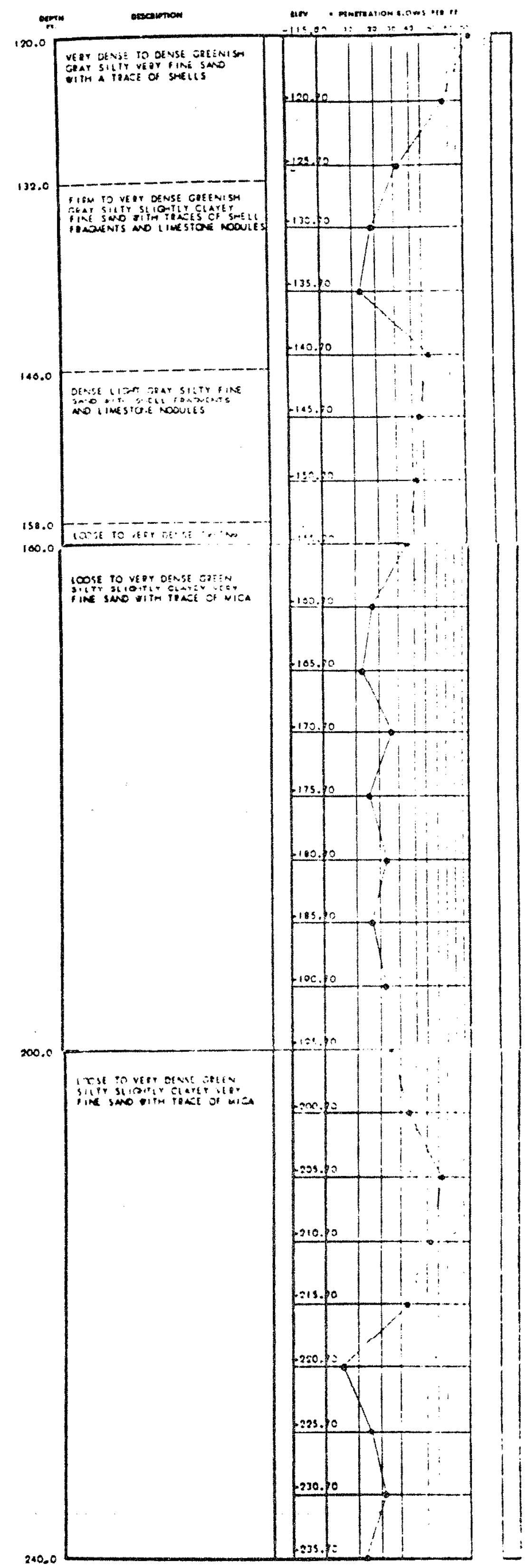
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

WATER TABLE, 24 HR.
UNDISTURBED SAMPLE
WATER TABLE, 1 HR.
% ROCK CORE RECOVERY
LOSS OF DRILLING WATER
"N" ROD USED IN PENETRATION TEST

BORING NO. B-6
DATE DRILLED 3/10/68
JOB NO. EC-163

LAW ENGINEERING TESTING CO.





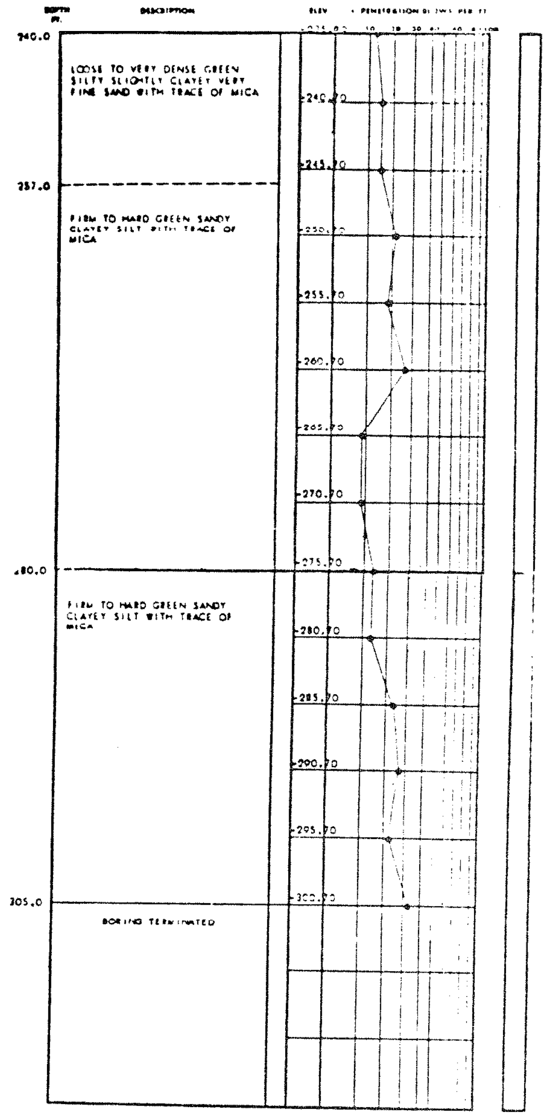
TEST BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE — WATER TABLE, 24 HR. — WATER TABLE, 1 HR.
% ROCK CORE RECOVERY — LOSS OF DRILLING WATER

BORING NO. B-6A
DATE DRILLED 1/3/69
JOB NO. 1-1127

LAW ENGINEERING TESTING CO.



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE — WATER TABLE, 24 HR.

% ROCK CORE RECOVERY — WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

TEST BORING RECORD

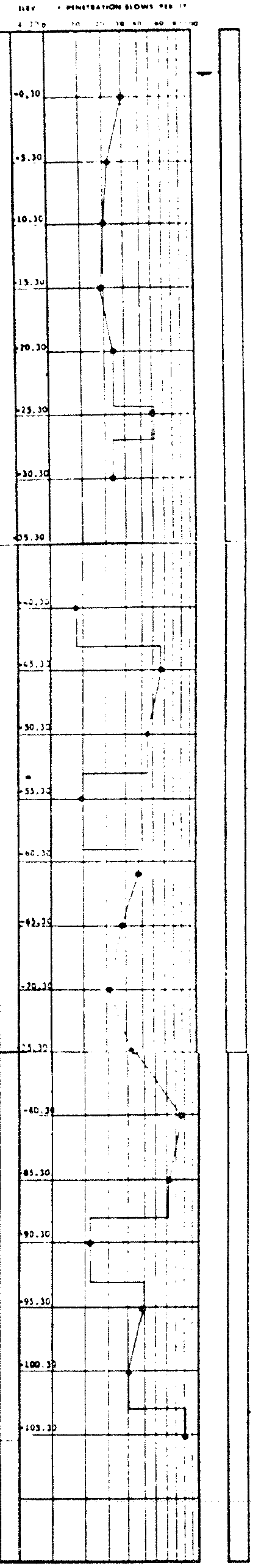
BORING NO. B-6A

DATE DRILLED 1/3/69

JOB NO. J-1127

LAW ENGINEERING TESTING CO.

DEPTH FT	DESCRIPTION
0	FILL-SAND SHELLS ROOTS WITH 1.5 TO [illegible] INCH LAYER OF [illegible]
[illegible]	LOOSE SAND AND SHELLS WITH ROOTS
4.5	DENSE TO FIRM DARK GRAY MEDIUM TO FINE SAND WITH TRACE OF SHELL FRAGMENTS
16.[illegible]	FIRM GRAY SILTY FINE SAND WITH SHELL FRAGMENTS
18.0	FIRM LIGHT GRAY CLAYEY SILTY SANDY HIGHLY FRAGMENTED SHELLS WITH FEW CEMENTED NODULES
29.5	VERY DENSE BROWN LIGHT GRAY SILTY FINE SAND WITH FEW CEMENTED NODULES
32.0	FIRM LIGHT BROWN LIMESTONE NODULES WITH TRACE OF SAND AND SHELL
40.0	FIRM LIGHT BROWN LIMESTONE NODULES WITH TRACE OF SAND AND SHELL
44.0	LOOSE LIGHT BROWN LIMESTONE NODULES WITH TRACE OF SAND
48.0	VERY DENSE AND DENSE LIGHT BROWN LIMESTONE NODULES WITH SILTY SAND
58.0	LOOSE DARK GRAY HIGHLY FRAGMENTED SHELLS WITH SHELLS AND SILTY FINE SAND
[illegible]	DENSE AND FIRM DARK GRAY HIGHLY FRAGMENTED SHELLS AND GRAY SILTY FINE SAND
80.0	DENSE AND FIRM DARK GRAY HIGHLY FRAGMENTED SHELLS AND GRAY SILTY FINE SAND
84.0	VERY DENSE DARK GRAY SILTY FINE SAND WITH NODULES OF CEMENTED SAND AND FRAGMENTED SHELL
93.0	FIRM DARK GRAY SILTY FINE SAND
98.0	DENSE DARK GRAY SLIGHTLY CLAYEY SILTY FINE SAND WITH TRACE OF SHELL FRAGMENTS
108.0	VERY DENSE GRAY SILTY FINE SAND WITH TRACE OF HIGHLY FRAGMENTED SHELL AND CEMENTED SAND
110.5	BORING TERMINATED



TEST BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE — WATER TABLE 24 HR. — WATER TABLE, 1 HR.

% ROCK CORE RECOVERY — LOSS OF DRILLING WATER

"N" ROD USED IN PENETRATION TEST

BORING NO. B-7

DATE DRILLED 3/12/68

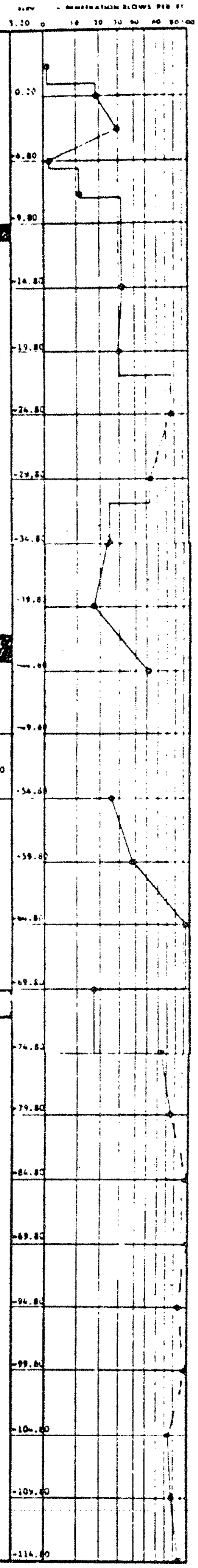
JOB NO. KC-163

LAW ENGINEERING TESTING CO.

TEST BORING RECORD

Depth ft	Description
0	VERY LOOSE ORGANIC SILT WITH DECOMPOSED WOOD, TRACE ROOTS (FILL)
4.0	FIRM DARK GRAY SLIGHTLY SILTY FINE SAND WITH TRACE ROOTS AND MICA
6.5	FIRM GRAY SLIGHTLY SILTY FINE SAND WITH TRACE MICA AND SHELL FRAGMENTS
10.5	VERY LOOSE PEAT AND CLAYEY SILTY FINE TO MEDIUM SAND WITH TRACE SHELLS
13.0	FIRM GRAY SILTY SAND AND HIGHLY FRAGMENTED SHELLS
20.0	DENSE LIGHT GRAY HIGHLY FRAGMENTED SHELLS WITH SLIGHTLY SILTY CLAYEY SAND AND SOME CEMENTED SAND NODULES
25.0	FIRM LIGHT GRAY HIGHLY FRAGMENTED SHELLS IN A MATRIX OF SILTY CLAYEY SAND WITH [illegible] CEMENTED SAND NODULES
27.0	VERY DENSE GRAY SLIGHTLY SILTY SAND WITH CEMENTED SAND LAYERS AND TRACE OF MICA
33.0	VERY DENSE GRAY FRAGMENTS OF CEMENTED SAND AND LIMESTONE FRAGMENTS WITH SILTY FINE SAND
37.0	FIRM BROWN LIMESTONE FRAGMENTS WITH SHELL INCLUSIONS-TRACE SILTY FINE SAND
40.0	FIRM BROWN LIMESTONE FRAGMENTS [illegible]
41.5	FIRM BROWN LIMESTONE NODULES WITH SHELL INCLUSIONS-TRACE CLAYEY FINE SANDY SILT
46.5	VERY DENSE GREENISH BROWN LIMESTONE NODULES WITH FEW SHELL INCLUSIONS AND TRACE SLIGHTLY SILTY SAND
60.0	FIRM GRAY FINE TO MEDIUM SLIGHTLY SILTY SAND WITH HIGHLY FRAGMENTED SHELLS
65.0	DENSE GRAY FINE SLIGHTLY SILTY SAND WITH SHELL FRAGMENTS
70.0	VERY DENSE GRAY VERY FINE SLIGHTLY SILTY SAND AND HIGHLY FRAGMENTED SHELLS
75.0	FIRM GRAY SILTY FINE SAND AND HIGHLY FRAGMENTED SHELLS
80.0	VERY DENSE GRAY SLIGHTLY SILTY FINE SAND WITH HIGHLY FRAGMENTED SHELLS AND TRACE CEMENTED SAND AND SHELL NODULES
83.0	VERY DENSE GRAY SILTY CLAYEY MEDIUM SAND WITH SHELL FRAGMENTS AND CEMENTED SAND AND SHELLS
90.0	VERY DENSE GRAY SILTY FINE TO VERY FINE SAND
91.5	VERY DENSE GRAY SILTY FINE TO MEDIUM SAND WITH HIGHLY FRAGMENTED SHELLS
113.0	VERY DENSE GREEN SILTY CLAYEY FINE SAND WITH HIGHLY FRAGMENTED SHELLS - TRACE CEMENTED SAND AND SHELLS
120.0	

ELEV. 5.20: 0.20, -4.80, -9.80, -14.80, -19.80, -24.80, -29.80, -34.80, -39.80, -44.80, -49.80, -54.80, -59.80, -64.80, -69.80, -74.80, -79.80, -84.80, -89.80, -94.80, -99.80, -104.80, -109.80, -114.80



Depth ft	Description
120.0	VERY DENSE GREEN SILTY CLAYEY FINE SAND WITH HIGHLY FRAGMENTED SHELLS TRACE CEMENTED SAND AND SHELLS
123.0	DENSE HIGHLY FRAGMENTED SHELLS WITH SLIGHTLY SILTY FINE TO VERY FINE SAND
127.0	FIRM LIGHT GRAY SLIGHTLY SILTY FINE TO VERY FINE SAND
133.0	DENSE GREEN SLIGHTLY CLAYEY SILTY FINE TO VERY FINE SAND WITH HIGHLY FRAGMENTED SHELLS
140.0	DENSE GREEN CLAYEY SILTY FINE SAND WITH TRACE OF MICA
160.0	FIRM GREEN CLAYEY SILTY FINE SAND, TRACE MICA
165.0	DENSE TO VERY DENSE GREEN SLIGHTLY CLAYEY SILTY FINE SAND, TRACE OF MICA
173.0	FIRM GREEN SILTY CLAYEY FINE SAND, TRACE MICA
190.5	BORING TERMINATED
200.0	

ELEV. -114.8	WEIGHT OF ROD AND HAMMER	MOVEMENT BY ROD WEIGHT	HAMMER BLOWS 2ND 6"	HAMMER BLOWS 3RD 6"
	758	0	44	4[illegible]
-119.80	781	0	12	20
-124.80	805	0	7	12
-129.80	828	0	13	1
-134.80	851	0	5	6
-139.80	874			
-144.80	898			
-149.80	921	0	1	6
-154.80	944	0	1	4
-159.60	977	0	10	12
[illegible]	991	4	25	[illegible]
-165.80	1014	0	10	12
-174.80	1037	0	6	12
-179.80	1060	0	5	[illegible]
-184.80	1084	0	3	12
-194.80	1107	0	2	1[illegible]

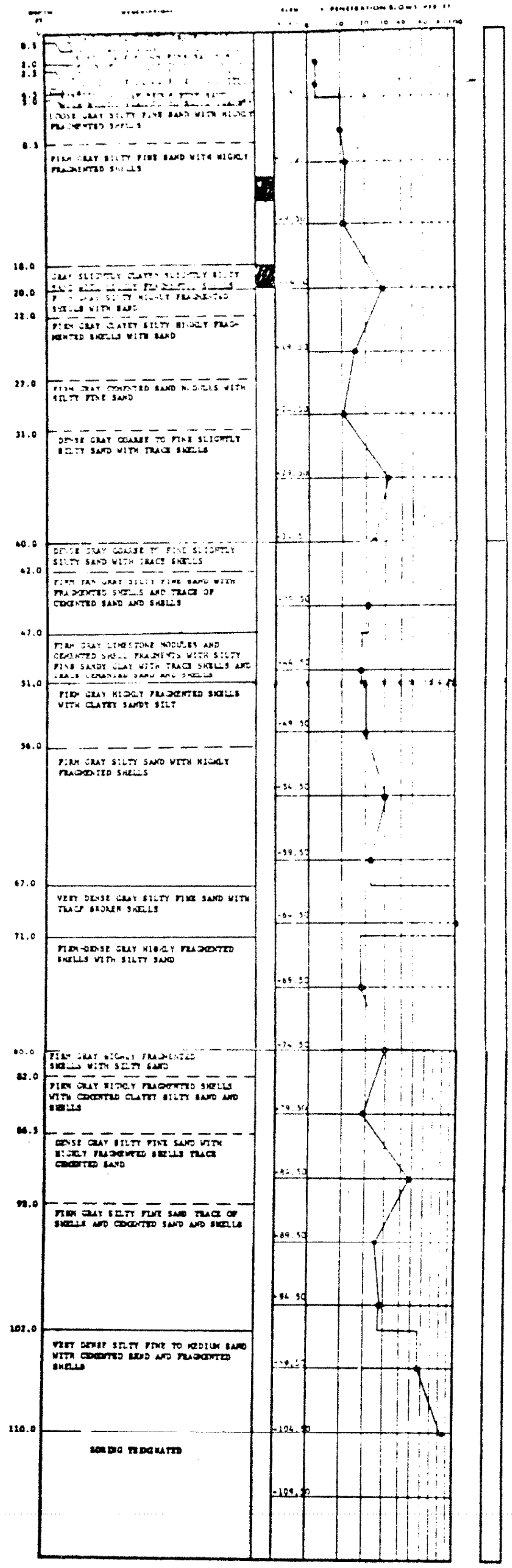
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
|50| % ROCK CORE RECOVERY
LOSS OF DRILLING WATER

"N" ROD USED IN PENETRATION TEST

BORING NO. B-8
DATE DRILLED 3/18/68
JOB NO. EC-163

LAW ENGINEERING TESTING CO.



* CLAY WITH ROOTS AND ORGANIC MATERIAL (FILL)
** OF DECOMPOSED WOOD
*** FILL

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

% ROCK CORE RECOVERY

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

"A" ROD USED IN PENETRATION TEST

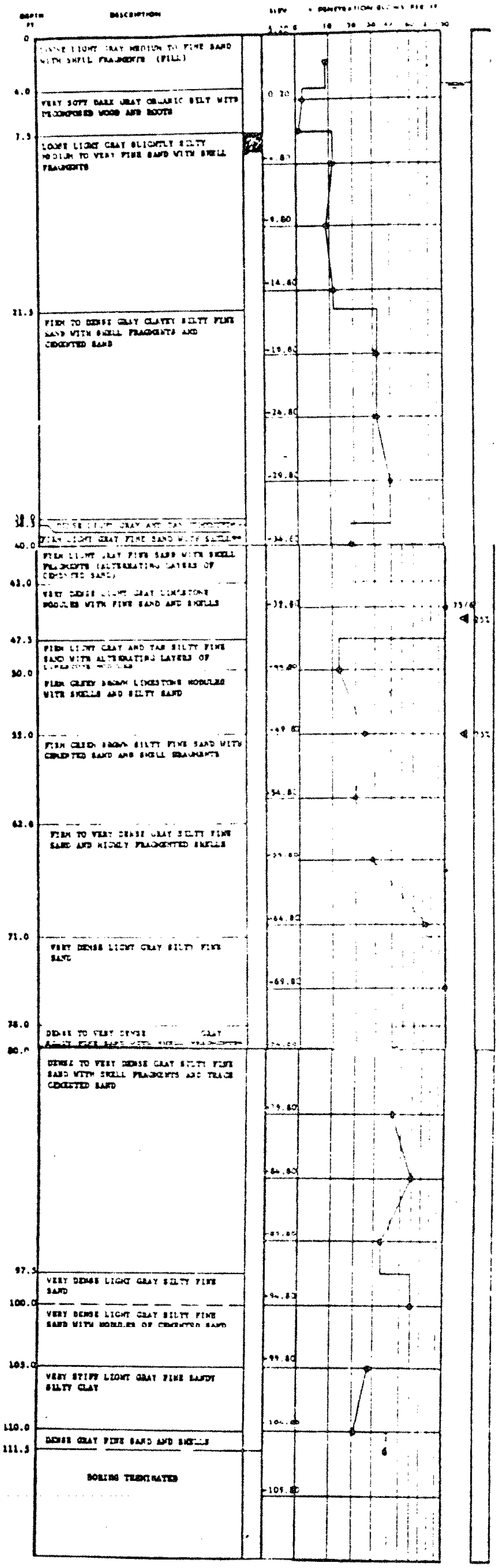
TEST BORING RECORD

BORING NO. B-9

DATE DRILLED 3/16/68

JOB NO. EC-163

LAW ENGINEERING TESTING CO



TEST BORING RECORD

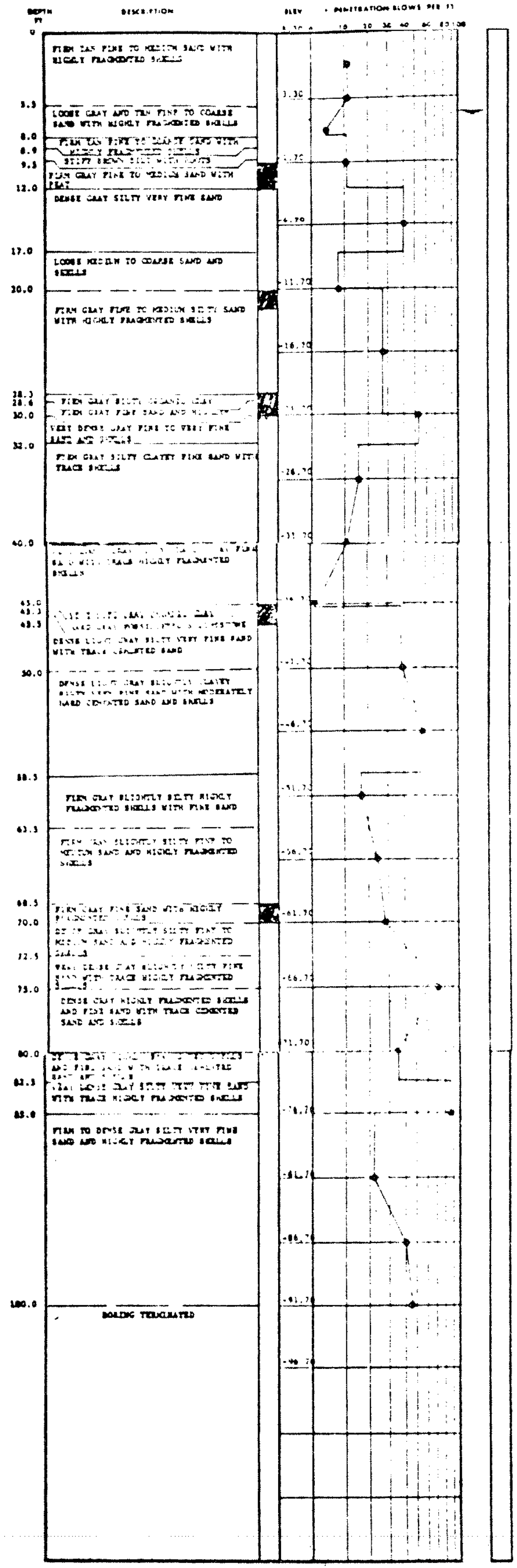
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE 24 HR.
WATER TABLE 1 HR.
LOSS OF DRILLING WATER

"A" ROD USED IN PENETRATION TEST

BORING NO. B-10
DATE DRILLED 3/11/68
JOB NO. EC-163

LAW ENGINEERING TESTING CO.



TEST BORING RECORD

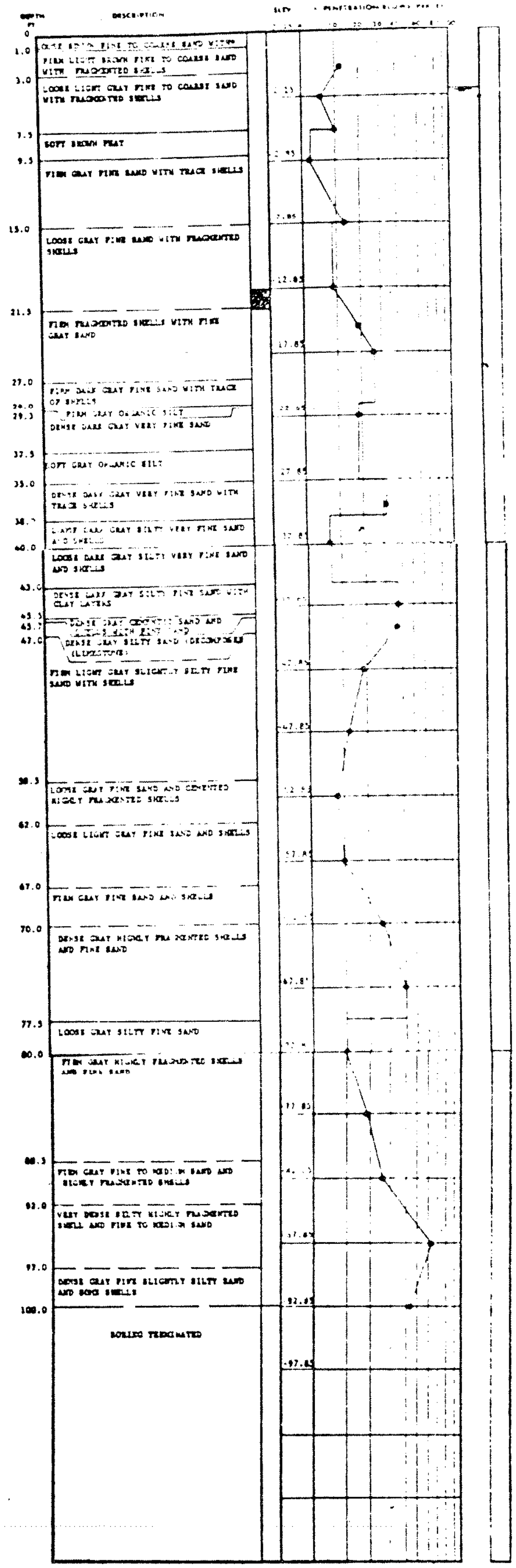
BORING NO. B-11

DATE DRILLED 3/15-16/68

JOB NO. EC-163

LAW ENGINEERING TESTING CO.

"A" ROD USED IN PENETRATION TEST



TEST BORING RECORD

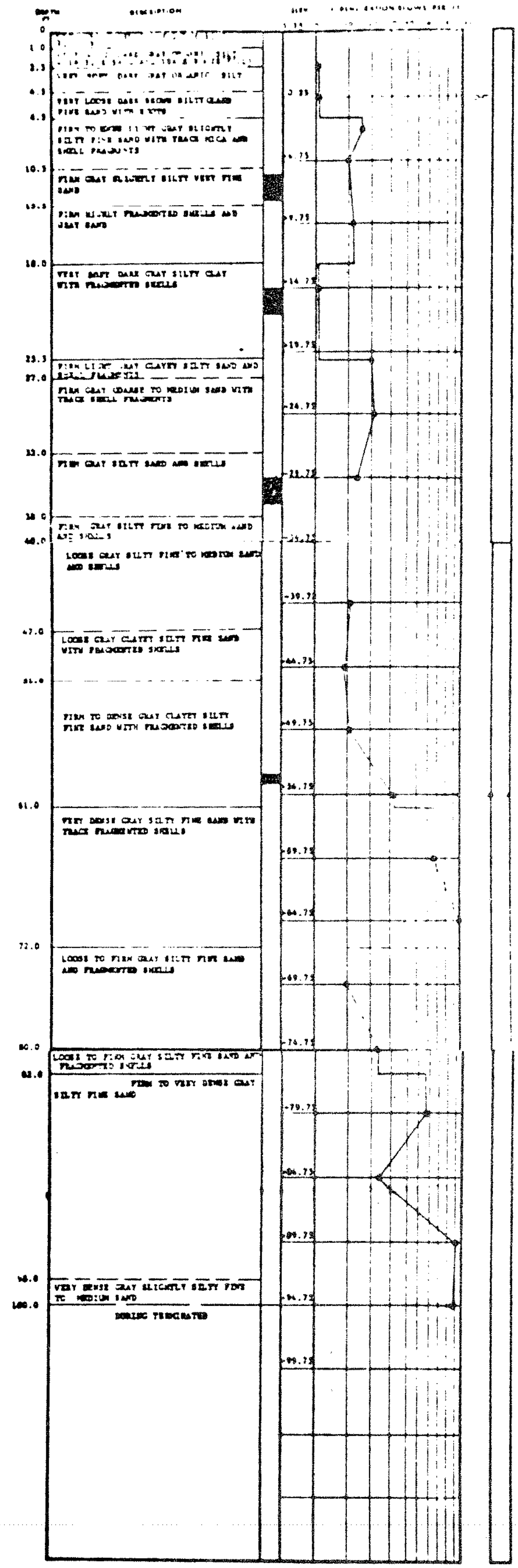
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

BORING NO. B-12
DATE DRILLED 3/14/68
JOB NO. EC-163

LAW ENGINEERING TESTING CO.

"A" ROD USED IN PENETRATION TEST



TEST BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

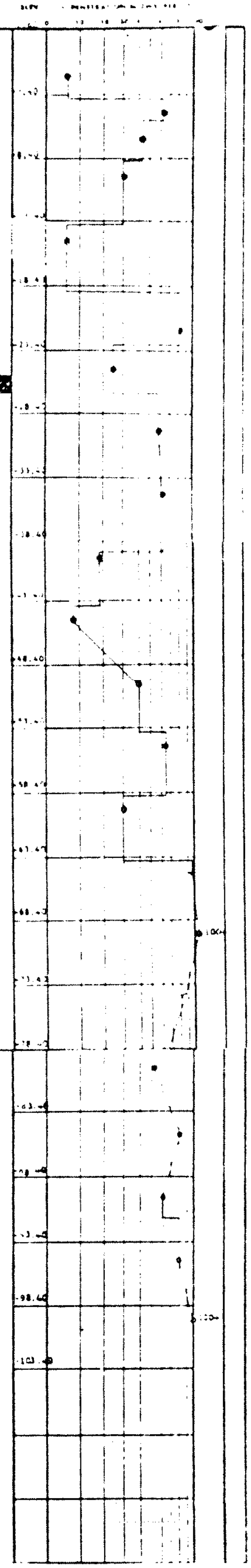
UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

"A" ROD USED IN PENETRATION TEST

BORING NO. B-13
DATE DRILLED 3/18/68
JOB NO. EC-163

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION
0 – 0.5	[illegible]
0.5 – 1.5	[illegible]
1.5 – 5.5	[illegible] LIGHT BROWN SILTY SAND WITH [illegible] FRAGMENTS
5.5 – 7.5	[illegible] LIGHT GRAY SAND AND HIGHLY [illegible] SHELLS, CEMENTED SHELLS, [illegible] SAND
7.5 – 10.5	DENSE LIGHT GRAY HIGHLY FRAGMENTED SHELLS WITH SAND
10.5 – 15.5	FIRM GRAY HIGHLY FRAGMENTED SHELLS WITH FINE SAND AND TRACE CEMENTED SAND
15.5 – 21.0	LOOSE GRAY AND WHITE HIGHLY FRAGMENTED SHELLS WITH SAND
21.0 – 26.5	VERY DENSE LIGHT GRAY FINE TO COARSE SAND AND HIGHLY FRAGMENTED SHELLS
26.5 – 28.5	FIRM GRAY SILTY CLAYEY HIGHLY FRAGMENTED SHELLS AND SAND
28.5 – 35.5	VERY DENSE BROWN SILTY FINE SAND WITH CEMENTED SAND
35.5 – 40.0	VERY DENSE GRAY CEMENTED SAND AND SHELLS WITH HIGHLY FRAGMENTED SHELLS
40.0 – 40.5	[illegible]
40.5 – 45.5	[illegible] BROWN SILTY FINE SAND WITH FRAGMENTS OF CEMENTED SAND WITH FEW SHELL [illegible]
45.5 – 55.5	LOOSE TO DENSE GRAY CLAYEY SLIGHTLY SILTY VERY FINE SAND WITH SHELL FRAGMENTS
55.5 – 60.5	VERY DENSE GREENISH GRAY VERY FINE SAND WITH HIGHLY FRAGMENTED SHELLS
60.5 – 65.5	FIRM GRAY SLIGHTLY SILTY VERY FINE SAND WITH HIGHLY FRAGMENTED SHELLS
65.5 – 75.5	VERY DENSE GRAY FINE TO COARSE SAND WITH HIGHLY FRAGMENTED SHELLS
75.5 – 80.0	VERY DENSE GRAY SILTY FINE SAND AND HIGHLY FRAGMENTED SHELLS
80.0 – 85.5	VERY DENSE GRAY SILTY FINE SAND AND HIGHLY FRAGMENTED SHELLS
85.5 – 87.5	VERY DENSE GRAY FINE TO COARSE SAND AND HIGHLY FRAGMENTED SHELLS
87.5 – 93.5	VERY DENSE GRAY SILTY FINE SAND TRACE HIGHLY FRAGMENTED SHELLS AND NODULES OF CEMENTED SAND
93.5 – 101.5	VERY DENSE LIGHT GRAY SILTY VERY FINE SAND WITH TRACE CEMENTED SAND
101.5	BORING TERMINATED



TEST BORING RECORD

BORING NO. B-14

DATE DRILLED 3/26/68

JOB NO. EC-163

LAW ENGINEERING TESTING CO.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

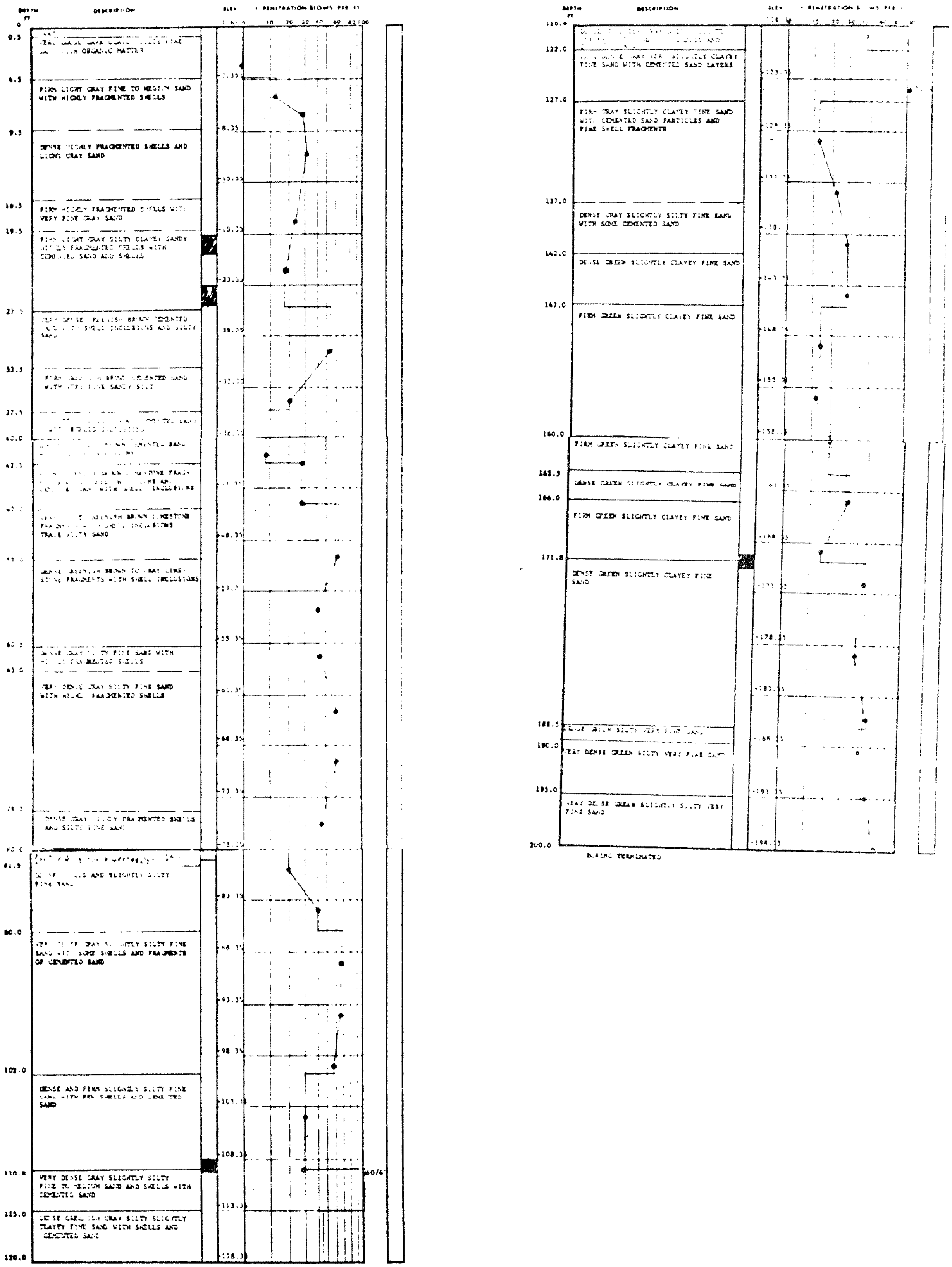
% ROCK CORE RECOVERY

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

"N" ROD USED IN PENETRATION TEST



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT

UNDISTURBED SAMPLE

% ROCK CORE RECOVERY

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

"N" ROD USED IN PENETRATION TEST

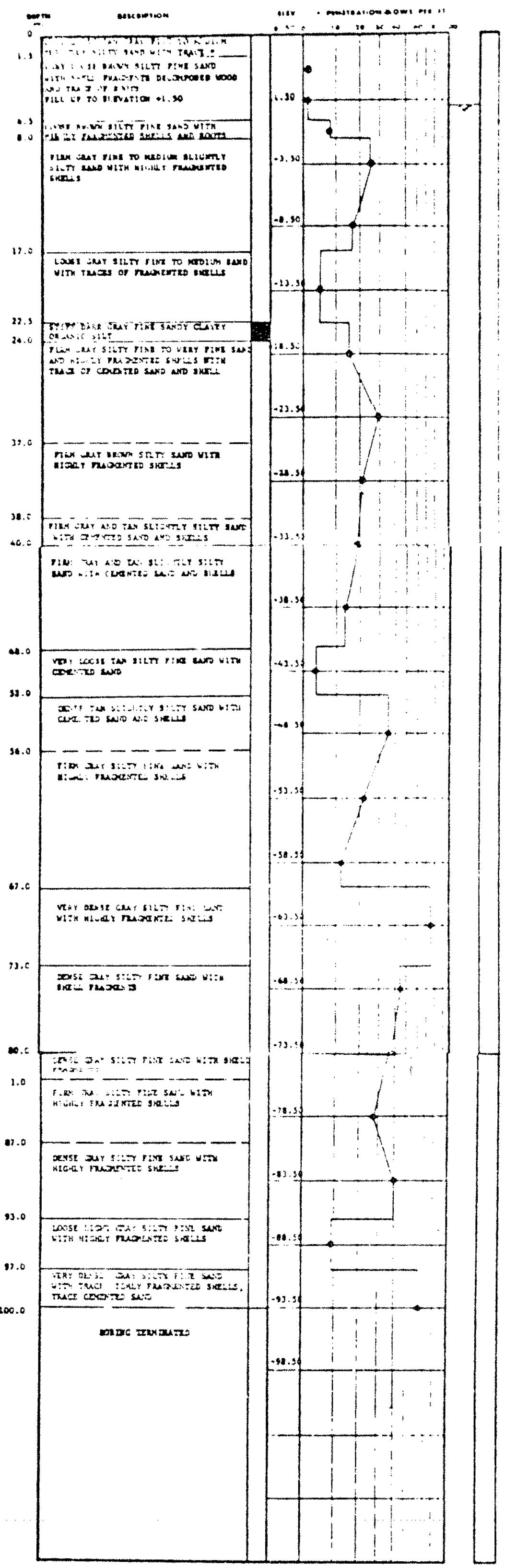
TEST BORING RECORD

BORING NO. B-15

DATE DRILLED 3/21/68

JOB NO. EC-163

LAW ENGINEERING TESTING CO



TEST BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

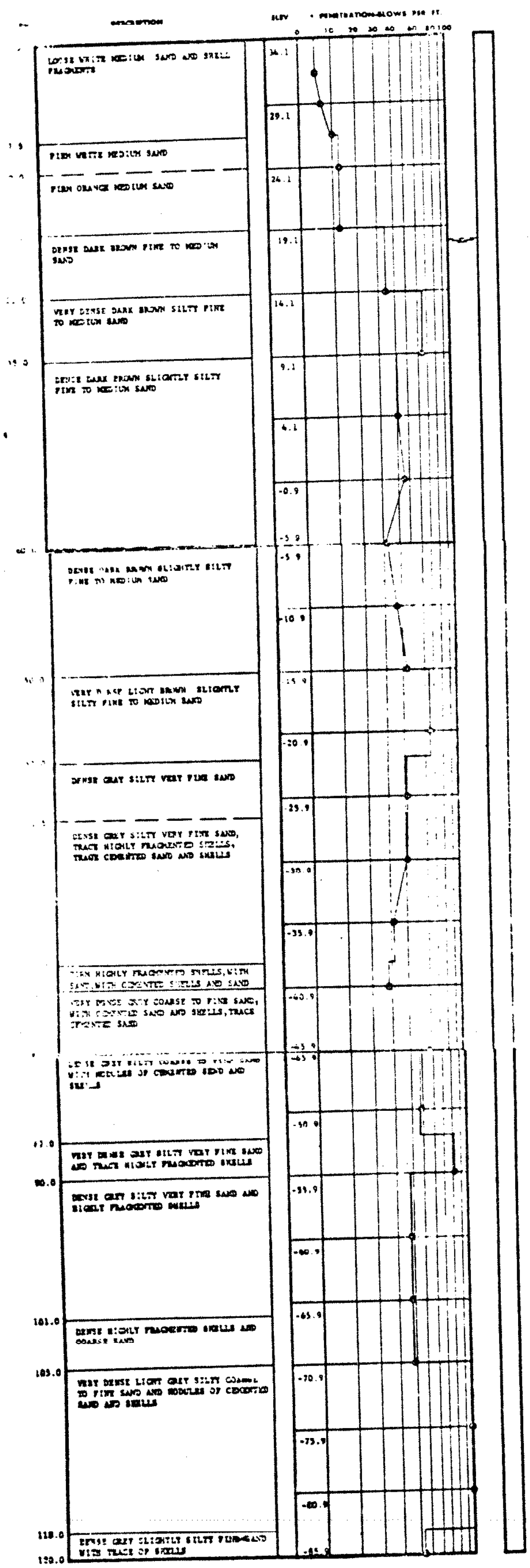
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

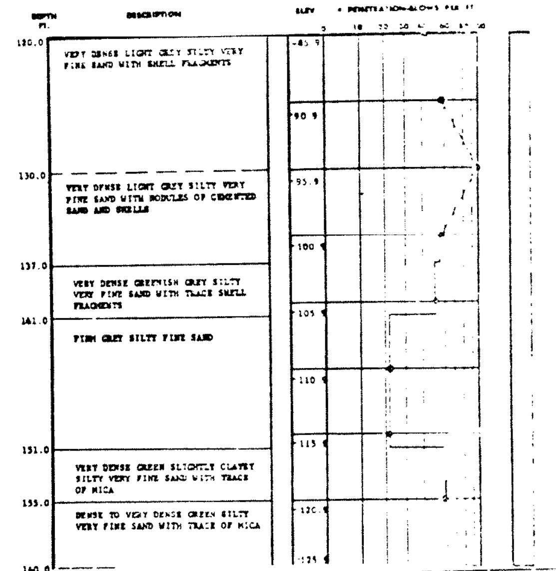
UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

"A" ROD USED IN PENETRATION TEST

BORING NO. B-16
DATE DRILLED 3/20/68
JOB NO. EC-163

LAW ENGINEERING TESTING CO.





Depth	Description	Elev	Weight of Rod and Hammer	Movement by Rod Weight	Hammer [illegible] 6"	Hammer [illegible] 6"
	DENSE TO VERY DENSE GREEN SILTY VERY FINE SAND WITH TRACE OF MICA	-125.9	776	0	15	28
		-130.9	794	0	23	29
		-135.9	812	0	26	36
172.0	DENSE GREEN CLAYEY SILTY VERY FINE SAND WITH TRACE OF MICA	-140.9	830	0	5	11
		-145.9	848	0	4	9
		-150.9	866	0	3	7
185.0	DENSE GREEN SLIGHTLY CLAYEY SILTY VERY FINE SAND WITH TRACE OF MICA	-155.9	884	0	4	11
		-160.9	902	0	5	10
200.0		-165.9	920	0	4	[illegible]

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE — WATER TABLE, 24 HR. — WATER TABLE, 1 HR.
% ROCK CORE RECOVERY — LOSS OF DRILLING WATER

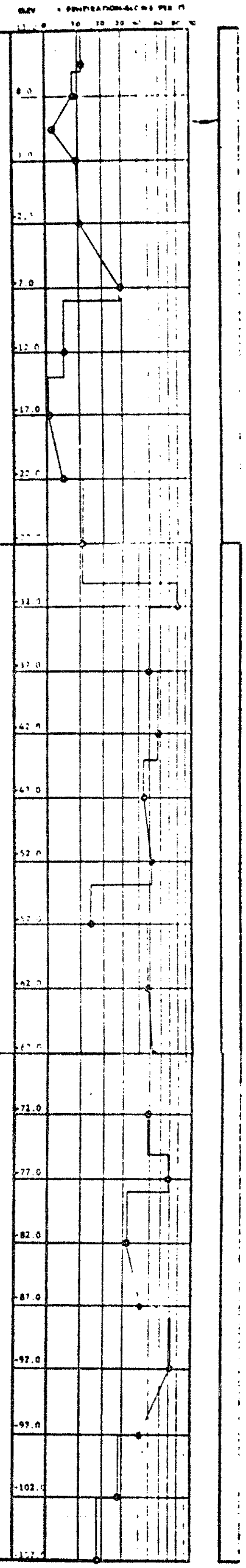
"A" ROD USED IN PENETRATION TEST

TEST BORING RECORD

BORING NO. B-17
DATE DRILLED 5/23-27/68
JOB NO. FC-163

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION
0	FIRM DARK BROWN SILTY SAND WITH ORGANIC MATTER
3.0	LOOSE TO VERY DENSE LIGHT BROWN SILTY FINE SAND
7.5	LOOSE LIGHT BROWN SLIGHTLY CLAYEY SILTY FINE SAND
12.0	FIRM LIGHT BROWN SLIGHTLY CLAYEY SILTY FINE SAND
15.0	FIRM WHITE FINE TO MEDIUM SAND
21.0	LOOSE LIGHT BROWN SLIGHTLY CLAYEY SILTY FINE SAND
27.0	VERY LOOSE TO LOOSE DARK GREY CLAYEY SILTY FINE SAND
34.5	FIRM BROWN SILTY FINE TO MEDIUM SAND
40.0	FIRM BROWN SILTY FINE TO MEDIUM SAND
43.0	VERY [illegible] FINE TO MEDIUM SAND
45.0	DENSE LIGHT GREY SLIGHTLY SILTY FINE TO COARSE SAND AND NODULES OF CEMENTED SAND
50.0	VERY DENSE LIGHT NODULES OF CEMENTED SAND AND SHELLS WITH FINE TO COARSE SAND
57.0	DENSE TO VERY DENSE LIGHT GREY SILTY VERY FINE SAND WITH NODULES OF CEMENTED SAND
67.0	FIRM GREY CLAYEY SILTY VERY FINE SAND
70.0	DENSE TO VERY DENSE LIGHT GREY SILTY VERY FINE SAND WITH TRACE HIGHLY FRAGMENTED SHELLS
80.0	DENSE LIGHT GREY SILTY VERY FINE SAND
88.0	VERY DENSE DARK GREY FINE TO COARSE SAND WITH HIGHLY FRAGMENTED SHELLS
91.0	DENSE GREY SILTY FINE TO COARSE SAND AND SHELLS WITH NODULES OF CEMENTED SAND
100.0	VERY DENSE TO DENSE GREY-GREEN SILTY FINE SAND WITH HIGHLY FRAGMENTED SHELLS
110.0	FIRM GREY-GREEN SILTY FINE SAND AND NODULES OF CEMENTED SAND AND SHELLS
115.0	FIRM GREY SILTY VERY FINE SAND
120.0	



DEPTH FT.	DESCRIPTION	ELEV	WEIGHT OF ROD AND HAMMER	MOVEMENT BY ROD WEIGHT	HAMMER BLOWS 6"	HAMMER BLOWS 6"
120.0	FIRM GRAY SILTY VERY FINE SAND	-107.0	632	0	7	0
122.0	FIRM GREEN CLAYEY SILTY VERY FINE SAND WITH TRACE OF MICA	-112.0	650	0	58	14
125.0	LOOSE GREEN CLAYEY SILTY VERY FINE SAND WITH MICA	-117.0	668	0	3	6
		-122.0	686	0	3	5
140.0	FIRM GREEN SLIGHTLY CLAYEY SILTY VERY FINE SAND WITH MICA	-127.0	704	0	4	6
145.0	FIRM GREEN CLAYEY SILTY VERY FINE SAND WITH MICA	-132.0	722	0	7	8
150.0	FIRM GREEN SLIGHTLY CLAYEY SILTY VERY FINE SAND WITH MICA	-137.0	740	0	4	6
		-142.0	758	0	4	11
160.0	FIRM GREEN CLAYEY SILTY VERY FINE SAND WITH MICA	-147.0	776	0	4	8
			776	0	4	8
		-152.0	794	0	5	7
168.0	STIFF GREEN CLAYEY SILT WITH MICA	-157.0	812	0	[illegible]	7
		-162.0	830	0	4	8
180.0	VERY STIFF GREEN VERY FINE SANDY CLAYEY SILT WITH MICA	-167.0	848	0	5	9
185.0	VERY STIFF TO STIFF GREEN CLAYEY SILT WITH MICA	-172.0	866	0	7	12
		-177.0	884	0	4	10
		-182.0	902	0	4	8
200.0		-187.0	920	0	3	[illegible]

SOIL AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

"A" ROD USED IN PENETRATION TEST

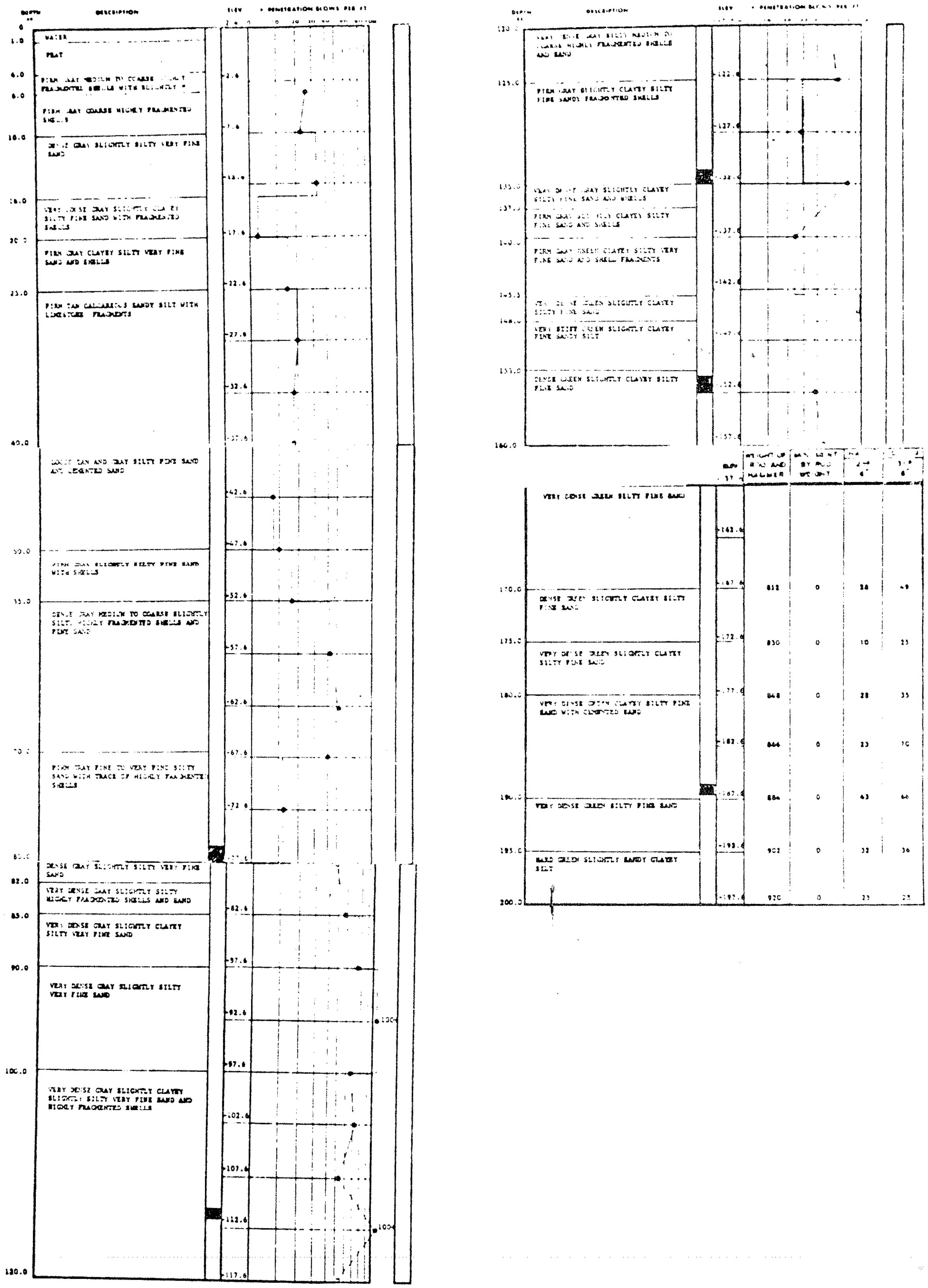
TEST BORING RECORD

BORING NO. B-18

DATE DRILLED 5/29-30-68

JOB NO. EC-163

LAW ENGINEERING TESTING CO.



ELEV. FT	WEIGHT OF ROD AND HAMMER	BLOWS SENT BY ROD WEIGHT	2-4 6"	3-4 6"
-167.6	812	0	28	49
-172.6	830	0	10	25
-177.6	848	0	28	35
-182.6	866	0	33	70
-187.6	884	0	43	66
-192.6	902	0	32	36
-197.6	920	0	23	23

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE — WATER TABLE, 24 HR.

% ROCK CORE RECOVERY — WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

TEST BORING RECORD

BORING NO. B-19

DATE DRILLED 8/13-19/68

JOB NO. EC-163

LAW ENGINEERING TESTING CO.

DEPTH FT	DESCRIPTION	ELEV
0	WATER	
1.0	PEAT, ROOTS, ORGANIC SILT	
3.5	FIRM GRAY MODERATE TO COARSE HIGHLY FRAGMENTED SHELLS	-2.6
7.0	FIRM GRAY SILTY VERY FINE TO FINE SAND WITH TRACE OF FRAGMENTED SHELLS	-7.6
		-12.6
		-17.6
20.0	VERY SOFT GRAY ORGANIC CLAYEY SILT WITH FRAGMENTED SHELLS AND OCCASIONAL LIMESTONE FRAGMENTS	-22.6
25.0	VERY STIFF GRAY SLIGHTLY SANDY CLAYEY SILT WITH OCCASIONAL SHELL FRAGMENTS	-27.6
29.0	[illegible] TAN [illegible] SAND AND	
30.1	[illegible] CEMENTED SAND PEBBLES	
30.4	DENSE TAN SILTY FINE SAND AND SHELLS WITH CEMENTED SAND	-32.6
35.0	FIRM TAN SILTY HIGHLY FRAGMENTED SHELLS AND FINE SAND	-37.6
40.0	FIRM TAN SILTY HIGHLY FRAGMENTED SHELLS AND FINE SILTY SAND	-42.6
45.0	HARD TAN LIMESTONE PEBBLES AND FINE SILTY SAND	
48.2	HARD CEMENTED SHELL AND DENSE TAN SLIGHTLY CLAYEY SILTY FINE SAND	-47.6
50.0	STIFF GRAY CLAYEY SILT WITH OCCASIONAL SHELL FRAGMENTS	-52.6
55.0	VERY DENSE GRAY SILTY SHELL FRAGMENTS AND FINE SILTY SAND	-57.6
60.0	VERY DENSE GRAY SILTY FINE SAND WITH OCCASIONAL SHELL FRAGMENTS	-62.6
		-67.6
72.0	FIRM GRAY SILTY FINE SAND	-72.6
75.0	VERY DENSE GRAY SILTY FINE SAND	-77.6
80.0	VERY DENSE GRAY SILTY VERY FINE SAND	-82.6
85.0	VERY DENSE GRAY SLIGHTLY CLAYEY SILTY VERY FINE SAND	-87.6
90.0	VERY DENSE GRAY SILTY VERY FINE SAND	-92.6
95.0	VERY DENSE GRAY SLIGHTLY CLAYEY SILTY VERY FINE SAND	-97.6
103.0	VERY DENSE GRAY SLIGHTLY CLAYEY SILTY VERY FINE SAND WITH OCCASIONAL SHELL FRAGMENTS AND CEMENTED SAND	-102.6
108.0	VERY DENSE GRAY MEDIUM SILTY SAND AND HIGHLY FRAGMENTED SHELLS	-107.6
110.0	VERY DENSE GRAY FINE SILTY SAND WITH OCCASIONAL SHELL FRAGMENTS	-112.6
118.0	VERY DENSE GRAY-GREEN CLAYEY FINE SILTY SAND WITH OCCASIONAL SHELLS	-117.6
120.0		

DEPTH FT	DESCRIPTION	ELEV
120.0	VERY DENSE GRAY CLAYEY SILTY VERY FINE SAND	-122.6
		-127.6
		-132.6
		-137.6
140.3	HARD GRAY-GREEN CLAYEY SANDY SILT WITH OCCASIONAL SHELL FRAGMENTS	
144.0	DENSE GRAY SLIGHTLY CLAYEY SILTY FINE SAND	-142.6
147.0	HARD GRAY-GREEN CLAYEY SANDY SILT	
149.0	DENSE GRAY-GREEN SILTY [illegible] CLAYEY [illegible] FINE SAND WITH OCCASIONAL SHELL FRAGMENTS	-147.6
155.0	FIRM GREEN SLIGHTLY SILTY FINE SAND	-152.6
160.0		-157.6

DEPTH	DESCRIPTION	ELEV	WEIGHT OF ROD AND HAMMER	MOVEMENT BY ROD WEIGHT	6"	6"
160.0	FIRM GREEN SLIGHTLY SILTY VERY FINE SAND		[illegible]	0	9	12
165.0	FIRM GREEN SLIGHTLY CLAYEY SILTY VERY FINE SAND	-162.6	967	0	6	9
		-167.6	991	0	6	18
175.0	[illegible] DENSE GREEN SILTY VERY FINE SAND	-172.6	1014	0	[illegible]	26
180.0	DENSE GREEN SILTY FINE TO VERY FINE SAND	-177.6	1037	0	21	42
		-182.6	1060	0	[illegible]	34
190.0	HARD GREEN SANDY CLAYEY SILT	-187.6	1084	0	8	33
195.0	VERY DENSE SILTY FINE SAND	-192.6	1107	0	3	34
200.0		-197.6	1130			[illegible]

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
[50] % ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

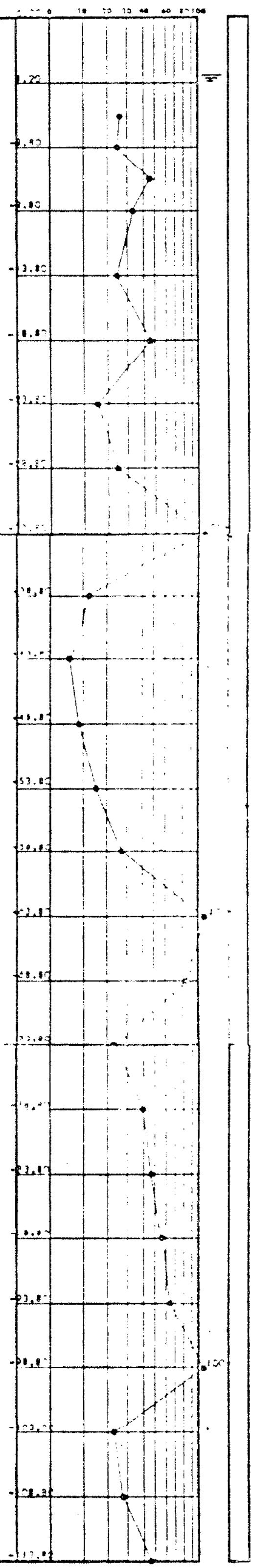
TEST BORING RECORD

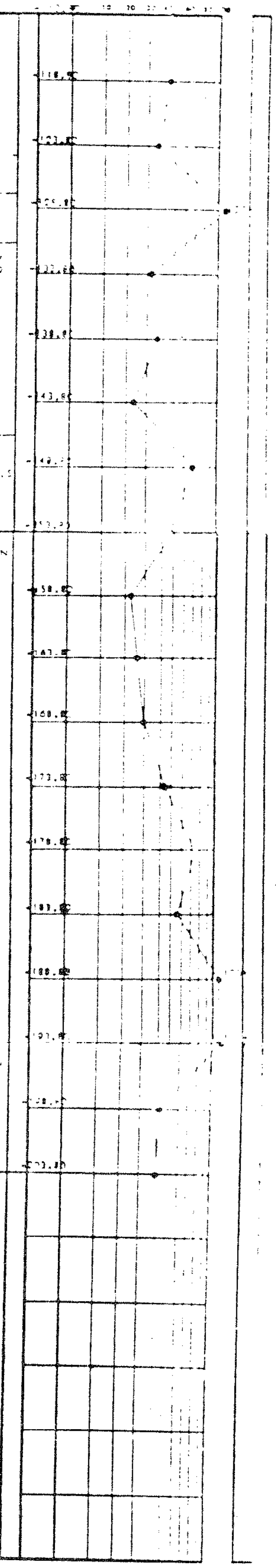
BORING NO. B-20
DATE DRILLED 8/13-20/68
JOB NO. EC-163

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION
0.0	GRAY SAND FILL (ROAD)
5.0	BROWN SANDY ORGANIC SILT (PEAT)
7.5	VERY FIRM TO DENSE GRAY TO BROWN SILTY FINE TO MEDIUM SAND AND FRAGMENTED SHELLS
28.0	FIRM LIGHT GRAY MEDIUM SAND AND NUMEROUS FRAGMENTED SHELLS
33.0	DENSE TO FIRM LIGHT TAN SILTY MEDIUM SAND WITH NODULES OF CEMENTED SAND AND SHELLS
40.0	DENSE TO FIRM LIGHT TAN SILTY MEDIUM SAND WITH NODULES OF CEMENTED SAND AND SHELLS
[illegible]	LOOSE TO VERY FIRM LIGHT GRAY SILTY MEDIUM SAND AND SHELLS WITH LIMESTONE NODULES
69.0	VERY DENSE GRAY SILTY MEDIUM SAND AND SHELLS
74.0	VERY DENSE TO VERY FIRM LIGHT GRAY SILTY FINE SAND AND SHELLS
80.0	VERY DENSE TO VERY FIRM LIGHT GRAY SILTY FINE SAND AND SHELLS
84.5	DENSE TO VERY DENSE GRAY SILTY FINE TO MEDIUM SAND AND SHELLS (SMALL CLAY SEAM 84.5-85.0)
97.0	VERY DENSE LIGHT GRAY SILTY VERY FINE SAND AND TRACE OF SHELLS
105.0	VERY DENSE TO VERY FIRM LIGHT GRAY SILTY MEDIUM SAND WITH FRAGMENTED SHELLS AND NODULES OF LIMESTONE
110.7	VERY FIRM GRAY SILTY, CLAYEY MEDIUM SAND AND SHELLS
119.5	DENSE GRAY SILTY MEDIUM SAND
120.0	DENSE GRAY SILTY MEDIUM SAND AND FRAGMENTED SHELLS
134.0	VERY DENSE GRAY SILTY MEDIUM SAND WITH SHELLS AND SMALL LIMESTONE NODULES
138.0	DENSE TO VERY FIRM GRAY SILTY SLIGHTLY CLAYEY FINE SAND AND SHELLS
153.0	VERY DENSE TO VERY FIRM GREEN SILTY SLIGHTLY CLAYEY FINE SAND WITH SMALL TRACE OF SHELLS
160.0	VERY DENSE TO VERY FIRM GREEN SILTY SLIGHTLY CLAYEY FINE SAND WITH SMALL TRACE OF SHELLS
200.0	VERY DENSE TO VERY FIRM GREEN SILTY SLIGHTLY CLAYEY FINE SAND WITH SMALL TRACE OF SHELLS
210.0	BORING TERMINATED

ELEV. • PENETRATION-BLOWS PER FT.





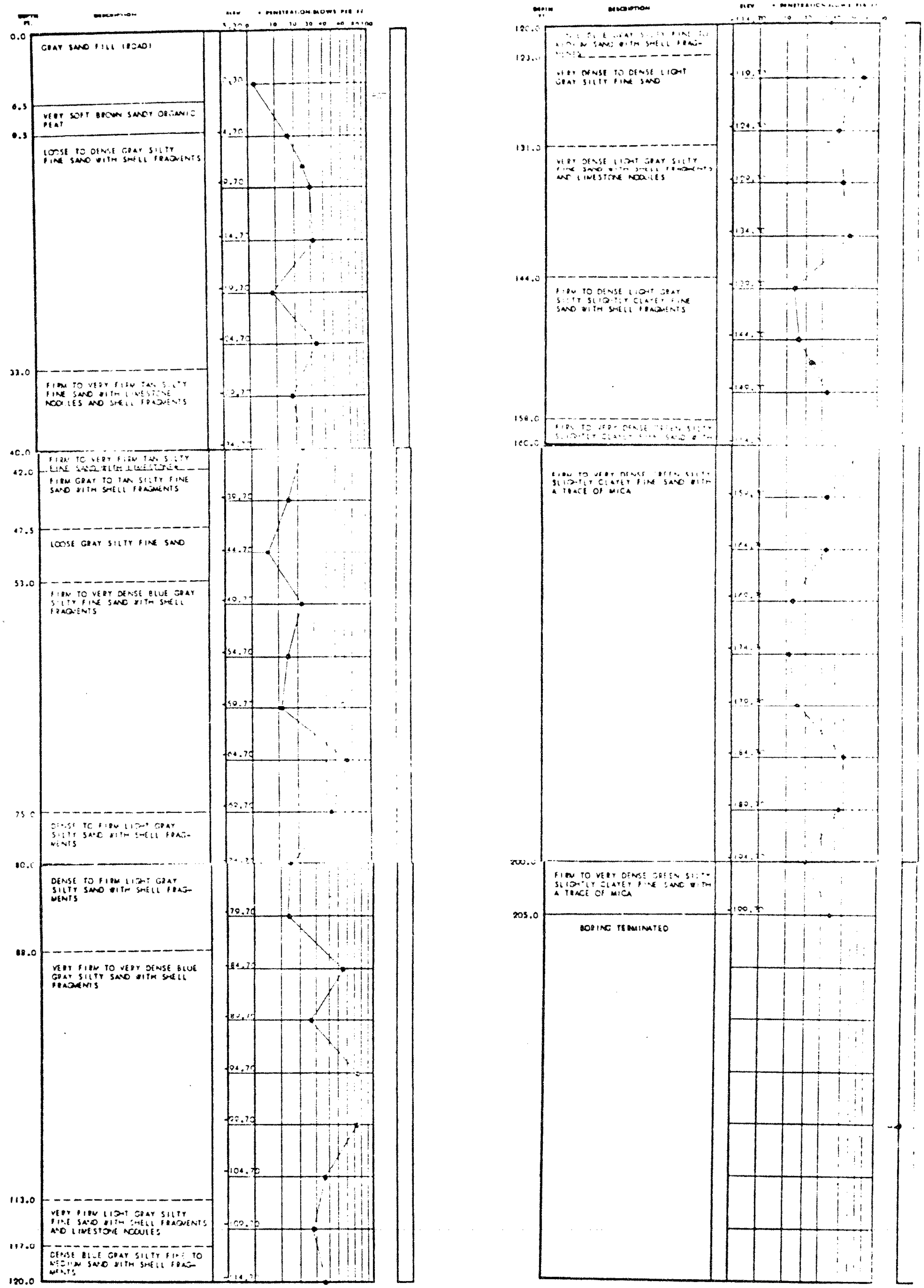
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

TEST BORING RECORD

BORING NO. B-101
DATE DRILLED 11/21/68
JOB NO. 1-1127

LAW ENGINEERING TESTING CO.



TEST BORING RECORD

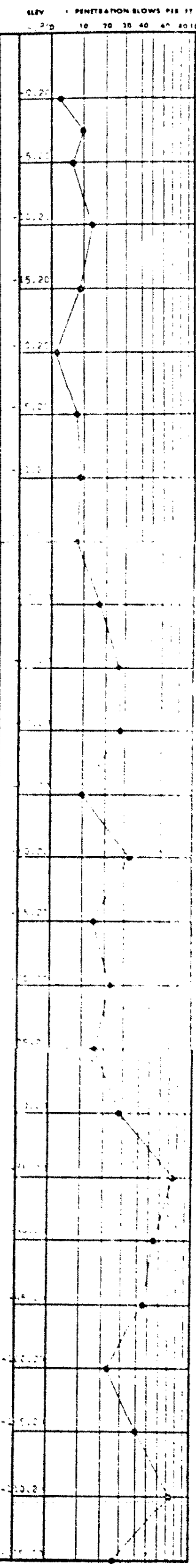
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

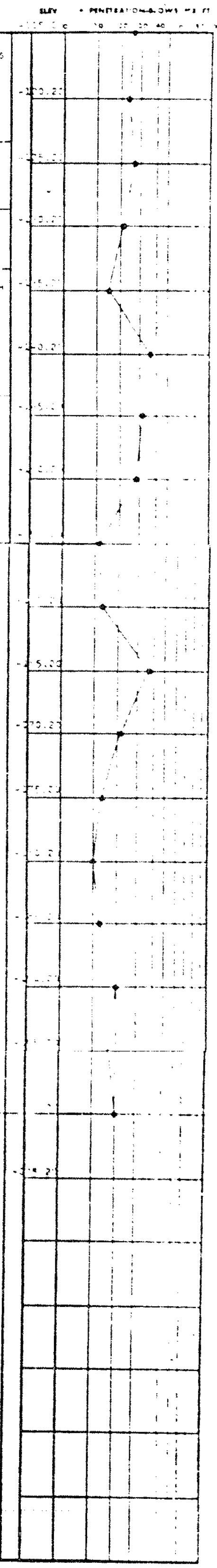
BORING NO. B-102
DATE DRILLED 12/4/68
JOB NO. 1-1127

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION
0.0	GRAY SAND FILL (ROAD)
5.0	SOFT BROWN SILTY CLAYEY ORGANIC PEAT
7.0	LOOSE BROWN TO GRAY SILTY FINE SAND WITH ORGANIC MATERIAL
14.5	FIRM TO LOOSE BLUE GRAY SILTY FINE TO MEDIUM SAND WITH SHELL FRAGMENTS
23.5	SOFT GRAY SILTY SLIGHTLY SANDY PLASTIC CLAY WITH TRACE OF SHELLS
28	LOOSE BROWN SILTY MEDIUM SAND WITH ORGANIC MATERIAL
32.7	LOOSE BROWN TO GRAY SILTY CLAYEY MEDIUM SAND WITH ORGANIC MATERIAL
[illegible]	[illegible] BROWN TO GRAY SILTY [illegible] SAND WITH ORGANIC MATERIAL
[illegible]	[illegible] BROWN TO GRAY SILTY PLASTIC CLAY WITH LIMESTONE LAYERS
[illegible]	VERY FIRM TAN SILTY FINE SAND WITH LIMESTONE [illegible]
[illegible]	VERY FIRM GRAY SILTY SLIGHTLY CLAYEY FINE SAND WITH SHELL FRAGMENTS
[illegible]	LOOSE TO DENSE GRAY SILTY FINE SAND WITH SHELL FRAGMENTS
[illegible]	VERY FIRM TO FIRM LIGHT GRAY SILTY VERY FINE SAND WITH SHELL FRAGMENTS
[illegible]	VERY FIRM TO FIRM LIGHT GRAY SILTY VERY FINE SAND WITH SHELL FRAGMENTS
85	VERY FIRM GRAY SILTY FINE SAND WITH CEMENTED SAND AND SHELLS
89	VERY DENSE GRAY SILTY VERY FINE SAND WITH SHELL FRAGMENTS
93	VERY DENSE TO FIRM LIGHT GRAY SILTY VERY FINE SAND WITH TRACE OF CEMENTED SAND AND SHELL FRAGMENTS
109.5	DENSE TO VERY DENSE GRAY SILTY FINE SAND WITH SHELLS
119.0	VERY FIRM GREENISH GRAY SILTY FINE SAND WITH SHELL FRAGMENTS
120.0	



DEPTH FT.	DESCRIPTION
120.0	VERY FIRM GREENISH GRAY SILTY FINE SAND WITH SHELL FRAGMENTS
128.0	VERY FIRM LIGHT GRAY SILTY FINE TO MEDIUM SAND WITH CEMENTED SAND AND TRACE OF SHELL FRAGMENTS
134.0	VERY FIRM GREENISH GRAY SILTY CLAYEY FINE SAND WITH SHELL FRAGMENTS
138.0	FIRM TO DENSE GRAY TO GREENISH GRAY SILTY FINE SAND
[illegible]	DENSE TO FIRM GREEN SILTY SLIGHTLY CLAYEY FINE TO VERY FINE SAND WITH TRACE OF MICA
160	DENSE TO FIRM GREEN SILTY SLIGHTLY CLAYEY FINE TO VERY FINE SAND WITH TRACE OF MICA
200.0	[illegible] GREEN SILTY SLIGHTLY CLAYEY FINE [illegible] FINE SAND WITH TRACE OF MICA
205.0	BORING TERMINATED



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

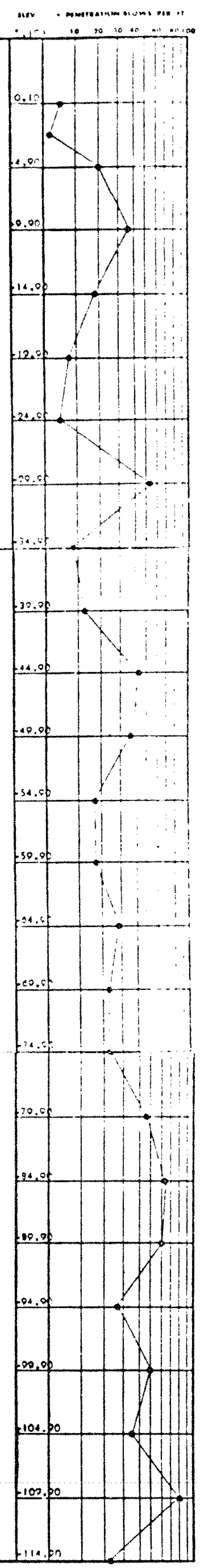
TEST BORING RECORD

BORING NO. B-103
DATE DRILLED 1/13/69
JOB NO. J-1127

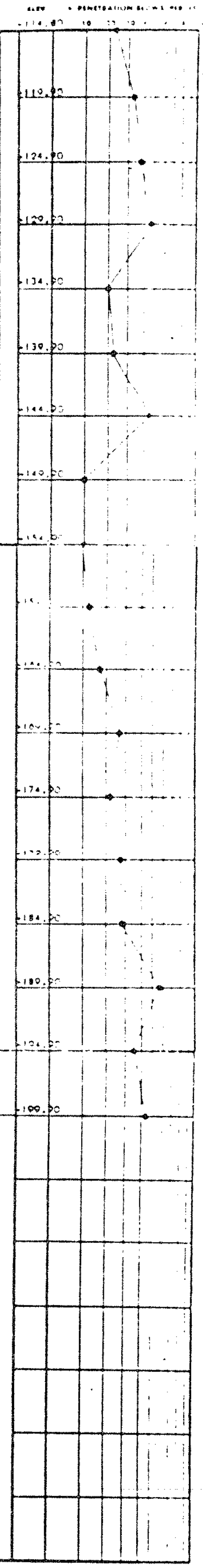
LAW ENGINEERING TESTING CO.

TEST BORING RECORD

DEPTH FT	DESCRIPTION
0.0	GRAY TO BROWN SAND FILL (ROAD)
5.0	VERY SOFT BROWN SANDY CLAYEY ORGANIC PEAT
8.0	FIRM TO DENSE GRAY SILTY FINE SAND WITH SHELL FRAGMENTS
22.0	FIRM GRAY SILTY SLIGHTLY SANDY VERY PLASTIC CLAY
29.0	LOOSE GRAY SILTY VERY CLAYEY FINE TO MEDIUM SAND WITH SHELL FRAGMENTS
33.0	VERY DENSE GRAY TO TAN SILTY CLAYEY FINE TO MEDIUM SAND WITH CEMENTED SANDS AND SHELLS
38.0	STIFF GRAY SILTY SANDY CLAY
40.0	STIFF GRAY SILTY SANDY CLAY WITH FRAGMENTED SHELLS AND LIMESTONE NODULES
48.0	DENSE GRAY TO TAN MEDIUM SAND WITH LIMESTONE NODULES
58.0	VERY STIFF GRAY SILTY VERY PLASTIC CLAY WITH LIMESTONE NODULES
68.0	VERY FIRM BLUE GRAY SILTY FINE TO MEDIUM SAND WITH FRAGMENTED SHELLS AND LIMESTONE NODULES
80.0	VERY FIRM BLUE GRAY SILTY FINE TO MEDIUM SAND WITH FRAGMENTED SHELLS AND LIMESTONE NODULES
84.5	DENSE LIGHT GRAY SILTY VERY FINE SAND WITH TRACE SHELL CONTENT AND LIMESTONE NODULES
88.0	VERY DENSE GRAY SILTY FINE SAND WITH LARGE SHELL CONTENT AND LIMESTONE NODULES
98.0	VERY FIRM TO VERY DENSE LIGHT GRAY SILTY VERY FINE SAND WITH TRACE OF SHELLS AND LIMESTONE NODULES
120.0	



DEPTH FT	DESCRIPTION
120.0	VERY FIRM TO VERY DENSE LIGHT GRAY SILTY VERY FINE SAND WITH TRACE OF SHELLS AND LIMESTONE NODULES
127.0	HARD LIGHT GRAY SANDY SLIGHTLY CLAYEY SILT WITH SHELL FRAGMENTS
139.0	VERY FIRM LIGHT GRAY SILTY FINE SAND WITH TRACE OF SHELLS
147.0	DENSE GREEN SILTY SLIGHTLY CLAYEY VERY FINE SAND WITH A TRACE OF MICA
153.0	STIFF GREEN SANDY SLIGHTLY CLAYEY SILT WITH A TRACE OF MICA
160.0	STIFF GREEN SANDY SLIGHTLY CLAYEY SILT WITH A TRACE OF MICA
163.0	FIRM TO VERY DENSE GREEN SILTY SLIGHTLY CLAYEY VERY FINE SAND WITH A TRACE OF MICA
200.0	FIRM TO VERY DENSE GREEN SILTY SLIGHTLY CLAYEY VERY FINE SAND WITH A TRACE OF MICA
205.0	BORING TERMINATED

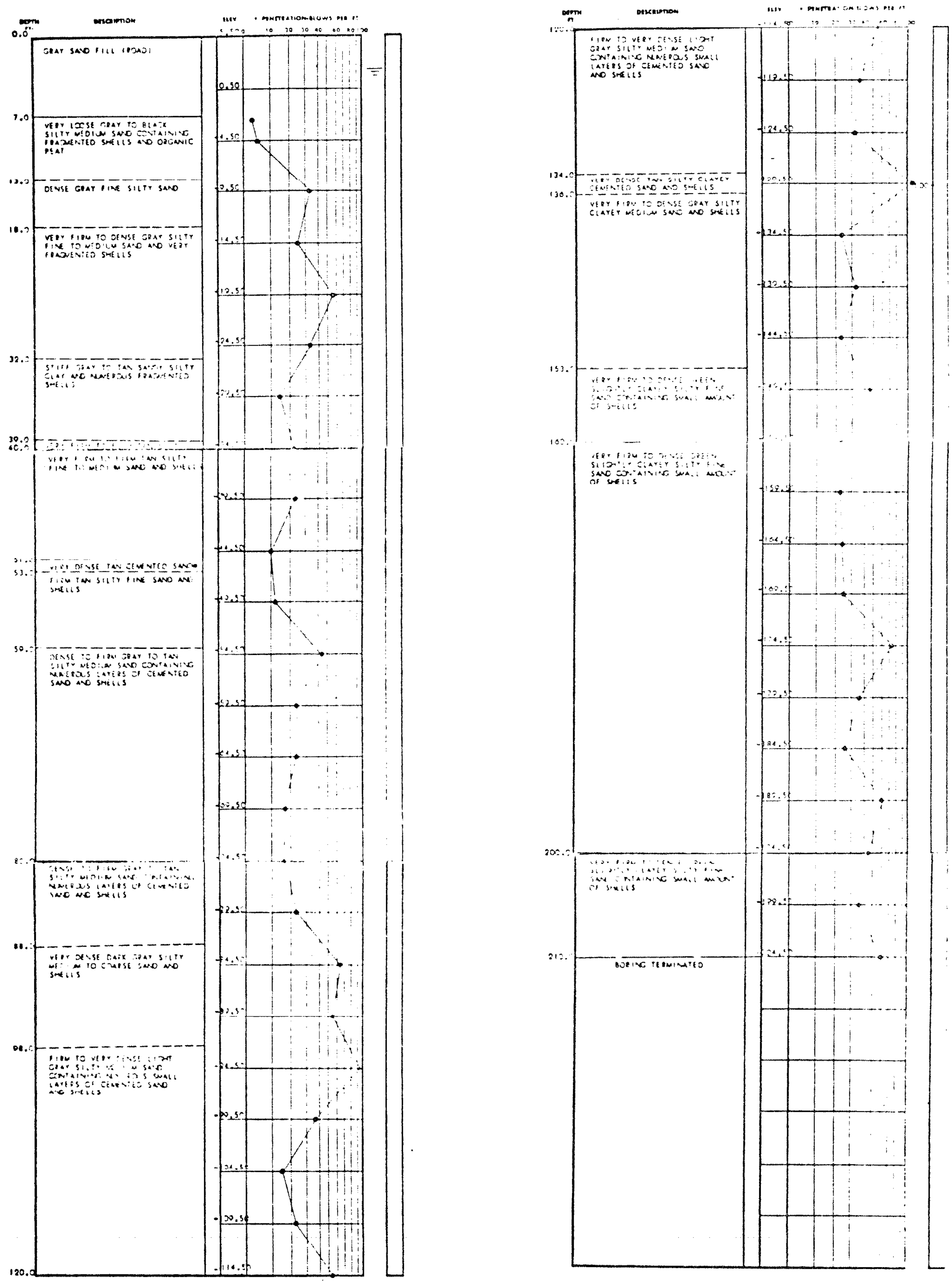


BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

BORING NO. B-104
DATE DRILLED 1/11/69
JOB NO. J-1127

LAW ENGINEERING TESTING CO.



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE 24 HR.
WATER TABLE 1 HR.
LOSS OF DRILLING WATER

TEST BORING RECORD

BORING NO. B-105
DATE DRILLED 11/18/68
JOB NO. J-1127

LAW ENGINEERING TESTING CO.

DEPTH FT	DESCRIPTION
0.0	GRAY SAND FILL (ROAD)
7.0	LOOSE GRAY TO BROWN SILTY SAND AND ORGANIC PEAT
9.5	VERY FIRM GRAY SILTY FINE SAND AND SHELL TRACES
23.0	DENSE TO VERY FIRM GRAY SILTY MEDIUM SAND AND NUMEROUS FRAGMENTED SHELLS
31.5	VERY STIFF GRAY GREEN SANDY SILTY CLAY AND FRAGMENTED SHELLS
35.0	VERY FIRM TO LOOSE GRAY TO TAN SILTY FINE SAND WITH NODULES OF LIMESTONE AND FRAGMENTED SHELLS-SOME SHELLS CEMENTED
40.0	VERY FIRM TO LOOSE GRAY TO TAN SILTY FINE SAND WITH NODULES OF LIMESTONE AND FRAGMENTED SHELLS-SOME SHELLS CEMENTED
61.0	FIRM TO DENSE DARK BLUE GRAY SILTY FINE TO MEDIUM SAND WITH FRAGMENTED SHELLS
80.0	FIRM TO DENSE DARK BLUE GRAY SILTY FINE TO MEDIUM SAND WITH FRAGMENTED SHELLS
89.0	VERY DENSE TO DENSE GRAY SILTY VERY FINE SAND WITH TRACE OF FRAGMENTED SHELLS AND LIMESTONE NODULES
114.0	VERY DENSE BLUE GRAY MEDIUM SAND AND FRAGMENTED SHELLS
120.0	VERY DENSE BLUE GRAY MEDIUM SAND AND FRAGMENTED SHELLS
123.0	DENSE TO VERY FIRM GRAY SILTY FINE SAND WITH FRAGMENTED SHELLS
133.0	MODERATELY HARD SANDY LIMESTONE
136.0	FIRM TO VERY FIRM GRAY SILTY SLIGHTLY CLAYEY FINE SAND CONTAINING SHELL FRAGMENTS AND LIMESTONE NODULES
154.0	FIRM TO VERY DENSE OLIVE GREEN SILTY SLIGHTLY CLAYEY VERY FINE SAND
160.0	FIRM TO VERY DENSE OLIVE GREEN SILTY SLIGHTLY CLAYEY VERY FINE SAND
200.0	FIRM TO VERY DENSE OLIVE GREEN SILTY SLIGHTLY CLAYEY VERY FINE SAND
205.0	BORING TERMINATED

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

TEST BORING RECORD

BORING NO. B-106
DATE DRILLED 11/21/68
JOB NO. 1-1127

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION
0.0	GRAY SAND FILL (ROAD)
5.0	LOOSE GRAY TO BROWN SILTY FINE SAND WITH SHELLS AND PEAT
8.0	FIRM TO DENSE GRAY SILTY FINE SAND WITH SHELL FRAGMENTS
19.0	VERY FIRM TO FIRM GRAY SILTY VERY FINE SAND WITH SHELL FRAGMENTS
26.0	VERY FIRM TO DENSE GRAY TO TAN SILTY FINE SAND WITH FRAGMENTED SHELLS AND CEMENTED SANDS AND SHELLS
34.0	DENSE TO LOOSE TAN SILTY FINE TO MEDIUM SAND WITH SHELL FRAGMENTS AND LIMESTONE NODULES
40.0	DENSE TO LOOSE TAN SILTY FINE TO MEDIUM SAND WITH SHELL FRAGMENTS AND LIMESTONE NODULES
60.0	VERY FIRM LIGHT GRAY SILTY VERY FINE SAND WITH TRACE OF SHELL FRAGMENTS
66.0	VERY DENSE BLUE GRAY SILTY FINE TO MEDIUM SAND WITH SHELL FRAGMENTS
69.0	
74.5	VERY DENSE TO FIRM GRAY SILTY VERY FINE SAND WITH SHELL FRAGMENTS
80.0	VERY DENSE TO FIRM GRAY SILTY VERY FINE SAND WITH SHELL FRAGMENTS
90.0	VERY FIRM TO VERY DENSE GRAY SILTY FINE TO MEDIUM SAND WITH SHELL FRAGMENTS
98.0	VERY DENSE TO DENSE LIGHT GRAY SILTY VERY FINE SAND WITH TRACE OF LIMESTONE NODULES AND SHELL FRAGMENTS
114.0	DENSE TO VERY DENSE GRAY SILTY FINE SAND WITH SHELL FRAGMENTS AND LIMESTONE NODULES
120.0	DENSE TO VERY DENSE GRAY SILTY FINE SAND WITH SHELL FRAGMENTS AND LIMESTONE NODULES
128.0	DENSE TO FIRM GREENISH GRAY SILTY SLIGHTLY CLAYEY FINE SAND WITH SHELL FRAGMENTS
154.0	LOOSE TO VERY DENSE GREEN SILTY SLIGHTLY CLAYEY VERY FINE SAND WITH TRACE OF MICA
160.0	LOOSE TO VERY DENSE GREEN SILTY SLIGHTLY CLAYEY VERY FINE SAND WITH TRACE OF MICA
200.0	LOOSE TO VERY DENSE GREEN SILTY SLIGHTLY CLAYEY VERY FINE SAND WITH TRACE OF MICA
228.0	FIRM TO VERY STIFF GREEN SANDY SLIGHTLY CLAYEY SILT WITH TRACE OF MICA
240.0	

ELEV: 4.70, -0.30, -5.30, -10.30, -15.30, -20.30, -25.30, -30.30, -35.30, -40.30, -45.30, -50.30, -55.30, -60.30, -65.30, -70.30, -75.30, -80.30, -85.30, -90.30, -95.30, -100.30, -105.30, -110.30, -115.30, -120.30, -125.30, -130.30, -135.30, -140.30, -145.30, -150.30, -155.30, -160.30, -165.30, -170.30, -175.30, -180.30, -185.30, -190.30, -195.30, -200.30, -205.30, -210.30, -215.30, -220.30, -225.30, -230.30, -235.30

PENETRATION-BLOWS PER FT: 10, 20, 30, 40, 60, 80, 100

100+

100+

TEST BORING RECORD

BORING NO. B-107

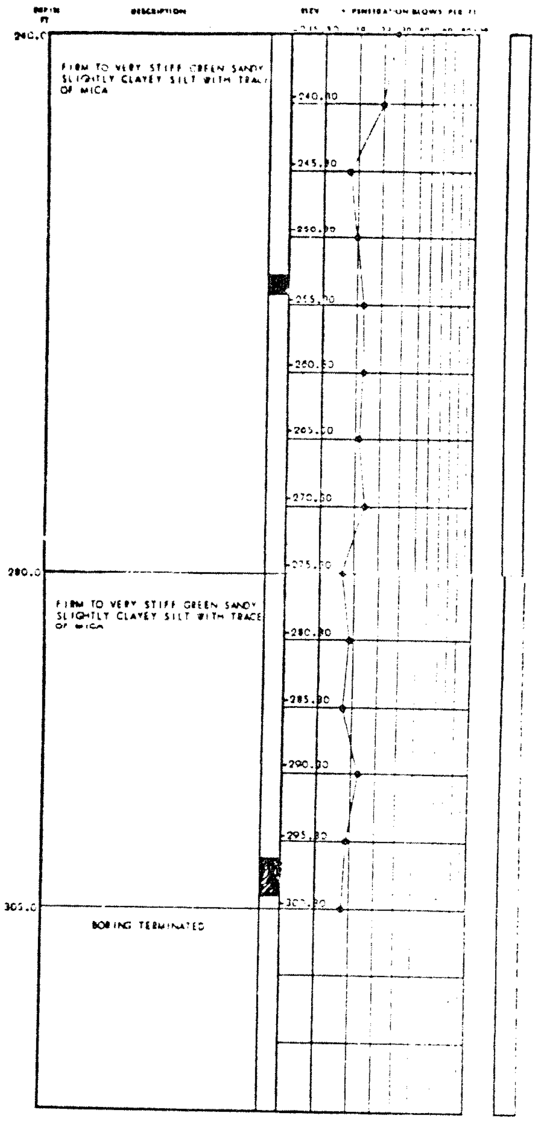
DATE DRILLED 12/17/68

JOB NO. J-1127

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE — WATER TABLE, 24 HR.
% ROCK CORE RECOVERY — WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

LAW ENGINEERING TESTING CO.



TEST BORING RECORD

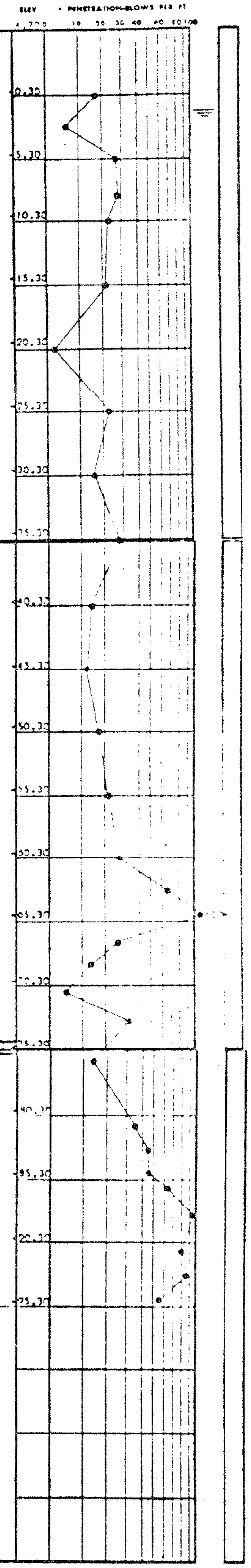
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

BORING NO. B-107
DATE DRILLED 12/17/68
JOB NO. J-1127

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION
0.0	GRAY SAND FILL (ROAD)
7.0	FIRM BROWN SILTY SANDY PEAT
9.5	VERY FIRM BLUE TO GRAY SILTY FINE TO MEDIUM SAND AND SHELL FRAGMENTS
16.0	VERY FIRM GRAY SILTY FINE SAND WITH A TRACE OF SHELLS
24.8	VERY SOFT GRAY SILTY SANDY CLAY
29.0	FIRM TO VERY FIRM GRAY SILTY FINE SAND WITH NODULES OF LIMESTONE AND CEMENTED SAND AND SHELLS
40.0	FIRM TO VERY FIRM GRAY SILTY FINE SAND WITH NODULES OF LIMESTONE AND CEMENTED SAND AND SHELLS
48.0	FIRM GRAY SILTY CLAYEY FINE SAND WITH SHELLS AND NODULES OF LIMESTONE
58.5	FIRM TO VERY DENSE GRAY TAN SILTY FINE SAND AND SHELL FRAGMENTS
74.5	LOOSE GRAY SILTY CLAYEY FINE SAND WITH SHELL FRAGMENTS
77.5	DENSE TO FIRM GRAY SILTY FINE SAND WITH SHELL FRAGMENTS
80.0	DENSE TO FIRM GRAY SILTY FINE SAND WITH SHELL FRAGMENTS
84.0	DENSE TO VERY DENSE BLUE GRAY SILTY FINE TO MEDIUM SAND WITH LARGE SHELL CONTENT
92.5	VERY DENSE LIGHT GRAY SILTY FINE SAND WITH LIMESTONE NODULES
100.0	BORING TERMINATED



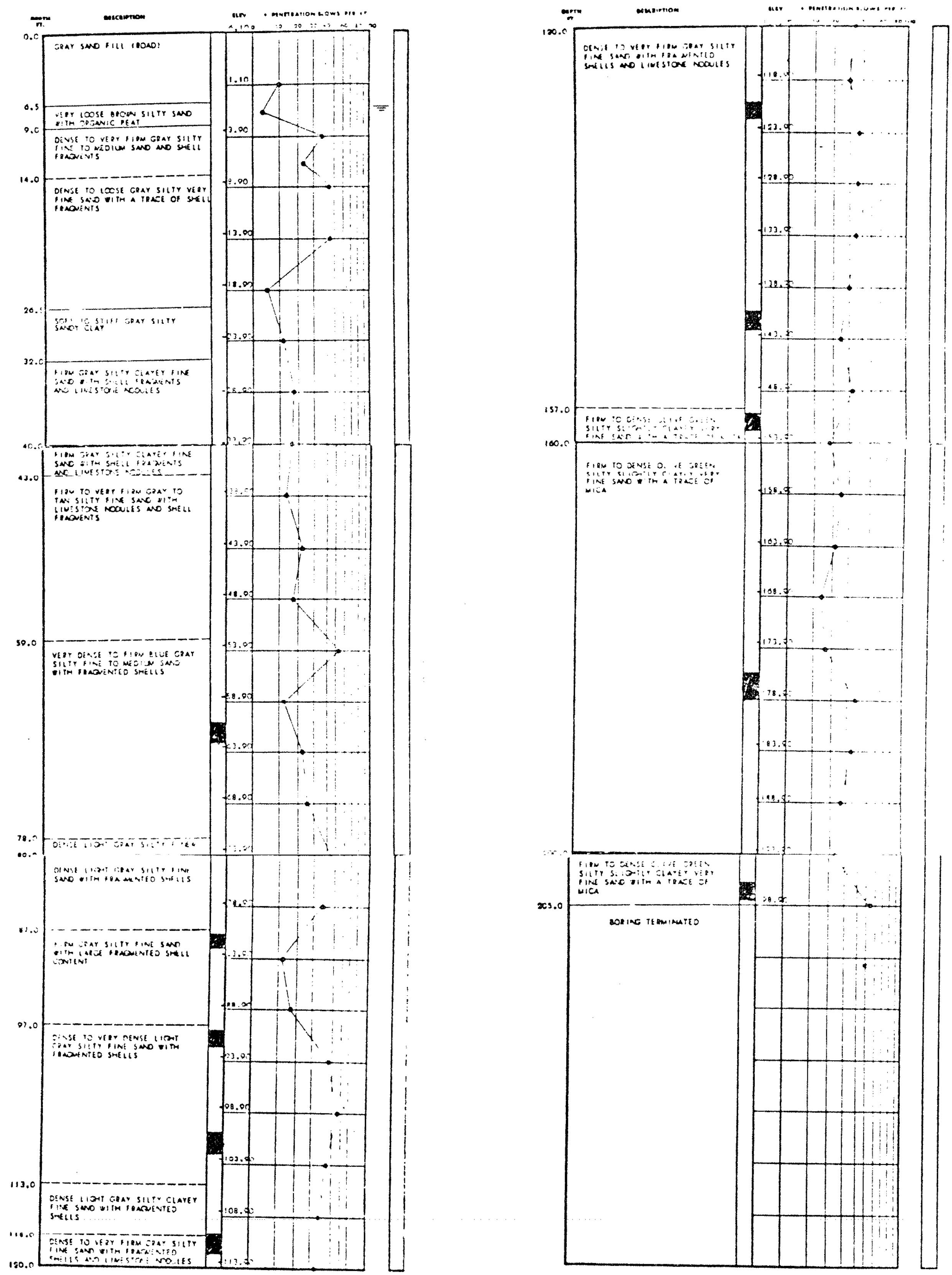
TEST BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

WATER TABLE, 24 HR.
UNDISTURBED SAMPLE
WATER TABLE, 1 HR.
% ROCK CORE RECOVERY
LOSS OF DRILLING WATER

BORING NO. B-107A
DATE DRILLED 12/10/68
JOB NO. J-1127

LAW ENGINEERING TESTING CO.



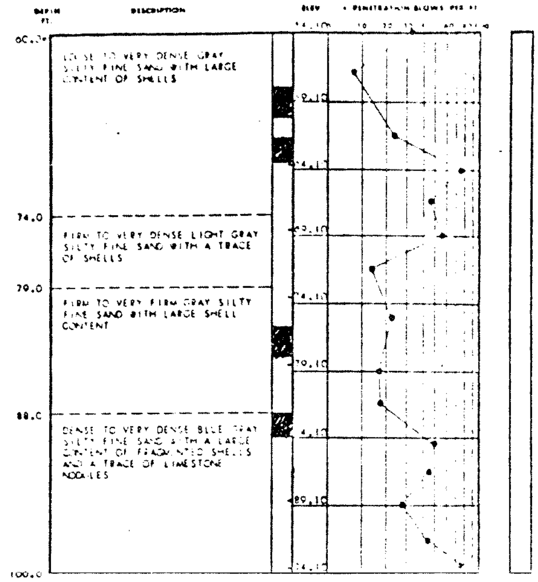
TEST BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

BORING NO. B-108
DATE DRILLED 12/4/68
JOB NO. J-1127

LAW ENGINEERING TESTING CO.



BORING TERMINATED

* BORING ADVANCED USING FISHTAIL DRILLING TOOLS FROM 0 TO 60 FEET

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

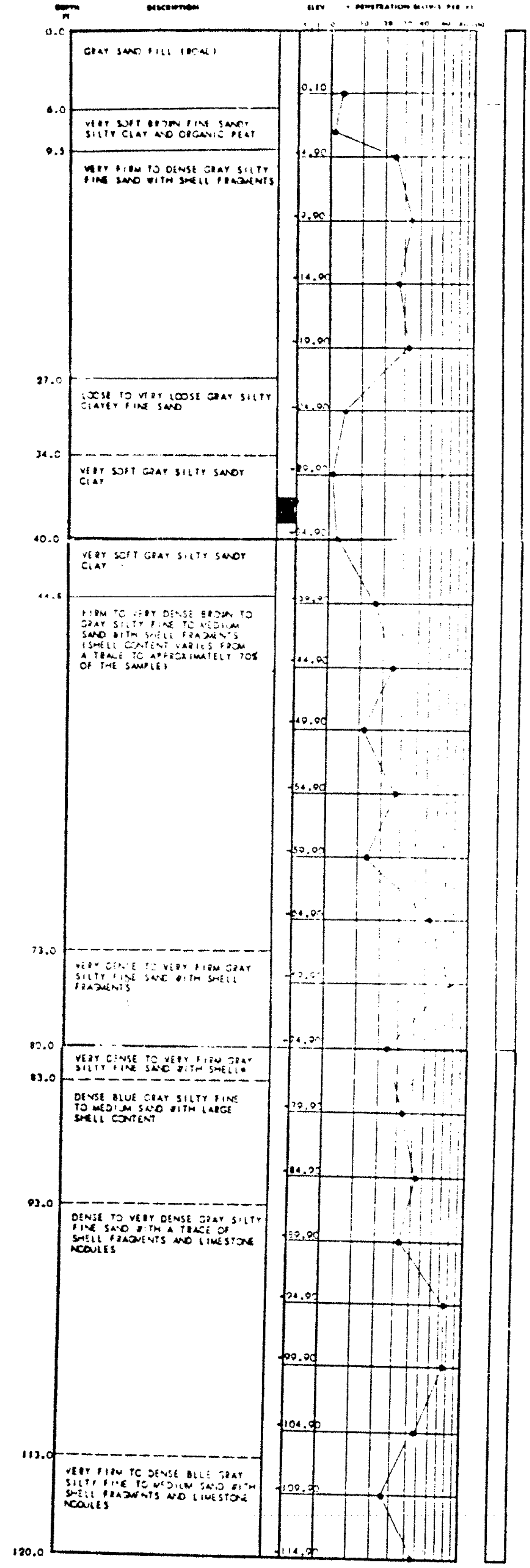
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

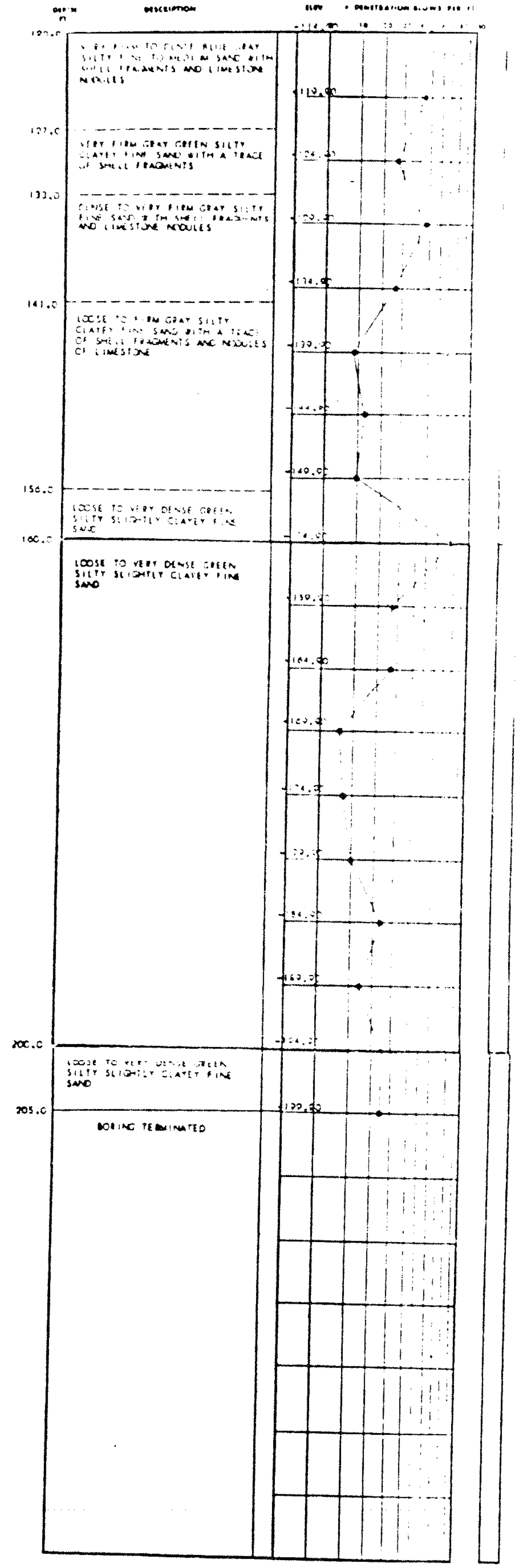
UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

TEST BORING RECORD

BORING NO. B-105A
DATE DRILLED 12/11/68
JOB NO. J-1127

LAW ENGINEERING TESTING CO.





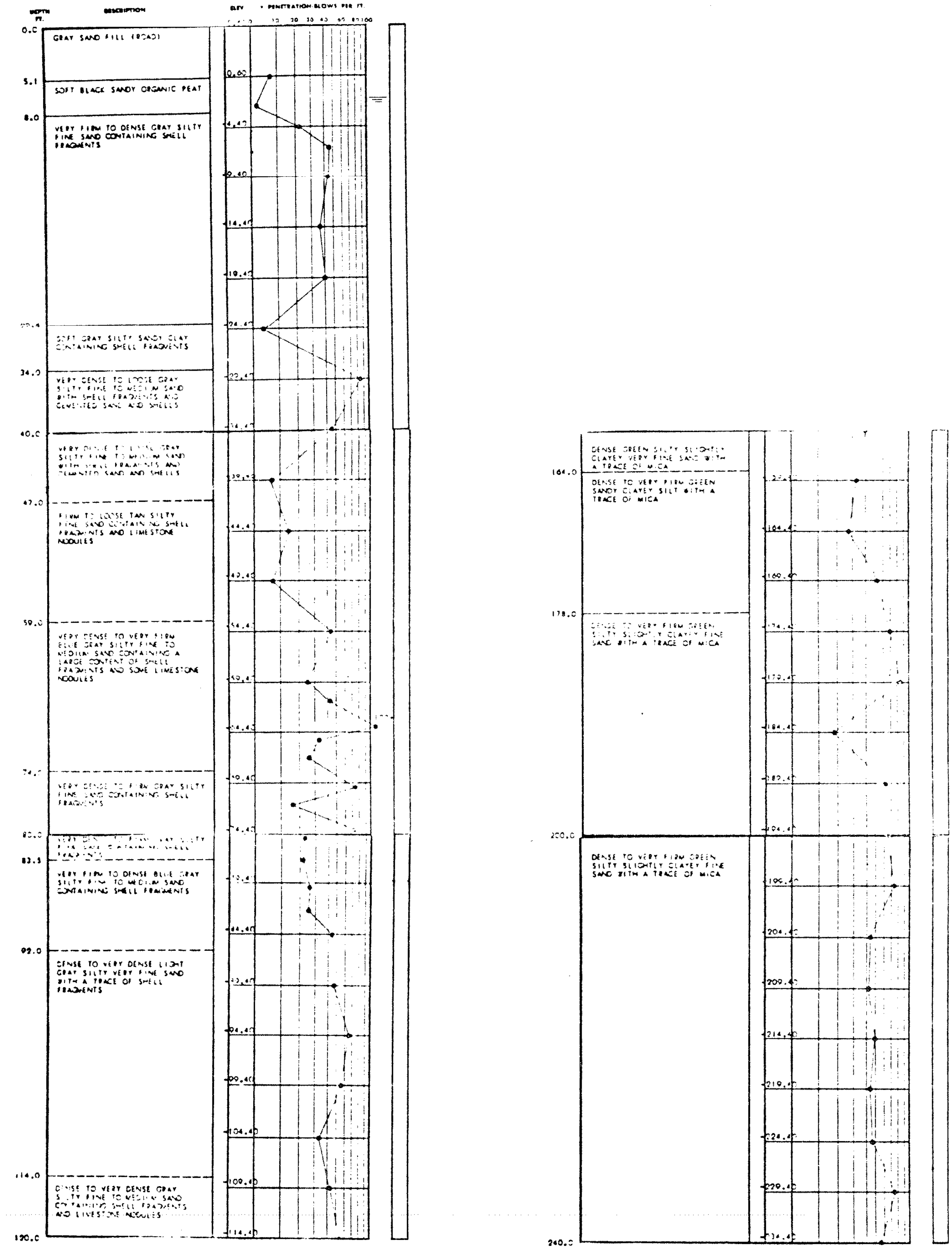
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

TEST BORING RECORD

BORING NO. B-109
DATE DRILLED 12/6/68
JOB NO. J-1127

LAW ENGINEERING TESTING CO.



TEST BORING RECORD

BORING NO. B-110

DATE DRILLED 11/26/69

JOB NO. J-1127

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

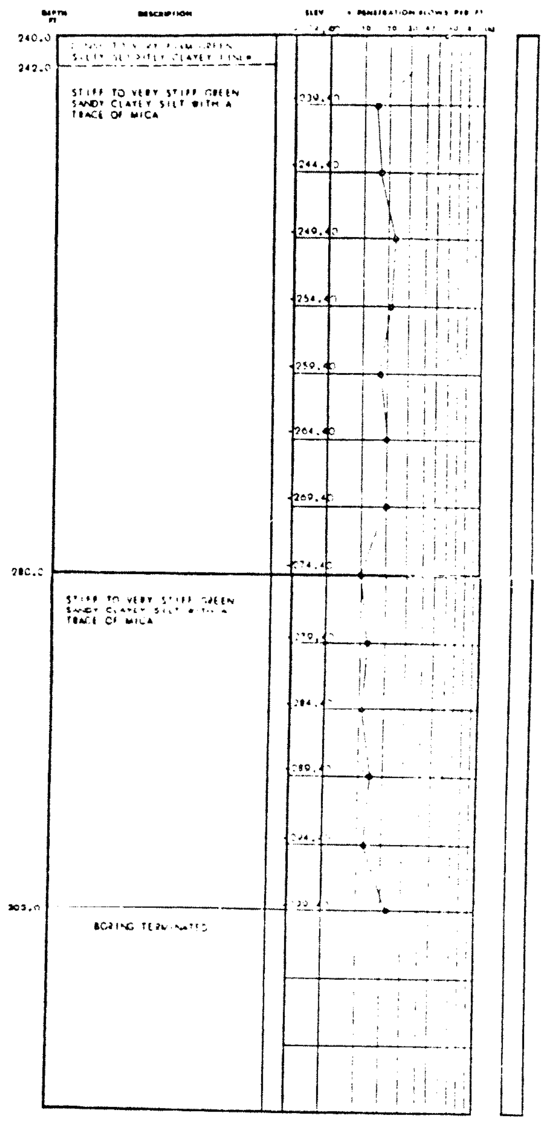
% ROCK CORE RECOVERY

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

LAW ENGINEERING TESTING CO.



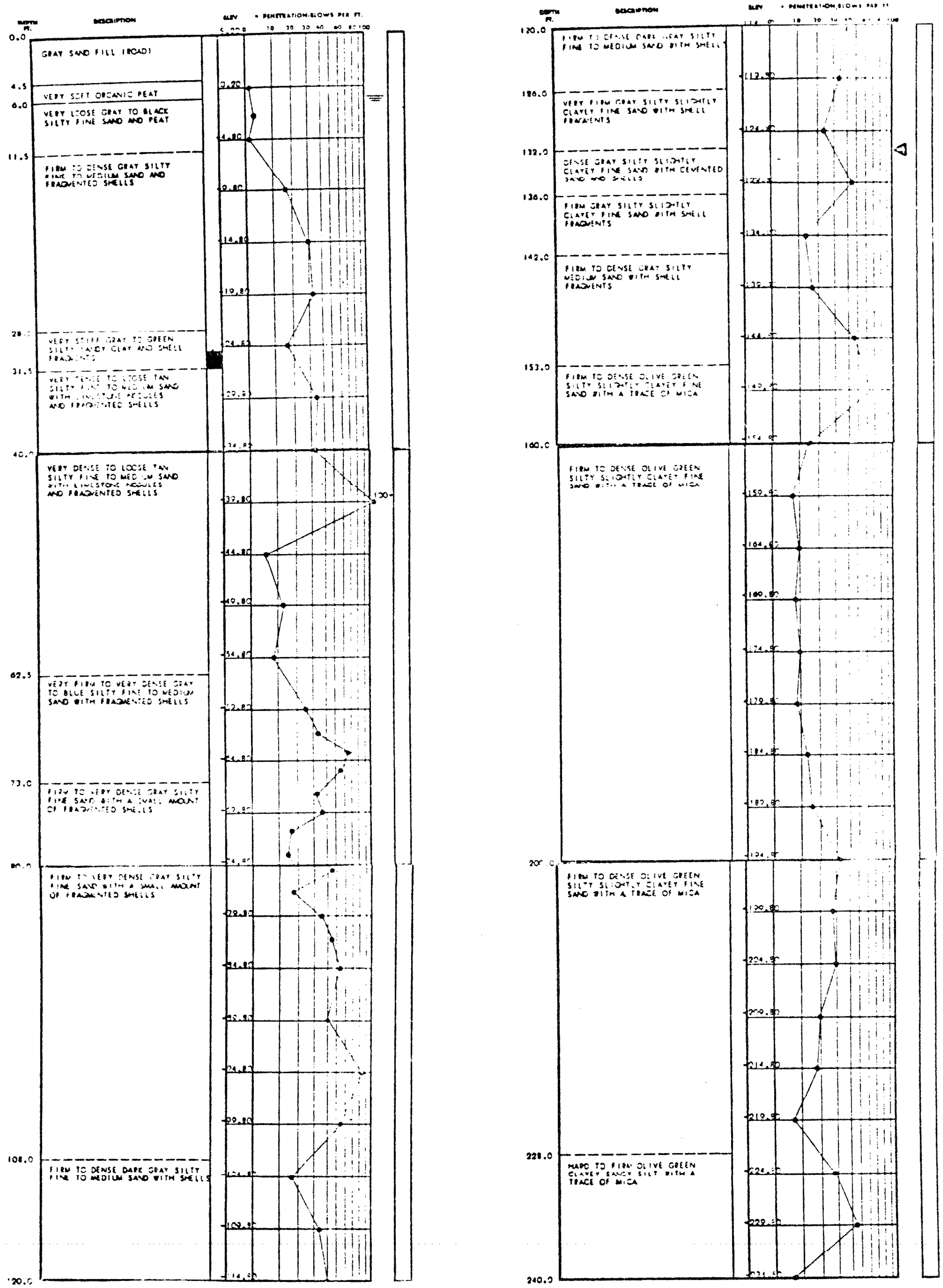
TEST BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER
FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

BORING NO. B-110
DATE DRILLED 11/26/82
JOB NO. 1-1127

LAW ENGINEERING TESTING CO.



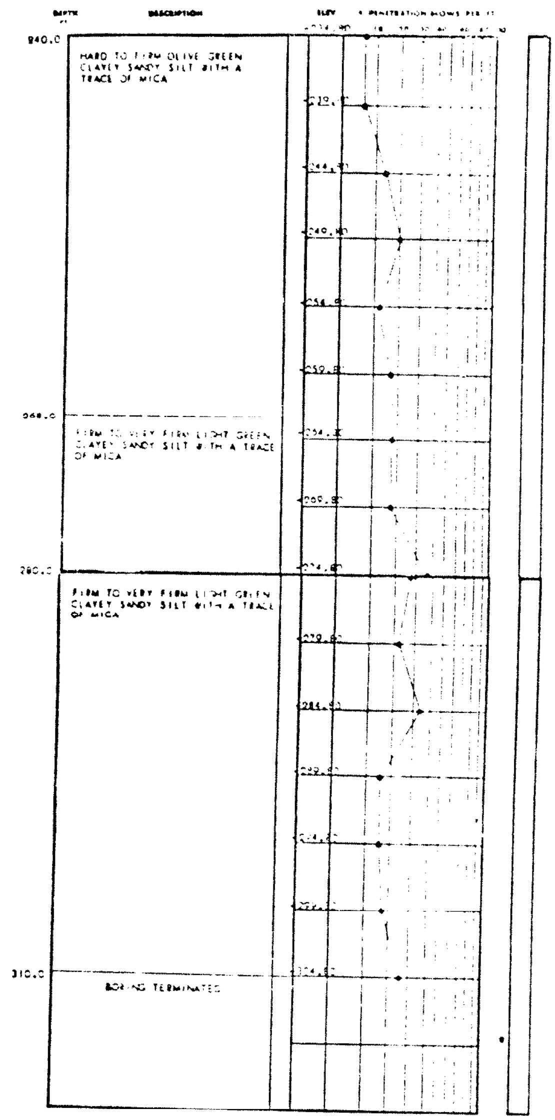
TEST BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

WATER TABLE, 24 HR.
UNDISTURBED SAMPLE
WATER TABLE, 1 HR.
% ROCK CORE RECOVERY
LOSS OF DRILLING WATER

BORING NO. B-111
DATE DRILLED 11/26/68
JOB NO. 1-1127

LAW ENGINEERING TESTING CO.



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

% ROCK CORE RECOVERY

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

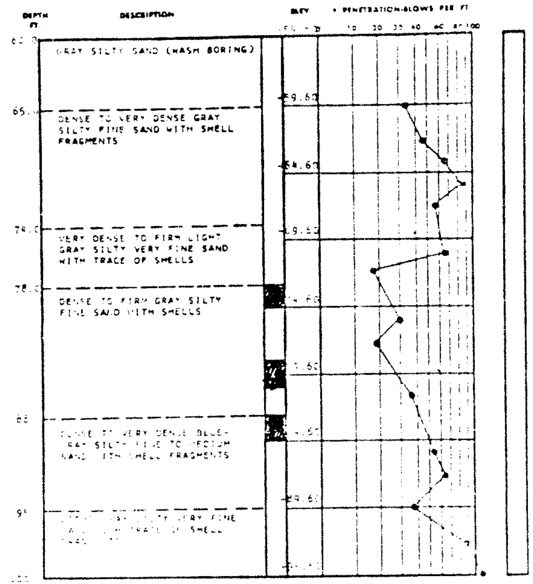
TEST BORING RECORD

BORING NO. B-111

DATE DRILLED 11/26/68

JOB NO. 1-1127

LAW ENGINEERING TESTING CO.



BORING ADVANCED USING FISHTAIL DRILLING TOOLS FROM 0 TO 60 FEET.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

% ROCK CORE RECOVERY

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

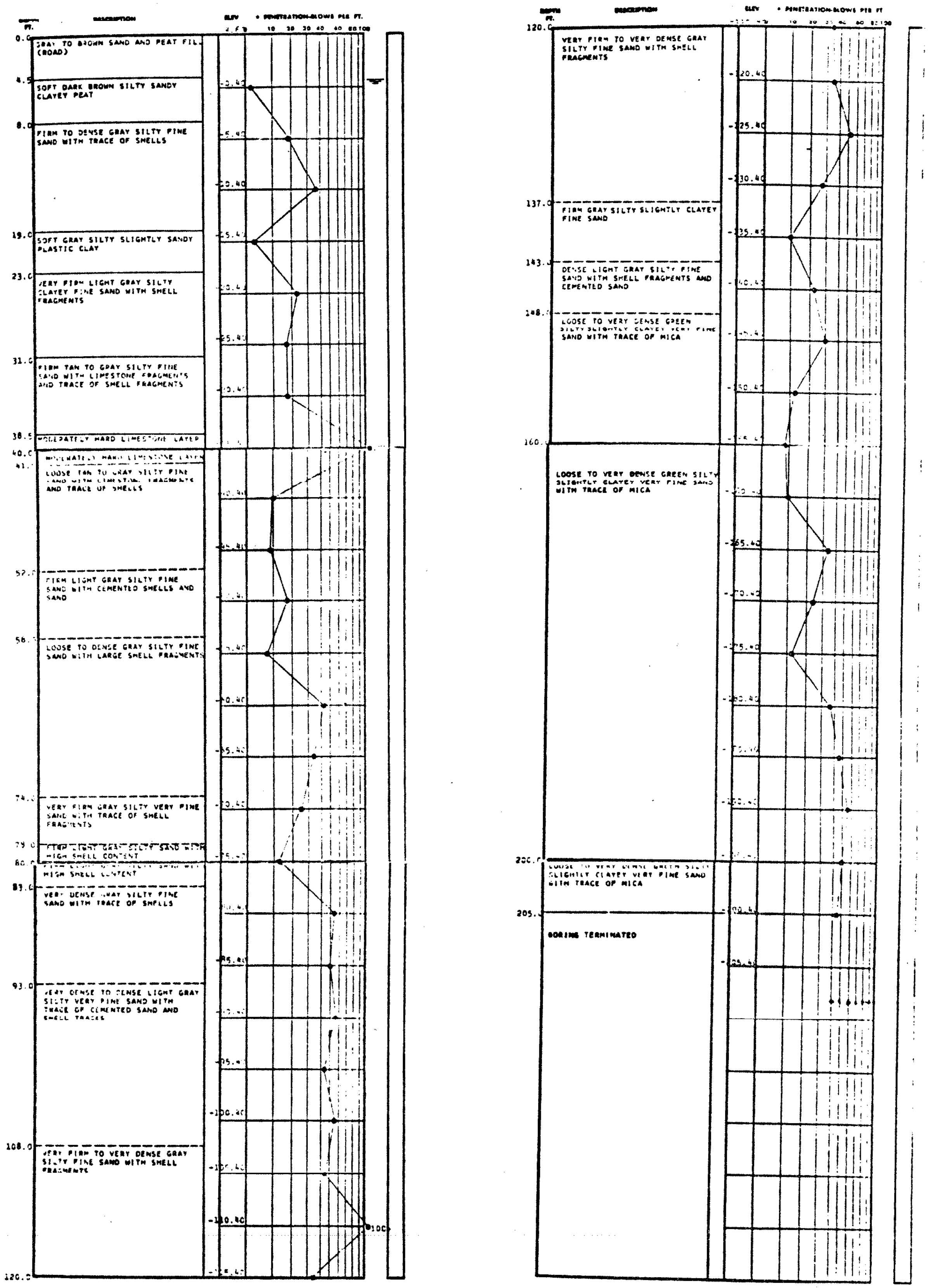
TEST BORING RECORD

BORING NO. B-111A

DATE DRILLED 1/22/69

JOB NO. J-1127

LAW ENGINEERING TESTING CO.



TEST BORING RECORD

BORING NO. B-112

DATE DRILLED 1/16/69

JOB NO. J-1127

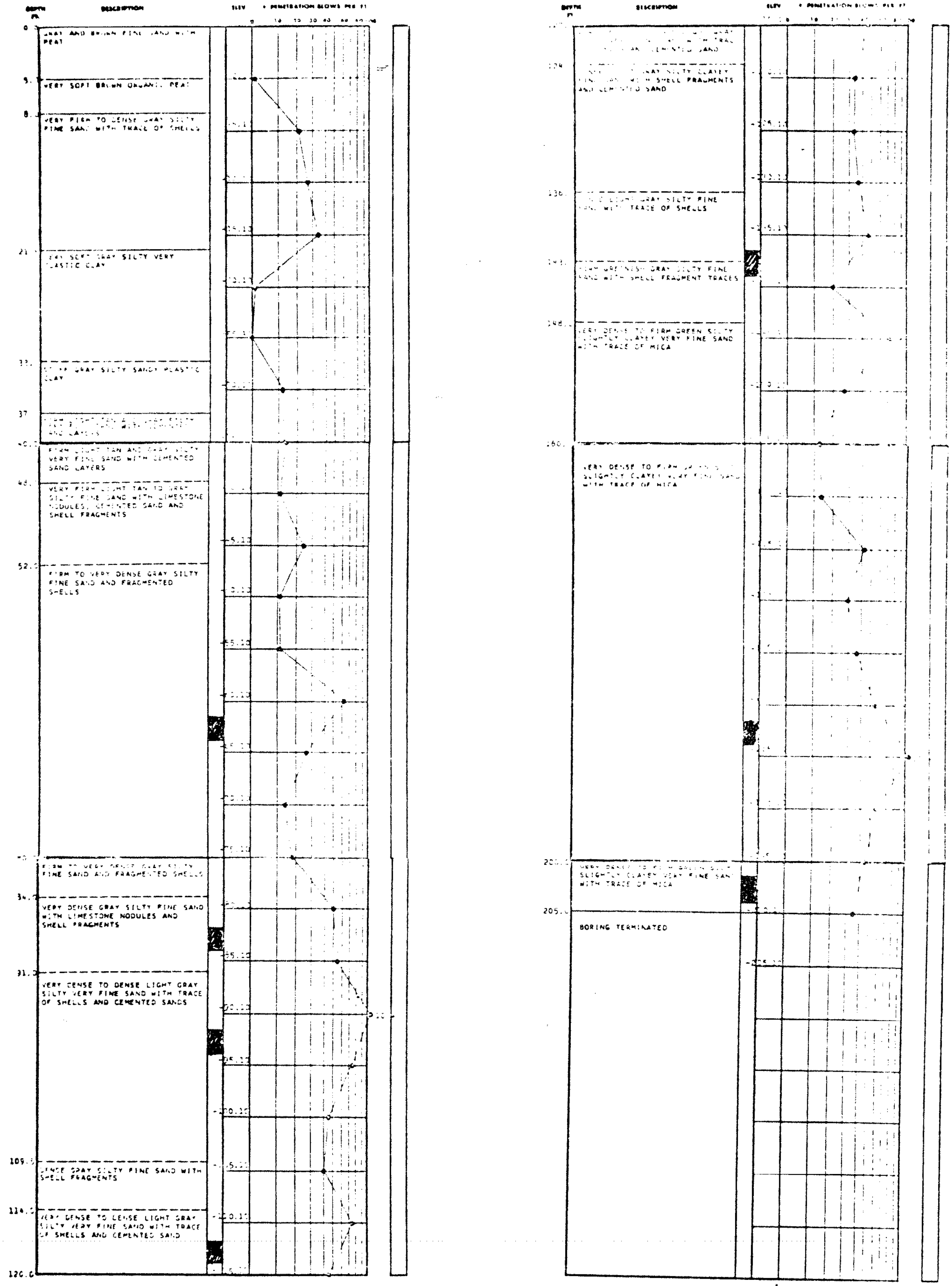
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE — WATER TABLE, 24 HR.

% ROCK CORE RECOVERY — WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

LAW ENGINEERING TESTING CO.



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

% ROCK CORE RECOVERY

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

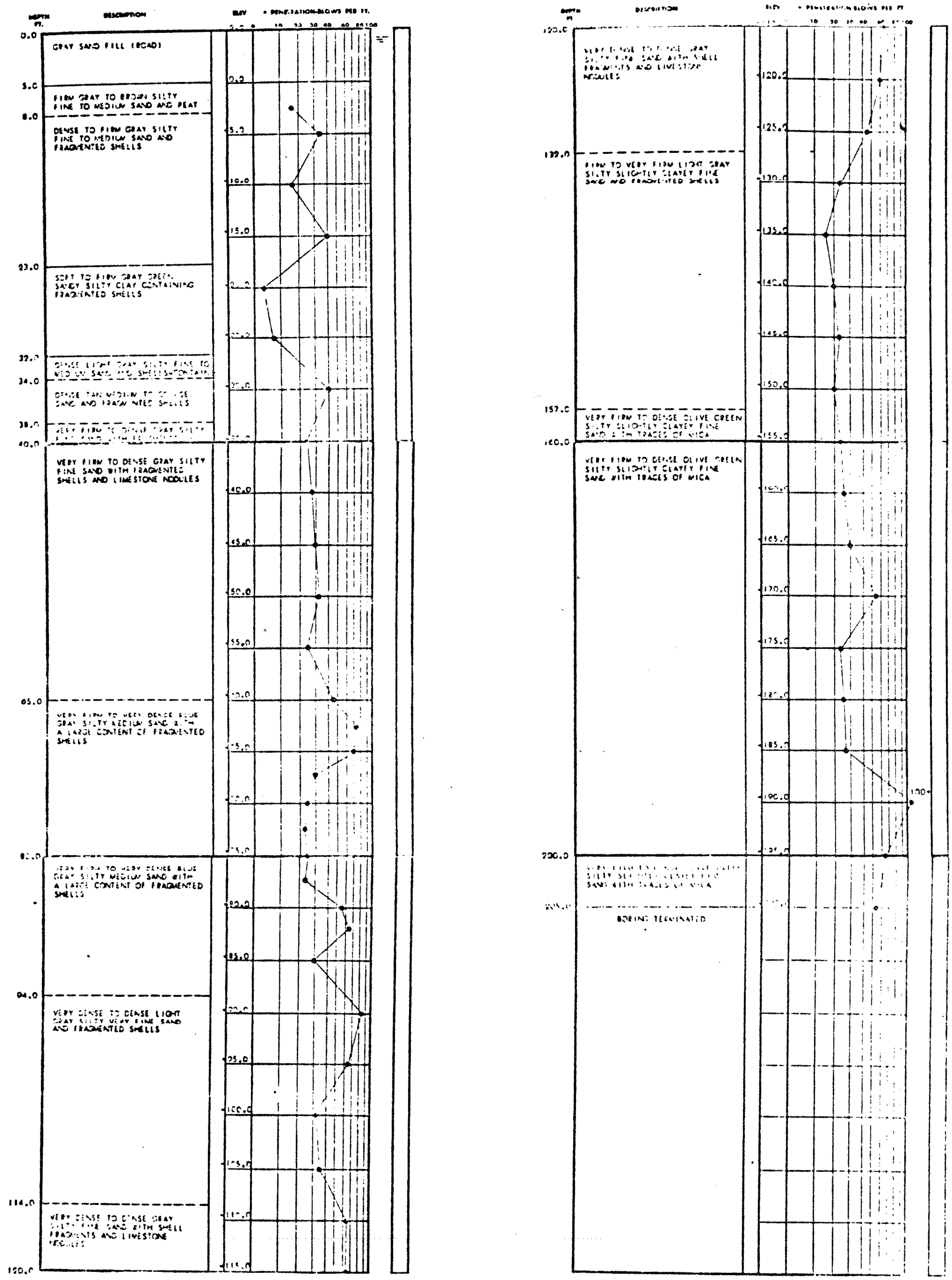
TEST BORING RECORD

BORING NO. B-113

DATE DRILLED 1/11/69

JOB NO. J-1127

LAW ENGINEERING TESTING CO.



*NODULES OF CEMENTED SAND AND SHELLS
**SHELLS AND LIMESTONE NODULES

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

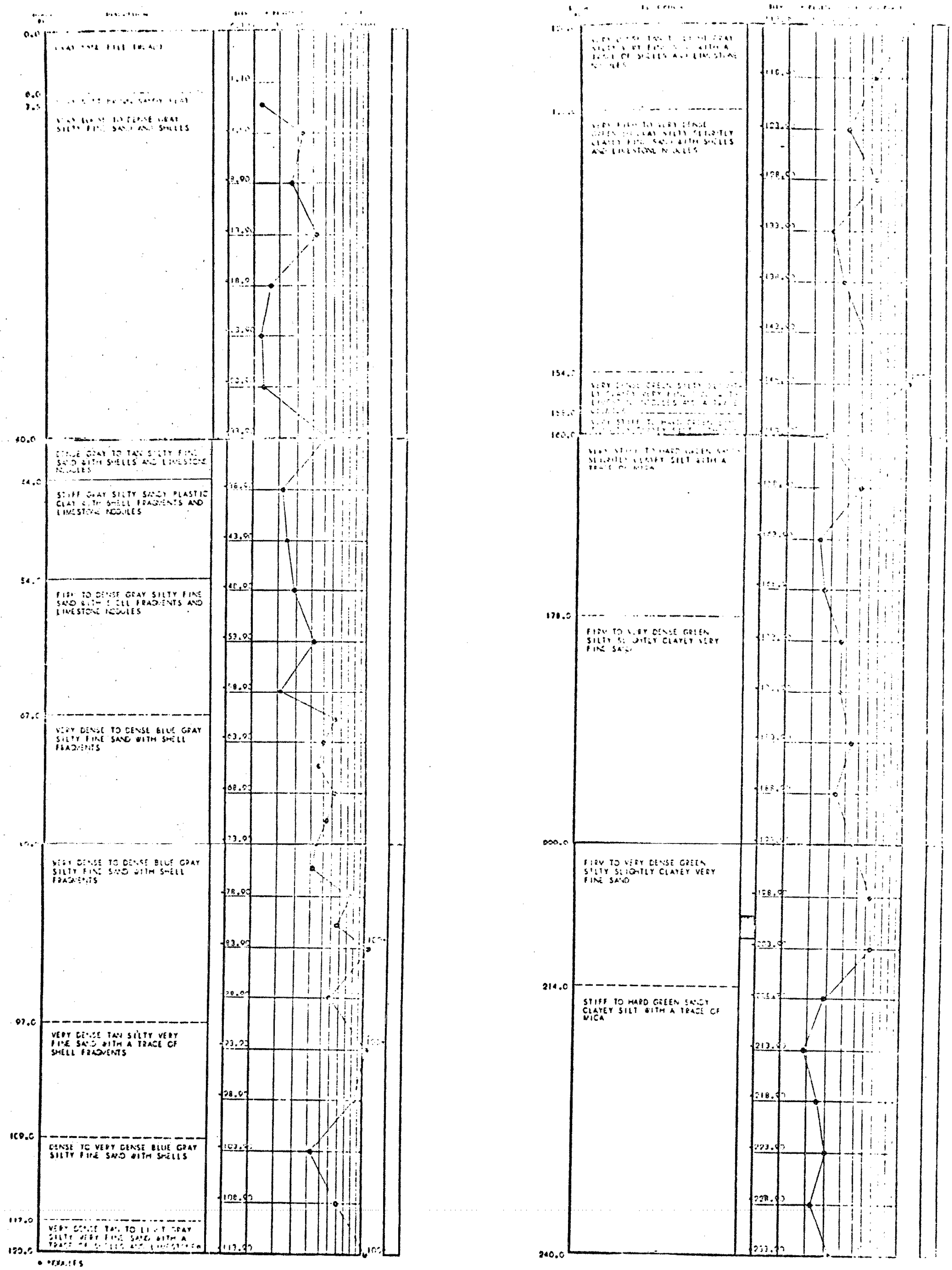
UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

2A-44

TEST BORING RECORD

BORING NO. B-114
DATE DRILLED 11/27/68
JOB NO. J-1127

LAW ENGINEERING TESTING CO.



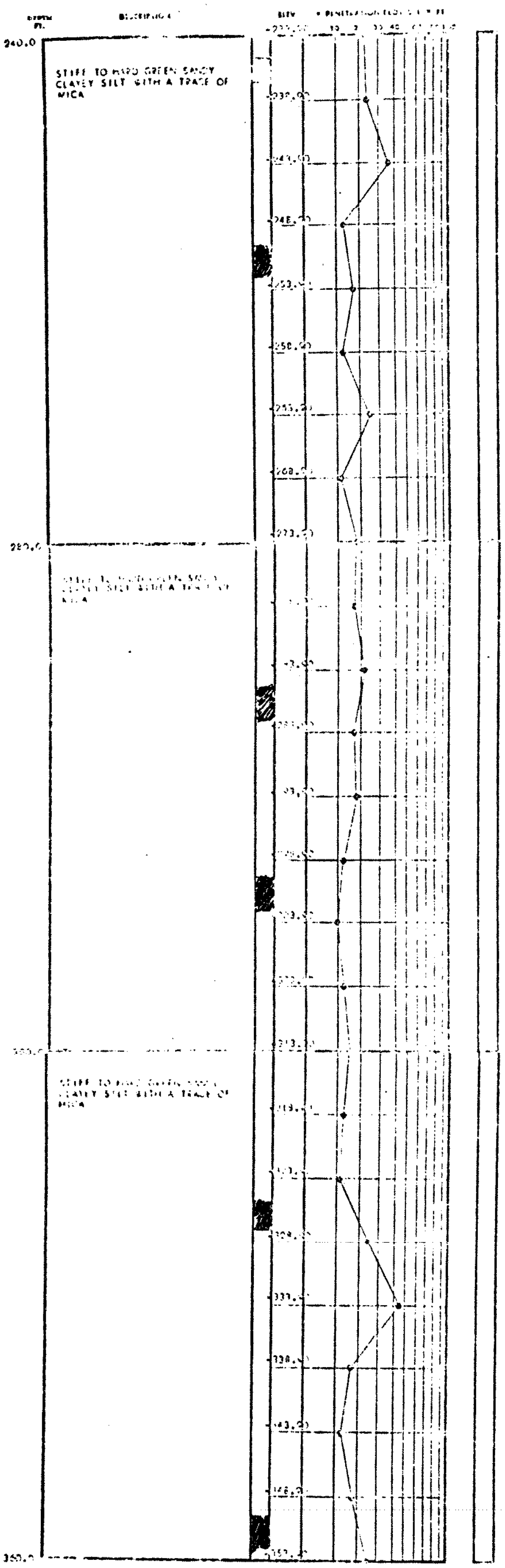
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

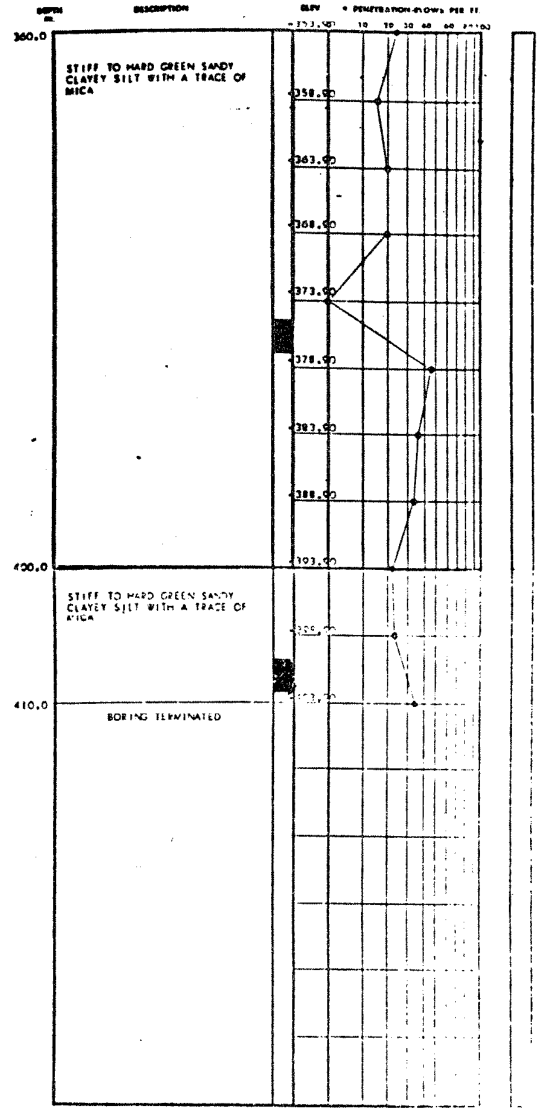
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
LOSS OF DRILLING WATER

TEST BORING RECORD

BORING NO. B-114A
DATE DRILLED 12/2/68
JOB NO. J-1127

LAW ENGINEERING TESTING CO.





TEST BORING RECORD

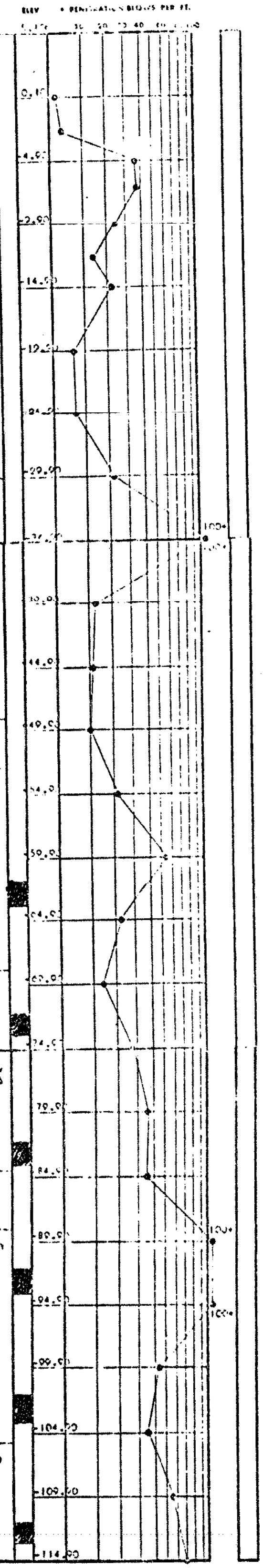
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

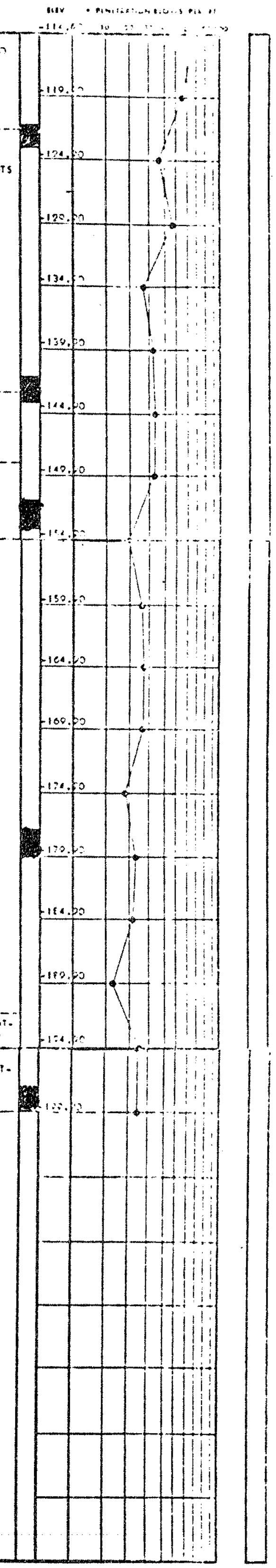
BORING NO. B-114A
DATE DRILLED 12/2/68
JOB NO. J-1127

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION
0.0	GRAY TO BROWN SAND AND PEAT FILL (ROAD)
5.0	VERY SOFT TO SOFT BROWN SANDY ORGANIC PEAT
8.0	DENSE BROWN TO GRAY SILTY FINE SAND WITH ORGANIC MATERIAL
10.5	FIRM TO VERY FIRM GRAY SILTY FINE SAND WITH FRAGMENTED SHELLS
24.5	FIRM GRAY SILTY SLIGHTLY SANDY VERY PLASTIC CLAY
35.0	FIRM TO VERY DENSE GRAY TO TAN SILTY FINE SAND WITH LIMESTONE NODULES AND TRACE OF SHELLS
40.0	FIRM TO VERY DENSE GRAY TO TAN SILTY FINE SAND WITH LIMESTONE NODULES AND TRACE OF SHELLS
54.7	LOOSE GRAY SILTY SLIGHTLY CLAYEY FINE SAND WITH NODULES OF LIMESTONE
58.0	VERY FIRM TO VERY DENSE BLUE GRAY SILTY FINE SAND WITH LARGE SHELL CONTENT
74.0	FIRM TO DENSE LIGHT GRAY SILTY FINE SAND WITH SHELL FRAGMENTS
80.0	FIRM TO DENSE LIGHT GRAY SILTY FINE SAND WITH SHELL FRAGMENTS
89.5	DENSE BLUE GRAY SILTY FINE TO MEDIUM SAND WITH SHELL FRAGMENTS
94.0	VERY DENSE TO DENSE LIGHT GRAY SILTY VERY FINE SAND WITH A TRACE OF SHELLS
111.5	VERY DENSE GRAY SILTY FINE TO MEDIUM SAND WITH LIMESTONE NODULES AND SHELL FRAGMENTS
120.0	



DEPTH FT.	DESCRIPTION
120.0	VERY DENSE GRAY SILTY FINE TO MEDIUM SAND WITH LIMESTONE NODULES AND SHELL FRAGMENTS
128.0	DENSE TO VERY FIRM GREENISH GRAY SILTY SLIGHTLY CLAYEY FINE SAND WITH SHELL FRAGMENTS
148.0	DENSE LIGHT GRAY SILTY FINE SAND WITH TRACE OF SHELLS
154.0	DENSE TO FIRM GREEN SILTY SLIGHTLY CLAYEY FINE SAND WITH TRACE OF MICA
160.0	DENSE TO FIRM GREEN SILTY SLIGHTLY CLAYEY FINE SAND WITH TRACE OF MICA
197.0	VERY STIFF GREEN SANDY SLIGHTLY CLAYEY SILT WITH A TRACE
200.0	VERY STIFF GREEN SANDY SLIGHTLY CLAYEY SILT WITH A TRACE OF MICA
205.0	BORING TERMINATED



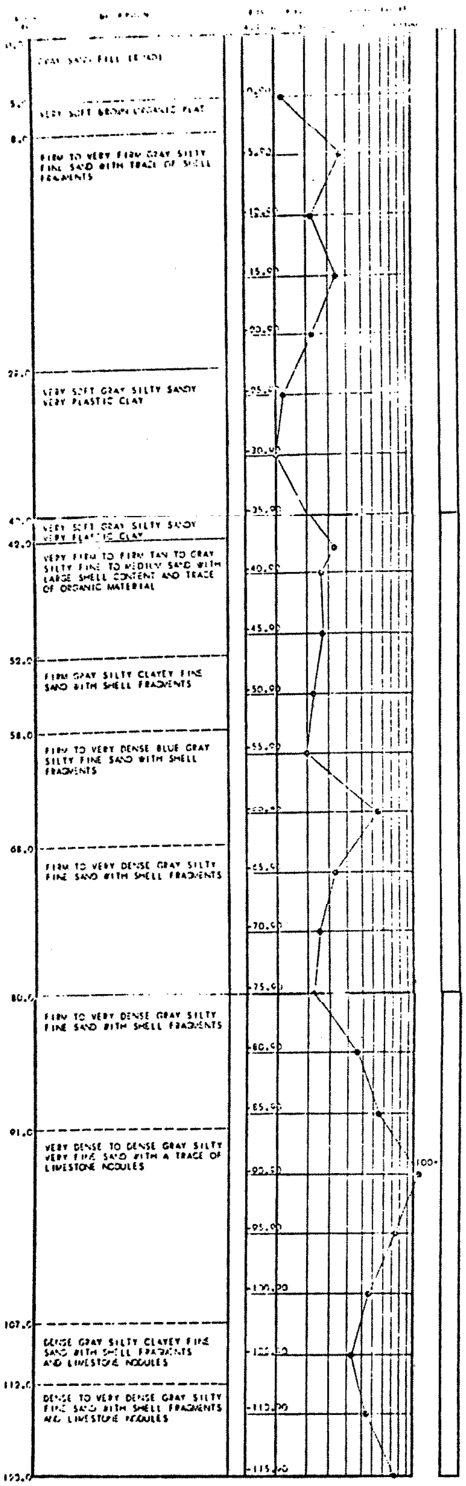
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

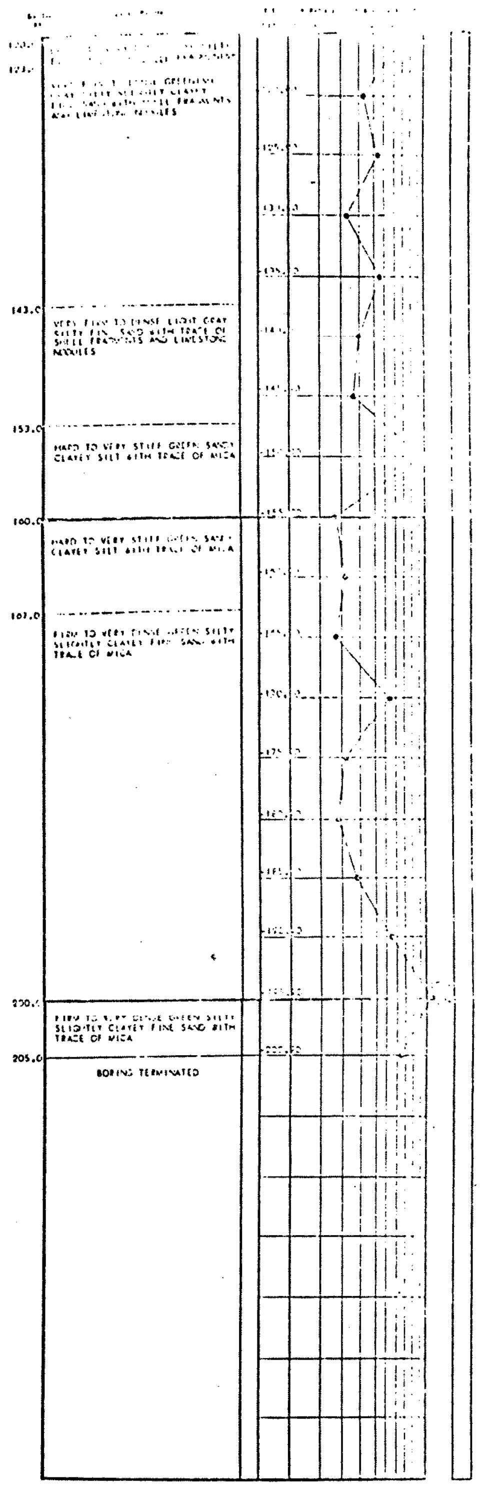
UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

TEST BORING RECORD

BORING NO. B-115
DATE DRILLED 1/7/69
JOB NO. 1-1127

LAW ENGINEERING TESTING CO.





TEST BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

BORING NO. B-115
DATE DRILLED 12/18/68
JOB NO. J-1127

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION
0.0	GRAY SAND FILL (ROAD)
7.0	SOFT BROWN ORGANIC PEAT
9.0	VERY FIRM TO LOOSE BLUE GRAY SILTY FINE SAND WITH SHELL FRAGMENTS
[illegible]	SOFT BLUE GRAY SILTY SLIGHTLY SANDY PLASTIC CLAY
34.0	VERY DENSE TO DENSE GRAY TO TAN SILTY FINE TO MEDIUM SAND WITH SHELL FRAGMENTS AND LIMESTONE NODULES
40.0	[illegible] GRAY SILTY FINE SAND WITH SHELL FRAGMENTS
55.0	FIRM GRAY SILTY FINE SAND WITH LARGE SHELL CONTENT
63.0	DENSE TO VERY FIRM BLUE GRAY SILTY FINE SAND WITH SHELL FRAGMENTS
[illegible]	LOOSE TO VERY DENSE GRAY SILTY FINE SAND WITH SHELL FRAGMENTS
[illegible]	[illegible] TO VERY DENSE GRAY SILTY FINE SAND WITH SHELL FRAGMENTS
92.0	VERY DENSE LIGHT GRAY SILTY VERY FINE SAND WITH LIMESTONE NODULES
107.0	VERY DENSE TO DENSE GRAY SILTY FINE SAND WITH SHELL FRAGMENTS
117.0	VERY DENSE TO DENSE LIGHT GREENISH GRAY SILTY VERY FINE SAND WITH SHELL FRAGMENTS
120.0	VERY DENSE TO DENSE LIGHT GREENISH GRAY SILTY VERY FINE SAND WITH SHELL FRAGMENTS
133.5	FIRM LIGHT GREENISH GRAY SILTY CLAYEY FINE SAND WITH SHELL FRAGMENTS
138.0	VERY FIRM TO DENSE GRAY SILTY FINE SAND WITH CEMENTED SANDS
148.0	LOOSE TO VERY DENSE GREEN SILTY SLIGHTLY CLAYEY VERY FINE SAND WITH TRACE OF MICA
160.0	LOOSE TO VERY DENSE GREEN SILTY SLIGHTLY CLAYEY VERY FINE SAND WITH TRACE OF MICA
200.0	LOOSE TO VERY DENSE GREEN SILTY SLIGHTLY CLAYEY VERY FINE SAND WITH TRACE OF MICA
205.0	BORING TERMINATED

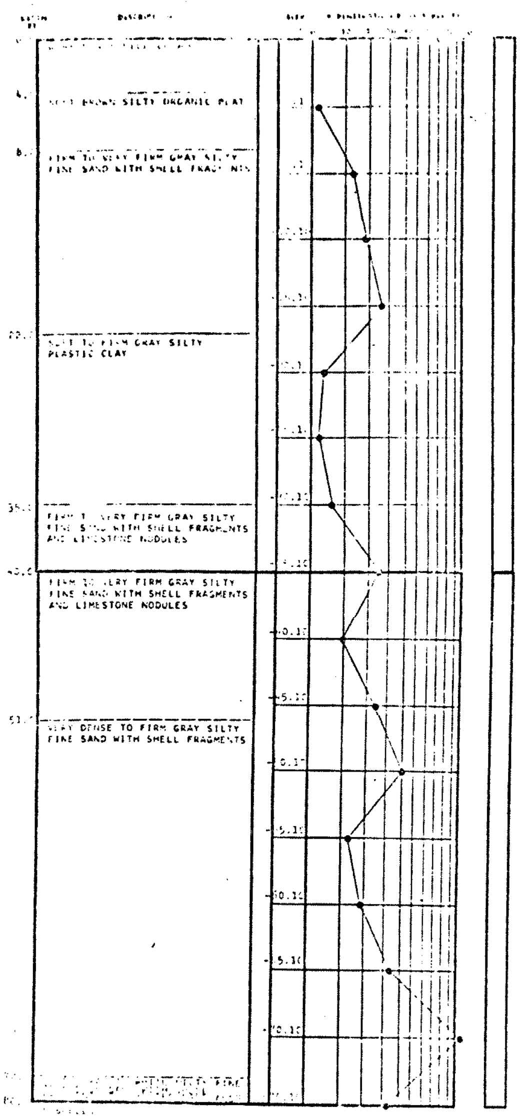
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

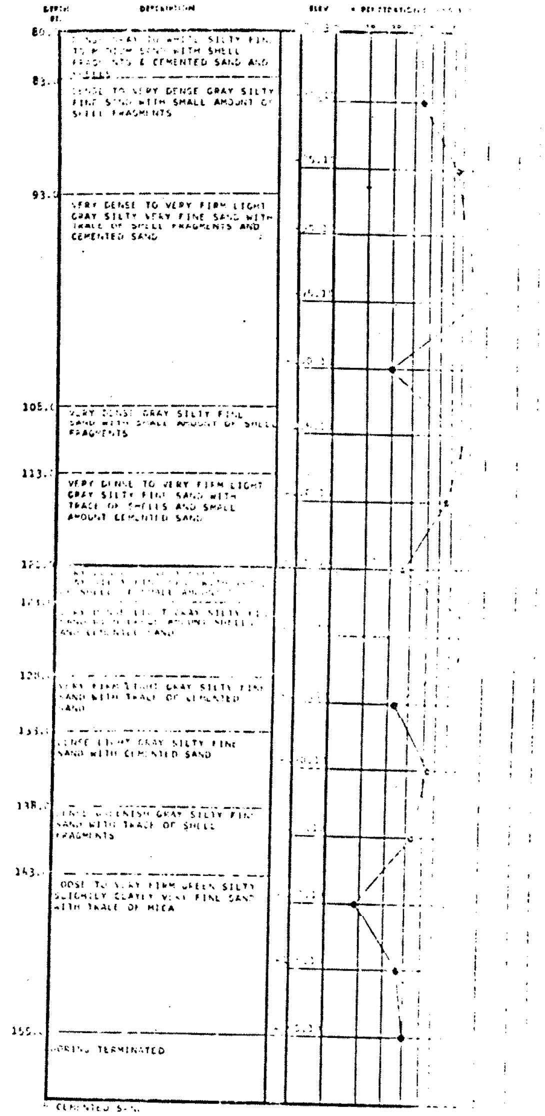
UNDISTURBED SAMPLE — WATER TABLE, 24 HR. — WATER TABLE, 1 HR.
% ROCK CORE RECOVERY — LOSS OF DRILLING WATER

TEST BORING RECORD

BORING NO. B-117
DATE DRILLED 1/08/69
JOB NO. J-1127

LAW ENGINEERING TESTING CO.





TEST BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

WATER TABLE, 24 HR.

UNDISTURBED SAMPLE

WATER TABLE, 1 HR.

% ROCK CORE RECOVERY

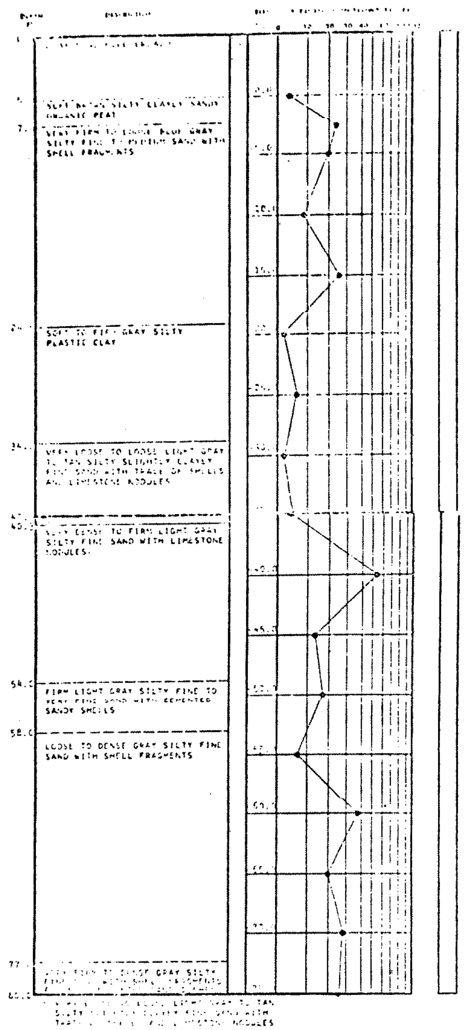
LOSS OF DRILLING WATER

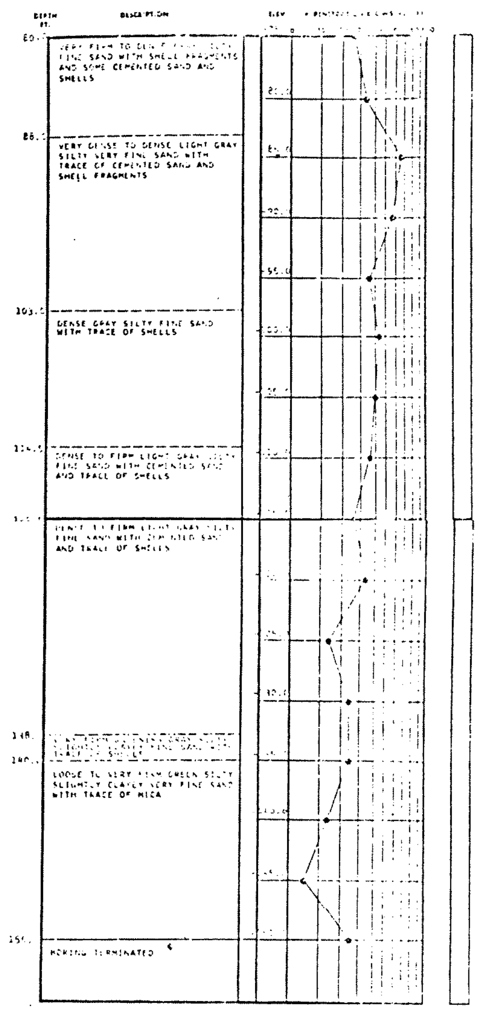
BORING NO. B-118

DATE DRILLED 1/18/69

JOB NO. J-1127

LAW ENGINEERING TESTING CO.





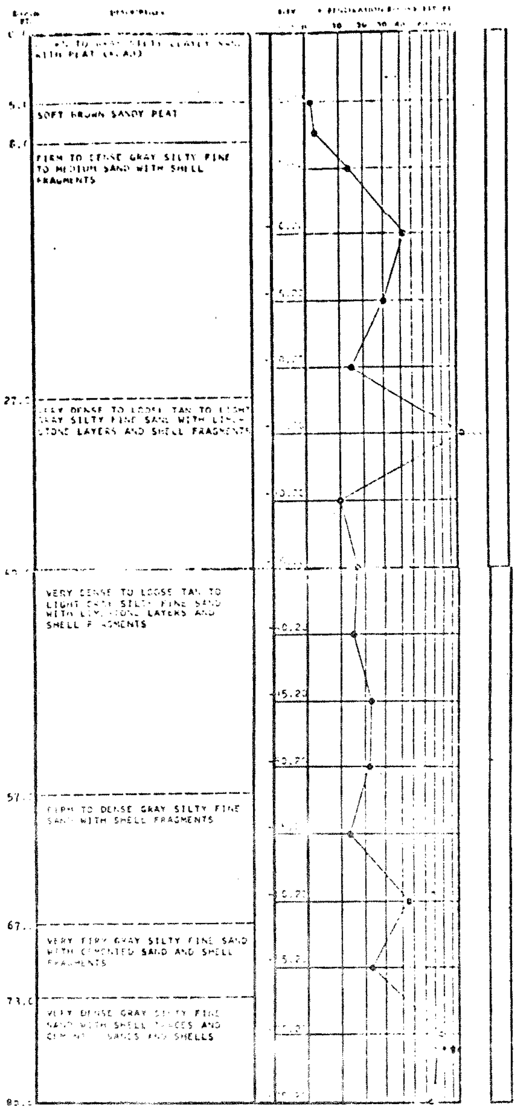
TEST BORING RECORD

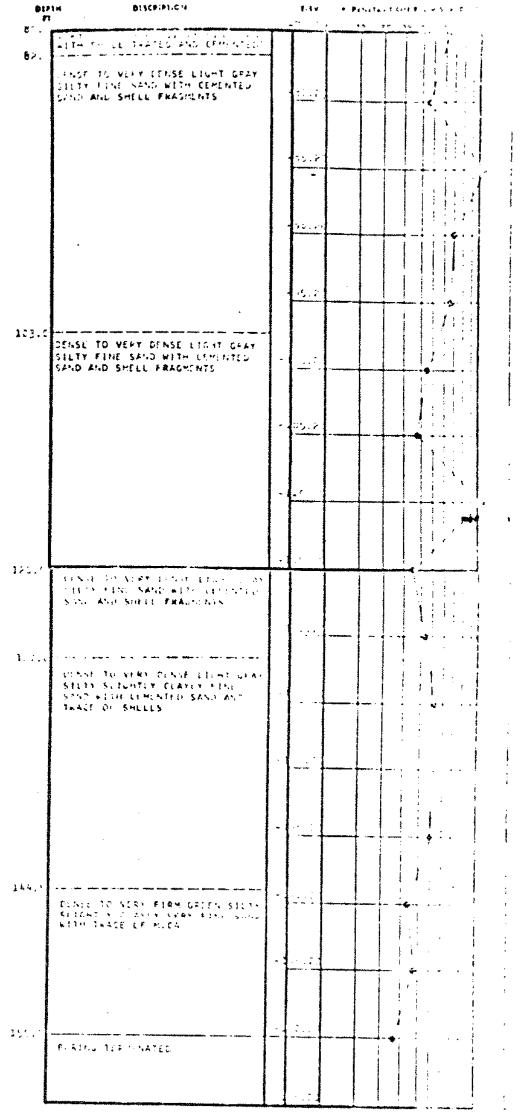
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

BORING NO. B-119
DATE DRILLED 1/18/69
JOB NO. J-1127

LAW ENGINEERING TESTING CO.





BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

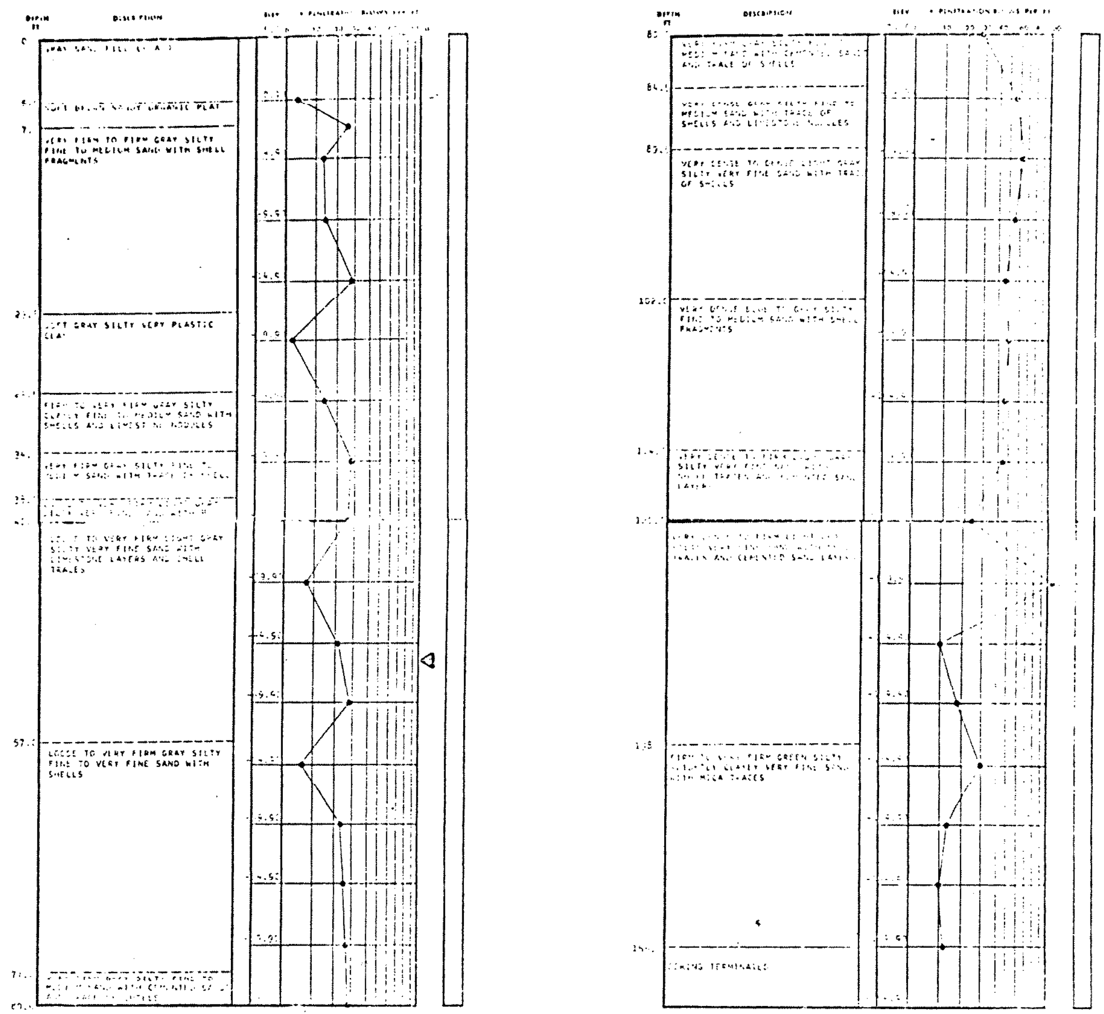
UNDISTURBED SAMPLE — WATER TABLE, 24 HR. — WATER TABLE, 1 HR.

% ROCK CORE RECOVERY — LOSS OF DRILLING WATER

TEST BORING RECORD

BORING NO. B-120
DATE DRILLED 1/18/69
JOB NO. J-1127

LAW ENGINEERING TESTING CO.



⁖ LIMESTONE LAYERS AND SHELL TRACES

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

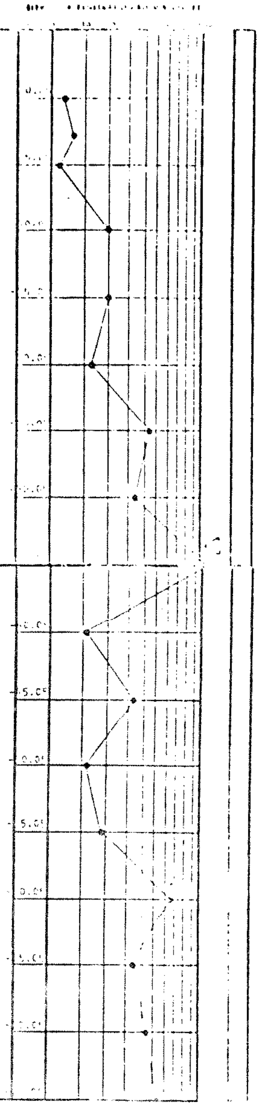
UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

TEST BORING RECORD

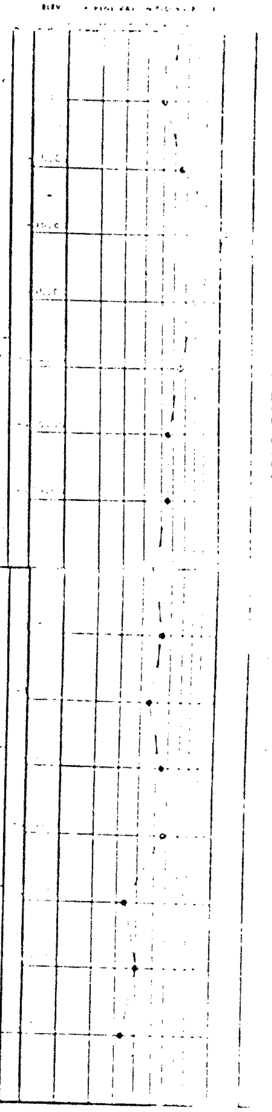
BORING NO. B-121
DATE DRILLED 1/16/69
JOB NO. J-1127

LAW ENGINEERING TESTING CO.

DEPTH FT	DESCRIPTION
0	[illegible] PEAT [illegible]
4.[illegible]	[illegible] SANDY ORGANIC PEAT
8.	VERY LOOSE GRAY TO BROWN SILTY CLAYEY FINE SAND WITH SHELLS AND PEAT
13.	FIRM GRAY SILTY FINE SAND WITH FRAGMENTED SHELLS
22.[illegible]	STIFF GRAY SILTY SLIGHTLY SANDY PLASTIC CLAY WITH TRACE OF SHELL FRAGMENTS
26.	DENSE LIGHT GRAY SILTY FINE TO MEDIUM SAND WITH CEMENTED SAND AND SHELL FRAGMENT TRACES
3[illegible].	VERY DENSE GRAY SILTY FINE SAND WITH SHELL FRAGMENTS AND [illegible]
[illegible]	VERY DENSE GRAY SILTY FINE SAND WITH SHELL FRAGMENTS AND CEMENTED SAND
43.[illegible]	FIRM TO DENSE TAN SILTY FINE TO COARSE SAND WITH SHELL TRACES AND LIMESTONE LAYERS
57.[illegible]	FIRM TO VERY DENSE GRAY SILTY FINE SAND WITH FRAGMENTED SHELLS



DEPTH FT	DESCRIPTION
[illegible]	FIRM TO [illegible] GRAY SILTY FINE SAND WITH FRAGMENTED [illegible]
[illegible]3.	DENSE TO VERY DENSE LIGHT GRAY SILTY VERY FINE SAND WITH TRACES OF SHELL AND CEMENTED SAND AND LIMESTONE NODULES
103.[illegible]	VERY DENSE TO DENSE LIGHT GRAY SILTY FINE SAND WITH SHELLS AND LIMESTONE NODULES
[illegible]	DENSE GREENISH GRAY SILTY FINE SAND WITH SHELLS
[illegible]	DENSE GREENISH GRAY SILTY FINE SAND WITH SHELLS
[illegible]	DENSE LIGHT GRAY SILTY FINE SAND WITH SHELLS AND LIMESTONE NODULES
[illegible]	VERY FIRM GRAY SILTY SLIGHTLY CLAYEY FINE SAND WITH TRACE OF MICA
[illegible]	BORING TERMINATED



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

% ROCK CORE RECOVERY

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

2A-54

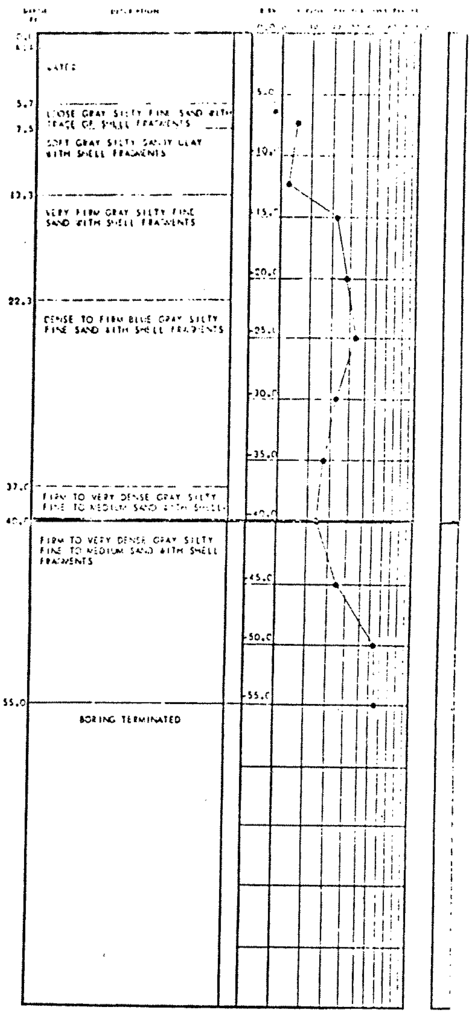
TEST BORING RECORD

BORING NO. B-122

DATE DRILLED 1/15/69

JOB NO. J-1127

LAW ENGINEERING TESTING CO.



FRAGMENTS

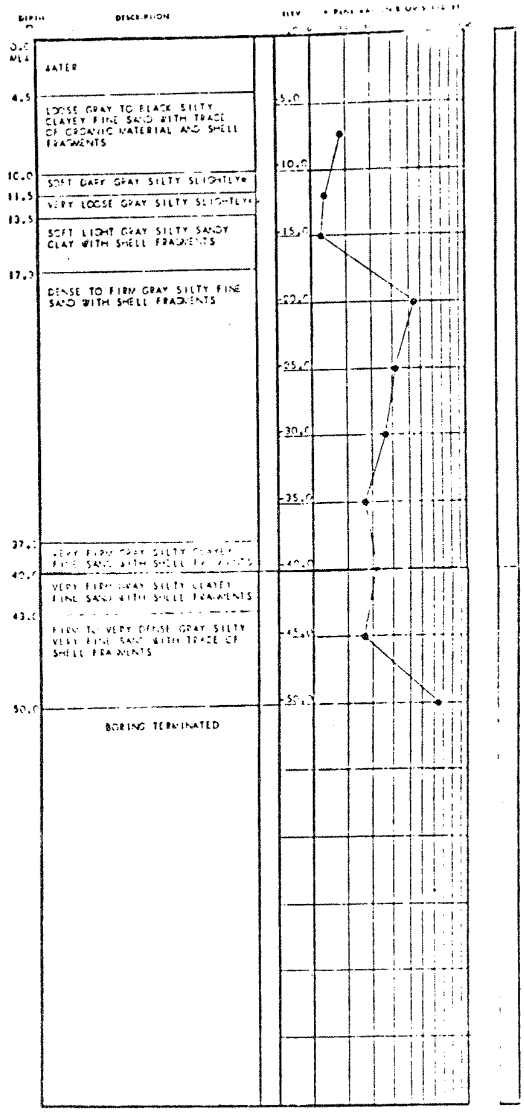
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

TEST BORING RECORD

BORING NO. B-123
DATE DRILLED 1/11/69
JOB NO. J-1127

LAW ENGINEERING TESTING CO.



* SANDY PLASTIC CLAY WITH TRACE OF SHELL FRAGMENTS

** CLAYEY FINE SAND WITH SHELLS

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE — WATER TABLE, 24 HR.

% ROCK CORE RECOVERY — WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

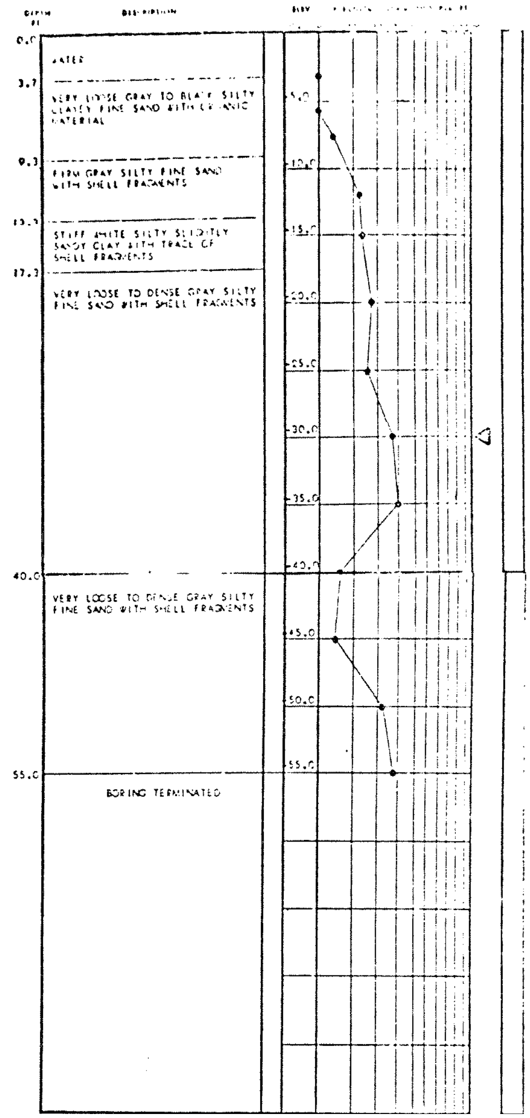
TEST BORING RECORD

BORING NO. B-124

DATE DRILLED 1/10/69

JOB NO. 1-1127

LAW ENGINEERING TESTING CO.



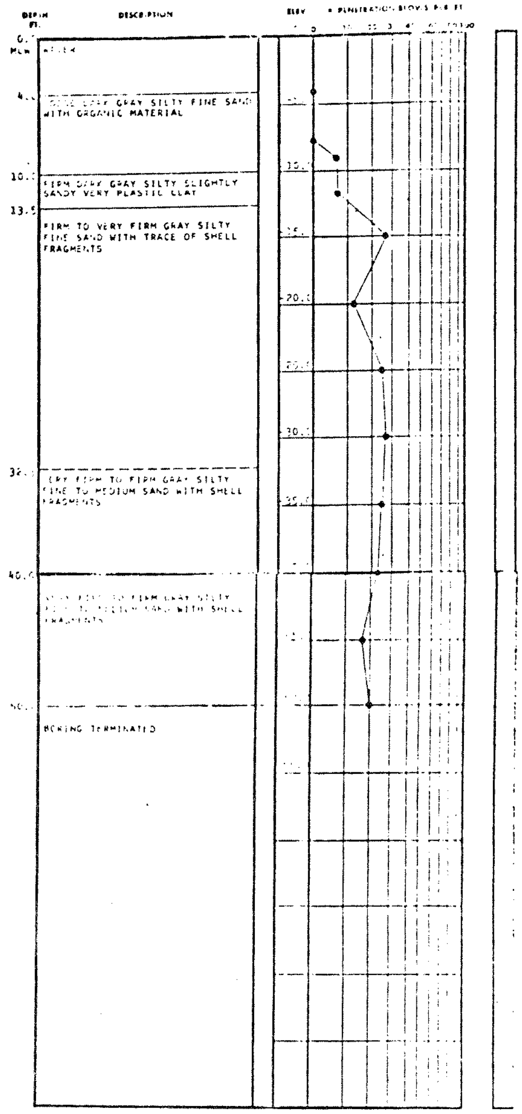
TEST BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

WATER TABLE, 24 HR.
UNDISTURBED SAMPLE
WATER TABLE, 1 HR.
% ROCK CORE RECOVERY
LOSS OF DRILLING WATER

BORING NO. B-125
DATE DRILLED 1/9/69
JOB NO. J-1127

LAW ENGINEERING TESTING CO.



TEST BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

% ROCK CORE RECOVERY

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

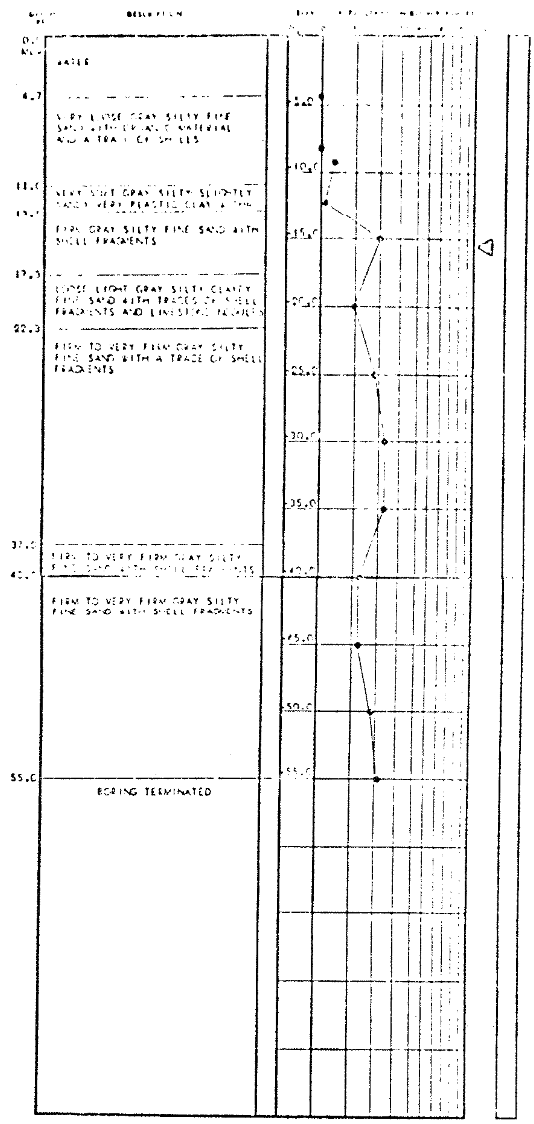
LOSS OF DRILLING WATER

BORING NO. B-126

DATE DRILLED 1/14/69

JOB NO. J-1127

LAW ENGINEERING TESTING CO.



* A TRACE OF SHELLS

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

% ROCK CORE RECOVERY

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

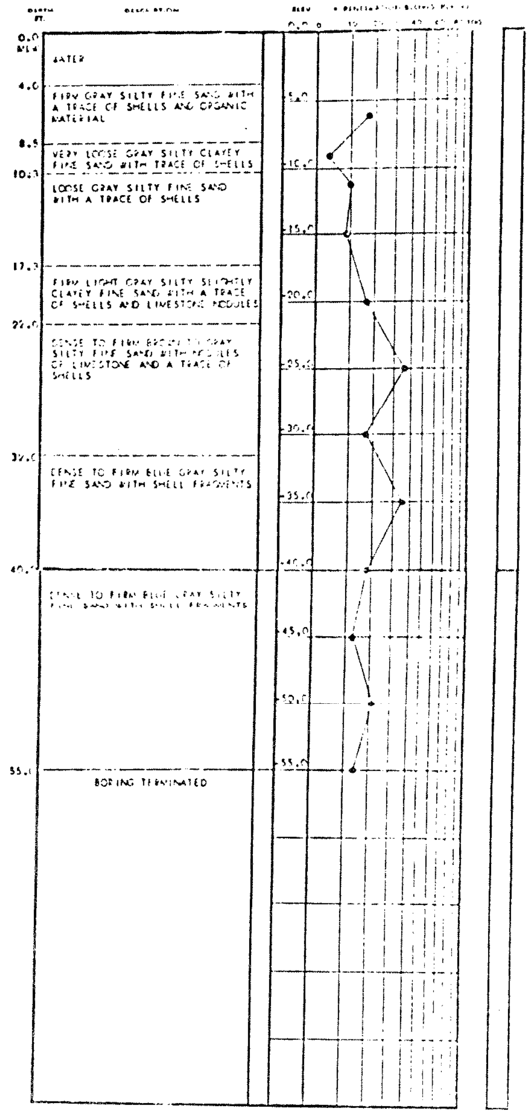
TEST BORING RECORD

BORING NO. B-127

DATE DRILLED 1/7/69

JOB NO. J-1127

LAW ENGINEERING TESTING CO.



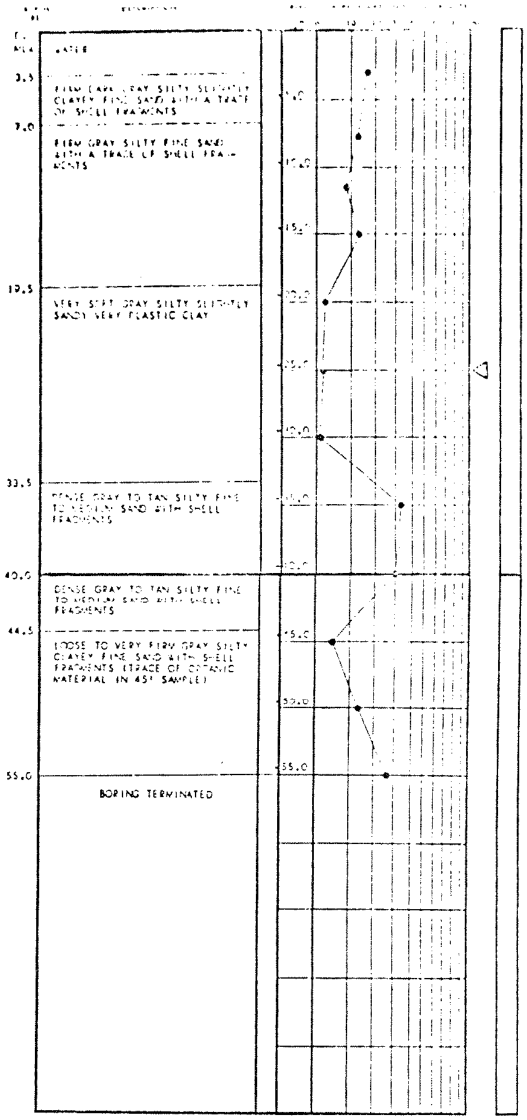
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

TEST BORING RECORD

BORING NO. B-128
DATE DRILLED 1/3/69
JOB NO. J-1127

LAW ENGINEERING TESTING CO.



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE — WATER TABLE, 24 HR.

% ROCK CORE RECOVERY — WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

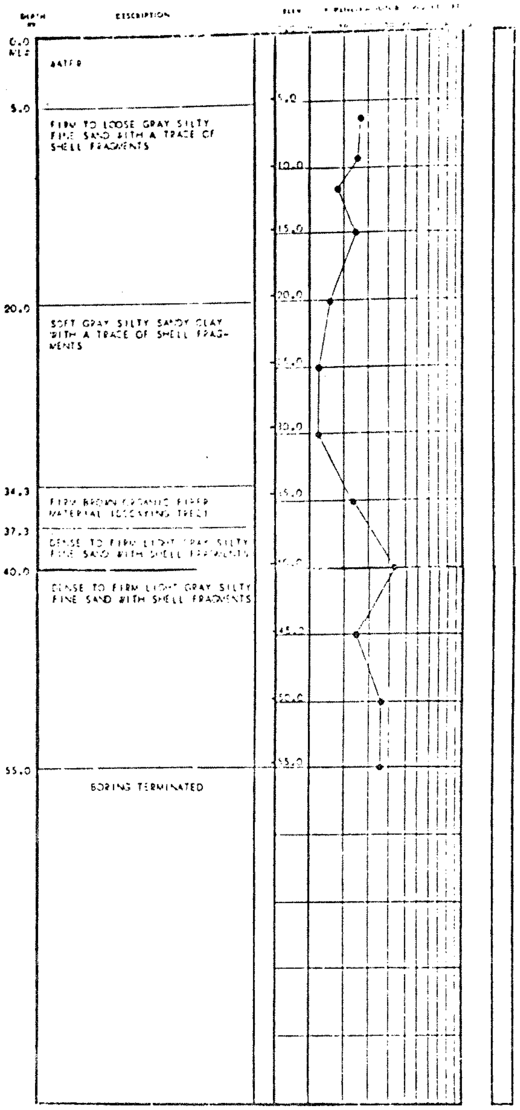
TEST BORING RECORD

BORING NO. B-129

DATE DRILLED 12/18/68

JOB NO. 1-1127

LAW ENGINEERING TESTING CO.



TEST BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE — WATER TABLE, 24 HR.

% ROCK CORE RECOVERY — WATER TABLE, 1 HR.

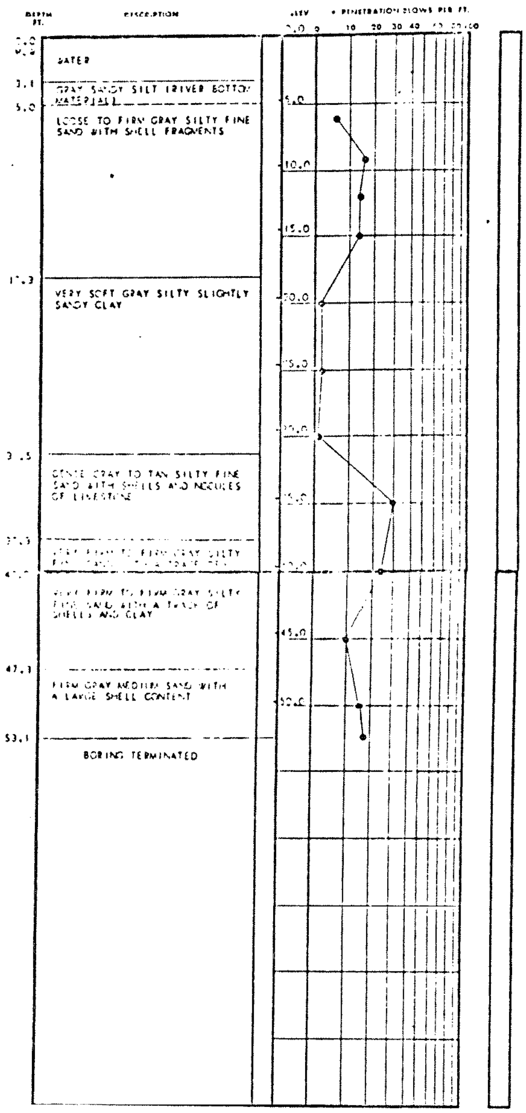
LOSS OF DRILLING WATER

BORING NO. R-130

DATE DRILLED 12/16/68

JOB NO. 1-1127

LAW ENGINEERING TESTING CO.



*SHELLS AND CLAY

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

% ROCK CORE RECOVERY

LOSS OF DRILLING WATER

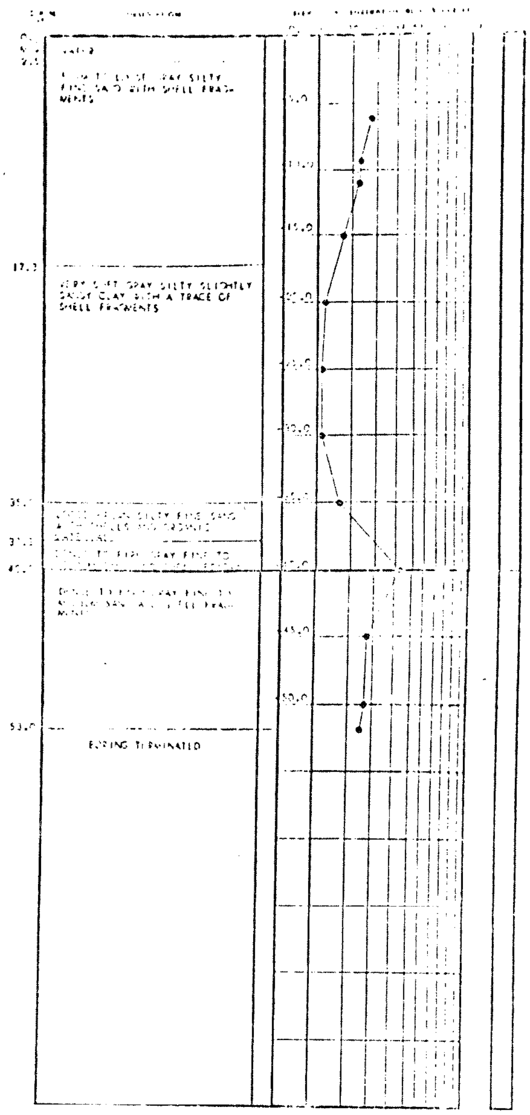
TEST BORING RECORD

BORING NO. B-131

DATE DRILLED 12/13/68

JOB NO. 1-1127

LAW ENGINEERING TESTING CO.



TEST BORING RECORD

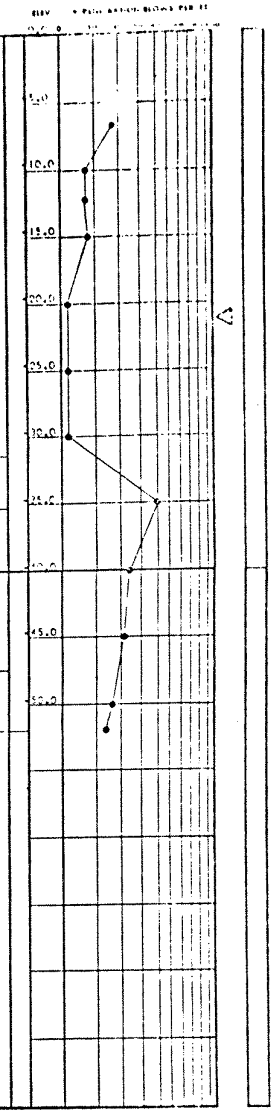
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

BORING NO. B-132
DATE DRILLED 12/12/68
JOB NO. 1-1127

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION
0.0	WATER
MLW	
2.9	LOOSE TO FIRM GRAY SILTY FINE SAND WITH A TRACE OF SHELL FRAGMENTS
17.5	VERY SOFT GRAY SILTY SLIGHTLY SANDY CLAY WITH A TRACE OF SHELL FRAGMENTS
32.3	VERY LOOSE TO DENSE TAN FINE SAND AND SHELLS WITH LAYERS OF CEMENTED SAND AND SHELLS
35.3	VERY FIRM GRAY SILTY FINE SAND WITH A TRACE OF SHELL FRAGMENTS
40.0	VERY FIRM GRAY SILTY FINE SAND WITH A TRACE OF SHELL FRAGMENTS
47.3	FIRM GRAY SILTY FINE TO MEDIUM SAND WITH FRAGMENTED SHELLS
53.0	BORING TERMINATED



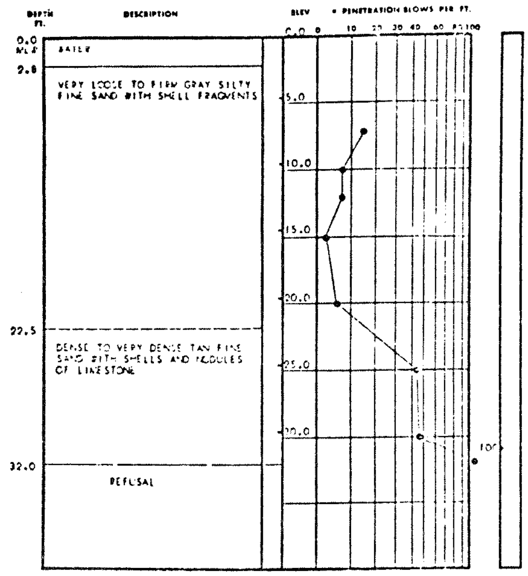
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

TEST BORING RECORD

BORING NO. B-133
DATE DRILLED 12/10/68
JOB NO. 1-1127

LAW ENGINEERING TESTING CO.



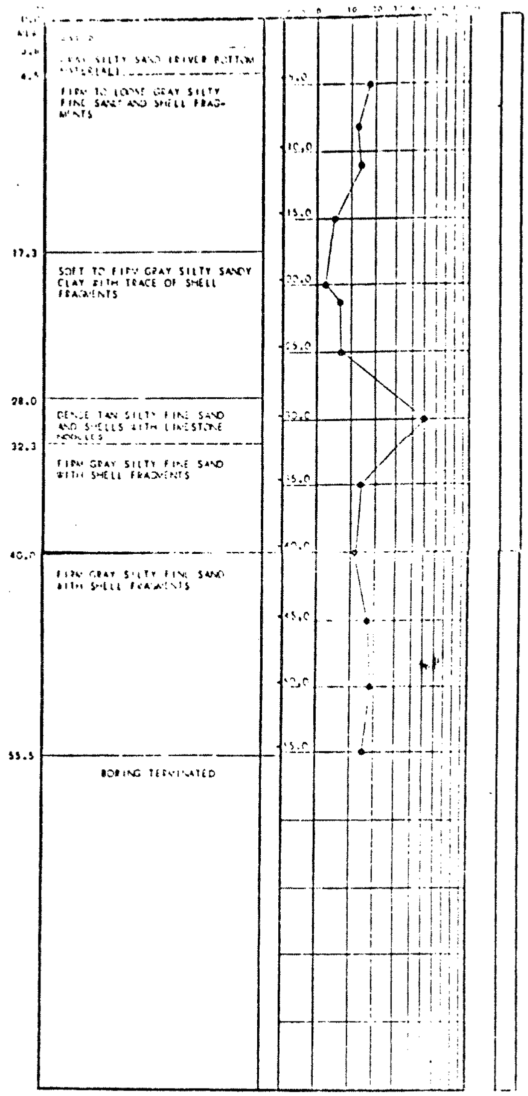
TEST BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

BORING NO. B-134
DATE DRILLED 12/9/68
JOB NO. J-1127

LAW ENGINEERING TESTING CO.



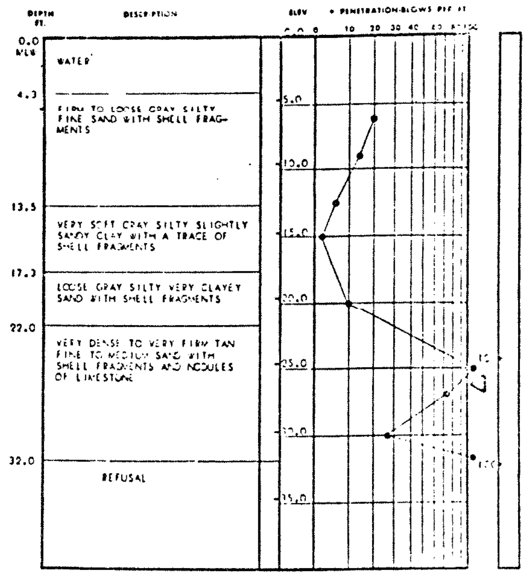
TEST BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

BORING NO. B-135
DATE DRILLED 12/6/68
JOB NO. J-1127

LAW ENGINEERING TESTING CO.



TEST BORING RECORD

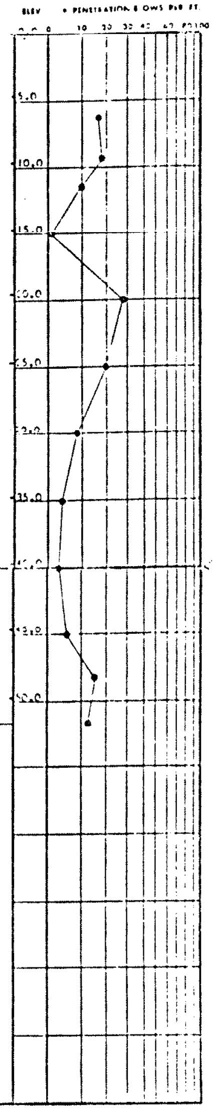
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

WATER TABLE, 24 HR.
UNDISTURBED SAMPLE
WATER TABLE, 1 HR.
% ROCK CORE RECOVERY
LOSS OF DRILLING WATER

BORING NO. B-136
DATE DRILLED 12/5/68
JOB NO. 1-1127

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION
0.0 MLW	WATER
4.0	FIRM GRAY SILTY FINE SAND WITH SHELL FRAGMENTS
13.3	VERY SOFT GRAY SILTY SLIGHTLY SANDY PLASTIC CLAY WITH A TRACE OF SHELL FRAGMENTS
19.5	VERY FIRM GRAY TO TAN MEDIUM SAND CONTAINING A LARGE SHELL CONTENT
22.3	VERY STIFF TAN SILTY VERY SANDY CLAY WITH SHELL FRAGMENTS
27.3	LOOSE TAN SILTY FINE SAND WITH A TRACE OF LIMESTONE NODULES
40.0	LOOSE TAN SILTY FINE SAND WITH A TRACE OF LIMESTONE NODULES
47.0	FIRM GRAY SILTY FINE SAND WITH SHELL FRAGMENTS
53.5	BORING TERMINATED



TEST BORING RECORD

BORING NO. B-137

DATE DRILLED 12/4/68

JOB NO. 1-1127

LAW ENGINEERING TESTING CO.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

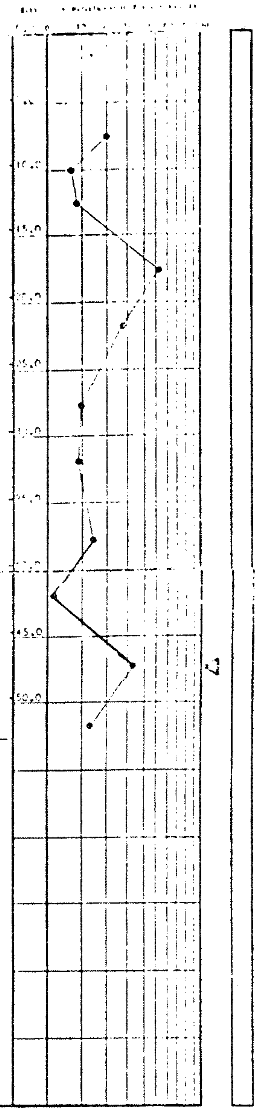
% ROCK CORE RECOVERY

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

DEPTH FT.	DESCRIPTION
0.0 MSL	WATER
4.0	GRAY SILTY FINE SAND (FILL MATERIAL)
7.0	LOOSE TO VERY DENSE GRAY SILTY VERY FINE SAND WITH FRAGMENTED SHELLS
19.0	VERY FIRM BROWN TO GRAY SILTY MEDIUM SAND AND FRAGMENTED SHELLS
26.0	STIFF TAN TO GRAY SILTY VERY SANDY CLAY WITH FRAGMENTED SHELLS
31.0	VERY LOOSE TO DENSE TAN TO GRAY SILTY FINE SAND CONTAINING FRAGMENTED SHELLS AND LIMESTONE NODULES
40.0	VERY LOOSE TO DENSE TAN TO GRAY SILTY FINE SAND CONTAINING FRAGMENTED SHELLS AND LIMESTONE NODULES
53.0	BORING TERMINATED



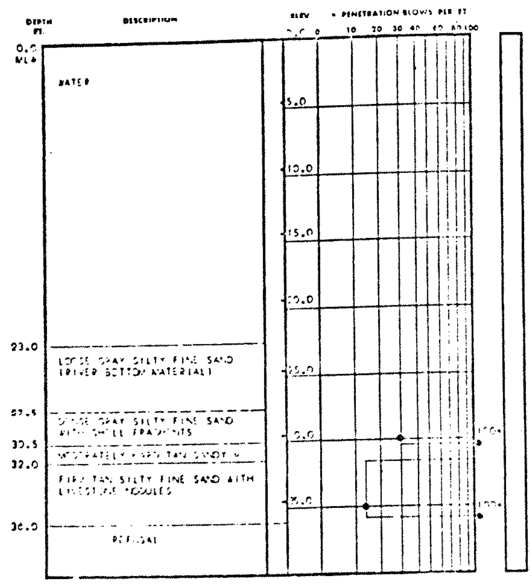
TEST BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
LOSS OF DRILLING WATER

BORING NO. B-138
DATE DRILLED 11/27/68
JOB NO. J-1127

LAW ENGINEERING TESTING CO.



*LIMESTONE LAYER

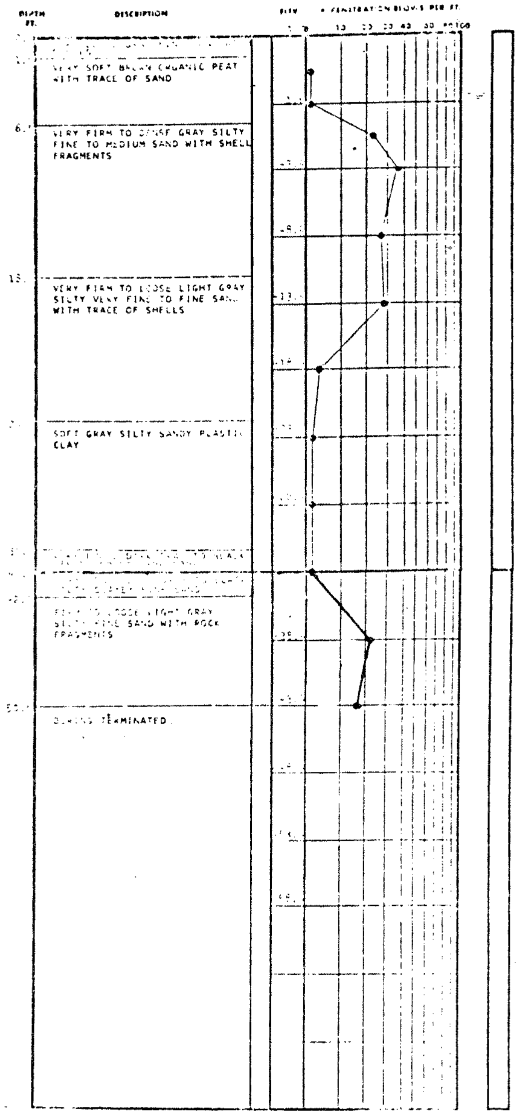
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

TEST BORING RECORD

BORING NO. B-139
DATE DRILLED 12/3/68
JOB NO. J-1127

LAW ENGINEERING TESTING CO.



TEST BORING RECORD

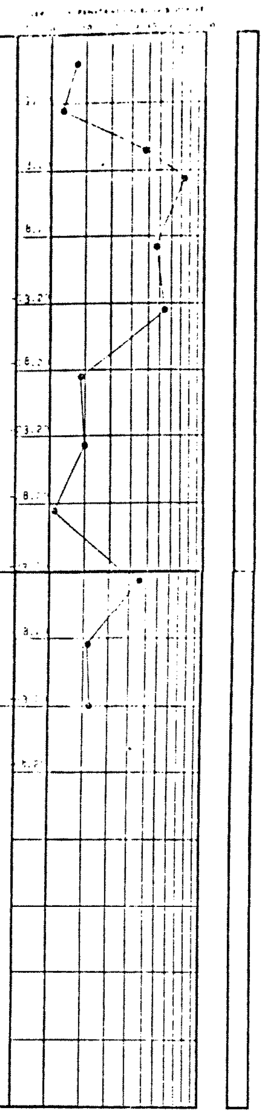
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

BORING NO. B-143
DATE DRILLED 1/23/69
JOB NO. J-1127

LAW ENGINEERING TESTING CO.

Depth (ft)	Description
[illegible]	[illegible] ORGANIC PEAT
5.[illegible]	LOOSE TO VERY DENSE GRAY SILTY FINE TO MEDIUM SAND WITH SHELL FRAGMENTS
12.[illegible]	LOOSE LIGHT GRAY SILTY VERY FINE SAND WITH TRACE OF SHELL FRAGMENTS
22.0	LOOSE GRAY SILTY CLAYEY FINE SAND WITH SHELL FRAGMENTS
25.0	LOOSE GRAY SILTY FINE SAND WITH TRACE OF SHELLS
32.[illegible]	VERY SOFT GRAY SILTY VERY PLASTIC CLAY
37.[illegible]	LOOSE TO FIRM GRAY SILTY FINE SAND WITH CEMENTED SAND AND TRACE OF SHELLS
40.0	LOOSE TO FIRM GRAY SILTY FINE SAND WITH CEMENTED SAND AND TRACE OF SHELLS
[illegible]	BORING TERMINATED



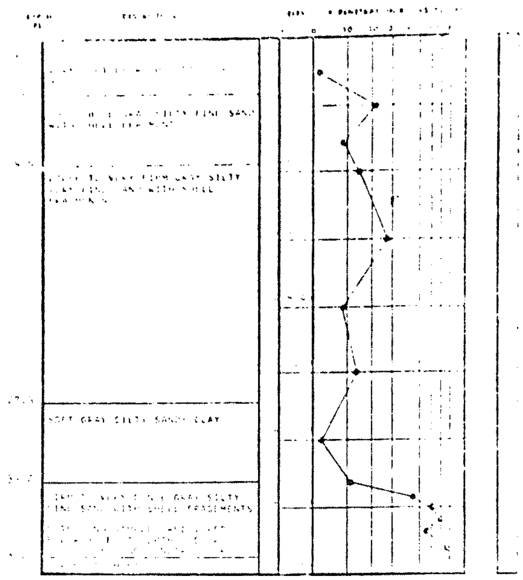
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

TEST BORING RECORD

BORING NO. B-[illegible]A
DATE DRILLED 1/1[illegible]/69
JOB NO. J-112[illegible]

LAW ENGINEERING TESTING CO.



NO GROUND WATER ENCOUNTERED

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

% ROCK CORE RECOVERY

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

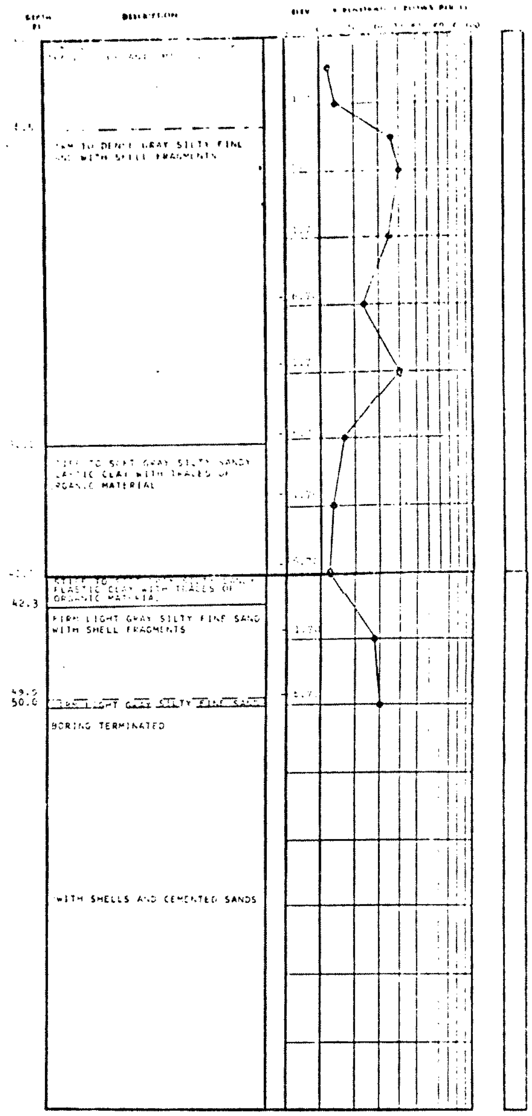
TEST BORING RECORD

BORING NO. B-[illegible]A

DATE DRILLED [illegible]-2-69

JOB NO. J-1[illegible]

LAW ENGINEERING TESTING CO.



TEST BORING RECORD

BORING NO. B-144B

DATE DRILLED 2-24-69

JOB NO. J1127

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

% ROCK CORE RECOVERY

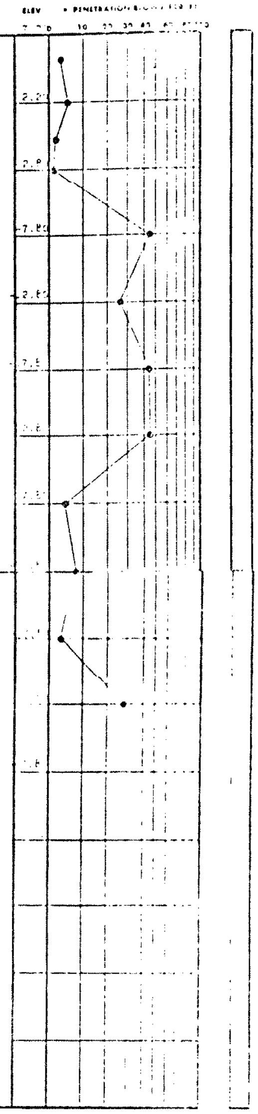
WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION
0.	[illegible] TAN TO [illegible] SILTY FINE TO MEDIUM SAND WITH SHELLS & ORGANIC MATERIAL
6.0	SOFT TO VERY SOFT BROWN ORGANIC PEAT
11.5	DENSE TO VERY FIRM GRAY TO TAN SILTY FINE SAND WITH SHELL FRAGMENTS
23.0	DENSE BLUE-GRAY SILTY FINE TO MEDIUM SAND WITH SHELL FRAGMENTS
34.5	FIRM GRAY SILTY SLIGHTLY SANDY PLASTIC CLAY
[illegible]	[illegible]
[illegible]	[illegible] SILTY VERY CLAYEY FINE SAND
[illegible]	[illegible]
[illegible]	BORING TERMINATED



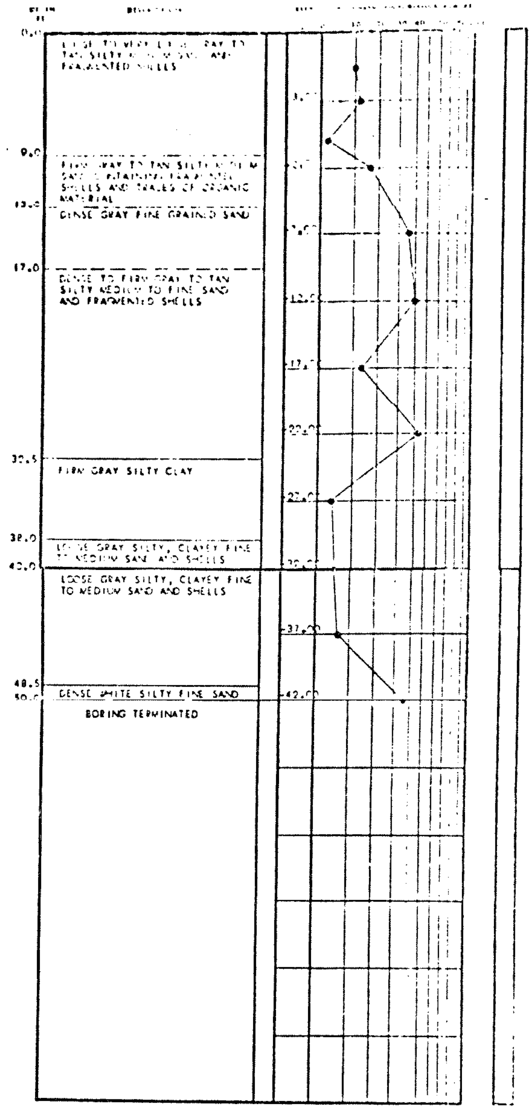
TEST BORING RECORD

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

BORING NO. B-145
DATE DRILLED 1/23/69
JOB NO. J-1127

LAW ENGINEERING TESTING CO.



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE — WATER TABLE, 24 HR.
% ROCK CORE RECOVERY — WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

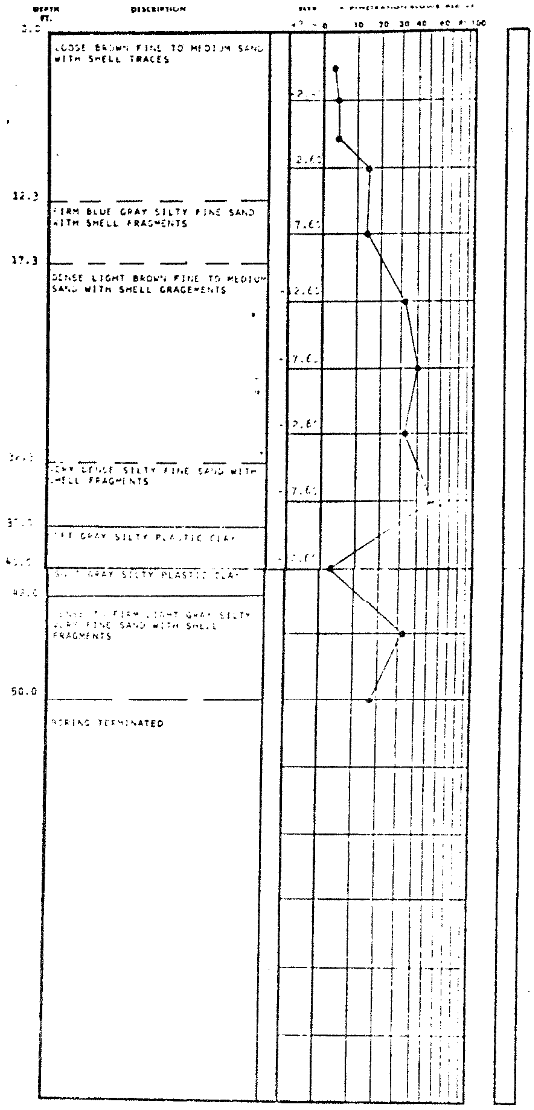
TEST BORING RECORD

BORING NO. B-145A

DATE DRILLED 11/13/68

JOB NO. 1-1127

LAW ENGINEERING TESTING CO.



TEST BORING RECORD

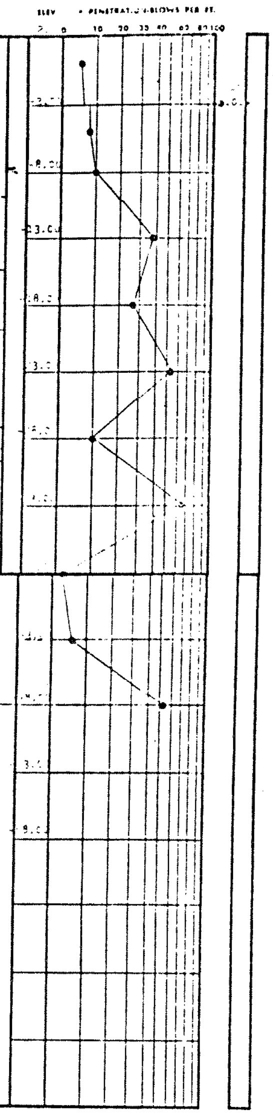
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, TIME OF BORING
LOSS OF DRILLING WATER

BORING NO. B146
DATE DRILLED 2-21-69
JOB NO. J1127

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION
0.	LOOSE BROWN TO TAN FINE TO MEDIUM SAND WITH SHELL FRAGMENTS
12.5	DENSE TAN SILTY VERY FINE SAND
17.5	VERY FIRM BROWN SILTY FINE TO MEDIUM SAND WITH SHELL FRAGMENTS
22.5	VERY DENSE TAN TO GRAY SILTY VERY FINE SAND WITH TRACE OF SHELL FRAGMENTS
29.	LOOSE GRAY SILTY VERY CLAYEY FINE SAND WITH SHELL FRAGMENTS
32.	VERY DENSE BLUE-GRAY SILTY FINE TO MEDIUM SAND WITH TRACE OF SHELLS
37.	SOFT TO FIRM GRAY SILTY [illegible] CLAY WITH TRACE OF SHELLS & ORGANIC MATERIAL
[illegible]	SOFT TO FIRM GRAY SILTY [illegible] WITH TRACE OF SHELLS & ORGANIC MATERIAL
47.	VERY DENSE LIGHT GRAY SILTY VERY FINE SAND WITH TRACE OF SHELLS
50.0	BORING TERMINATED
80.0	



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

TEST BORING RECORD

BORING NO. B-146A
DATE DRILLED 1/15/69
JOB NO. J-1127

LAW ENGINEERING TESTING CO.

SOIL DATA REPORT
HUTCHINSON ISLAND
JOB NO. J-1127

BORING NUMBER	SAMPLE NUMBER	DEPTH (FT.)	WATER CONTENT PERCENT	PCF (WET)	SPECIFIC GRAVITY	VISUAL CLASSIFICATION
B-106	UD	87.5	25.9		2.72	GRAY FINE TO MEDIUM SAND
B-106	UD	137.5	24.1	130.02	2.69	GRAY FINE TO MEDIUM CLAYEY SAND
B-106	UD	147.5	30.8	118.55	2.68	GRAY FINE TO MEDIUM CLAYEY SAND
B-106	UD	157.5	28.5	118.2	2.67	GRAY FINE TO MEDIUM SILTY SAND
B-107A	UD	79.5	26.1	131.25	2.74	GRAY SHELLY SILTY FINE SAND
B-108	UD	67.5	16.2	132.3	2.75	GRAY FINE TO MEDIUM SAND WITH SHELL
B-108	UD	87.5	23.6	132.6	2.70	GRAY FINE TO MEDIUM SAND
B-108	UD	97.5	18.6	137.9	2.70	GRAY FINE TO MEDIUM SAND WITH SHELL
B-108A	UD	64.0	19.6	142.8	2.73	GRAY FINE TO MEDIUM SAND WITH SHELL
B-108A	UD	68.0	20.6	112.0	2.71	GRAY FINE TO MEDIUM SAND WITH SHELL
B-108A	UD	82.0	24.1	111.0	2.77	GRAY SHELL WITH SAND
B-108A	UD	88.0	30.3	120.3	2.73	GRAY FINE TO MEDIUM SAND WITH SHELL
B-114A	UD	207.5	26.1	117.7	2.67	DARK GRAY CLAYEY FINE SANDY SILT
B-114A	UD	257.5	41.0	110.7	2.64	GRAY CLAYEY FINE SILTY SAND
B-114A	UD	282.5	23.8	105.9	2.63	GRAY FINE TO MEDIUM SANDY CLAYEY SILT
B-114A	UD	307.5	35.4	112.2	2.62	GRAY FINE SANDY SILTY CLAY

SOIL DATA REPORT
HUTCHINSON ISLAND
JOB NO. J-1127

BORING NUMBER	SAMPLE NUMBER	DEPTH (FT.)	WATER CONTENT PERCENT	PCF (WET)	SPECIFIC GRAVITY	VISUAL CLASSIFICATION
B-107	UD	83.5	20.4	132.5	2.79	GRAY FINE TO MEDIUM SANDY SHELL
B-107	UD	152.5	18.8	133.0	2.76	GRAY AND GREEN FINE TO MEDIUM SILTY CLAYEY SAND
B-111A	UD	78.0	19.9	104.2	2.74	GRAY FINE TO MEDIUM SANDY SHELL
B-111A	UD	84.0	20.7	102.6	2.75	GRAY FINE TO MEDIUM SANDY SHELL
B-111A	UD	88.0	21.2	103.0	2.74	GRAY FINE TO MEDIUM SILTY SAND
B-113	UD	67.0	17.8	132.6	2.75	GRAY FINE TO MEDIUM SAND WITH SHELL
B-114A	UD	357.5	35.7	116.2	2.62	DARK GREEN FINE SANDY SILTY CLAY
B-114A	UD	407.5	43.7	109.9	2.60	DARK GREEN FINE SANDY SLIGHTLY MICACEOUS CLAYEY SILT
B-115	UD	67.5	18.0	132.0	2.76	GRAY FINE TO MEDIUM SANDY SHELL
B-115	UD	77.5	20.6	133.1	2.74	GRAY FINE TO MEDIUM SANDY SHELL
B-115	UD	87.5	16.1	105.7	2.76	GRAY FINE TO MEDIUM SAND WITH SHELL
B-115	UD	107.5	24.7	127.9	2.72	GRAY FINE SILTY SAND WITH SHELL
B-115	UD	107.5			2.69	GRAY FINE SILTY SAND
B-115	UD	137.5	27.2	125.2	2.69	GRAY FINE TO MEDIUM SILTY SAND
B-115	UD	147.5	26.0	127.2	2.67	GRAY FINE TO MEDIUM SAND
B-117	UD	77.0	19.9	103.4	2.74	GRAY FINE TO MEDIUM SANDY SHELL

LAW ENGINEERING TESTING COMPANY
BIRMINGHAM ALABAMA

HUTCHINSON ISLAND
J-1127 (UD SAMPLES)
REPORT OF RELATIVE DENSITY

BORING NUMBER	DEPTH (FT.)	MIN. DENSITY (#cf) (γmin)	MAX. DENSITY (#cf) (γmin)	IN PLACE DENSITY (#cf) (γ)	RELATIVE DENSITY (%) DRY METHOD	RELATIVE DENSITY (%) WET METHOD
B-106	137.5	69.70	95.47	104.77	122.94	
B-106	147.5	73.70	101.50	90.63	68.20	75.81
B-106	157.5	68.70	91.29	91.98	102.28	
B-107	152.5	72.31	113.25	111.95	97.95	
B-107	83.5	72.91	112.83	110.05	95.39	
B-107A	79.5	73.86	95.60	104.08	127.68	
B-108	67.5	72.00	113.87	113.86	99.98	
B-108	87.5	83.62	105.76	107.28	105.35	101.44
B-108	97.5	85.80	117.65	116.27	96.80	
B-108A	64.0	84.28	111.66	119.40	119.95	
B-108A	68.0	85.24	117.00	92.87	30.27	
B-108A	82.0	83.87	117.16	85.19	54.54	53.33
B-108A	88.0	75.82	97.01	96.97	99.63	
B-111A	78.0	79.80	111.53	86.91	28.76	
B-111A	84.0	72.90	107.78	85.00	43.99	
B-111A	88.0	84.00	107.78	84.98	5.23	
B-113	67.0	80.64	113.66	112.56	97.61	
B-114A	207.5	75.60	98.08	93.34	82.92	
B-114A	257.5	71.40	92.44	78.51	39.79	38.77
B-114A	282.5	73.25	91.22	85.54	72.95	
B-114A	307.5	68.89	89.01	82.87	74.63	68.59
B-115	67.5	85.20	113.85	111.86	94.71	
B-115	77.5	76.21	107.86	110.36	105.45*	
B-115	87.5	86.40	114.54	91.04	20.75	
B-115	147.5	76.65	109.92	100.95	79.53	
B-117	77.0	72.18	111.89	86.24	45.94	

*CEMENTED SAND AND SHELL

LAW ENGINEERING TESTING COMPANY
BIRMINGHAM, ALABAMA

2A-83

SUMMARY OF LABORATORY TEST DATA

Boring Number	Sample Number	Feet	Soil Type	Natural Water Content, w - %	Specific Gravity G	Grain Size: Gravel %	Grain Size: Sand %	Grain Size: Fines %	Compaction Test AASHO,T-180-57: Type of Test	Compaction Test AASHO,T-180-57: Optimum Moisture	Compaction Test AASHO,T-180-57: Max. Dry Density	ASTMD 2049-64T Max., Min. Den.: Max. Density	ASTMD 2049-64T Max., Min. Den.: Min. Density
A-1	S-1	12'-27'	Gy med to fi sa	24.9	2.68	0.6	96.9	2.5	D	14.5	97.6	102.3	76.5
A-1	S-2	27'-47'	Gy & tan si med to fi sa w/shell	22.1	2.69	15.7	85.8	8.5	C	14.1	118.3	106.5	80.3
A-1	S-3	47'-67'	Gy & tan si fi sa w/shell & LS	24.2	2.75	3.5	87.5	9.0	D	14.2	118.0	115.1	83.8
A-3	S-4	12'-27'	Gy fi sa w/ shell	28.0	2.65	1.0	96.4	2.6	D	14.6	100.3	102.1	77.1
A-3	S-5	27'-47'	Tan cl si fi sa w/shell & LS	16.9	2.65	50.8	37.0	12.2	D	9.6	126.3	114.3	80.6
A-3	S-6	47'-67'	Tan cl si fi sa w/shell & LS	9.6	2.65	20.0	60.3	19.7	D	11.0	123.3	113.4	72.4
A-4	S-7	12'-27'	Gy fi sa w/shell	12.9	2.68	1.8	95.0	3.2	D	15.9	103.4	103.2	76.0
A-4	S-8	27'-47'	Gy & tan si fi sa w/shell & LS	16.3	2.69	12.0	75.0	13.0	D	10.0	121.0	113.3	78.8
A-4	S-9	47'-67'	Gy & tan si fi sa w/shell & LS	13.6	2.72	18.0	68.7	13.3	D	8.2	129.7	122.1	78.5
A-2	S-10	12'-27'	Dk gy med to co sa	20.2	2.65	2.9	94.4	2.7	D	13.7	106.0	107.4	79.7
A-2	S-11	27'-47'	Br & tan sl clsi sa w/shell & LS	25.4	2.70	5.9	68.3	25.8	A	13.4	120.1	107.8	78.8
A-2	S-12	47'-67'	Gy & tan si cl fi sa w/LS & shell	20.9	2.65	17.0	78.3	4.7	D	9.0	125.4	117.8	76.9

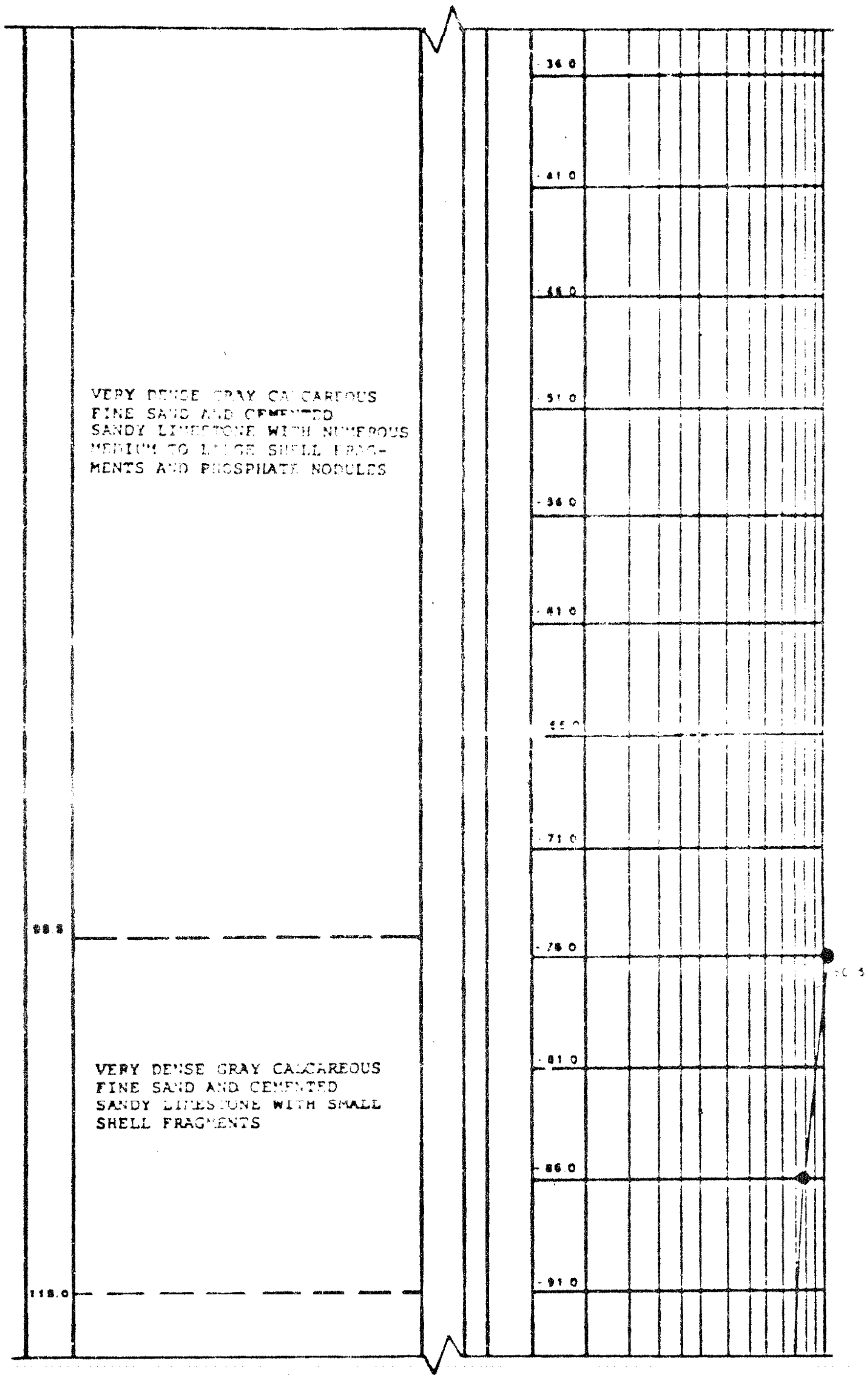
LAW ENGINEERING TESTING COMPANY Jacksonville, Florida	HUTCHINSON ISLAND	J-1127	File No. Table No.

TEST BORING RECORD

DEPTH FEET	DESCRIPTION	ELEV.	PENETRATION-BLOWS PER FOOT 0 5 10 15 20 30 40 60 80 100
0.0		24.0	
		19.0	
		14.0	
		9.0	
		4.0	
		-1.0	
	WASH BORING FROM 0 - 98.5 FEET, NO SAMPLES TAKEN	-6.0	
		-11.0	
		-16.0	
		-21.0	
		-26.0	
		-31.0	

BORING NUMBER AG 100
ST. LUCIE PLANT
JOB NUMBER SA-737

PAGE 1 OF 4



BORING NUMBER AG 100
ST. LUCIE PLANT
JOE NUMBER SA-737

Depth	Description	Elevation
	VERY DENSE GRAY CALCAREOUS FINE SAND AND CEMENTED SANDY LIMESTONE WITH NUMEROUS MEDIUM TO LARGE SHELL FRAGMENTS AND PHOSPHATE NODULES	-96.0
		-101.0
		-106.0
135.0		-111.0
	VERY DENSE TO DENSE GRAY CALCAREOUS FINE TO MEDIUM SAND AND CEMENTED SANDY LIMESTONE WITH SOME SHELL FRAGMENTS	-116.0 (50/5")
		-121.0

Depth	Description	Elevation	Weight of Rod and Hammer	Movement by Rod Weight	Hammer Blows 2nd 6"	Hammer Blows 3rd 6"
150.0	VERY DENSE TO DENSE GRAY CALCAREOUS FINE TO MEDIUM SAND AND CEMENTED SANDY LIMESTONE WITH SOME SHELL FRAGMENTS	-126.0	770	0	18	17
	BASE OF UNDIFFERENTIATED SANDS	-131.0				
155.0	TOP OF HAWTHORNE FORMATION					
	VERY DENSE LIGHT GREEN TO GREEN FINE SAND WITH SOME SMALL PHOSPHATE NODULES	-136.0	812	0	50/6"	-
165.0		-141.0				
	VERY FIRM GREEN SLIGHTLY SILTY FINE SAND	-146.0	854	0	10	17
175.0		-151.0				

BORING NUMBER AG 100
ST. LUCIE PLANT
JOB NUMBER SA-737

FIRM TO DENSE GREEN SLIGHTLY CLAYEY SILTY FINE SAND

193.0

STIFF TO VERY STIFF GREEN SLIGHTLY SANDY, SILTY CLAY AND SILTY CLAY

215.0

STIFF TO VERY STIFF GREEN CALCAREOUS SLIGHTLY SANDY SILTY CLAY WITH SOME SMALL SHELL FRAGMENTS

240.0

BORING TERMINATED

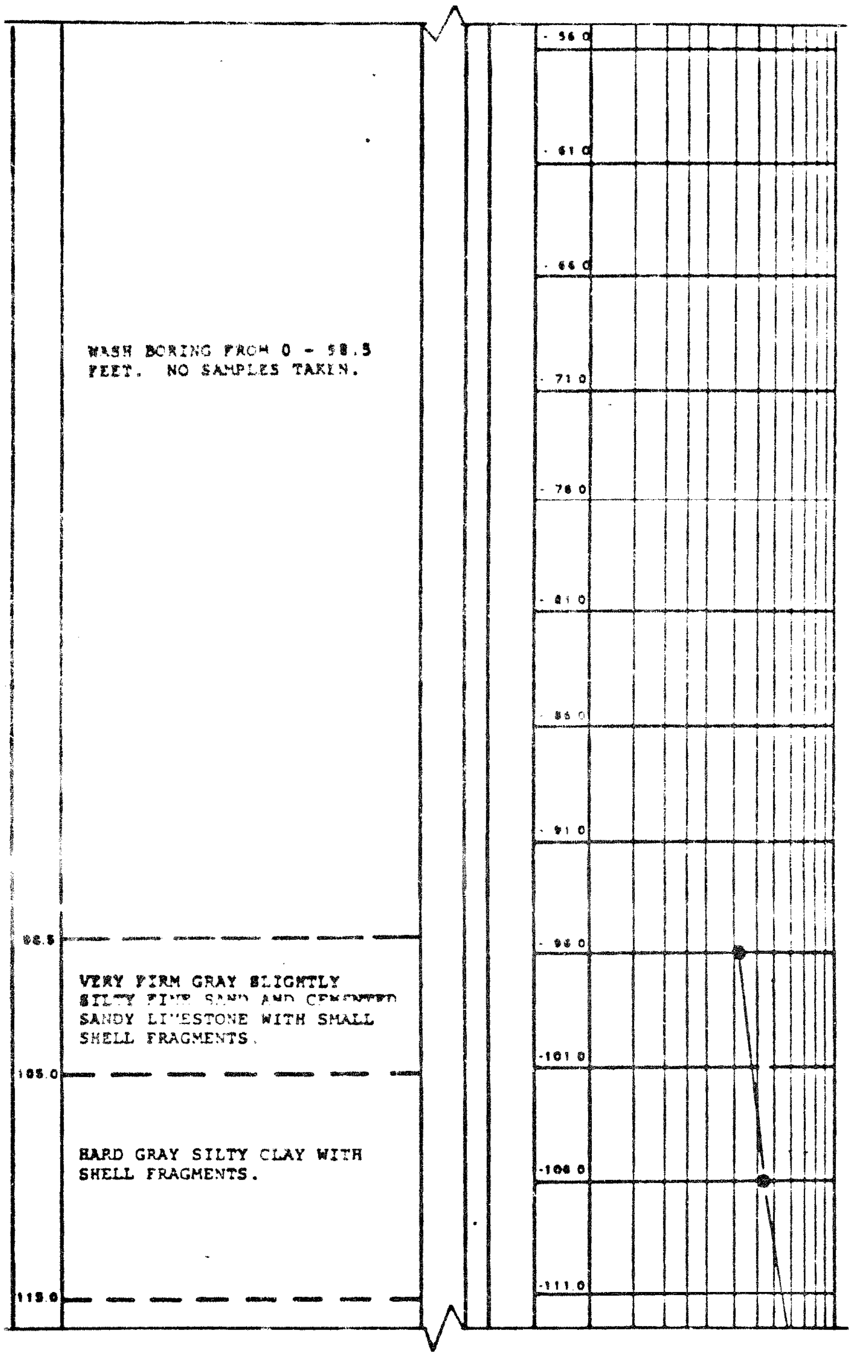
-156.0	896	0	3	9
-161.0				
-166.0	938	0	9	34
-171.0				
-176.0	980	0	2	18
-181.0				
-186.0	1022	0	3	6
-191.0				
-196.0	1064	0	4	10
-201.0				
-206.0	1106	0	0	11
-211.0				
-216.0	1148	0	8	13

BORING NUMBER AG 100
ST. LUCIE PLANT
JOB NUMBER SA-737

TEST BORING RECORD

DEPTH FEET	DESCRIPTION	ELEV.	PENETRATION-BLOWS PER FOOT [illegible]
0.0		4.0	
		- 1.0	
		- 6.0	
		- 11.0	
		- 16.0	
		- 21.0	
	WASH BORING FROM 0 - 98.5 FEET. NO SAMPLES TAKEN.		
		- 26.0	
		- 31.0	
		- 36.0	
		- 41.0	
		- 46.0	
		- 51.0	

BORING NUMBER AG 101
ST. LUCIE PLANT
JOB NUMBER SA-737

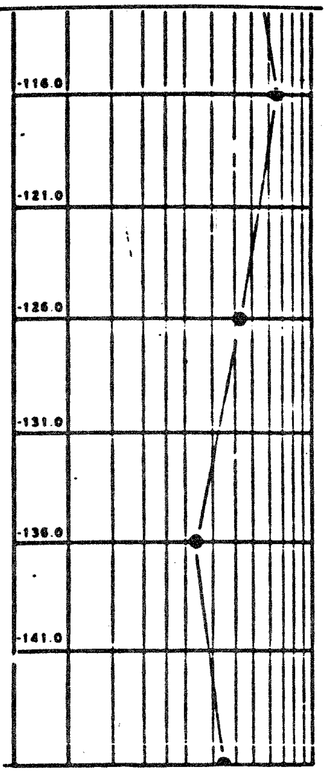


BORING NUMBER AG 101
ST. LUCIE PLANT
JOB NUMBER SA-737

VERY DENSE TO DENSE GRAY FINE SAND AND SANDY LIMESTONE WITH SHELL FRAGMENTS.

135.0

VERY FIRM TO DENSE GRAY AND BLACK SPECKLED CALCAREOUS FINE SAND AND SLIGHTLY SILTY FINE SAND AND CEMENTED SANDY LIMESTONE WITH SMALL SHELL FRAGMENTS.



Depth	Description	Elevation	Weight of rod and hammer	Movement by rod weight	Hammer blows 2nd 6"	Hammer blows 3rd 6"
150.0	VERY FIRM TO DENSE GRAY AND BLACK SPECKLED CALCAREOUS FINE SAND AND SLIGHTLY SILTY FINE SAND AND CEMENTED SANDY LIMESTONE WITH SMALL SHELL FRAGMENTS	-146.0				
155.0	BASE OF UNDIFFERENTIATED	-151.0				
	TOP OF HAWTHORNE FORMATION					
	VERY FIRM GREEN SLIGHTLY SILTY FINE SAND.	-156.0	812	9	14	16
165.0		-161.0				
	LOOSE TO FIRM GREEN SILTY FINE SAND	-166.0	854	0	3	5

BORING NUMBER AG 101
ST. LUCIE PLANT
JOB NUMBER SA-737

175.0		-171.0				
	LOOSE TO FIRM GREEN SILTY VERY FINE SAND	-176.0	896	0	2	6
		-181.0				
		-186.0	938	0	4	7
195.0		-191.0				
	VERY FIRM GREEN SLIGHTLY CALCAREOUS SILTY FINE SAND WITH SOME SMALL SHELL FRAGMENTS	-196.0	980	0	9	13
203.0		-201.0				
	DENSE GREEN SILTY FINE SAND	-206.0	1022	0	16	31
		-211.0				
		-216.0	1064	[illegible]	11	28
		-221.0				
		-226.0	1106	12"	0	11

BORING NUMBER AG 101
ST. LUCIE PLANT
JOB NUMBER SA-737

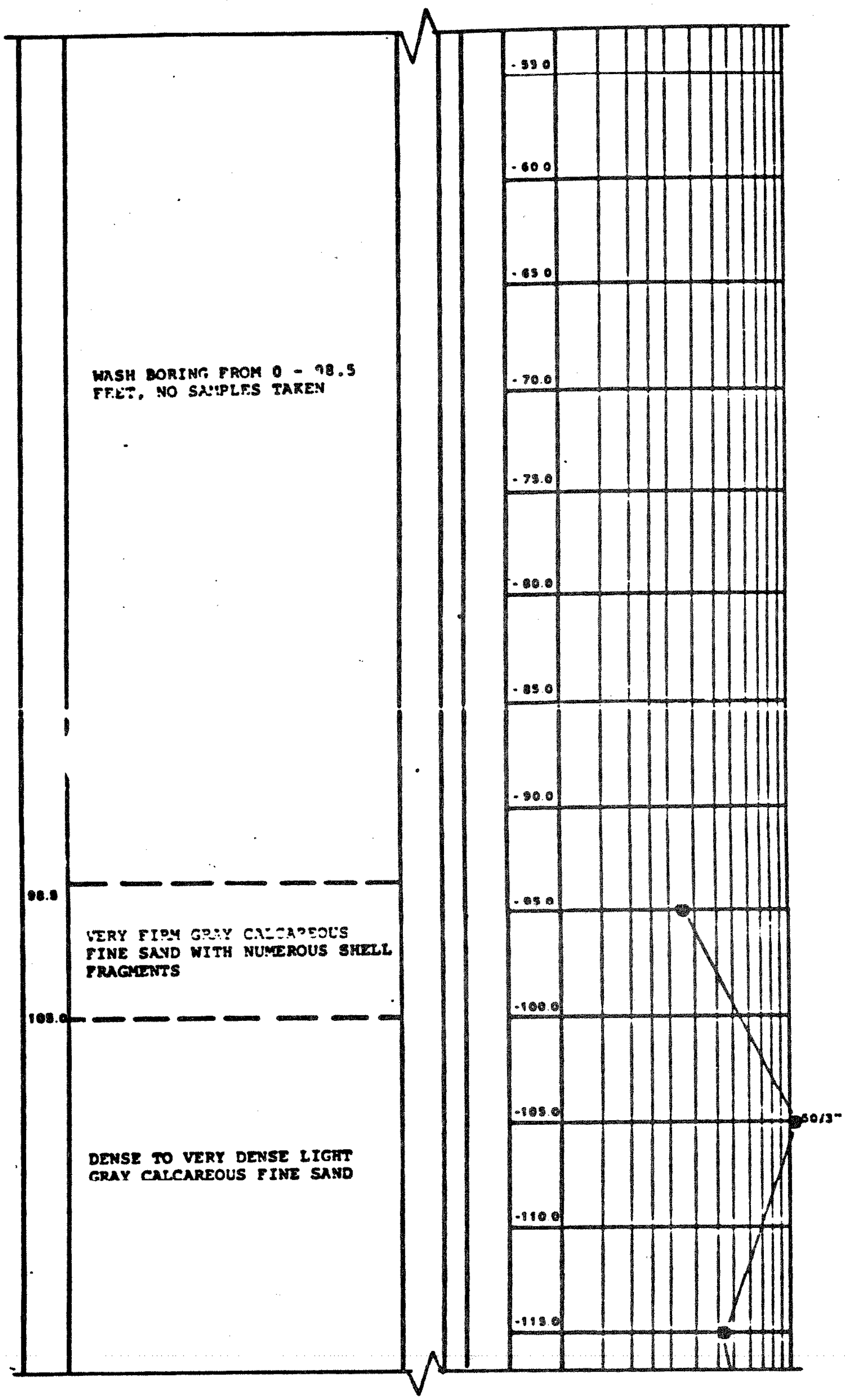
235.0		-231.0				
	DENSE GREEN SLIGHTLY CAL-CAREOUS FINE SAND WITH SOME SMALL SHELL FRAGMENTS	-236.0	1148	18"	0	8
		-241.0				
250.0		-246.0	1190	12"	0	11
	BORING TERMINATED					

BORING NUMBER AG 101
ST. LUCIE PLANT
JOB NUMBER SA-737

TEST BORING RECORD

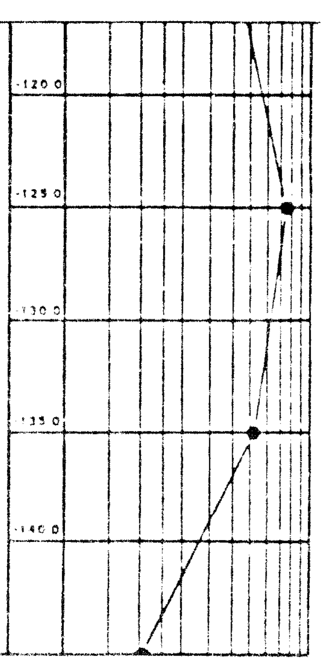
DEPTH FEET	DESCRIPTION	ELEV.	PENETRATION-BLOWS PER FOOT
0.0		5.0	
		0.0	
		-5.0	
		-10.0	
		-15.0	
	WASH BORING FROM 0 - 18.5 FEET, NO SAMPLES TAKEN	-20.0	
		-25.0	
		-30.0	
		-35.0	
		-40.0	
		-45.0	
		-50.0	

BORING NUMBER AG 102
ST. LUCIE PLANT
JOB NUMBER SA-737



BORING NUMBER AG 102
ST. LUCIE PLANT
JOB NUMBER SA-737

Depth	Description	Elevation
125.0		-120.0
	DENSE TO VERY DENSE GRAY AND BLACK SPECKLED CALCAREOUS FINE SAND AND CEMENTED SANDY LIMESTONE WITH NUMEROUS SHELL FRAGMENTS	-125.0
		-130.0
		-135.0
145.0	FIRM GRAY AND BLACK SPECKLED CALCAREOUS FINE SAND (VERY FINE SHELL FRAGMENTS)	-140.0



Depth	Description	Elevation	WEIGHT OF ROD AND HAMMER	MOVEMENT BY ROD WEIGHT	HAMMER BLOWS 2ND 6"	HAMMER BLOWS 3RD 6"
	FIRM GRAY AND BLACK SPECKLED CALCAREOUS FINE SAND (VERY FINE SHELL FRAGMENTS)	-145.0				
155.0		-150.0				
	FIRM GRAY AND BLACK SPECKLED CALCAREOUS FINE SAND (VERY FINE SHELL FRAGMENTS)	-155.0	812	0	7	9
162.0	BASE OF UNDIFFERENTIATED SANDS / TOP OF HAWTHORNE FORMATION					
		-160.0				
	DENSE GREEN SILTY FINE SAND	-165.0	854	8	12	24
175.0		-170.0				
	LOOSE GREEN SILTY FINE SAND	-175.0	896	6	2	8

BORING NUMBER AG 102
ST. LUCIE PLANT
JOB NUMBER SA-737

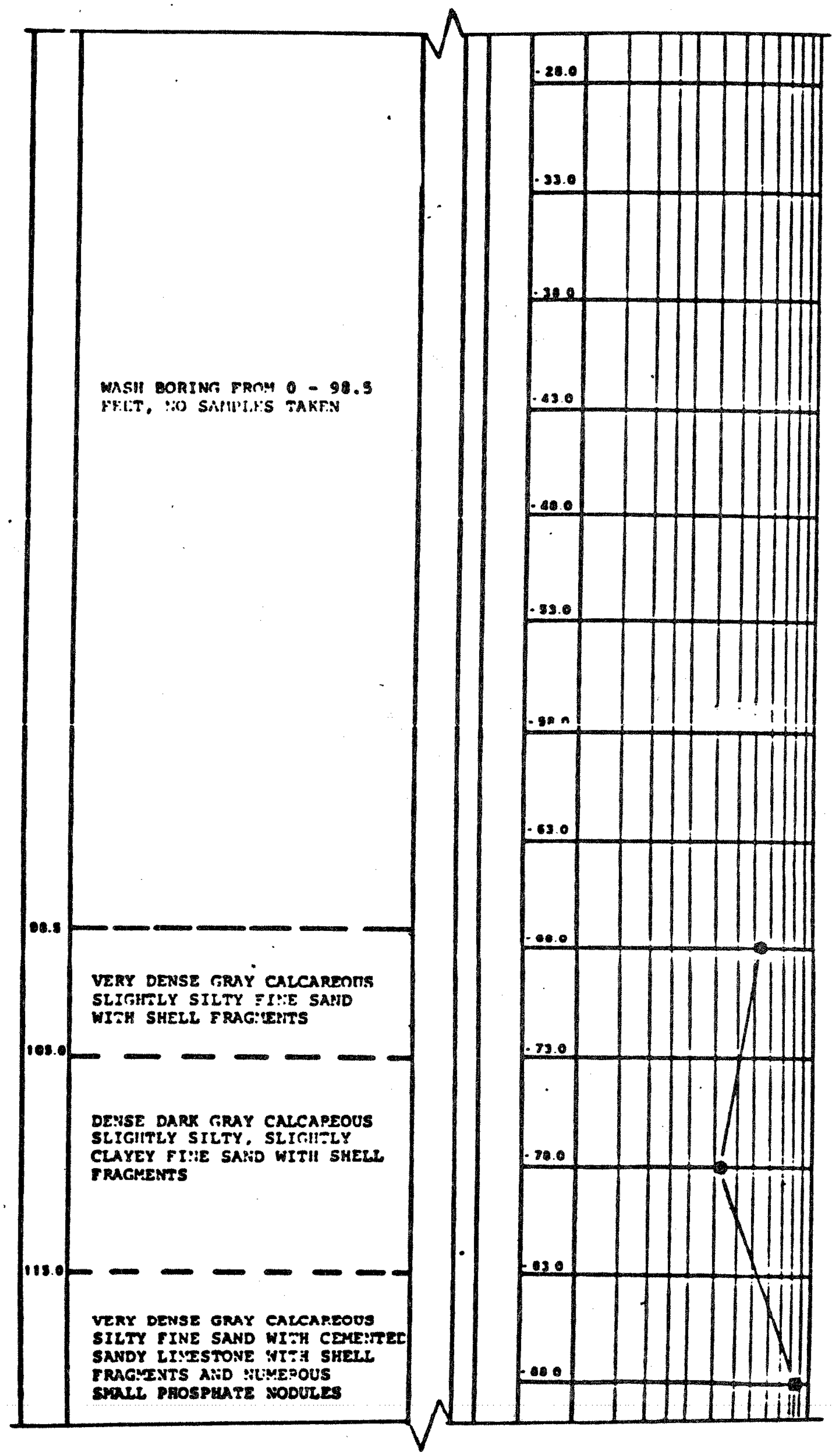
Depth	Description	Elevation	Sample			
185.0		-180.0				
	LOOSE GREEN SLIGHTLY CALCAREOUS SLIGHTLY CLAYEY SILTY FINE SAND	-185.0	938	0	3	7
195.0		-190.0				
	VERY DENSE GREEN SLIGHTLY SILTY FINE SAND	-195.0	980	0	18	35
205.0		-200.0				
	DENSE GREEN SLIGHTLY CALCAREOUS SLIGHTLY CLAYEY SILTY FINE SAND WITH SOME SHELL FRAGMENTS AND BROWN NODULES	-205.0	1022	0	8	26
215.0		-210.0				
	DENSE TO VERY DENSE GREEN SLIGHTLY TO NON-CALCAREOUS FINE SAND AND SLIGHTLY SILTY FINE SAND WITH A FEW SMALL SHELL FRAGMENTS	-215.0	1064	0	24	50/4"
		-220.0				
		-225.0	1106	0	13	26
		-230.0				
		-235.0	1148	6"	1	2
		-240.0				
		-245.0	1190	12"	0	7
250.0	BORING TERMINATED					

BORING NUMBER AG 102
ST. LUCIE PLANT
JOB NUMBER SA-737

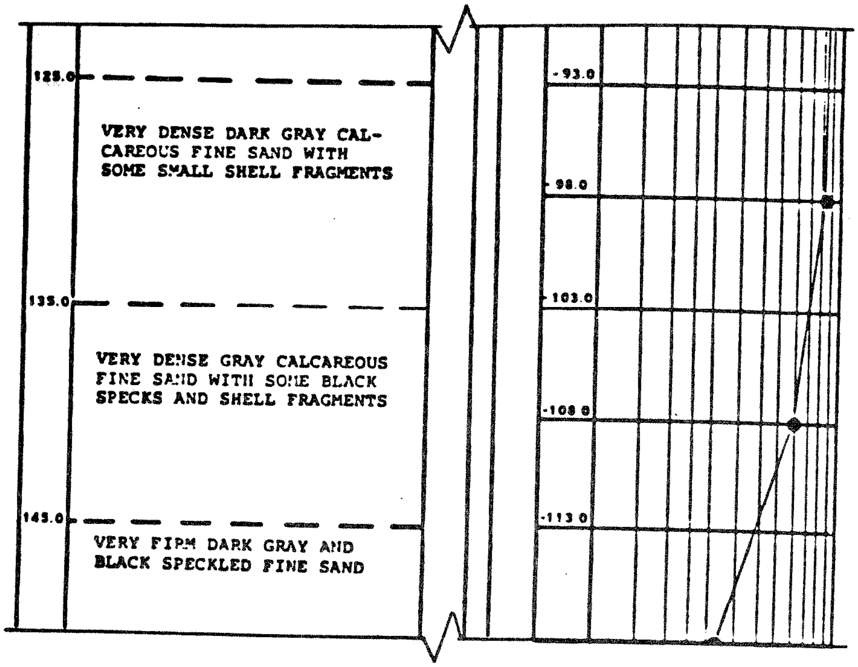
TEST BORING RECORD

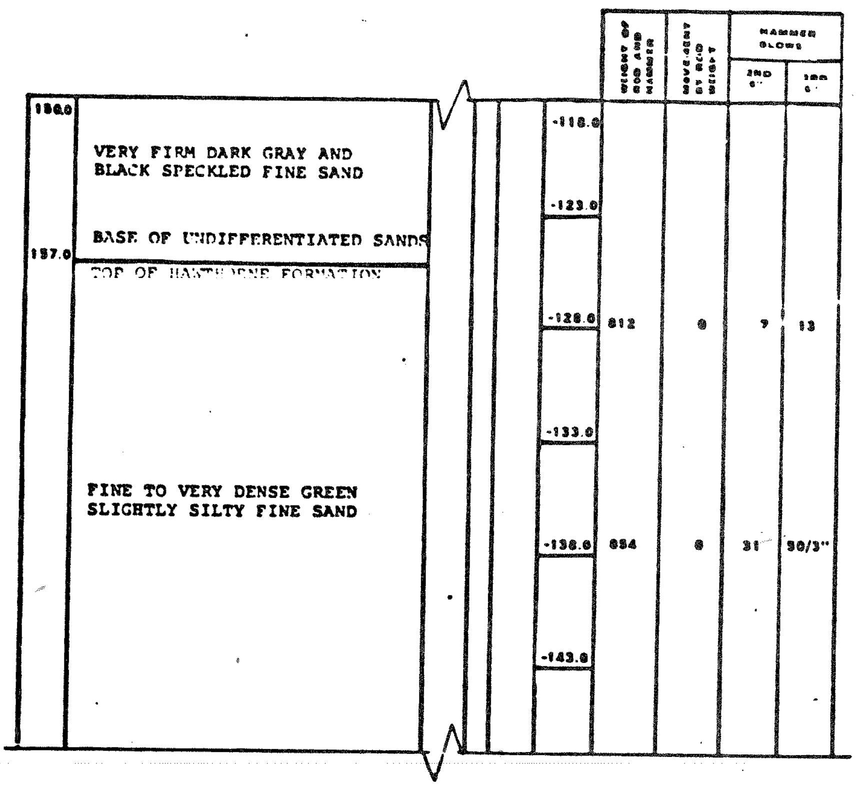
DEPTH FEET	DESCRIPTION	ELEV.	PENETRATION-BLOWS PER FOOT 0 5 10 15 20 30 40 60 80 100
0.0		32.0	
		27.0	
		22.0	
		17.0	
		12.0	
		7.0	
	WASH BORING FROM 0 - [illegible]8.5 FEET, NO SAMPLES TAKEN	2.0	
		-3.0	
		-8.0	
		-13.0	
		-18.0	
		-23.0	

BORING NUMBER AG 103
ST. LUCIE PLANT
JOB NUMBER SA-737



BORING NUMBER AG 103
ST. LUCIE PLANT
JOB NUMBER SA-737





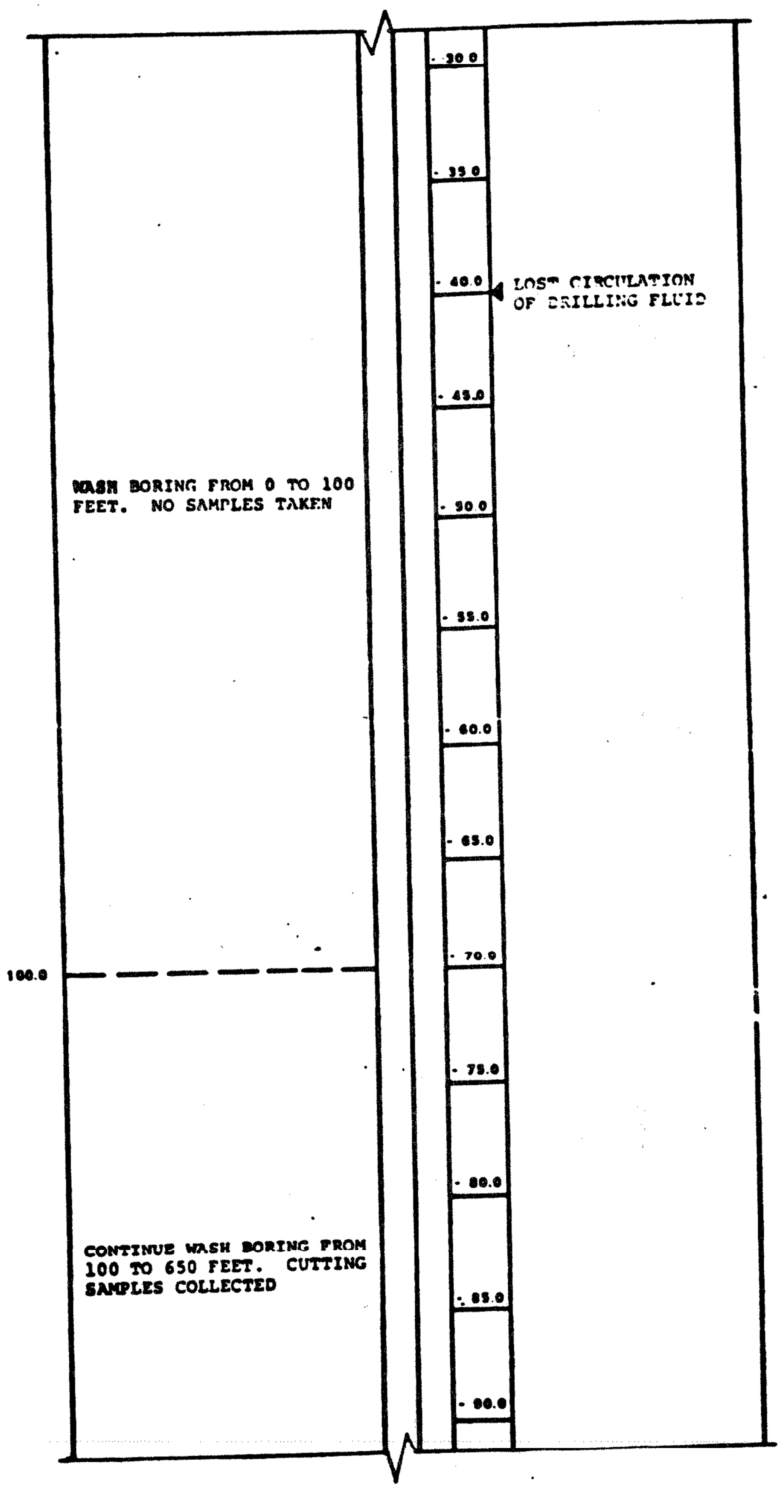
BORING NUMBER AG 103
ST. LUCIE PLANT
JOB NUMBER SA-737

	FINE TO VERY DENSE GREEN SLIGHTLY SILTY FINE SAND	-148.0	896	0	6	13
		-153.0				
185.0	FIRM GREEN SLIGHTLY SILTY CLAYEY FINE SAND	-158.0	938	0	3	9
		-163.0				
		-168.0	980	0	5	13
		-173.0				
205.0	STIFF GREEN SLIGHTLY SANDY SILTY CLAY	-178.0	1022	0	5	7
210.0	BORING TERMINATED	-183.0				

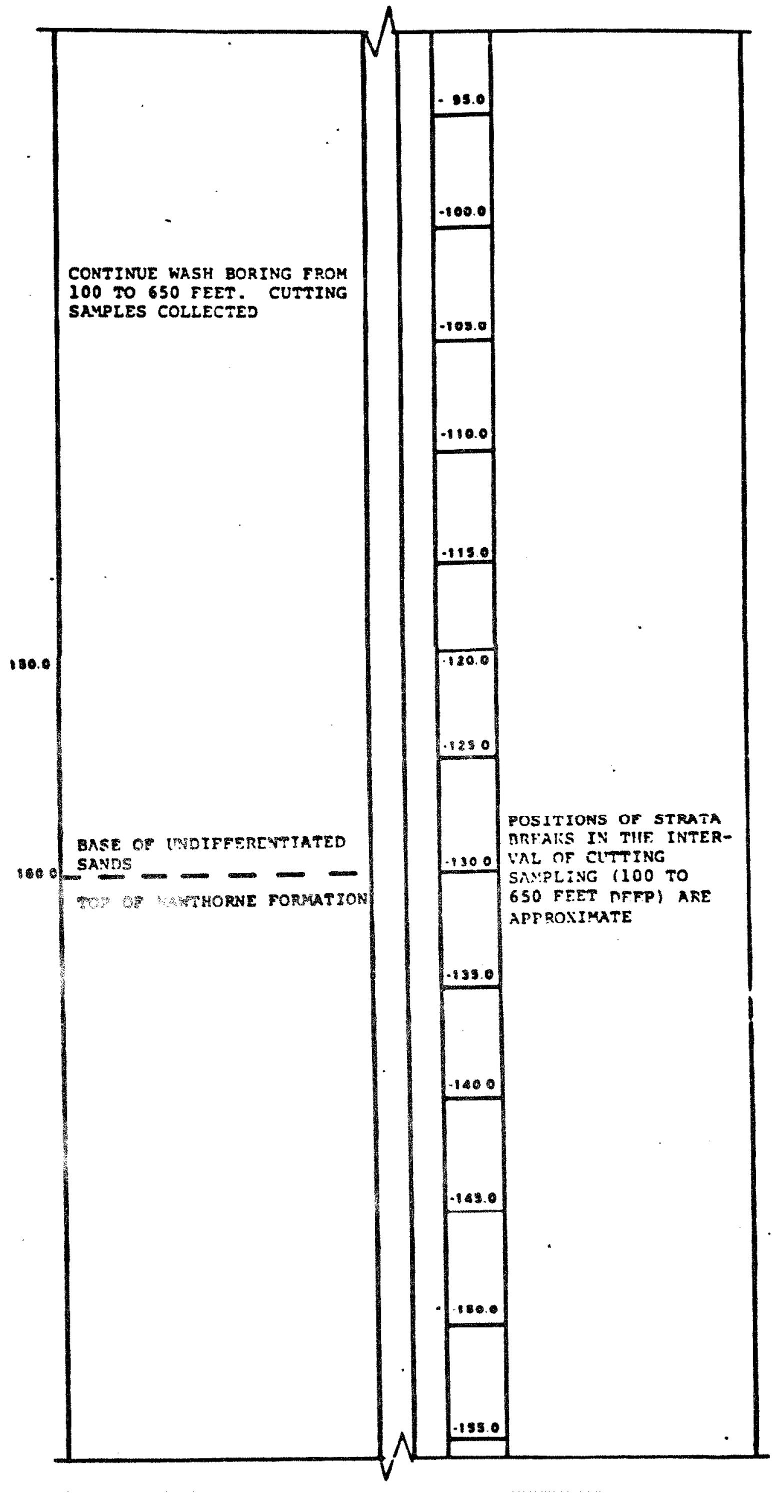
BORING NUMBER AG 103
ST. LUCIE PLANT
JOB NUMBER SA-737

DEPTH FT.	DESCRIPTION	CORE %	BIT SIZE	ELEV.	REMARKS
0.0				30.0	
				25.0	
				20.0	
				15.0	
				10.0	
				5.0	
	WASH BORING FROM 0 TO 100 FEET. NO SAMPLES TAKEN				
				0.0	
				- 5.0	
				- 10.0	
				- 15.0	
				- 20.0	
				- 25.0	

BORING NUMBER AG 104
ST. LUCIE PLANT
JOB NUMBER SA-737



BORING NUMBER AG 104
ST. LUCIE PLANT
JOB NUMBER SA-737



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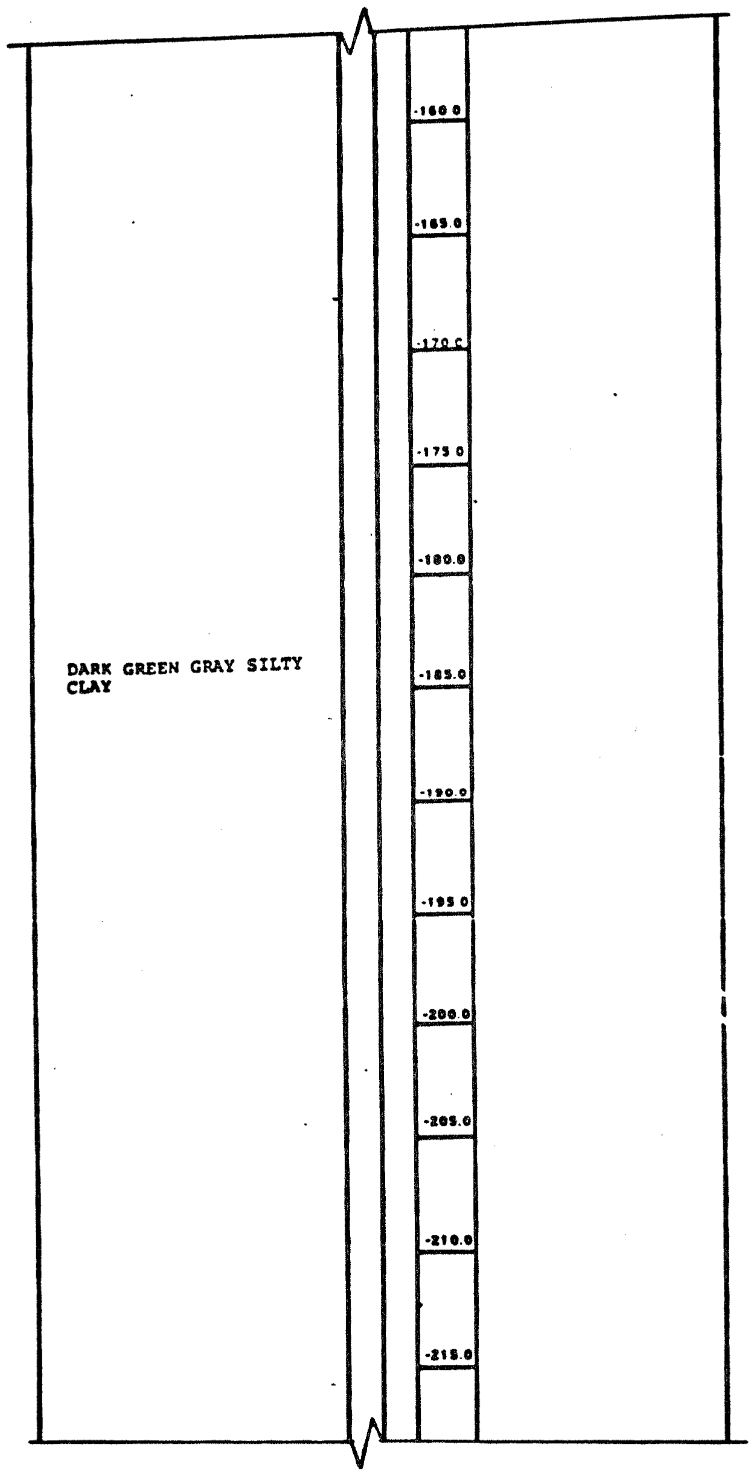
ST. LUCIE PLANT
JOB NUMBER SA-737



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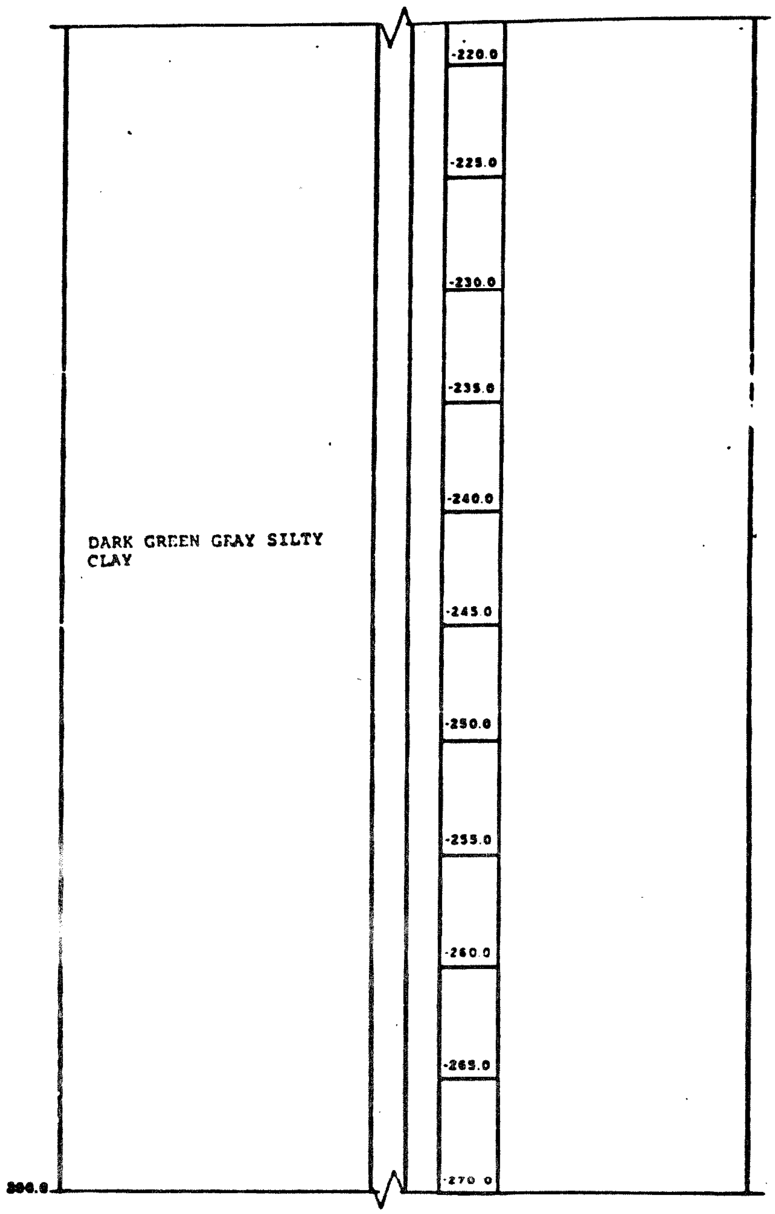
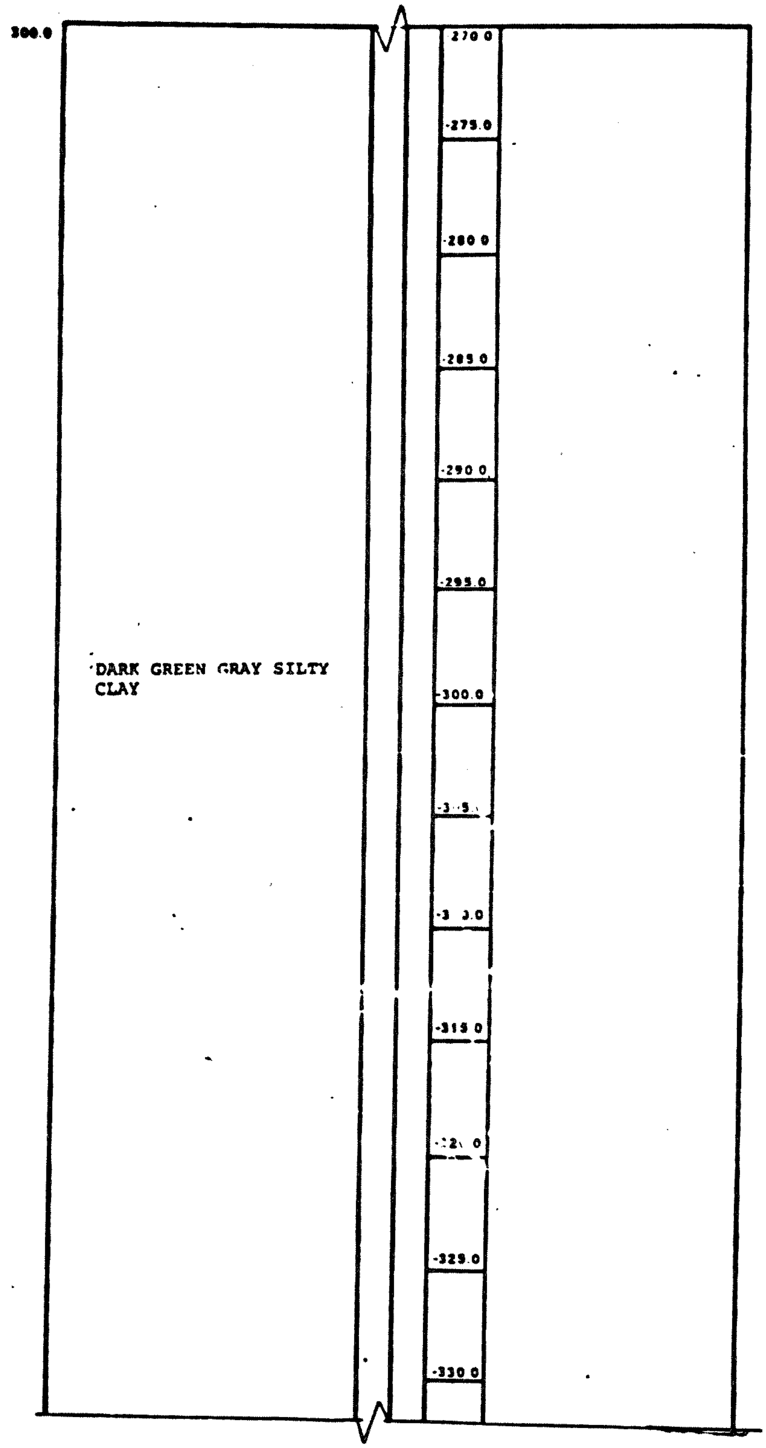
ST. LUCIE PLANT
JOB NUMBER SA-737



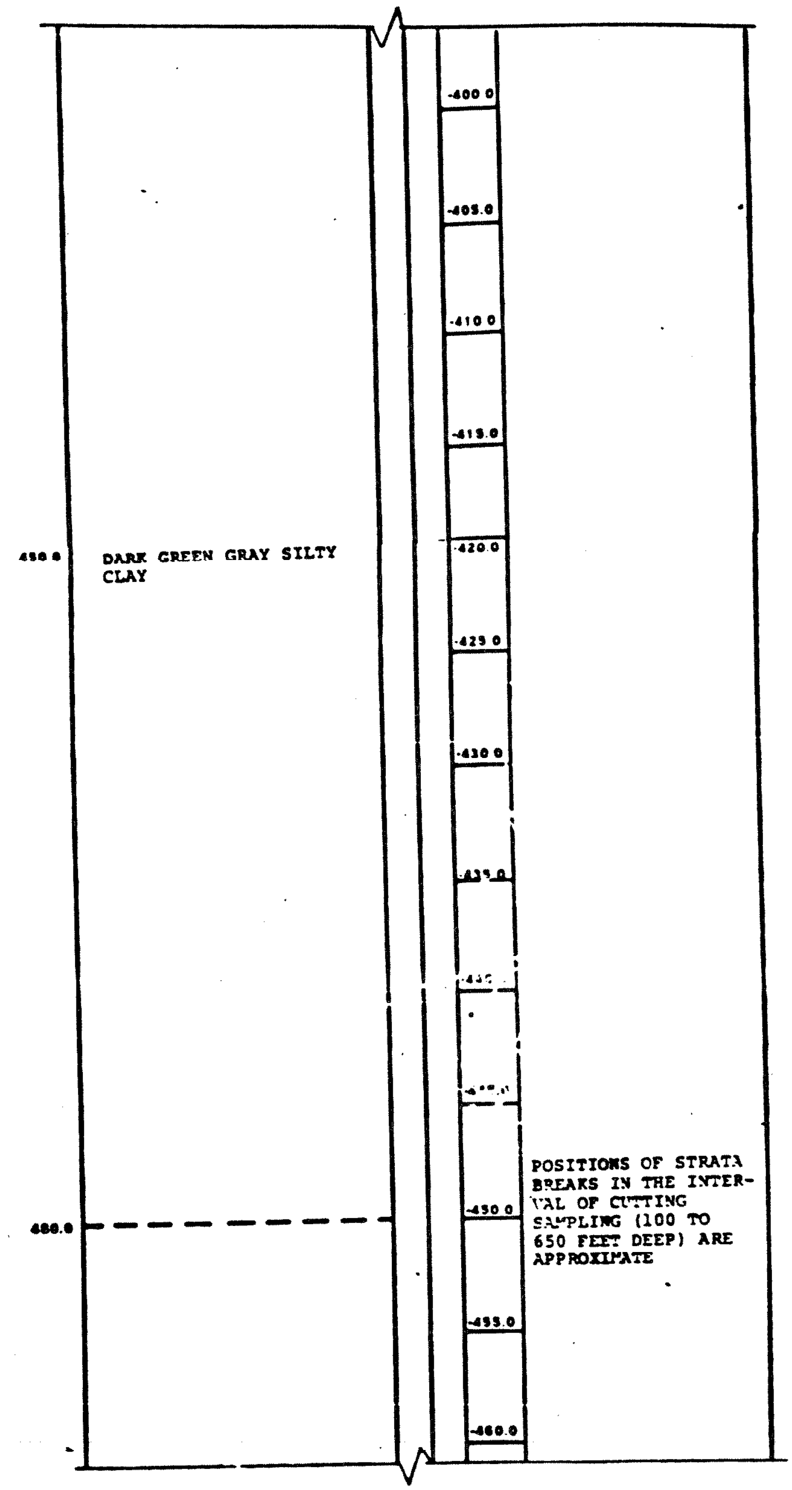
BORING NUMBER AG 104
ST. LUCIE PLANT
JOB NUMBER SA-737



BORING NUMBER AG 104
ST. LUCIE PLANT
JOB NUMBER SA-737

Description	Elevation
	-335.0
	-340.0
	-345.0
	-350.0
	-355.0
	-360.0
DARK GREEN GRAY SILTY CLAY	
	-365.0
	-370.0
	-375.0
	-380.0
	-385.0
	-390.0
	-395.0

BORING NUMBER AG 104
ST. LUCIE PLANT
JOB NUMBER SA-737



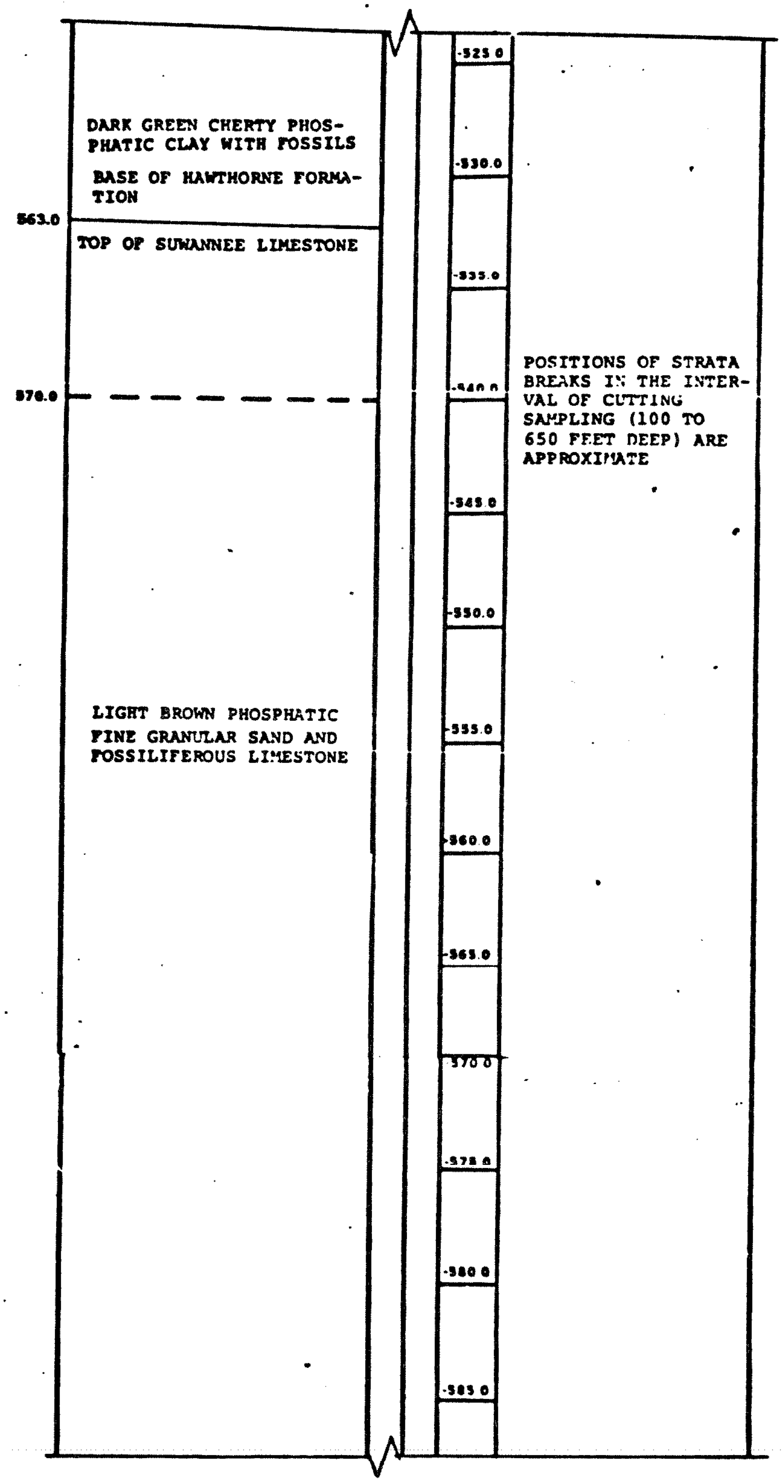
BORING NUMBER AG 104
ST. LUCIE PLANT
JOB NUMBER SA-737

-465.0
-470.0
-475.0
-480.0
-485.0

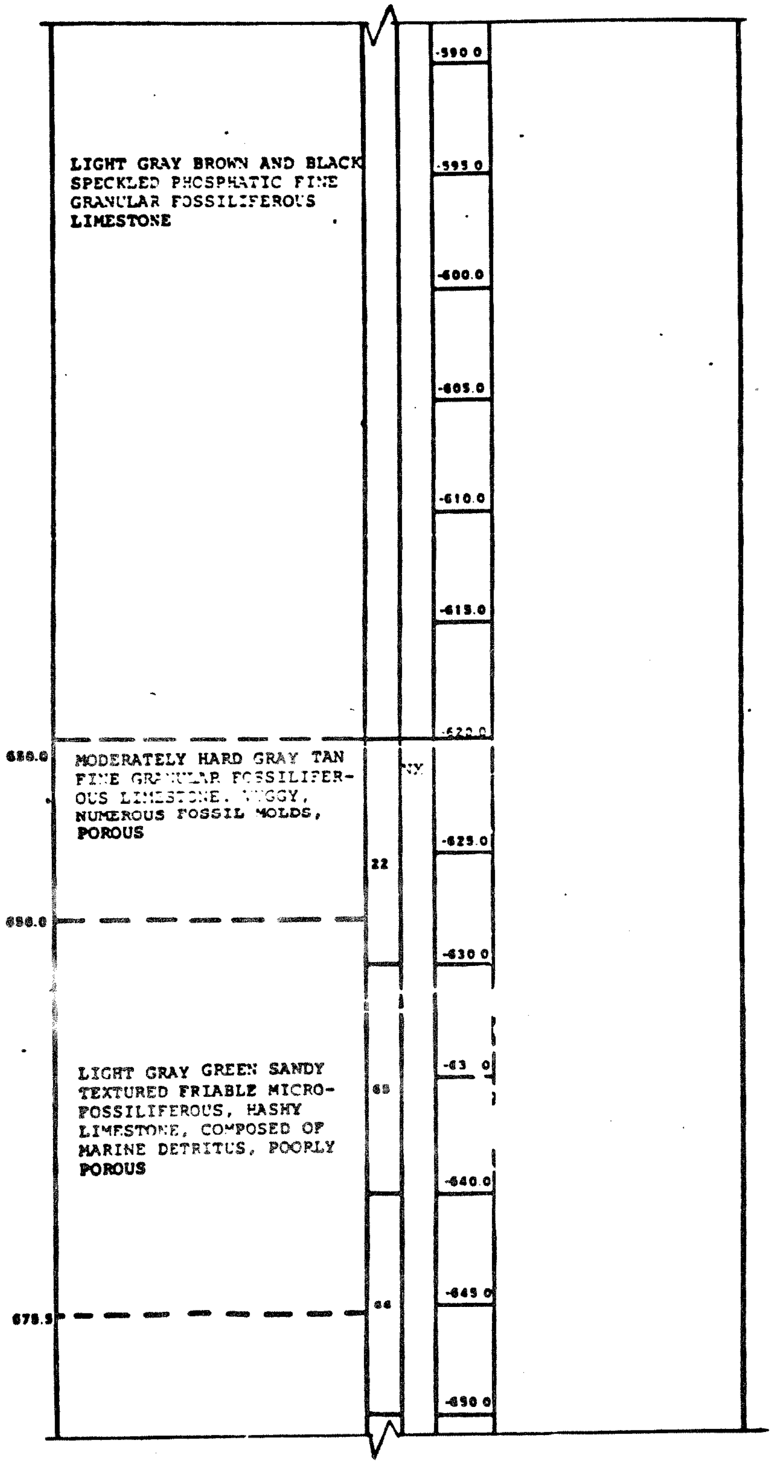
DARK GREEN CHERTY PHOS-
PHATIC CLAY WITH FOSSILS

-490.0
-495.0
-500.0
-505.0
-510.0
-515.0
-520.0

BORING NUMBER AG 104
ST. LUCIE PLANT
JOB NUMBER SA-737



BORING NUMBER AG 104
ST. LUCIE PLANT
JOB NUMBER SA-737



BORING NUMBER AG 104
ST. LUCIE PLANT
JOB NUMBER SA-737

Depth	Description	Recovery	Elevation
	VERY SOFT TO SOFT LIGHT TAN TO CREAM SILTY TEXTURED CHALKY LIMESTONE, MODERATELY FOSSILIFEROUS, SOME VUGS	53	-655.0
	BECOMES MORE GRANULAR DARK GRAY GREEN GLAUCONITIC MOTTLED AND TAN CHALKY LIMESTONE		-660.0
		96	-665.0
			-670.0
		28	-675.0
	BASE OF SUWANNEE LIMESTONE		
708.0	TOP OF OCALA LIMESTONE		-680.0
	MODERATELY HARD TAN FINE TO MEDIUM GRANULAR GRAY TO DARK GRAY MOTTLED FOSSILIFEROUS LIMESTONE, WITH FOSSIL MOLDS, POROUS	17	-685.0
			-690.0
72 ..0	BORING TERMINATED		
			-695.0

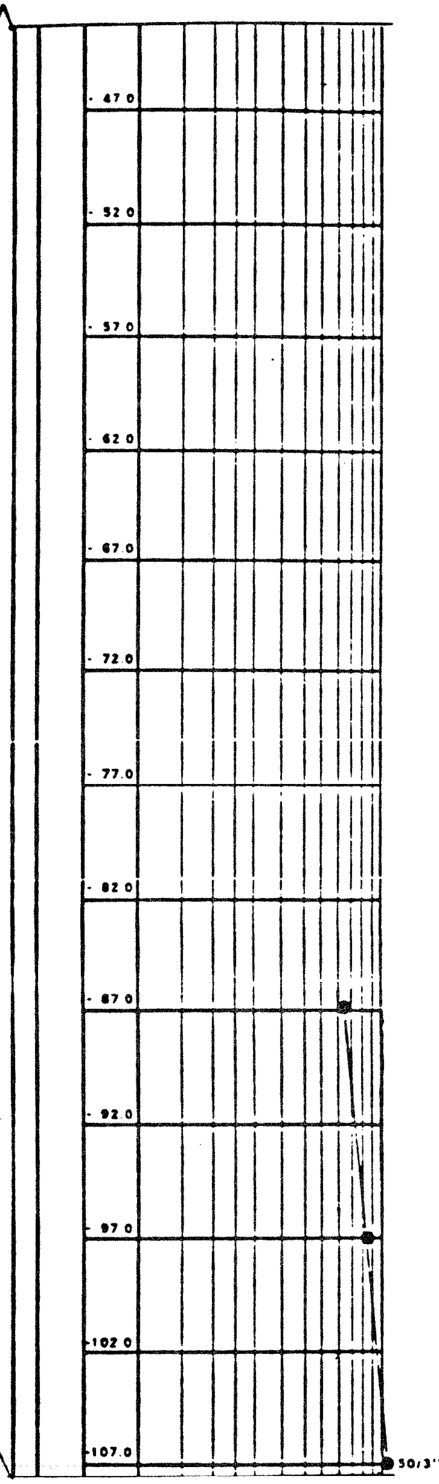
BORING NUMBER AG 104
ST. LUCIE PLANT
JOB NUMBER SA-737

TEST BORING RECORD

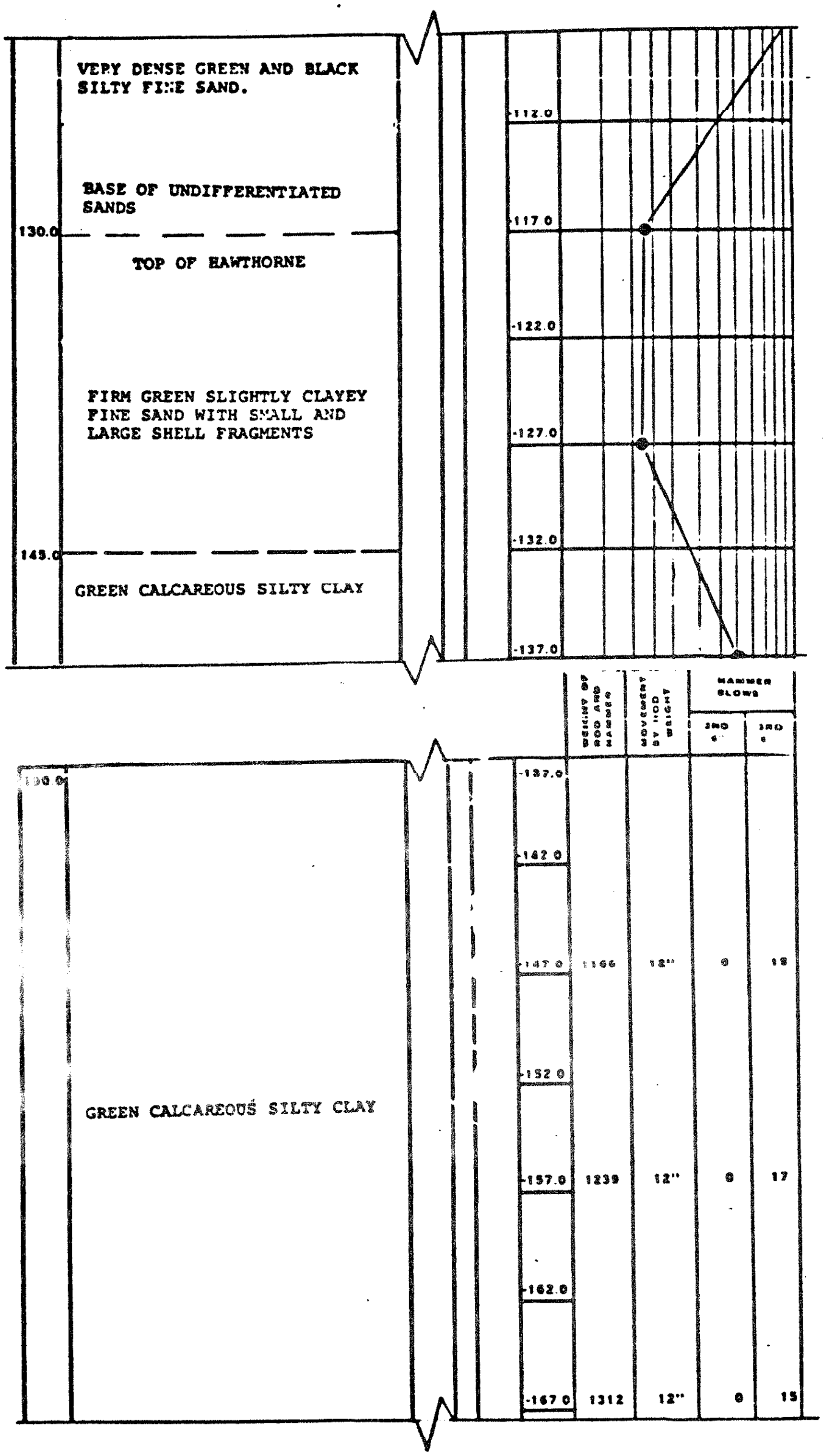
DEPTH FEET	DESCRIPTION	ELEV.	PENETRATION-BLOWS PER FOOT 0 5 10 15 20 30 40 60 80 100
0.0		13.0	
		8.0	
		3.0	
		-2.0	
		-7.0	
	WASH BORING FROM 0 - 98.5 FEET. NO SAMPLES TAKEN.	-12.0	
		-17.0	
		-22.0	
		-27.0	
		-32.0	
		-37.0	
		-42.0	

BORING NUMBER AG 105
ST. LUCIE PLANT
JOB NUMBER SA-737

Depth	Description
	WASH BORING FROM 0 - 98.5 FEET. NO SAMPLES TAKEN.
98.5	VERY DENSE GRAY CALCAREOUS SAND AND CEMENTED SANDY LIME-STONE FRAGMENTS WITH NUMEROUS SHELL FRAGMENTS.
107.0	VERY DENSE LIGHT GREEN AND BLACK SPECKLED SLIGHTLY CAL-CAREOUS SILTY SAND WITH SHELL FRAGMENTS.
115.0	VERY DENSE GREEN AND BLACK SILTY FINE SAND.



BORING NUMBER AG 105
ST. LUCIE PLANT
JOB NUMBER SA-737



BORING NUMBER AG 105
ST. LUCIE PLANT
JOB NUMBER SA-737

GREEN CALCAREOUS SILTY CLAY

-172.0				
-177.0	1385	12"	0	17
-182.0				
-187.0	1458	12"	0	13
-192.0				
-197.0	1531	18"	0	0
-202.0				
-207.0	1604	12"	0	6
-212.0				
-217.0	1677	18"	0	0
-222.0				
-227.0	1780	18"	0	0

BORING NUMBER AG 105
ST. LUCIE PLANT
JOB NUMBER SA-737

		-232.0				
	GREEN CALCAREOUS SILTY CLAY	-237.0	1823	18"	0	0
		-242.0				
255.0	DARK GREEN SLIGHTLY CLAYEY SLIGHTLY SILTY VERY FINE SAND					
		-247.0	1896	18"	0	0
		-252.0				
265.0	GREEN CALCAREOUS SILTY CLAY					
		-257.0	1969	18"	0	0
		-262.0				
275.0						
		-267.0	2042	18"	0	0
		-272.0				
	GREEN CALCAREOUS SLIGHTLY CLAYEY SILT					
		-277.0	2115	18"	0	0
		-282.0				
		-287.0	2188	18"	0	0
300.0						
305.0		-292.0				
	GREEN SLIGHTLY CALCAREOUS SLIGHTLY SILTY CLAY	-297.0	2261	18"	0	0

BORING NUMBER AG 105
ST. LUCIE PLANT
JOB NUMBER SA-737

	GREEN SLIGHTLY CALCAREOUS SLIGHTLY SILTY CLAY					
318.0		-302.0				
		-307.0	2334	18"	0	0
	GREEN CALCAREOUS SILTY CLAY					
		-312.0				
		-317.0	2407	18"	0	0
		-322.0				
335.0						
		-327.0	2480	18"	0	0
		-332.0				
		-337.0	2553	18"	0	0
	GREEN SLIGHTLY CALCAREOUS SLIGHTLY CLAYEY SILT	-342.0				
		-347.0	2626	18"	0	0
		-352.0				
		-357.0	2699	18"	0	0

BORING NUMBER AG 105
ST. LUCIE PLANT
JOB NUMBER SA-737

399.0

GREEN SLIGHTLY CALCAREOUS
SLIGHTLY CLAYEY SILT

GREEN SILT

418.0

GREEN SLIGHTLY CALCAREOUS
SILTY CLAY WITH NUMEROUS
PHOSPHATE NODULES AND LARGE
DARK GRAY CHERT PEBBLES

-362.0				
-367.0	2772	18"	0	0
-372.0				
-377.0	2845	18"	0	0
-382.0				
-387.0	2918	18"	0	0
-392.0				
-397.0	2991	18"	0	0
-402.0				
-407.0	3064	18"	0	0
-412.0				
-417.0	3137	12"	0	-

BORING NUMBER AG 105
ST. LUCIE PLANT
JOB NUMBER SA-737

435.0

LIGHT GREEN AND TAN SPECKLED VERY CALCAREOUS SLIGHTLY CLAYEY SILT AND FINE SAND WITH SOME SHELL FRAGMENTS

450.0

LIGHT GREEN AND TAN SPECKLED VERY CALCAREOUS SLIGHTLY CLAYEY SILT AND FINE SAND WITH SOME SHELL FRAGMENTS

465.0

LIGHT GREEN SLIGHTLY CALCAREOUS SLIGHTLY SILTY CLAY WITH SOME SMALL SHELL FRAGMENTS

485.0

LIGHT GREEN CALCAREOUS SILTY CLAYEY FINE SAND WITH SOME PHOSPHATE GRANULES

-422.0				
-427.0	3210	18"	0	0
-432.0				
-43[illegible]	3823	18"	0	0
-442.0				
-447.0	3356	18"	0	0
-452.0				
-457.0	3429	18"	0	0
-462.0				
-467.0	3502	18"	0	0
-472.0				
-477.0	3575	18"	0	0
-482.0				
-487.0	3648	18"	0	0

BORING NUMBER AG 105
ST. LUCIE PLANT
JOB NUMBER SA-737

DEPTH FT.	DESCRIPTION	CORE %	BIT SIZE	ELEV.	REMARKS
500.0			NX	-487.0	
	SOFT GREEN TO DARK GREEN SANDY TO CLAYEY TEXTURED SILTSTONE, MICROFOSSILI- FEROUS WITH INTERBEDDED ZONES OF DARK GRAY CHERT BOULDERS AND THIN BEDS OF ABUNDANT PHOSPHATE NODULES OCCASIONALLY LAMINATED, GENERALLY NON-POROUS	77		-492.0	
		44		-497.0	
		57		-502.0	
		92		-507.0	
				-512.0	
		21		-517.0	
533.0	SOFT TO MEDIUM SOFT GREEN, TAN AND BLACK SPECKLED CALCAREOUS SANDY SILT- STONE, PHOSPHATIC, SOME SMALL FOSSILS	91		-522.0	
				-52[illegible]	
542.0	VERY SOFT DARK GREEN CAL- CAREOUS CLAY	41		-5[illegible]	
548.0	SOFT TO MEDIUM HARD GRAY TO GRAY TAN VERY PHOS- PHATIC FINE GRANULAR MODERATELY FRIABLE FOSSIL- IFEROUS LIMESTONE, SLIGHTLY POROUS			-537.0	
		50		-542.0	

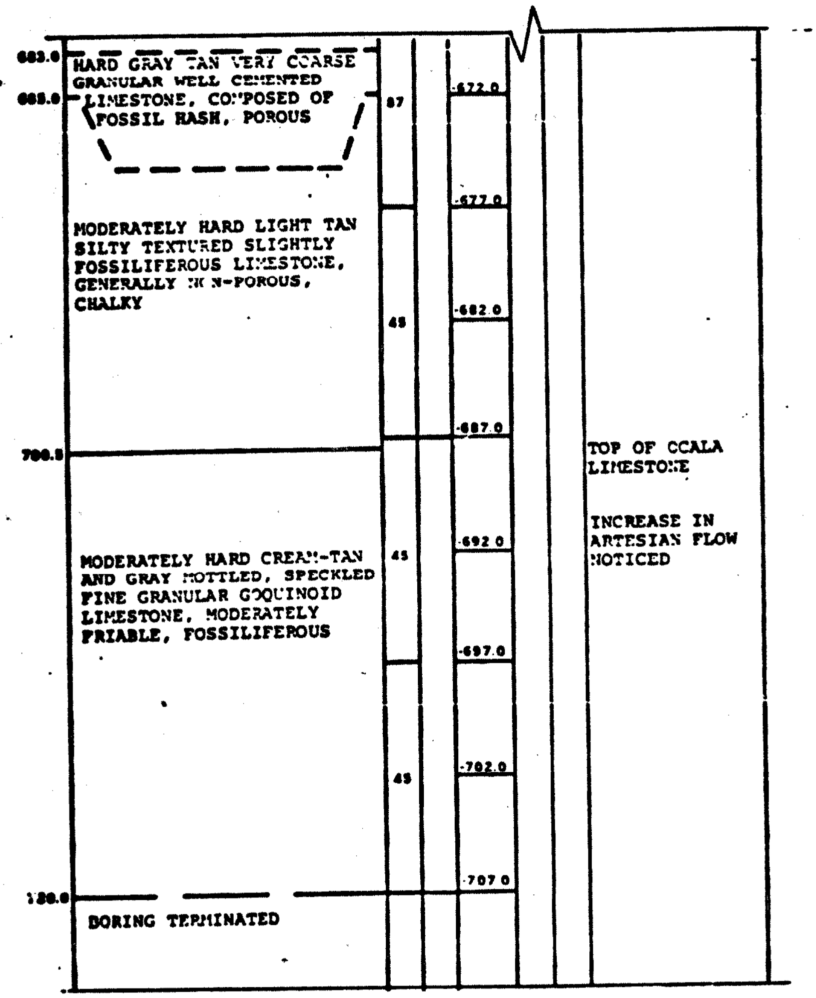
BORING NUMBER AG 105
ST. LUCIE PLANT
JOB NUMBER SA-737

Depth	Description	Recovery	Elevation	Remarks
			-547.0	
560.0	HARD TO MODERATELY SOFT GRAY GREEN SANDY TO SILTY TEXTURED SLIGHTLY PHOS-PHATIC FOSSILIFEROUS LIMESTONE, POROUS, SMALL VUGS	13	-552.0	TOP OF SUWANNEE LIMESTONE
			-557.0	
	GRADES INTO A SOFT GRAY GREEN FRIABLE SILTY TEXTURED LIMESTONE, MICRO-FOSSILIFEROUS, POORLY POROUS, SOME SMALL BLACK SPECKS	49	-562.0	
			-567.0	
		100	-572.0	
		60		
589.0	VERY SOFT DARK GREEN SILTY CLAY		-577.0	
590.0	MODERATELY HARD TO HARD GRAY TAN SANDY TO SILTY TEXTURED FOSSILIFEROUS SOMEWHAT FRIABLE TO WELL CEMENTED LIMESTONE, NUMEROUS FOSSIL MOLDS AND CASTS, MODERATELY TO VERY POROUS, OCCASIONALLY VUGGY, OCCASIONAL THIN COQUINOID ZONES	61	-582.0	
			-587.0	
		50	-592.0	
			-597.0	
614.0	MODERATELY SOFT GRAY GREEN FRIABLE SILTY TEXTURED LIMESTONE, GENERALLY POROUS TO NON-POROUS, SOME FOSSIL CASTS	100	-602.0	
			-607.0	

BORING NUMBER AG 105
ST. LUCIE PLANT
JOB NUMBER SA-737

Depth	Description	Recovery (%)	Elevation
		100	-612.0
628.0	VERY HARD GRAY TAN FINE GRANULAR, VERY WELL CEMENTED LIMESTONE, VERY VUGGY, NUMEROUS FOSSIL CASTS, VERY POROUS		-617.0
632.0		50	-622.0
	MODERATELY SOFT GRAY TAN FRIABLE SILTY TEXTURED LIMESTONE, SOME FOSSILS, VERY FINE GRANULAR POORLY POROUS		-627.0
		100	-632.0
			-637.0
652.0	HARD LIGHT TAN-WHITE CO-QUINOID FINE GRANULAR, VERY VUGGY LIMESTONE, NUMEROUS CASTS AND MOLDS VERY POROUS	100	-642.0
6[illegible]3.0	MODERATELY SOFT GRAY GREEN FRIABLE SILTY TEXTURED LIMESTONE, POORLY POROUS. FINE GRANULAR, SLIGHTLY FOSSILIFEROUS		-647.0
		39	-652.0
	MODERATELY SOFT GRAY GREEN FRIABLE SILTY TEXTURED LIMESTONE, POORLY POROUS, FINE GRANULAR, SLIGHTLY FOSSILIFEROUS		-657.0
			-662.0
		100	-667.0

BORING NUMBER AG 105
ST. LUCIE PLANT
JOB NUMBER SA-737



BORING NUMBER AG 105
ST. LUCIE PLANT
JOB NUMBER SA-737

TEST BORING RECORD

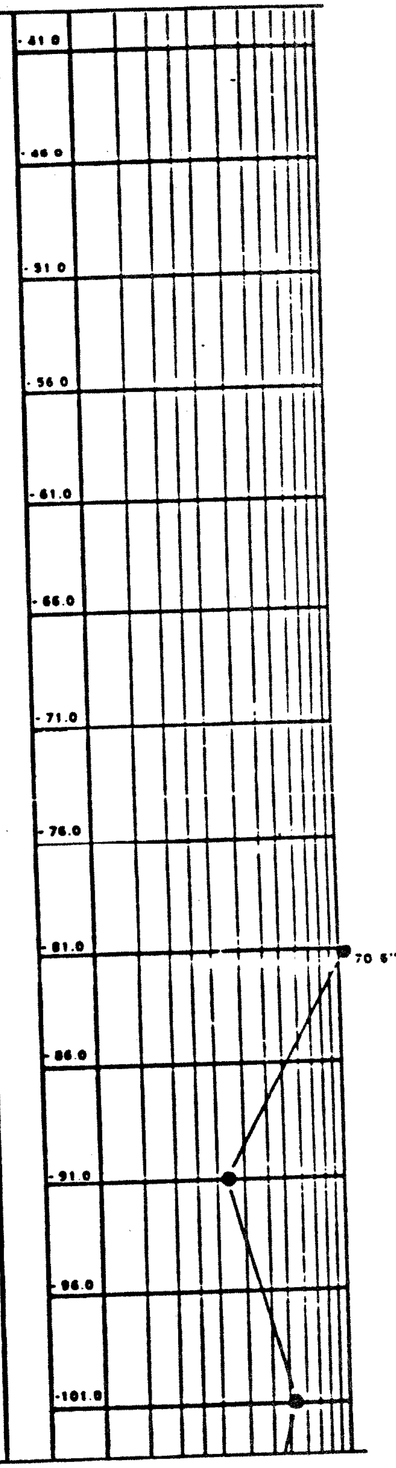
DEPTH FEET	DESCRIPTION	ELEV.	PENETRATION-BLOWS PER FOOT
0.0		19.0	
		14.0	
		9.0	
		4.0	
		-1.0	
	WASH BORING [illegible] TO [illegible] 38.0 FEET [illegible]	-6.0	
		-11.0	
		-16.0	
		-21.0	
		-26.0	
		-31.0	
		-36.0	

BORING NUMBER AG 106
ST. LUCIE PLANT
JOB NUMBER SA-737

WASH BORING FROM 0 TO 98.5
FEET. NO SAMPLES TAKEN.

VERY DARK GRAY AND BLACK
SPECKLED CALCAREOUS SLIGHTLY
SILTY FINE SAND AND CEMENTED
SANDY LIMESTONE WITH SHELL
FRAGMENTS

VERY FIRM GRAY AND BLACK
SPECKLED CALCAREOUS SLIGHTLY
SILTY FINE SAND



BORING NUMBER AG 106
ST. LUCIE PLANT
JOB NUMBER SA-737

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	BASE OF [illegible]DIFFERENTIATED SANDS
123.0	TOP OF HAWTHORN[illegible] FORMATION
	VER[illegible] STIFF GREE[illegible] SLIGHTLY CA[illegible]REOUS FINE SAND[illegible] CLAY
125.0	[illegible] DENSE TO [illegible]ARD GREE[illegible] CA[illegible]ARE[illegible]S FI[illegible] SA[illegible]Y CLAY WI[illegible] SHELL FRA[illegible]MENTS

	-1?5.0	
	-1?1.0	
	-1?4.0	
	-1?7.0	
	-1?8.0	
	-1?1.0	

		[illegible]	[illegible]	HAMMER BLOWS	
				[illegible]	[illegible]
	-13?.0				
[illegible]	-1?8.0				
	-1?[illegible]	66	[illegible]	[illegible]	[illegible]
	-14?.0				
HARD TO VERY HARD GREEN CALCAREOUS SILTY CLAY	-151.0	1239	0	24	21
	-153.0				
	-161.0	1312	0	23	30

BORING NUMBER AG 106
ST. LUCIE PLANT
JOB NUMBER SA-737

185.0

HARD TO VERY HARD GREEN
CALCAREOUS SILTY CLAY WITH
SOME SHELL FRAGMENTS

245.0

-166.0				
-171.0	1392	0	22	27
-176.0				
-181.0	1458	0	30	26
-186.0				
-191.0	1531	0	22	27
-196.0				
-201.0	1604	0	18	23
-206.0				
-211.0	1677	0	25	33
-216.0				
-221.0	1750	0	27	34
-226.0				

BORING NUMBER AG 106
ST. LUCIE PLANT
JOB NUMBER SA-737

	VERY HARD GREEN SLIGHTLY CALCAREOUS SILTY CLAY	-231.0	1823	0	28	36
		-236.0				
		-241.0	1895	0	50/5"	-
		-246.0				
		-251.0	1968	0	50/4"	-
		-256.0				
		-261.0	2041	0	50/3"	-
		-266.0				
		-271.0	2114	0	50/6"	-
		-276.0				
		-281.0	2187	0	42	50/2"
300.0						
		-286.0				
		-291.0	2250	0	37	50

BORING NUMBER AG 106
ST. LUCIE PLANT
JOB NUMBER SA-737

	VERY HARD GREEN SLIGHTLY CALCAREOUS SILTY CLAY	-296 0				
		-301 0	2333	10"	10/2"	35
		-306 0				
		-311 0	2406	11"	5/1"	60
		-316 0				
		-321.0	2479	12"	0	46
		-326 0				
345.0	HARD TO VERY HARD GREEN SILTY CLAY					
		-331.0	2552	18"	0	0
		-336.0				
		-341.0	2624	6"	15	50/4"
		-346 0				
		-351.0	2697	6"	5	50/3"
		-356 0				
375.0						
		-361 0	2770	18"	0	0

BORING NUMBER AG 106
ST. LUCIE PLANT
JOB NUMBER SA-737

Depth	Description	Elevation				
	GREEN SLIGHTLY CALCAREOUS SILTY CLAY WITH SOME LARGE SHELL FRAGMENTS	-366.0				
		-371.0	2843	18"	0	0
395.0		-376.0				
	GREEN SLIGHTLY CALCAREOUS SILTY CLAY WITH FINE TO MEDIUM SAND SEAMS AND PHOSPHATE NODULES	-381.0	2916	18"	0	0
405.0		-386.0				
	LIGHT GREEN CALCAREOUS SILTY CLAY WITH FINE SAND SEAMS	-391.0	2989	18"	0	0
415.0		-396.0				
	LIGHT GREEN SANDY SILTY CLAY WITH SOME LARGE GRAY CHERT PEBBLES	-401.0	3062	18"	0	0
425.0		-406.0				
	LIGHT GREEN CALCAREOUS CLAYEY SILTY SAND WITH SHELL FRAGMENTS AND SOME SMALL PHOSPHATE GRANULES	-411.0	3134	18"	0	0
435.0		-416.0				
		-421.0	3208	18"	0	0

BORING NUMBER AG 106
ST. LUCIE PLANT
JOB NUMBER SA-737

	LIGHT GREEN SLIGHTLY CAL-CAREOUS SILTY CLAY						
			-426.0				
450.0			-431.0	3281	18"	0	0
	LIGHT GREEN SLIGHTLY CALCAREOUS SILTY CLAY						
			-436.0				
455.0							
			-441.0	3353	-18"	0	0
	LIGHT GREEN CALCAREOUS SLIGHTLY CLAYEY FINE SAND WITH SOME PHOSPHATE GRANULES	41 NX					
			-446.0				
			-451.0				
			-456.0				
475.0							
	LIGHT GREEN SLIGHTLY CAL-CAREOUS SILTY CLAY WITH SOME LARGE DARK GRAY CHERT PEBBLES		-461.0	3499	18"	0	0
			-466.0				
485.0							
	LIGHT GREEN CALCAREOUS SLIGHTLY CLAYEY SANDY SILT		-471.0	3572	18"	0	0
			-476.0				
495.0							
	GREEN CALCAREOUS CLAYEY SANDY SILT		-481.0	3645	18"	0	0

BORING NUMBER AG 106
ST. LUCIE PLANT
JOB NUMBER SA-737

Depth	Description	Elev.				
		-486.0				
505.0	LIGHT GREEN CALCAREOUS SILTY CLAY					
510.0		-491.0	3718	18"	0	0

DEPTH FT.	DESCRIPTION	CORE %	BIT SIZE	ELEV.	REMARKS
510.0	SOFT GRAY GREEN SILTY TO FINE SANDY TEXTURED MODERATELY PHOSPHATIC LIMESTONE	85	NX	-491.0 -496.0 -501.0	
520.5	MODERATELY HARD GRAY GREEN SILTY TEXTURED LIMESTONE NUMEROUS MOLDS AND CASTS	100			
523.0	VUGGY, POROUS, SLIGHTLY PHOSPHATIC MODERATELY SOFT GRAY GREEN			-506.0	
526.0	SILTY TEXTURED SLIGHTLY PHOSPHATIC LIMESTONE MODERATELY SOFT LIGHT GRAY SILTY TO SANDY TEXTURED SLIGHTLY FOSSILIFEROUS LIMESTONE, VERY PHOSPHATIC, SMALL AND LARGE PHOSPHATE PEBBLES, GENERALLY NON-POROUS	100		-511.0 -516.0	
238.0	HARD LIGHT GRAY COQUINOID LIMESTONE, VUGGY, POROUS, ALTERNATING WITH MODERATELY HARD SANDY TO SILTY TEXTURED LIMESTONE, MODERATELY TO SLIGHTLY POROUS, SOME PHOSPHATE NODULES.	90		-521.0 -526.0	TOP OF SUWANNEE LIMESTONE
	GRADUALLY BECOMES SOFT GRAY GREEN PHOSPHATIC SANDY TEXTURED LIMESTONE, POORLY POROUS, SLIGHTLY FOSSILIFEROUS.	100		-531.0 -536.0	

BORING NUMBER AG 106
ST. LUCIE PLANT
JOB NUMBER SA-737

560.0

MODERATELY HARD TO MODERATELY SOFT LIGHT GRAY SANDY TEXTURED FOSSILIFEROUS LIMESTONE, SOME SMALL VUGS, NUMEROUS FOSSIL CASTS.

BECOMES MODERATELY HARD TO HARD CHALKY TO VERY FINE GRANULAR COQUINOID LIMESTONE, VERY VUGGY, POROUS, MANY FOSSIL MOLDS AND CASTS.

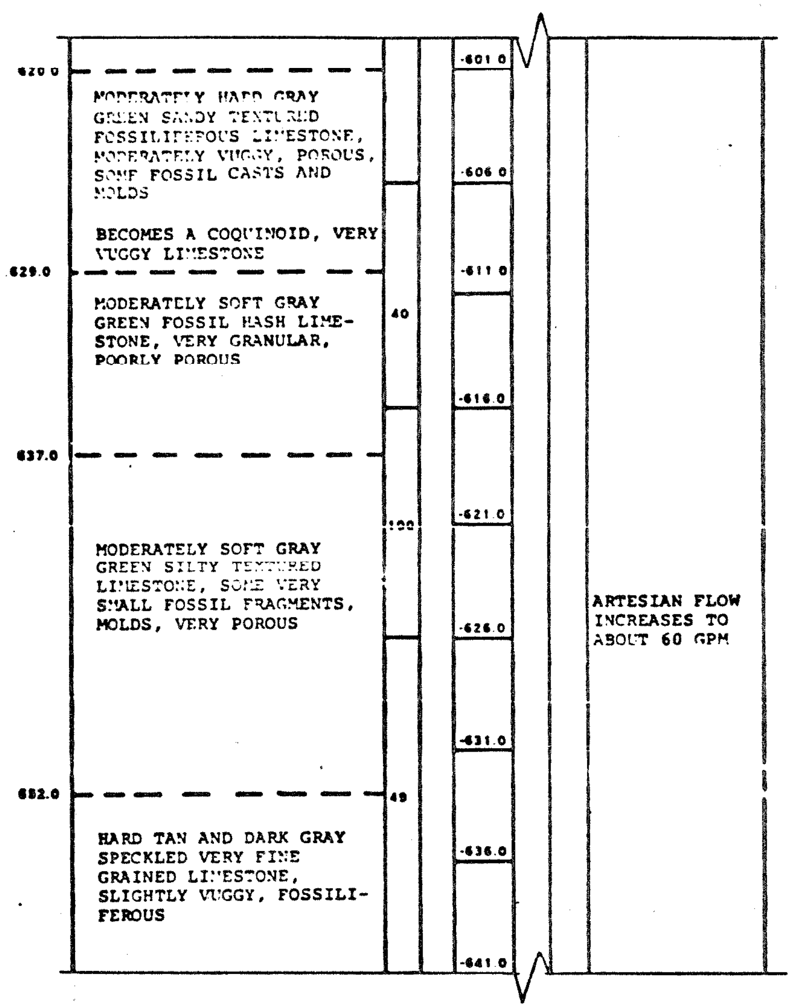
579.5

MODERATELY SOFT TO SOFT SANDY TO SILTY TEXTURED, SLIGHTLY FOSSILIFEROUS LIMESTONE, POORLY POROUS, SOME ZONES OF FOSSIL MOLDS

Recovery	Elevation
100	-541.0
	-546.0
22	-551.0
	-556.0
50	-561.0
	-566.0
	-571.0
100	-576.0
	-581.0
45	-5[illegible]0
	-591.0
	-596.0
23	

ARTESIAN WATER FLOW APPROXIMATELY 30 GALLONS PER MINUTE (GPM) AT 610 FEET DEEP

BORING NUMBER AG 106
ST. LUCIE PLANT
JOB NUMBER SA-737



BORING NUMBER AG 106
ST. LUCIE PLANT
JOB NUMBER SA-737

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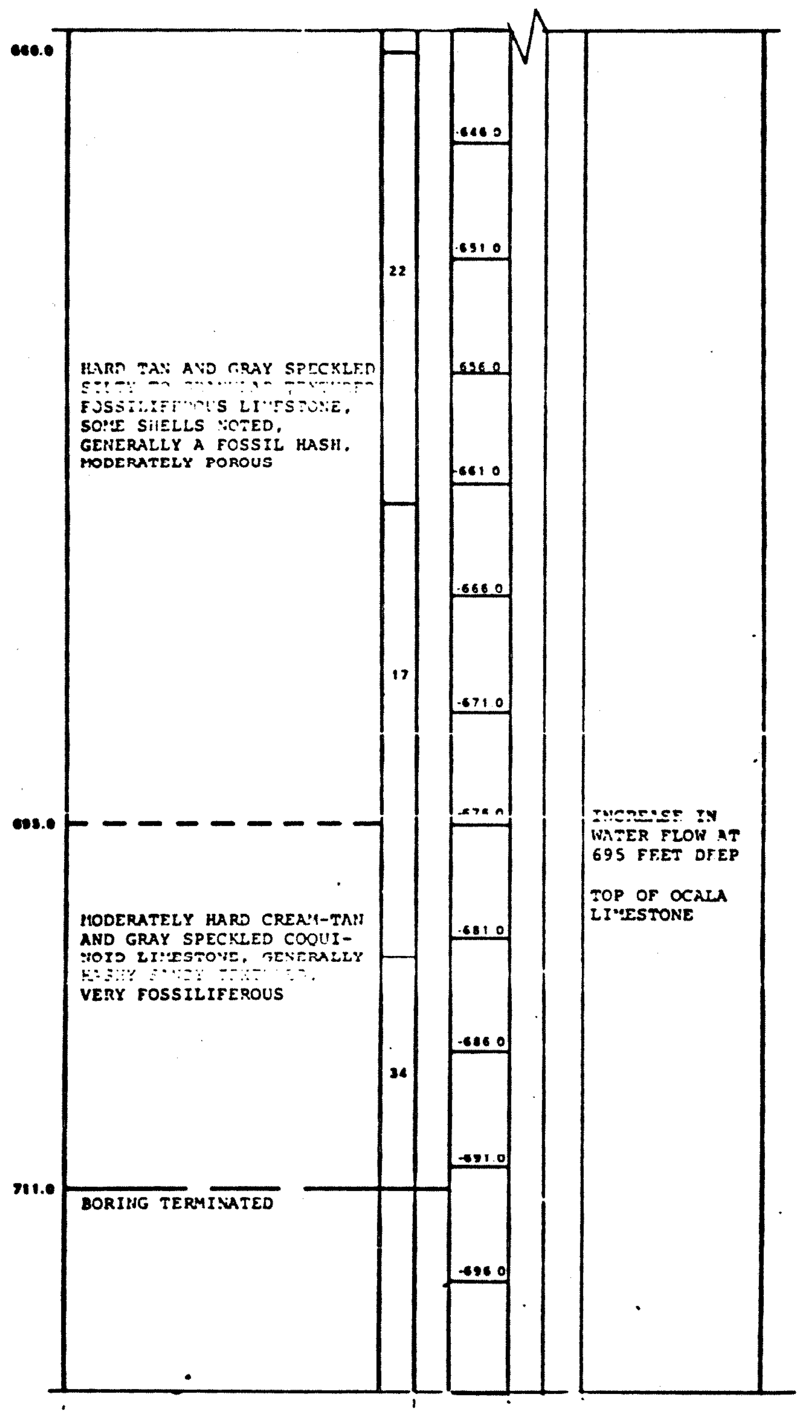

BORING NUMBER AG 106
ST. LUCIE PLANT
JOB NUMBER SA-737

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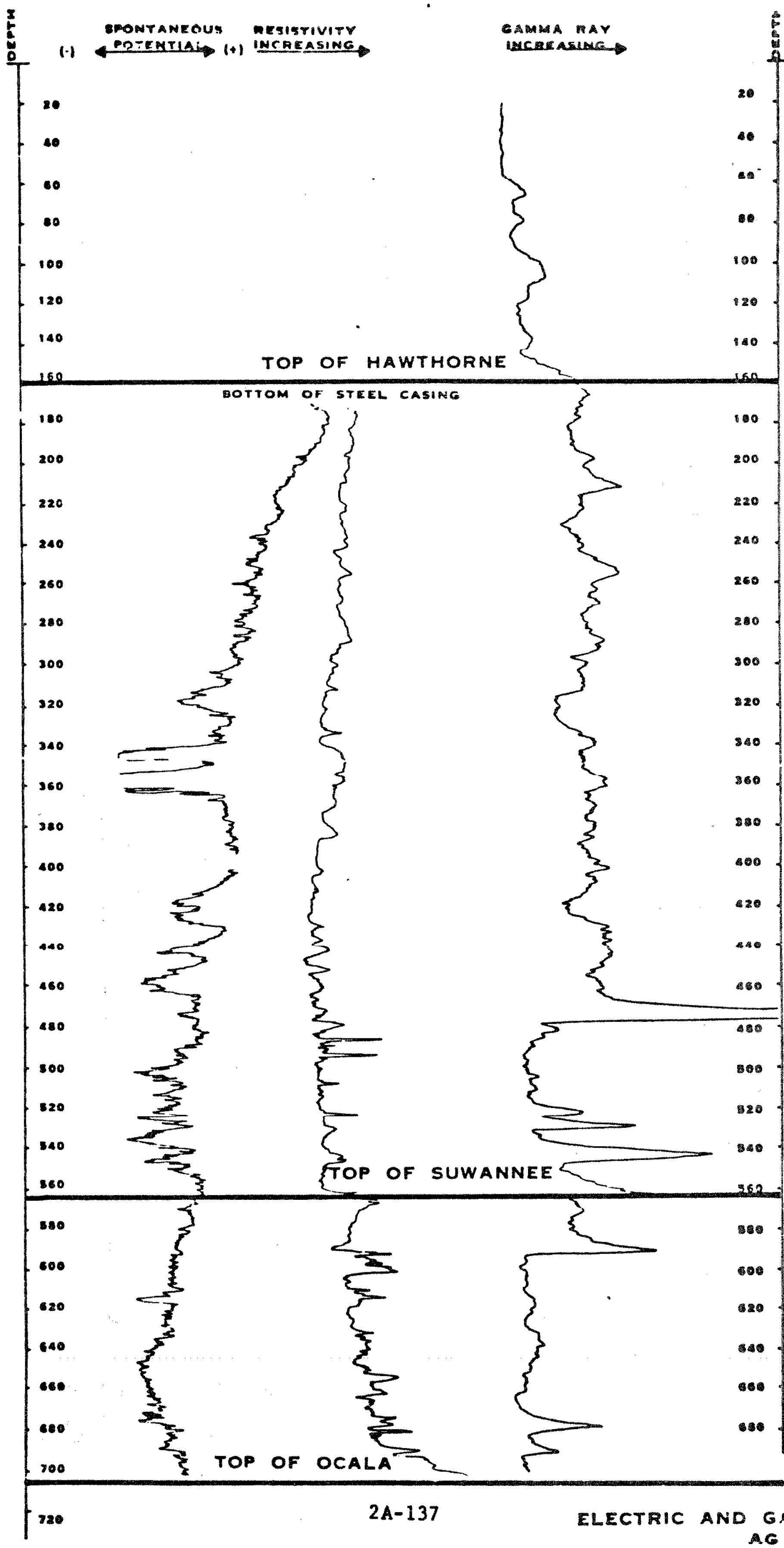
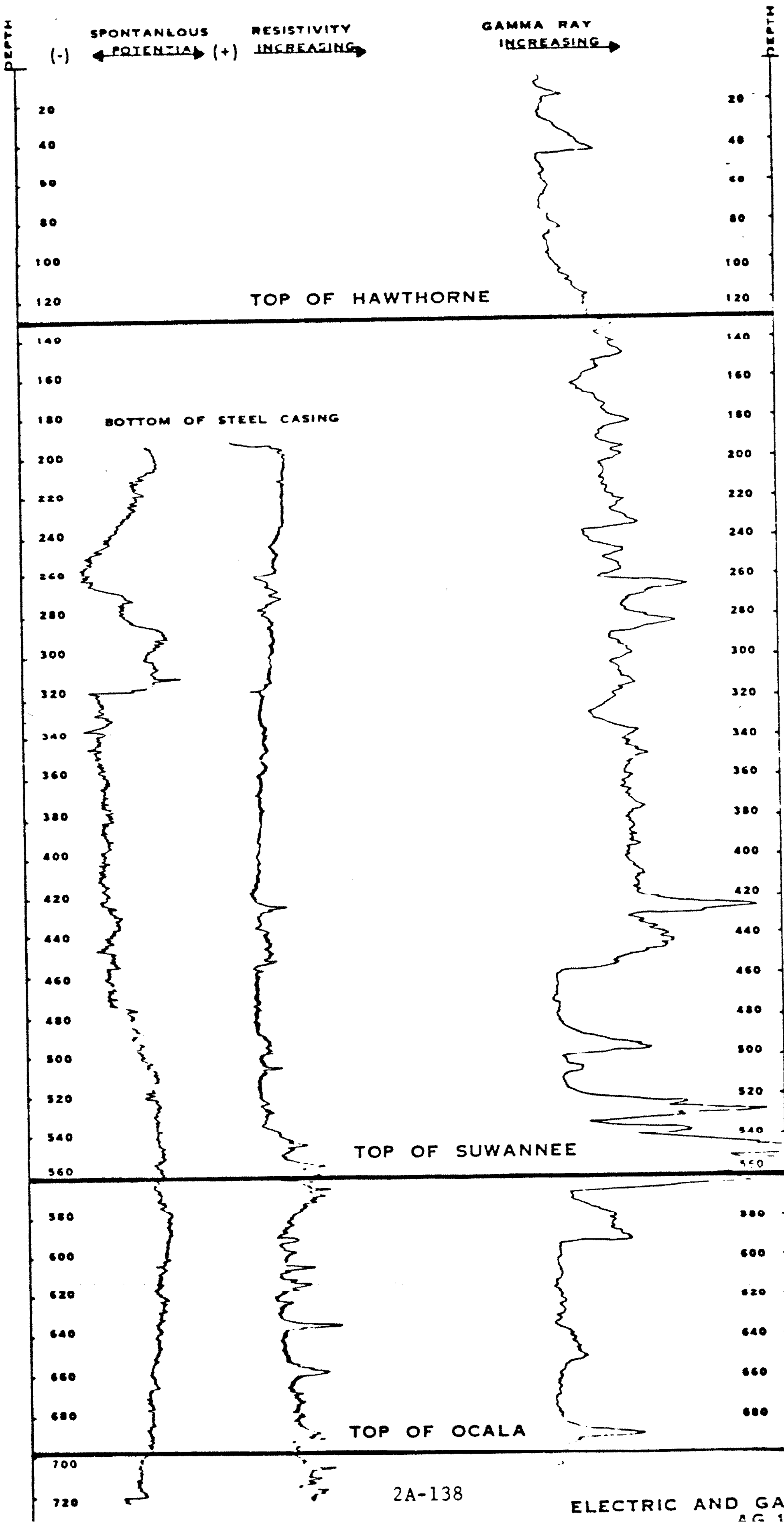


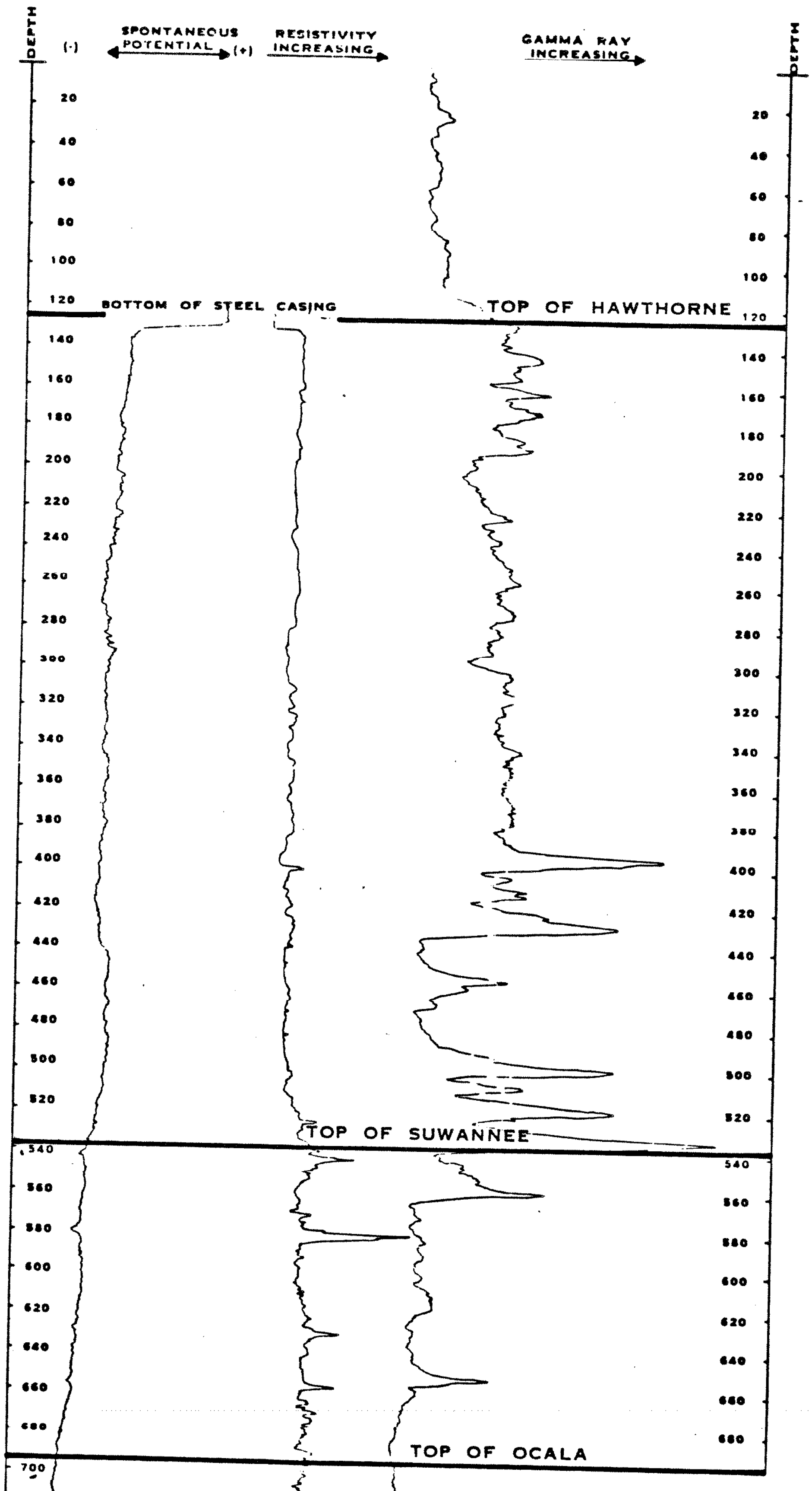
2A-137

ELECTRIC AND GAMMA RAY LOG
AG 104



2A-138

ELECTRIC AND GAMMA RAY LOG
AG 105



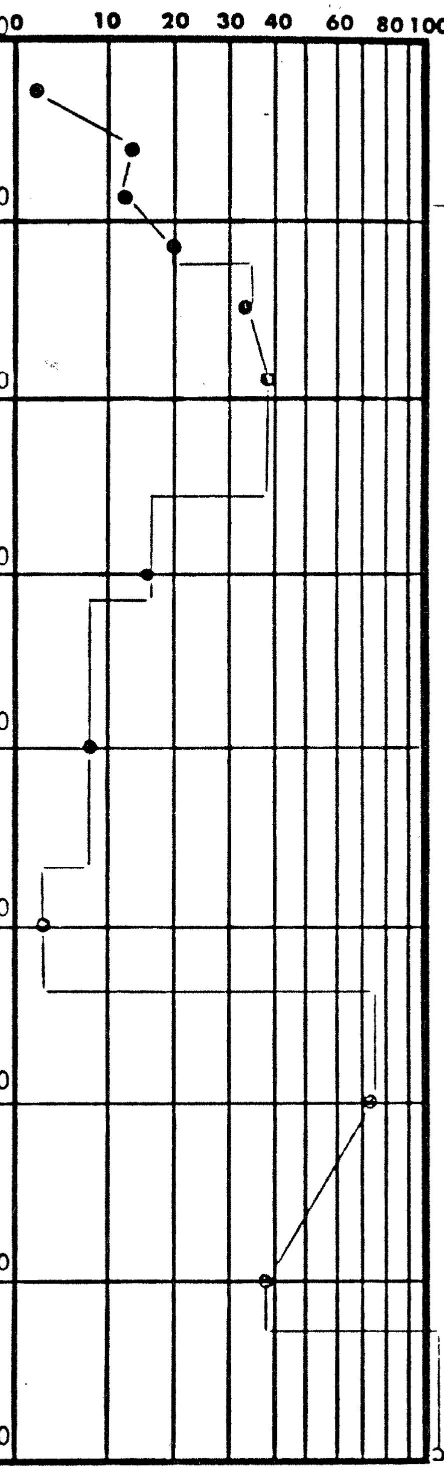
ELECTRIC AND GAMMA RAY LOG
AG 106

DEPTH FT.	DESCRIPTION	
0		
1.0	VERY LOOSE light grey fine	(1)
	VERY LOOSE to FIRM grey silty to slightly silty slightly clayey fine SAND	
5.0	FIRM grey fine SAND	
6.5	DENSE blue-grey silty slightly clayey fine SAND with shell fragments and some small silty clay seams	
13.0	FIRM blue-grey fine SAND and clayey silt SEAMS	
16.0	LOOSE to FIRM blue-grey slightly silty fine SAND with trace of silty clay lenses	
23.5	SOFT blue-grey silty fine slightly sandy CLAY with shell fragments	
27.0	DENSE to VERY DENSE grey slightly silty fine SAND with cemented fine sand seams	
33.5	DENSE grey slightly silty fine SAND	
36.5	FIRM to VERY FIRM tan fine SAND with some shell fragments and VERY DENSE grey cemented fine sand SEAMS	
40.0		

ELEV

+ 7.0
+ 2.0
- 3.0
- 8.0
-13.0
-18.0
-23.0
-28.0
-33.0

• PENETRATION-BLOWS PER FT.



TEST BORING RECORD

(Page 1 of 2 Pages)

BORING NO. SB-1

DATE DRILLED 1/5/72

JOB NO. J-1540

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

% ROCK CORE RECOVERY

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

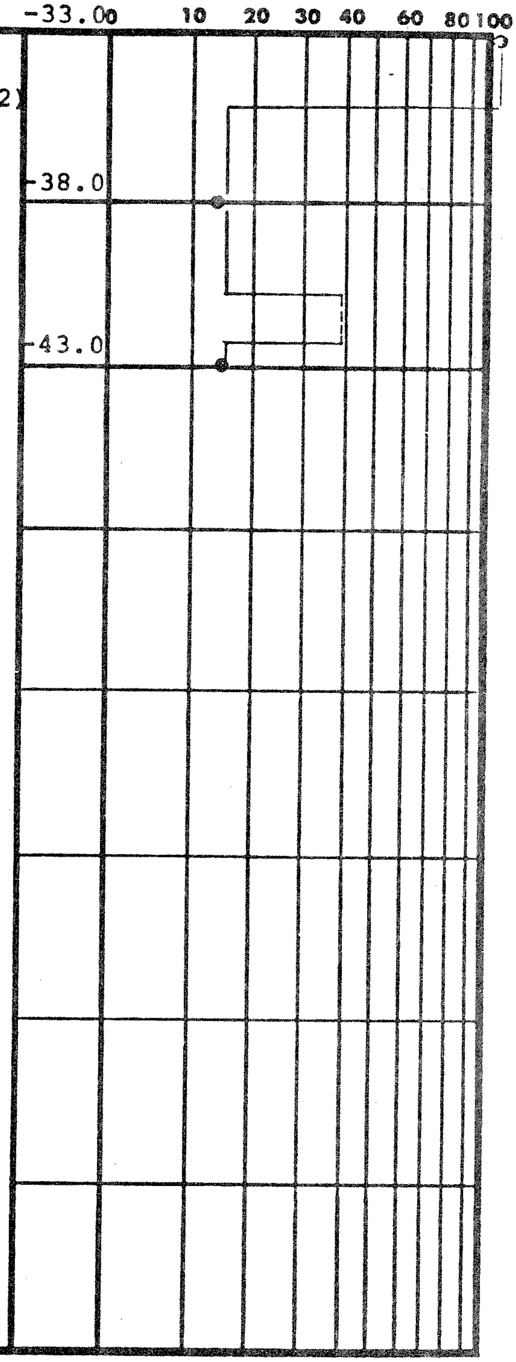
2A-140

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION		ELEV
40.0	FIRM to VERY FIRM tan fine SAND with some shell fragments and VERY DENSE grey	(2)	-33.0
42.5	VERY FIRM to FIRM grey slightly silty fine SAND with small cemented fine sand seams		-38.0
48.0	DENSE grey cemented fine SAND		
49.5	FIRM grey slightly silty (3)		-43.0
50.5	BORING TERMINATED		

(1) SAND
(2) cemented fine sand SEAMS
(3) fine SAND

• PENETRATION-BLOWS PER FT.



TEST BORING RECORD

(Page 2 of 2 Pages)

BORING NO. SB-1

DATE DRILLED 1/5/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

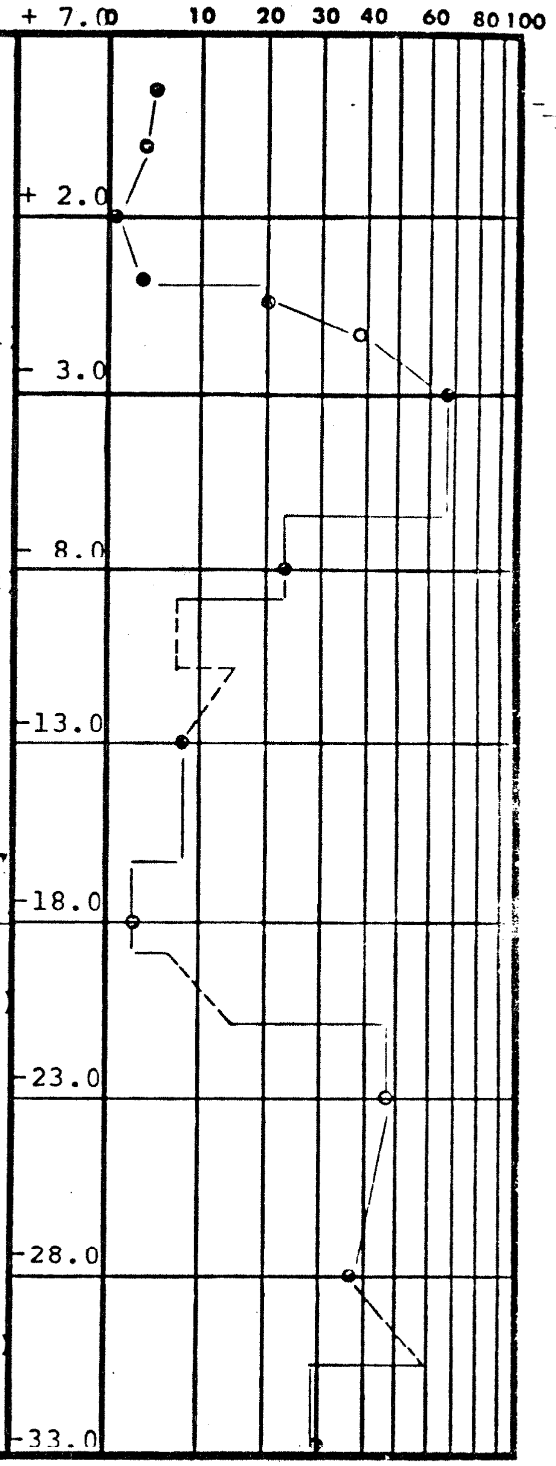
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
% ROCK CORE RECOVERY
LOSS OF DRILLING WATER

2A-141

DEPTH FT.	DESCRIPTION	
0		
2.0	VERY LOOSE to LOOSE grey fine SAND	
3.5	VERY LOOSE grey slightly silty fine SAND	
7.0	VERY LOOSE grey slightly silty slightly clayey fine SAND	
9.0	FIRM to DENSE grey slightly silty slightly clayey	(1)
13.5	VERY DENSE grey slightly silty fine SAND with shell fragments	
16.0	VERY FIRM grey slightly silty fine SAND with small fine sandy clayey silt	(2)
18.0	LOOSE grey silty clayey fine SAND	
23.5	FIRM to LOOSE grey slightly silty fine SAND with some shell fragments and decayed organic matter	
26.0	SOFT blue-grey silty CLAY with some shell fragments and trace of fine sand lenses	
28.0	LOOSE to FIRM grey slightly silty slightly clayey	(3)
36.5	DENSE blue-grey to grey fine SAND	
37.5	DENSE to VERY DENSE grey	(4)
40.0	VERY FIRM grey fine SAND with some shell fragments & cemented fine sand fragments	

ELEV • PENETRATION-BLOWS PER FT.



TEST BORING RECORD

(Page 1 of 2 Pages)

BORING NO. SB-2

DATE DRILLED 1/4/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

50 % ROCK CORE RECOVERY

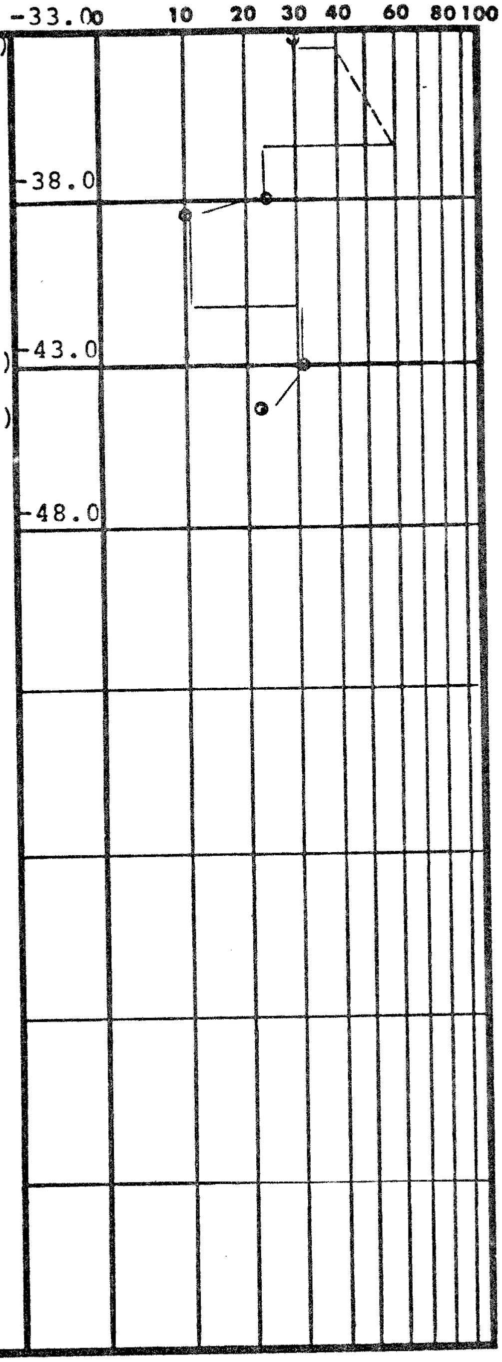
LOSS OF DRILLING WATER

2A-142

DEPTH FT.	DESCRIPTION		ELEV
40.0	VERY FIRM grey fine SAND	(5)	-33.00
40.5	DENSE to VERY DENSE grey cemented fine SAND		
43.5	VERY FIRM grey cemented fine SAND with loose slightly silty fine sand seams		-38.0
48.5	DENSE grey cemented QUARTZ, LIMESTONE & FINE SAND with	(6)	-43.0
50.5	VERY FIRM grey cemented QUARTZ, LIMESTONE, SHELL &	(7)	
52.0	BORING TERMINATED		-48.0

(1) fine SAND with shell fragments
(2) lenses
(3) fine SAND
(4) cemented fine SAND
(5) with some shell fragments and cemented fine sand fragments
(6) small slightly silty fine sand seams
(7) FINE SAND with small silty fine sand lenses

• PENETRATION-BLOWS PER FT.



TEST BORING RECORD
(Page 2 of 2 Pages)

BORING NO. SB-2
DATE DRILLED 1/4/72
JOB NO. J-1540

LAW ENGINEERING TESTING CO.

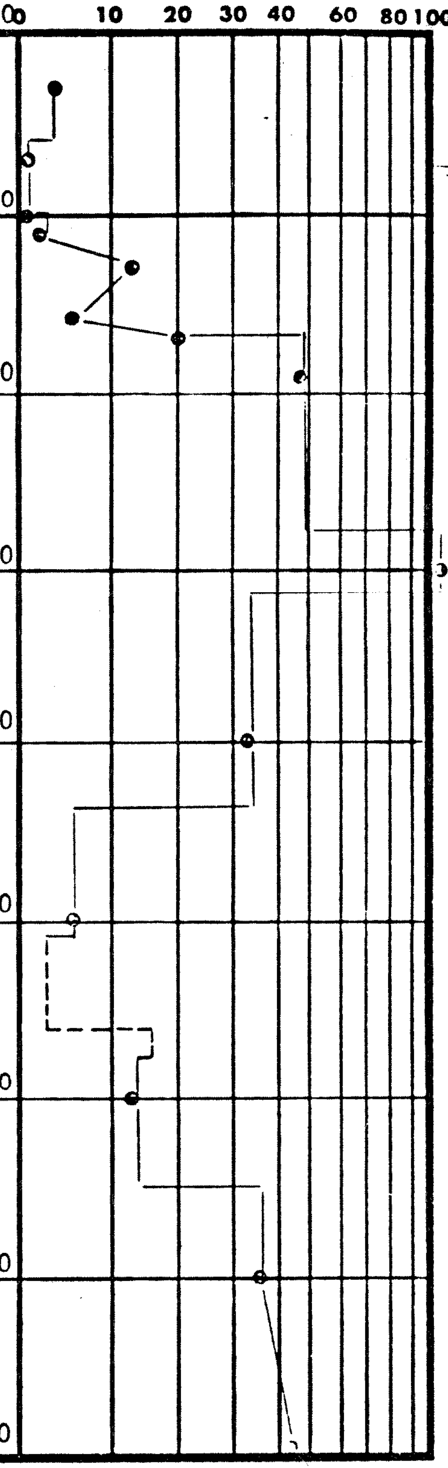
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
LOSS OF DRILLING WATER

2A-143

DEPTH FT.	DESCRIPTION		ELEV
0	VERY LOOSE grey fine SAND		+ 7.0
2.5	VERY LOOSE grey slightly	(1)	
3.0	VERY SOFT blue-grey to grey clayey fine sandy to	(2)	+ 2.0
5.0	FIRM grey slightly silty fine SAND and VERY LOOSE to LOOSE grey silty clayey to slightly clayey fine SAND		
8.5	DENSE grey slightly silty fine SAND with small shell & cemented fine sand fragments & small silty clay lenses		- 3.0
14.0	VERY DENSE tan fine SAND with cemented fine sand	(3)	- 8.0
16.0	DENSE grey slightly silty very slightly clayey fine SAND		-13.0
22.0	FIRM blue-grey fine sandy silty CLAY with fine sand lenses		-18.0
25.5	SOFT blue-grey silty CLAY		
28.0	FIRM grey slightly silty	(4)	
29.0	STIFF blue-grey fine slightly sandy CLAY with calcareous silt lenses		-23.0
32.5	DENSE grey fine SAND		-28.0
36.5	DENSE grey tan cemented LIMESTONE & FINE SAND with silty fine sand seams		-33.0
40.0			

PENETRATION-BLOWS PER FT.



15 hrs

TEST BORING RECORD

(Page 1 of 2 Pages)

BORING NO. SB-3

DATE DRILLED 1/6/72

JOB NO. J-1540

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE — WATER TABLE, 24 HR. — WATER TABLE, 1 HR. — % ROCK CORE RECOVERY — LOSS OF DRILLING WATER

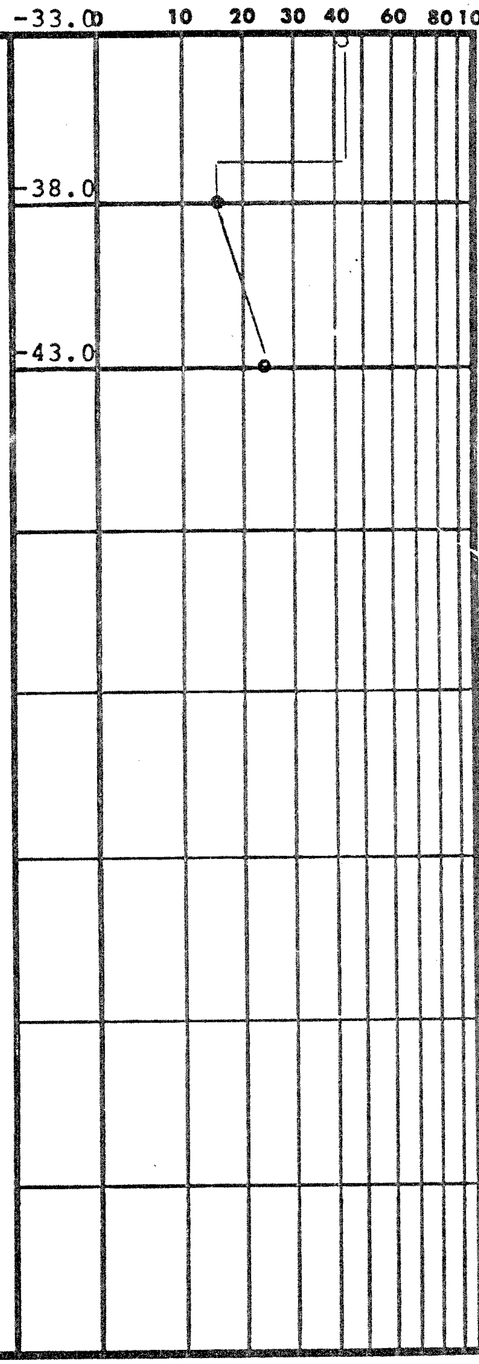
2A-144

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION
40.0	DENSE grey tan cemented LIMESTONE & FINE SAND with silty fine sand seams
44.0	FIRM grey silty fine SAND & VERY FIRM grey cemented LIMESTONE, SHELL & FINE SAND
50.5	BORING TERMINATED

(1) silty slightly clayey fine SAND
(2) slightly sandy SILT
(3) fragments
(4) slightly clayey fine SAND

ELEV • PENETRATION-BLOWS PER FT.



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
50 % ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

TEST BORING RECORD

(Page 2 of 2 Pages)

BORING NO. SB-3

DATE DRILLED 1/6/72

JOB NO. J-1540

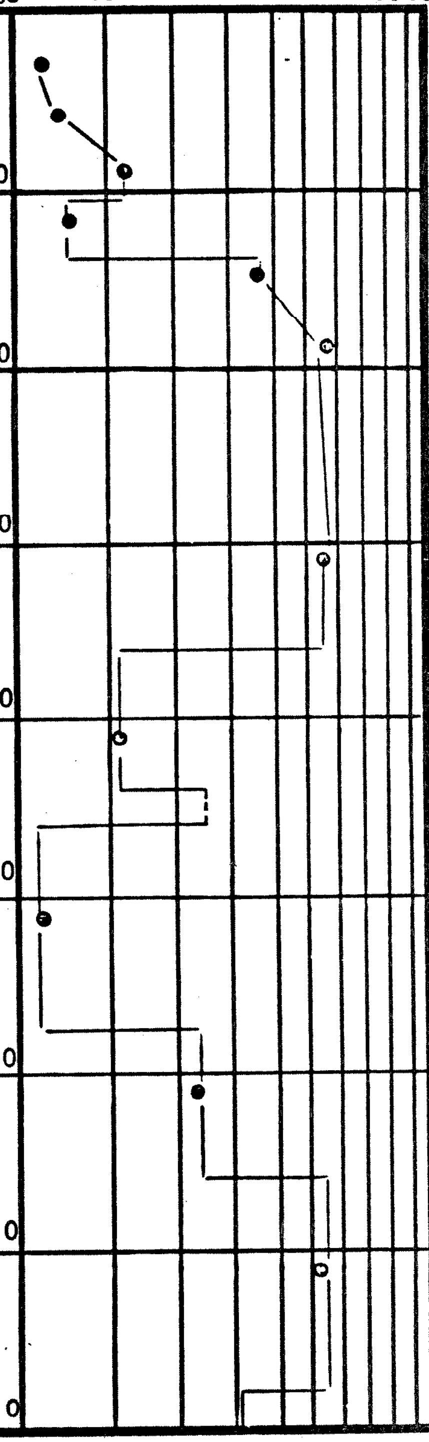
LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION	
0	VERY LOOSE grey fine SAND	
2.0	LOOSE to FIRM grey slightly silty fine SAND	
5.5	LOOSE silty clayey fine SAND	
7.0	DENSE grey slightly silty slightly clayey fine SAND	(1)
8.5	VERY DENSE grey slightly silty to very slightly silty fine SAND with small shell fragments & trace of silt lenses	
18.0	FIRM to LOOSE grey silty slightly clayey fine SAND	
22.0	VERY FIRM grey fine SAND	(2)
23.0	SOFT blue-grey silty CLAY with small fine sand lenses	
29.0	VERY FIRM grey fine SAND with cemented fine sand fragments	
33.0	VERY DENSE grey fine SAND with cemented shell & fine sand seams	
39.0	VERY FIRM to FIRM grey slightly silty fine SAND	(3)
40.0		

ELEV: + 7.00, + 2.0, - 3.0, - 8.0, -13.0, -18.0, -23.0, -28.0, -33.0

• PENETRATION-BLOWS PER FT.

10 20 30 40 60 80 100



20 hrs

TEST BORING RECORD

(Page 1 of 2 Pages)

BORING NO. SB-4

DATE DRILLED 1/6/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

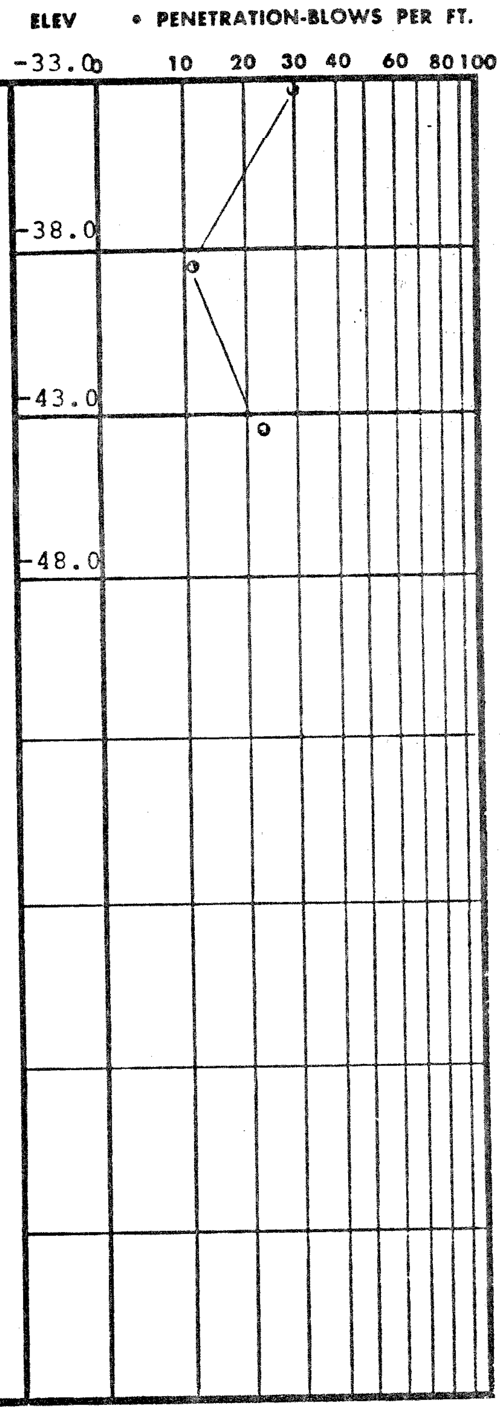
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
LOSS OF DRILLING WATER

2A-146

DEPTH FT.	DESCRIPTION
40.0	VERY FIRM to FIRM grey slightly silty fine SAND with cemented shell, limestone & fine sand seams
47.0	VERY FIRM grey tan partially cemented QUARTZ, SHELL, LIMESTONE & silty FINE SAND
51.0	BORING TERMINATED

(1) with some shell fragments
(2) with cemented fine sand fragments
(3) with cemented shell, limestone & fine sand seams



TEST BORING RECORD
(Page 2 of 2 Pages)

BORING NO. SB-4
DATE DRILLED 1/6/72
JOB NO. J-1540

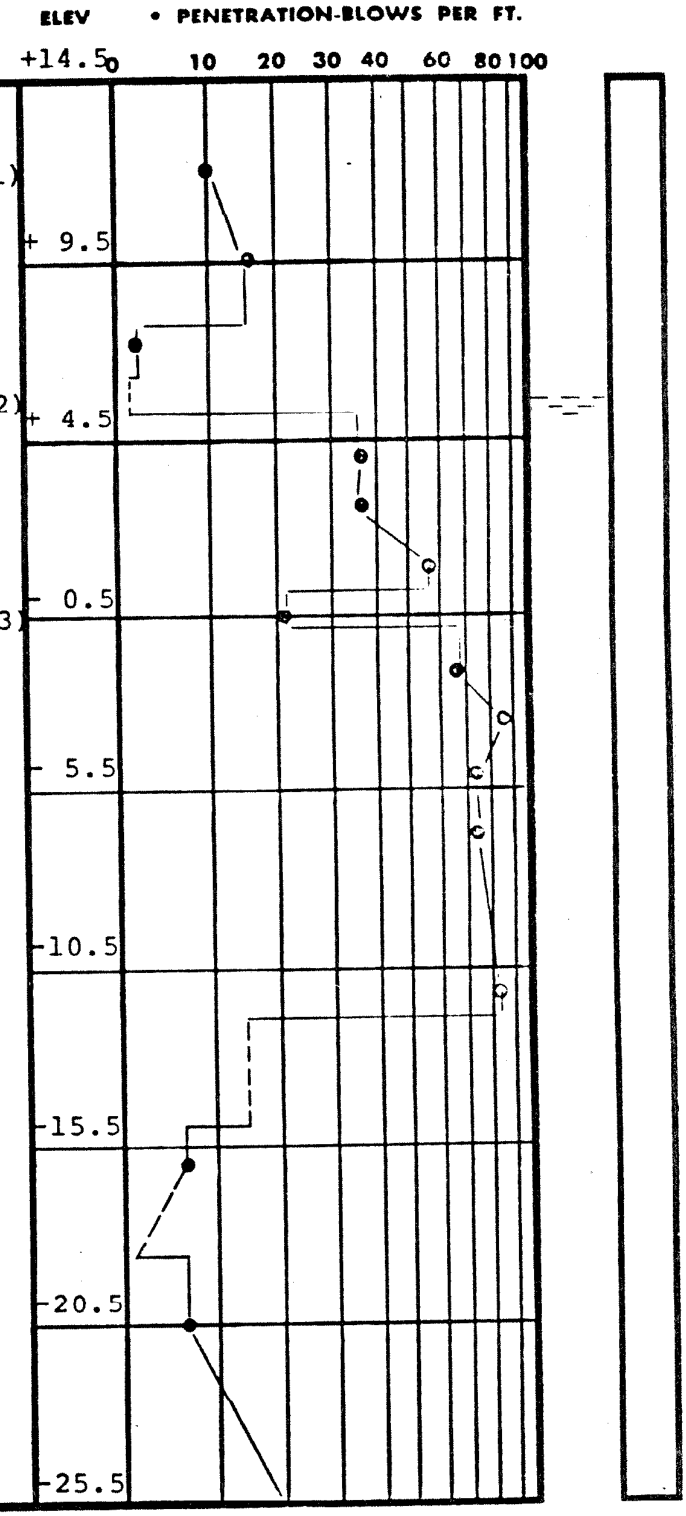
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
50 % ROCK CORE RECOVERY
LOSS OF DRILLING WATER

2A-147

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION	
0	LOOSE light tan fine SAND	
2.0	LOOSE grey slightly silty	(1)
3.0	FIRM light tan fine SAND	
7.0	VERY LOOSE grey slightly silty fine SAND	
8.5	VERY SOFT blue-grey silty	(2)
9.5	DENSE to VERY DENSE grey slightly silty fine SAND	
14.5	VERY FIRM blue-grey silty	(3)
15.5	VERY DENSE blue-grey to dark grey slightly silty fine SAND with some small shell fragments, silty clay lenses & cemented fine sand fragments	
20.5	VERY DENSE grey slightly silty fine SAND	
26.5	FIRM grey silty clayey fine SAND	
29.5	FIRM to VERY SOFT blue-grey silty CLAY	
33.0	FIRM to LOOSE dark grey clayey slightly silty fine SAND	
36.0	LOOSE to FIRM grey slightly silty fine SAND with some cemented fine sand fragments	
40.0		



TEST BORING RECORD
(Page 1 of 2 Pages)

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

BORING NO. SB-5
DATE DRILLED 1/7/72
JOB NO. J-1540

WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
UNDISTURBED SAMPLE
50 % ROCK CORE RECOVERY
LOSS OF DRILLING WATER

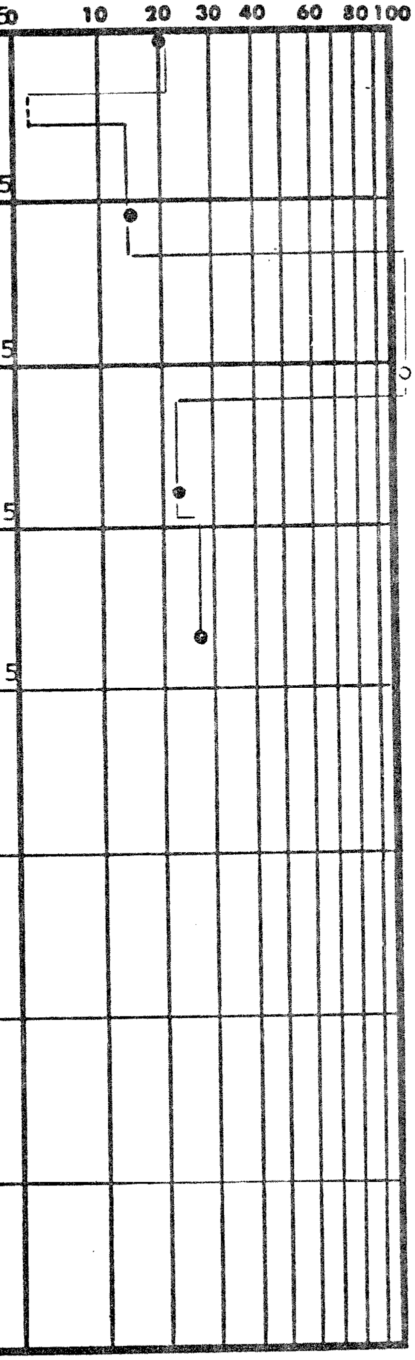
2A-148

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION		ELEV
40.0	LOOSE to FIRM grey slightly silty fine SAND with	(4)	-25.5
42.0	VERY SOFT blue-grey silty	(5)	
43.0	FIRM blue-grey brown very clayey slightly silty fine SAND		-30.5
47.0	VERY DENSE cemented fine SAND with fine sand seams		-35.5
51.5	VERY FIRM grey slightly silty fine SAND with cemented limestone & fine sand seams		-40.5
55.0	VERY FIRM grey cemented QUARTZ, LIMESTONE & FINE SAND with silty fine sand lenses		
59.0	BORING TERMINATED		-45.5

(1) fine SAND
(2) CLAY
(3) clayey fine SAND
(4) some cemented fine sand fragments
(5) CLAY

• PENETRATION-BLOWS PER FT.



TEST BORING RECORD
(Page 2 of 2 Pages)

BORING NO. SB-5

DATE DRILLED 1/7/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

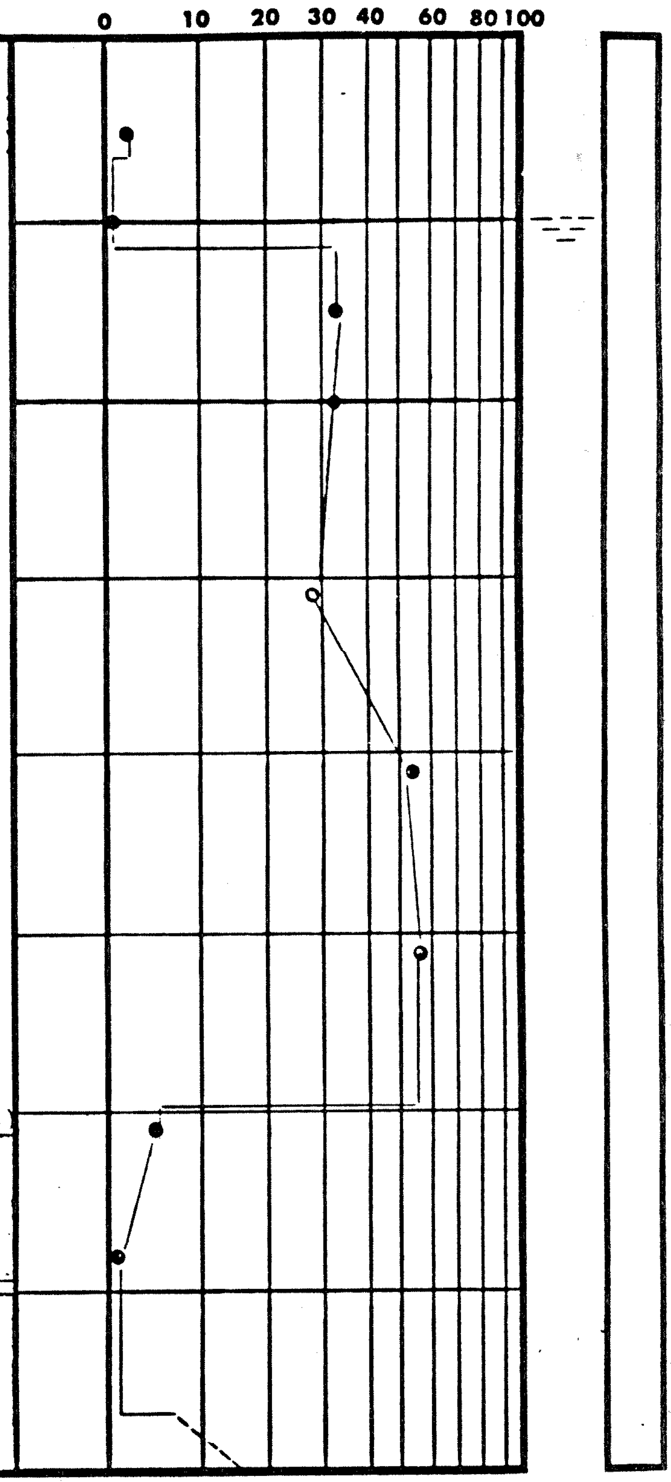
UNDISTURBED SAMPLE — WATER TABLE, 24 HR. — WATER TABLE, 1 HR.

% ROCK CORE RECOVERY — LOSS OF DRILLING WATER

2A-149

DEPTH FT.	DESCRIPTION		ELEV
0	FIRM light tan fine SAND with shell fragments (Fill)		
1.5	LOOSE blue-grey slightly silty	(1)	
2.0	VERY SOFT dark brown fine	(2)	
3.5	VERY LOOSE dark grey slightly silty fine SAND with decayed roots		
6.0	DENSE to VERY FIRM blue-grey fine SAND & SHELL FRAGMENTS		
19.0	VERY DENSE blue-grey fine SAND with shell fragments		
30.0	FIRM dark blue-grey slightly	(3)	
31.0	VERY SOFT dark blue-grey clayey fine very slightly sandy to sandy SILT		
38.5	LOOSE to FIRM grey slightly silty fine SAND		
40.0			

• PENETRATION-BLOWS PER FT.



TEST BORING RECORD

(Page 1 of 3 Pages)

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

BORING NO. B-154

DATE DRILLED 1/24,25/72

JOB NO. J-1540

UNDISTURBED SAMPLE

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

% ROCK CORE RECOVERY

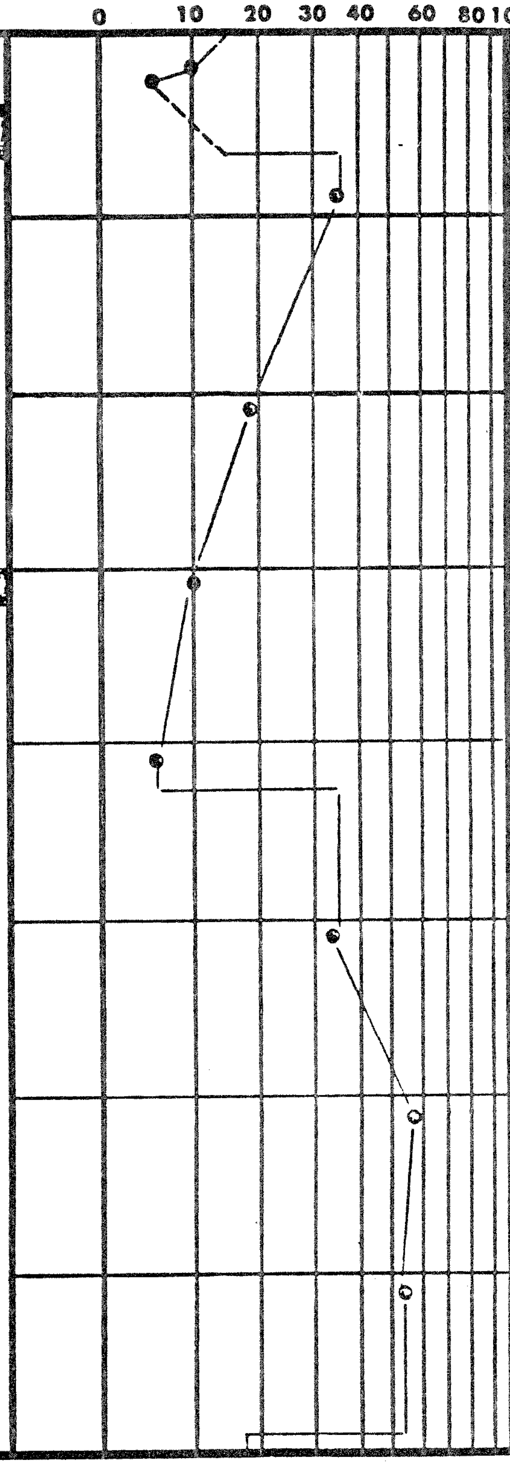
LOSS OF DRILLING WATER

2A-150

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION		ELEV
40.0			
41.0	LOOSE blue-grey slightly silty	(4)	
42.0	LOOSE grey silty clayey fine	(5)	
43.5	LOOSE to FIRM blue-grey fine SAND with shell fragments		
	DENSE to FIRM blue-grey very slightly silty to slightly silty fine SAND with some shell fragments		
51.0	LOOSE & FIRM blue-grey slightly silty fine SAND & SHELL FRAGMENTS		
61.5	DENSE to VERY DENSE blue-grey slightly silty fine SAND with shell fragments		
72.0	VERY DENSE blue-grey fine SAND		
79.5	FIRM blue-grey slightly silty	(6)	
80.0			

• PENETRATION-BLOWS PER FT.



TEST BORING RECORD

(Page 2 of 3 Pages)

BORING NO. B-154

DATE DRILLED 1/24,25/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

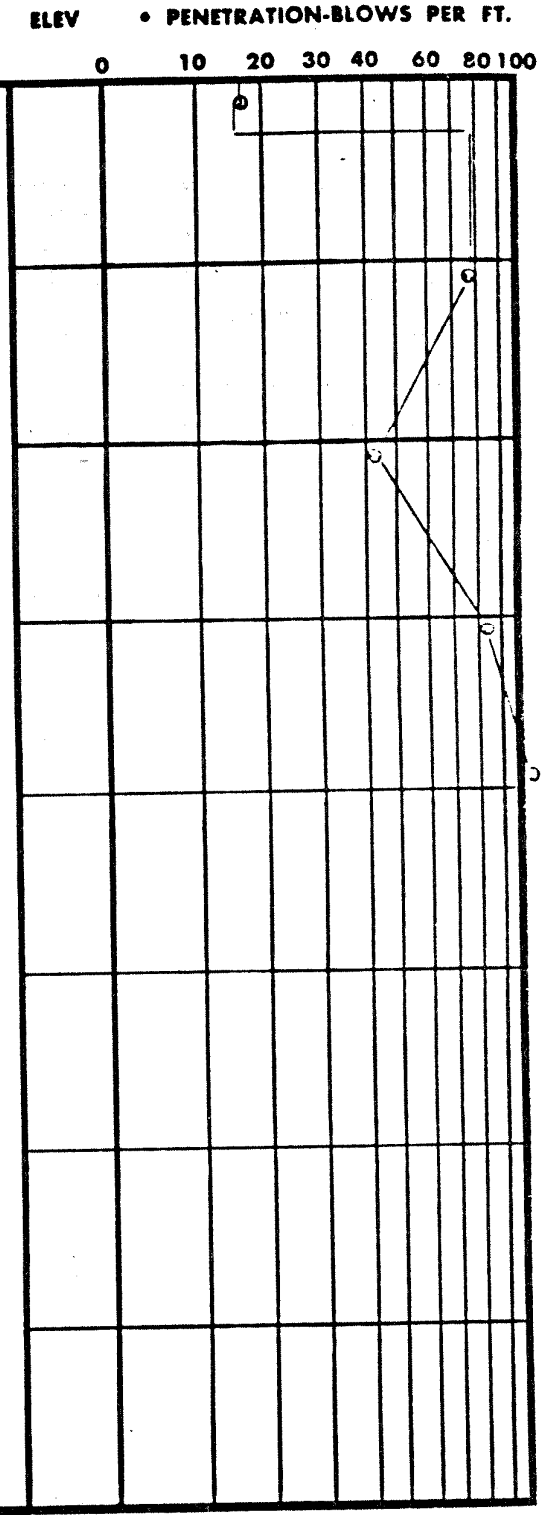
50 % ROCK CORE RECOVERY

LOSS OF DRILLING WATER

2A-151

DEPTH FT.	DESCRIPTION
80.0	FIRM blue-grey slightly silty fine SAND & SHELL FRAGMENTS
81.5	VERY DENSE blue-grey very slightly silty fine SAND
89.0	DENSE blue-grey slightly silty fine SAND with shell & cemented fine sand & shell fragments
93.5	VERY DENSE grey very slightly silty fine SAND
100.0	BORING TERMINATED

ELEV



(1) fine SAND with shell fragments
(2) sandy slightly clayey SILT (Peat)
(3) organic SILT
(4) fine SAND with shell & cemented fine sand fragments
(5) SAND with some shell fragments
(6) fine SAND & SHELL FRAGMENTS

TEST BORING RECORD
(Page 3 of 3 Pages)

BORING NO. B-154

DATE DRILLED 1/24,25/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

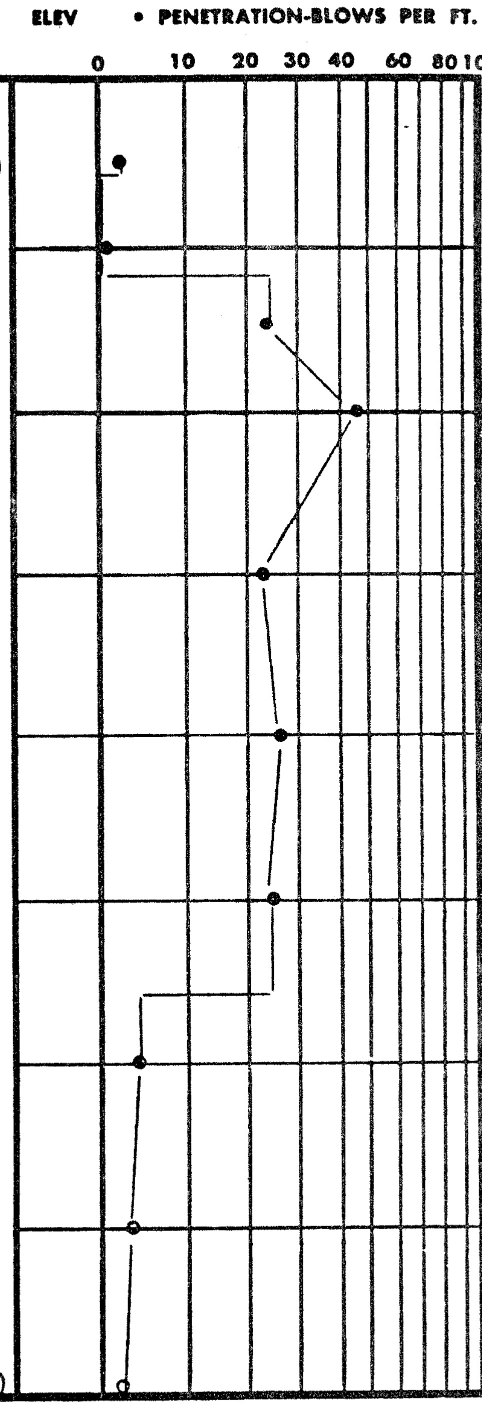
BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE — WATER TABLE, 24 HR. — WATER TABLE, 1 HR.

|50| % ROCK CORE RECOVERY — LOSS OF DRILLING WATER

2A-152

DEPTH FT.	DESCRIPTION	ELEV
0		
2.0	LOOSE light tan fine SAND with shell fragments (Fill)	
3.0	VERY LOOSE blue-grey silty	(1)
6.0	VERY SOFT dark brown organic fine sandy clayey SILT (Peat)	
8.5	VERY FIRM blue-grey fine SAND with small shell fragments	
17.0	DENSE to FIRM blue-grey fine SAND	
28.0	VERY FIRM blue-grey fine SAND & SHELL FRAGMENTS	
39.0	VERY LOOSE blue-grey silty slightly clayey fine SAND	
40.0	VERY LOOSE dark grey silty	(2)



TEST BORING RECORD

(Page 1 of 3 Pages)

BORING NO. B-155H

DATE DRILLED 1/11,12/72

JOB NO. J-1540

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

% ROCK CORE RECOVERY

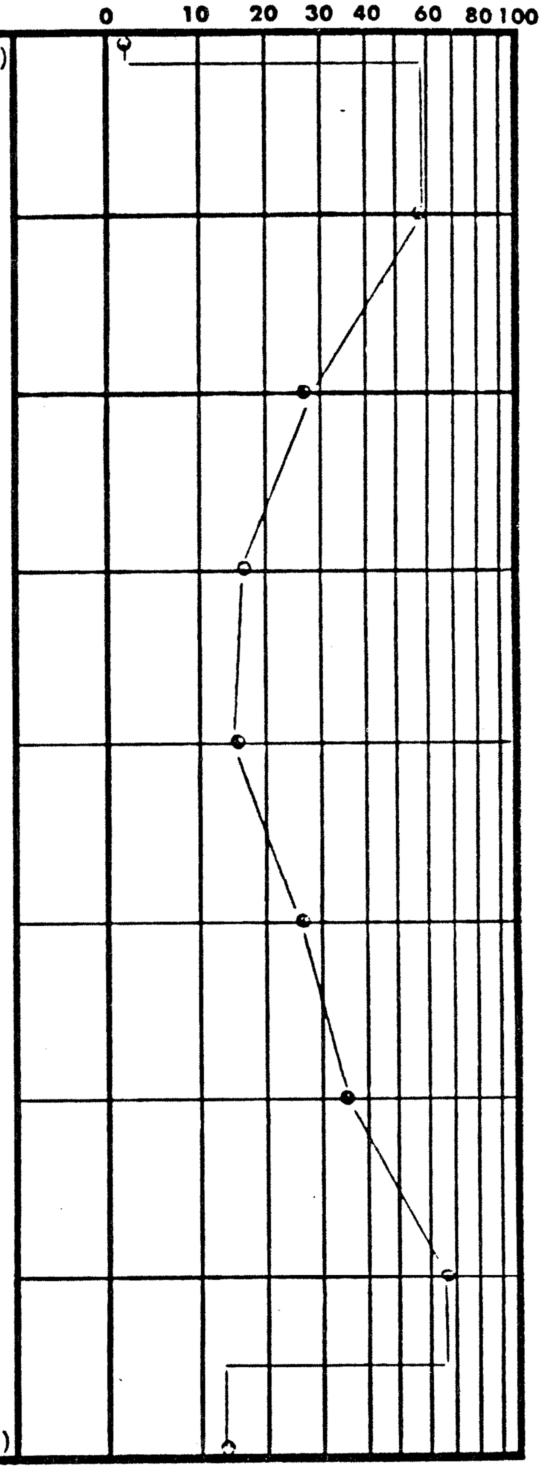
LOSS OF DRILLING WATER

2A-153

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION		ELEV
40.0			
41.0	VERY LOOSE dark grey silty	(3)	
	VERY DENSE to VERY FIRM grey slightly silty fine SAND		
52.5			
	FIRM to DENSE blue-grey slightly silty to very slightly silty fine SAND & SHELL FRAGMENTS		
72.5			
	VERY DENSE blue-grey slightly silty fine SAND with some shell fragments		
77.5			
	FIRM blue-grey slightly silty fine SAND & SHELL FRAGMENTS with firm blue-grey	(4)	
80.0			

• PENETRATION-BLOWS PER FT.



TEST BORING RECORD

(Page 2 of 3 Pages)

BORING NO. B-155

DATE DRILLED 1/11,12/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

50 % ROCK CORE RECOVERY

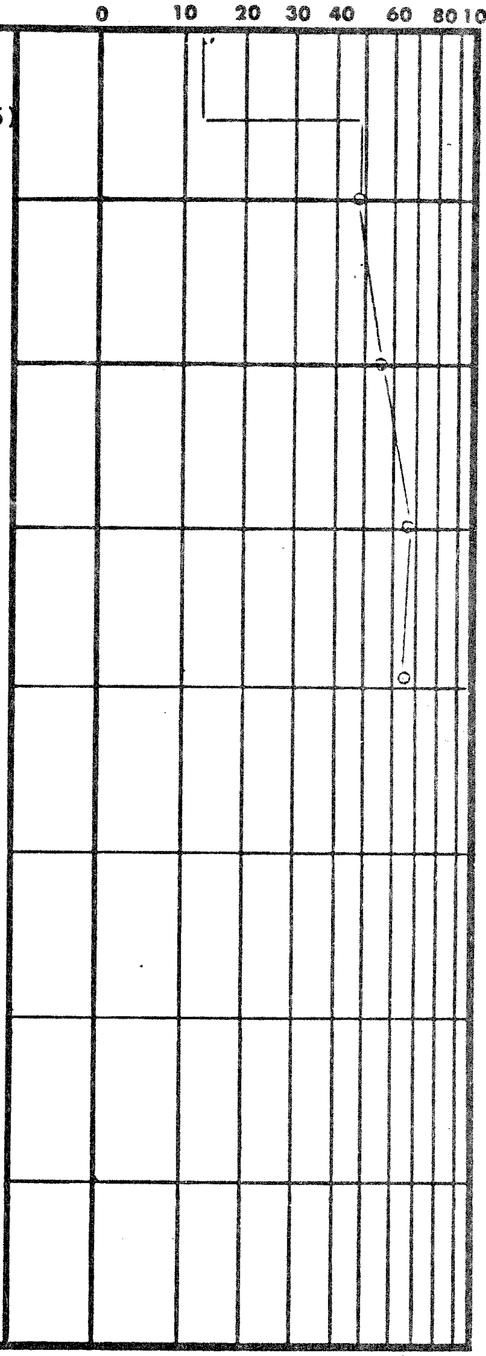
LOSS OF DRILLING WATER

2A-154

DEPTH FT.	DESCRIPTION	ELEV
80.0	FIRM blue-grey slightly silty fine SAND & SHELL FRAGMENTS with firm blue-grey shell & silty slightly	
83.0		(5)
	DENSE blue-grey fine SAND & SHELL FRAGMENTS & Cemented fine SAND FRAGMENTS	
89.5	VERY DENSE blue-grey slightly silty fine SAND with shell fragments	
92.0	VERY DENSE grey slightly silty fine SAND with some cemented fine sand fragments	
97.0	VERY DENSE grey slightly silty fine SAND	
100.0	BORING TERMINATED	

(1) fine SAND with shell fragments (Fill)
(2) slightly clayey fine SAND with clayey silt seams
(3) slightly clayey fine SAND with clayey silt seams
(4) shell & silty slightly clayey fine sand layers
(5) clayey fine sand layers

• PENETRATION-BLOWS PER FT.



TEST BORING RECORD

(Page 3 of 3 Pages)

BORING NO. B-155

DATE DRILLED 1/11,12/72

JOB NO. J-1540

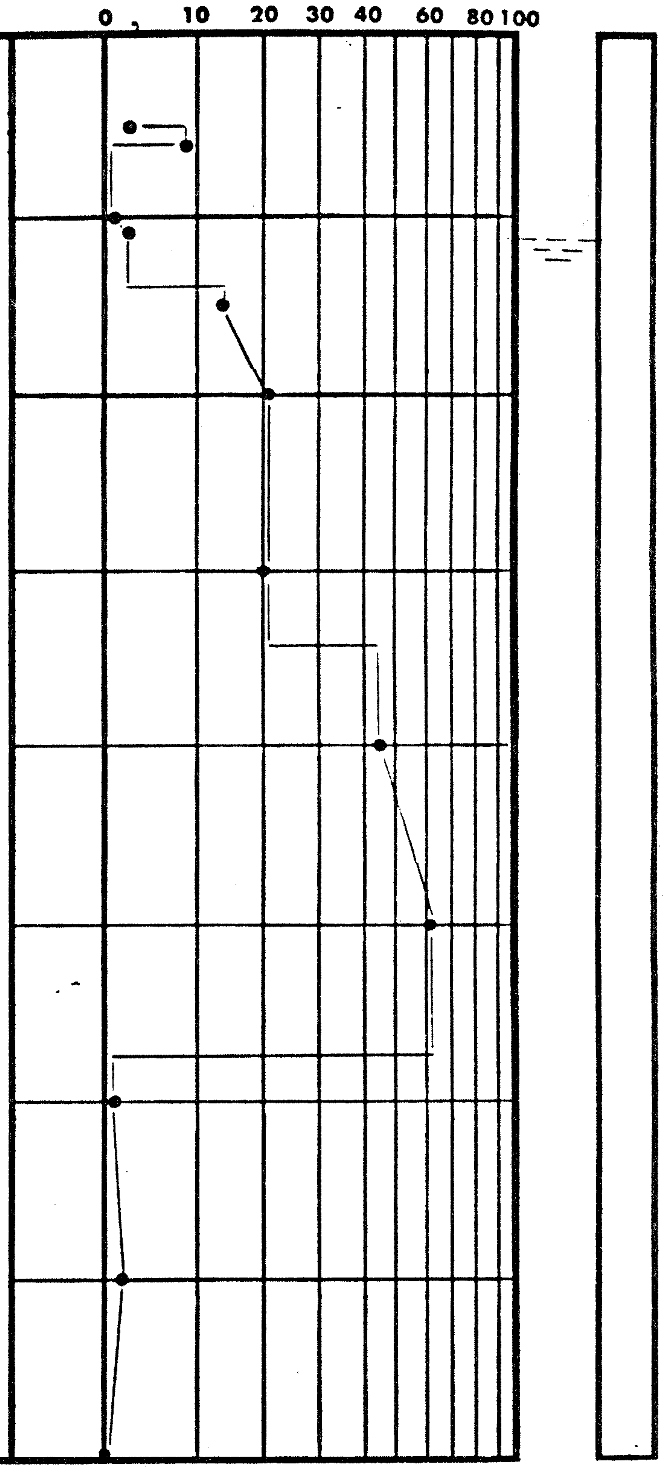
LAW ENGINEERING TESTING CO.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
50 % ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

DEPTH FT.	DESCRIPTION	ELEV
0	LOOSE light tan fine SAND with shell fragments (Fill)	
2.0	VERY SOFT blue-grey clayey	(1
2.5	LOOSE blue-grey slightly	(2
3.0	VERY SOFT dark brown organic SILT (Peat)	
5.0	VERY SOFT dark brown highly organic fine sandy SILT (Peat)	
7.0	FIRM to VERY FIRM grey to blue-grey slightly silty to very slightly silty fine SAND with small shell fragments	
11.0	FIRM blue-grey fine SAND & SHELL FRAGMENTS	
17.0	DENSE blue-grey fine SAND with shell fragments	
23.0	VERY DENSE blue-grey fine SAND & SHELL FRAGMENTS & DENSE blue-grey slightly silty fine sand LAYERS	
28.5	VERY LOOSE blue-grey to dark grey silty clayey fine SAND	
40.0		

• PENETRATION-BLOWS PER FT.



TEST BORING RECORD
(Page 1 of 3 Pages)

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

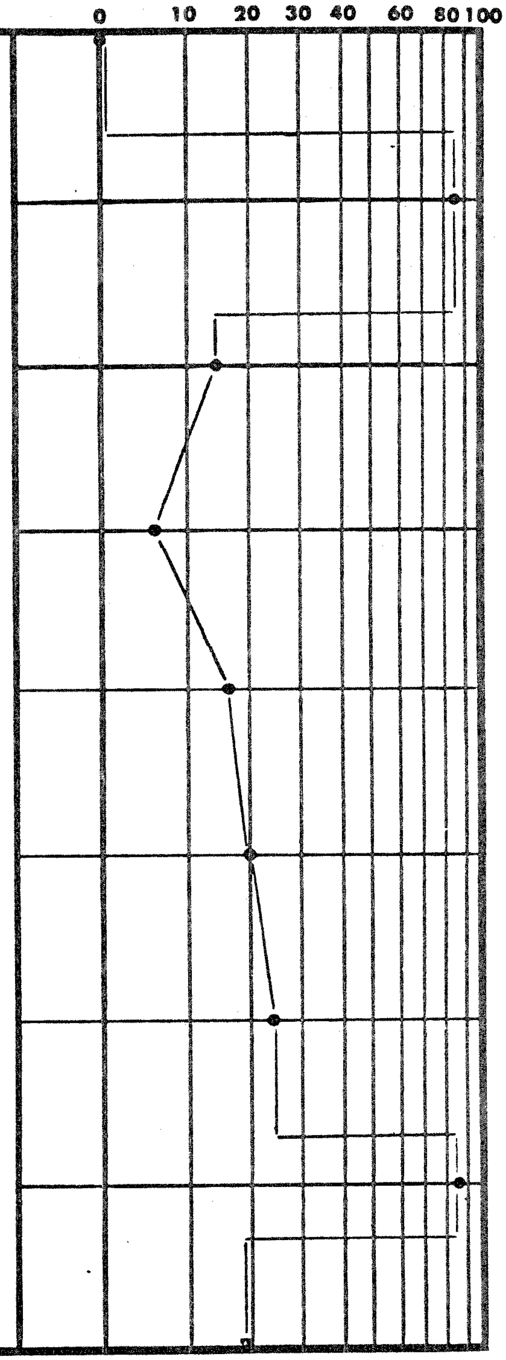
2A-156

BORING NO. B-156 H
DATE DRILLED 1/12,13/72
JOB NO. J-1540

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION	ELEV
40.0	VERY LOOSE blue-grey to dark grey silty clayey fine SAND	
43.0	VERY DENSE blue-grey slightly silty fine SAND	
48.5	FIRM to LOOSE blue-grey slightly silty slightly clayey fine SAND with some shell fragments	
58.0	FIRM blue-grey slightly silty fine SAND with shell fragments	
62.0	FIRM to VERY FIRM blue-grey slightly silty fine SAND & SHELL FRAGMENTS	
73.5	VERY DENSE blue-grey slightly silty fine SAND	
76.5	FIRM blue-grey silty slightly clayey fine SAND & SHELL FRAGMENTS	
80.0		

• PENETRATION-BLOWS PER FT.



TEST BORING RECORD

(Page 2 of 3 Pages)

BORING NO. B-156 H

DATE DRILLED 1/12,13/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

50 % ROCK CORE RECOVERY

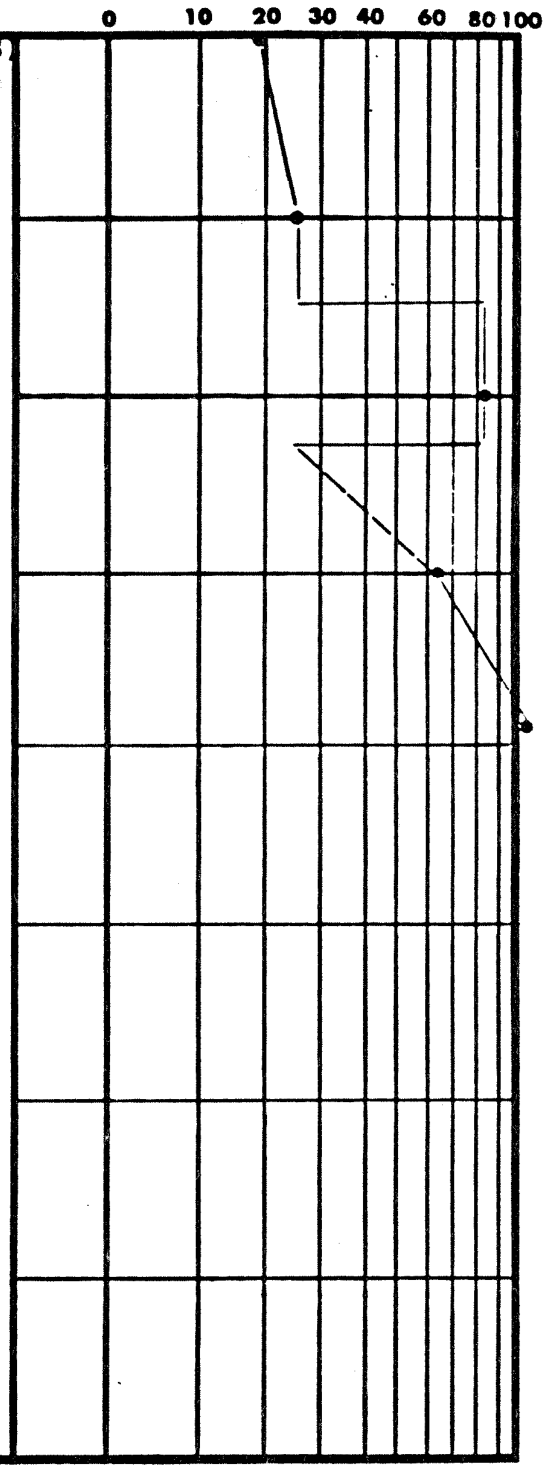
LOSS OF DRILLING WATER

2A-157

DEPTH FT.	DESCRIPTION	ELEV
80.0 / 80.5	FIRM blue-grey silty slightly (3)	
	VERY FIRM blue-grey fine SAND & SHELL FRAGMENTS	
87.5	VERY DENSE blue-grey fine SAND	
91.5	VERY FIRM to VERY DENSE grey slightly silty fine SAND with dense seam of shell fragments	
100.0	BORING TERMINATED	

(1) fine slightly sandy SILT
(2) silty fine SAND with shell fragments
(3) clayey fine SAND & SHELL FRAGMENTS

• PENETRATION-BLOWS PER FT.



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
% ROCK CORE RECOVERY
LOSS OF DRILLING WATER

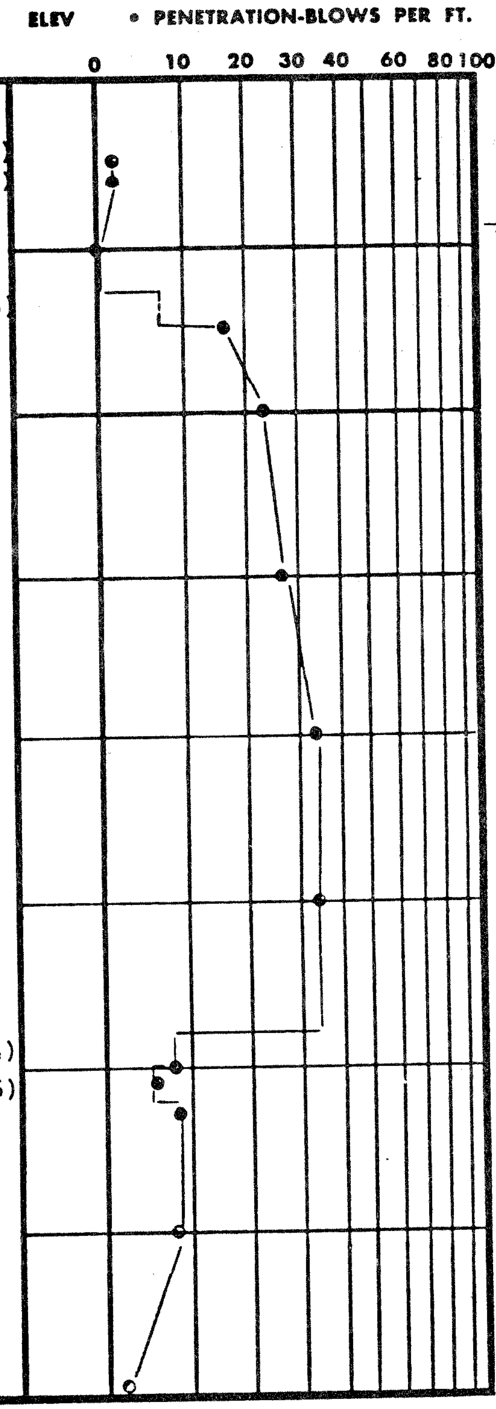
2A-158

TEST BORING RECORD
(Page 3 of 3 Pages)

BORING NO. B-156H
DATE DRILLED 1/12,13/72
JOB NO. J-1540

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION	ELEV
0	LOOSE tan fine SAND with shell fragments (Fill)	
2.0	VERY LOOSE blue-grey slightly	(1)
2.5	VERY SOFT dark brown highly	(2)
3.5	VERY SOFT dark brown organic slightly clayey SILT (Peat)	
6.5	LOOSE dark grey slightly	(3)
7.5	FIRM to DENSE blue-grey fine SAND with shell fragments	
22.0	DENSE blue-grey fine SAND & SHELL FRAGMENTS	
29.0	LOOSE blue-grey fine SAND &	(4)
30.0	FIRM blue-grey slightly	(5)
31.0	LOOSE and VERY LOOSE blue-grey to dark grey silty clayey and slightly clayey fine SAND	
40.0		



TEST BORING RECORD

(Page 1 of 3 Pages)

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

WATER TABLE, 24 HR.
UNDISTURBED SAMPLE
WATER TABLE, 1 HR.
% ROCK CORE RECOVERY
LOSS OF DRILLING WATER

2A-159

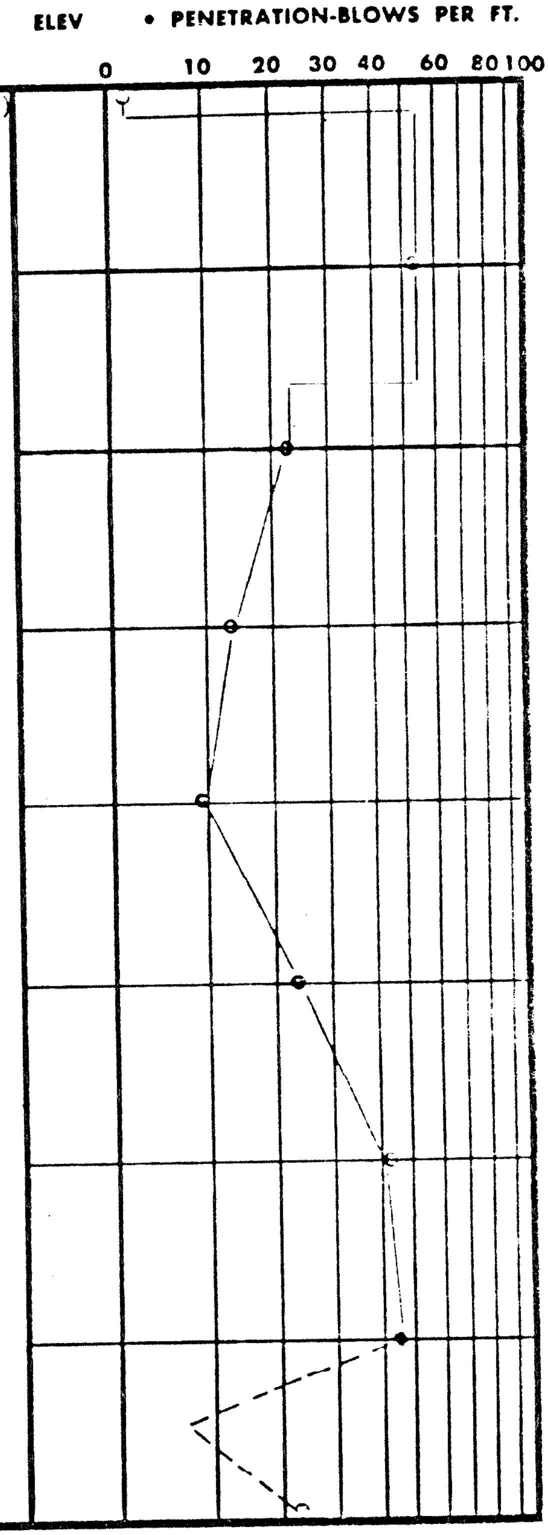
BORING NO. B-157

DATE DRILLED 1/19,20/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION	ELEV
40.0	LOOSE and VERY LOOSE blue-	(6)
41.0	VERY DENSE blue-grey fine SAND with seams of fine sand & shell fragments	
48.5	VERY FIRM blue-grey slightly silty fine SAND	
51.0	LOOSE and FIRM blue-grey silty to slightly silty fine SAND & SHELL FRAGMENTS	
64.0	LOOSE to DENSE blue-grey slightly silty fine SAND with shell fragments	
80.0		



TEST BORING RECO[R]

(Page 2 of 3 Pages)

BORING NO. B-157 H

DATE DRILLED 1/19,20/7[2]

JOB NO. J-1540

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

WATER TABLE, 24 HR.

UNDISTURBED SAMPLE

WATER TABLE, 1 HR.

% ROCK CORE RECOVERY

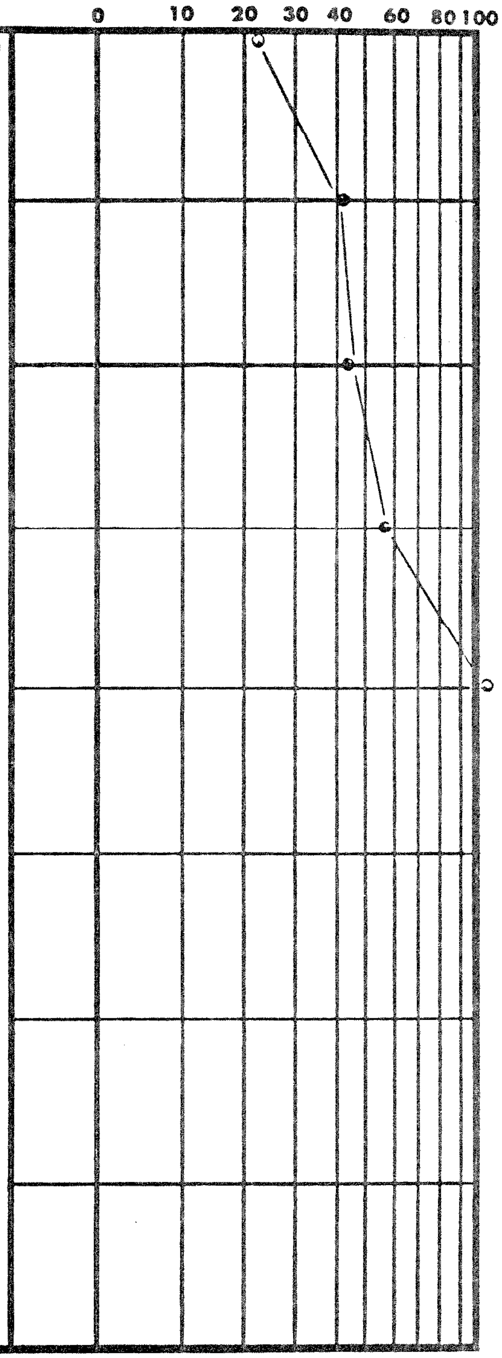
LOSS OF DRILLING WATER

2A-160

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION	ELEV
80.0		
81.0	LOOSE to DENSE blue-grey	(7)
	DENSE blue-grey slightly silty fine SAND & SHELL FRAGMENTS	
94.0		
	VERY DENSE grey slightly silty fine SAND	
100.0		
	BORING TERMINATED	

● PENETRATION-BLOWS PER FT.



(1) silty fine SAND
(2) organic SILT (Peat)
(3) silty fine SAND with some decayed roots
(4) SHELL FRAGMENTS
(5) organic fine sandy clayey SILT
(6) grey to dark grey silty clayey & slightly clayey fine SAND
(7) slightly silty fine SAND with shell fragments

TEST BORING RECORD

(Page 3 of 3 Pages)

BORING NO. B-157 H

DATE DRILLED 1/19,20/72

JOB NO. J-1540

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

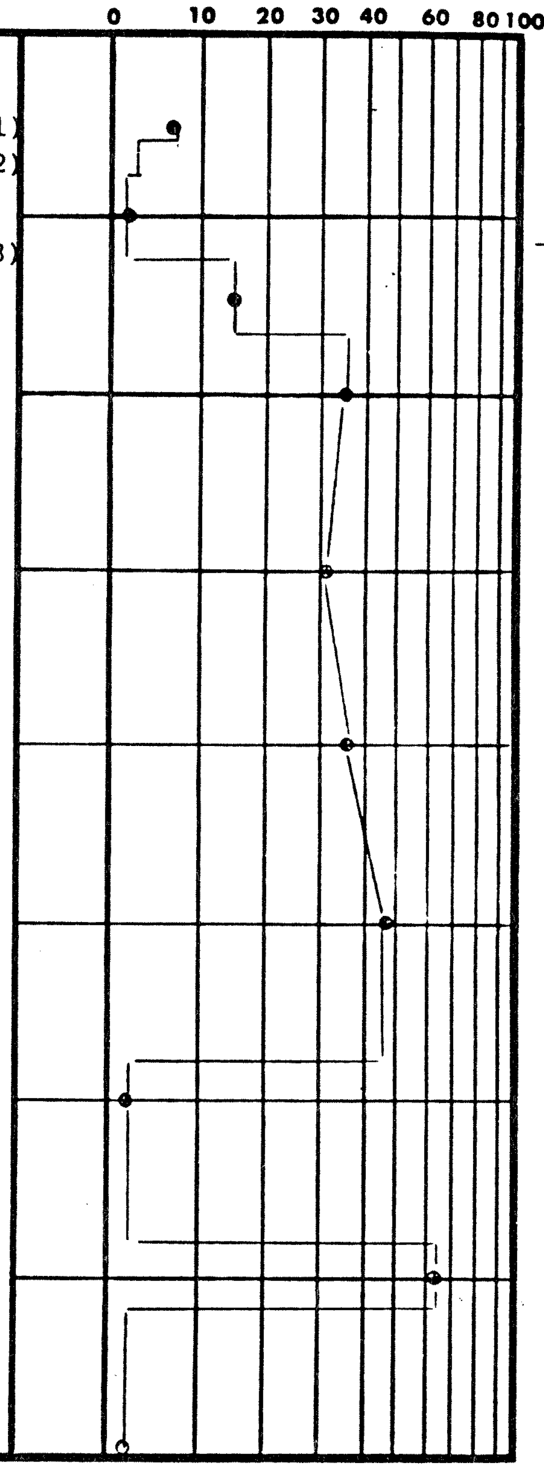
50 % ROCK CORE RECOVERY

LOSS OF DRILLING WATER

2A-161

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION	ELEV
0 – 2.0	LOOSE light tan fine SAND with shell fragments (Fill)	
2.0 – 3.0	LOOSE grey fine SAND & SHELL	(1)
3.0 – 4.0	VERY LOOSE grey silty	(2)
4.0 – 6.5	VERY SOFT dark brown to dark grey organic fine sandy clayey to slightly clayey SILT	(3)
6.5 – 8.5	FIRM grey fine SAND	
8.5 – 11.0	DENSE blue-grey fine SAND & fine SAND & SHELL LAYERS	
11.0 – 19.0	DENSE blue-grey fine SAND	
19.0 – 21.0	DENSE blue-grey fine SAND with small shell fragments	
21.0 – 29.0	DENSE grey fine SAND & SHELL FRAGMENTS	
29.0 – 34.0	VERY SOFT blue-grey fine sandy slightly clayey SILT	
34.0 – 36.0	VERY DENSE blue-grey fine SAND	
36.0 – 40.0	VERY LOOSE dark grey slightly silty slightly clayey fine SAND & VERY SOFT blue-grey slightly organic silty clay LAYERS	



TEST BORING RECORD

(Page 1 of 3 Pages)

BORING NO. B-158 H

DATE DRILLED 1/10,11/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

UNDISTURBED SAMPLE

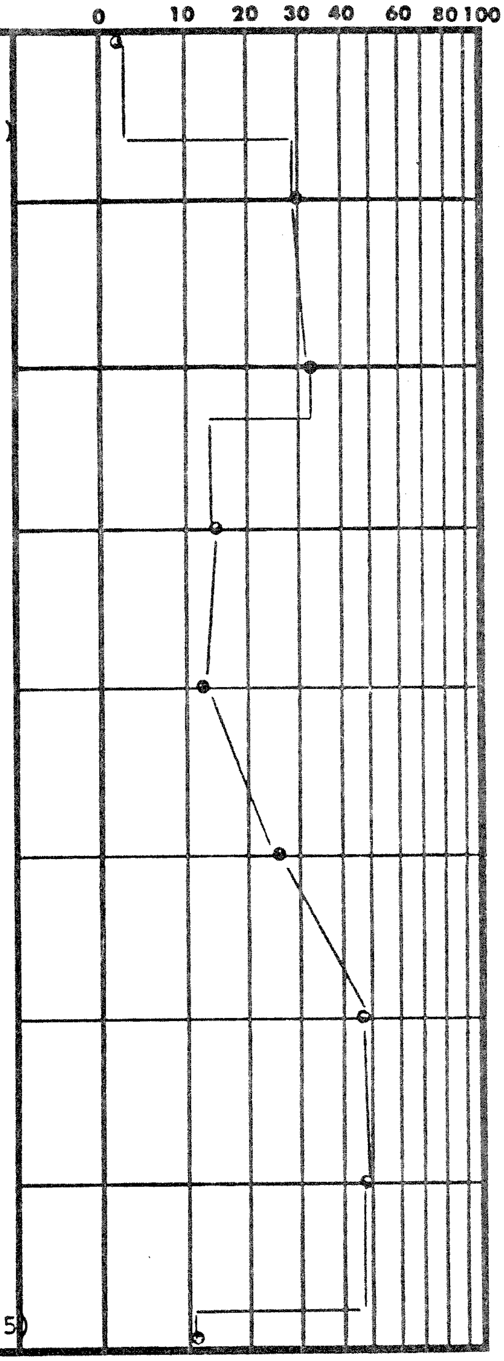
50 % ROCK CORE RECOVERY

LOSS OF DRILLING WATER

2A-162

DEPTH FT.	DESCRIPTION	ELEV
40.0	VERY LOOSE dark grey slightly silty slightly clayey fine SAND & VERY SOFT blue-grey slightly organic silty	(4)
43.5	VERY FIRM blue-grey slightly silty fine SAND & SHELL FRAGMENTS	
49.0	DENSE blue-grey slightly silty fine SAND with some small cemented fine sand fragments	
52.0	FIRM blue-grey slightly silty fine SAND & SHELL FRAGMENTS	
63.0	VERY FIRM blue-grey slightly silty fine SAND with shell fragments	
68.0	DENSE blue-grey fine SAND with shell fragments	
73.5	DENSE blue-grey slightly silty fine SAND with some small shell fragments	
79.0	FIRM blue-grey slightly silty	(5)
80.0		

• PENETRATION-BLOWS PER FT.



TEST BORING RECORD
(Page 2 of 3 Pages)

BORING NO. B-158 H

DATE DRILLED 1/10,11/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

% ROCK CORE RECOVERY

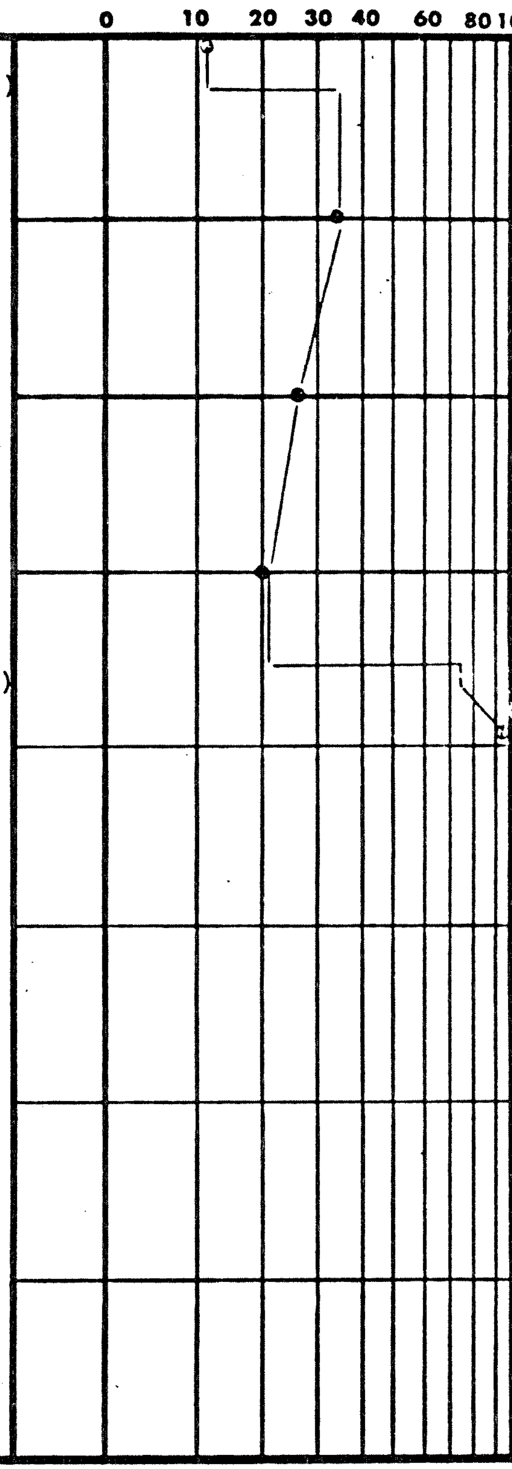
LOSS OF DRILLING WATER

2A-163

DEPTH FT.	DESCRIPTION	ELEV	
80.0	FIRM blue-grey slightly silty slightly clayey fine SAND	(6)	
81.5	DENSE grey fine SAND & SHELL FRAGMENTS		
87.5	VERY FIRM blue-grey slightly silty fine SAND & SHELL FRAGMENTS with FIRM seam of grey fine sandy clayey SILT with small cemented fine sand fragments		
93.5	FIRM grey slightly silty fine SAND with small shell & cemented fine sand fragments		
98.0	VERY DENSE grey cemented	(7)	
98.5	VERY DENSE grey slightly silty fine SAND		
100.0	BORING TERMINATED		

(1) FRAGMENTS with small silty clay lenses (Fill)
(2) slightly clayey fine SAND with shell fragments
(3) (Peat)
(4) clay LAYERS
(5) slightly clayey fine SAND with some small shell fragments & silty clay seams
(6) with some small shell fragments & silty clay seams
(7) fine SAND

• PENETRATION-BLOWS PER FT.



TEST BORING RECORD
(Page 3 of 3 Pages)

BORING NO. B-158 H

DATE DRILLED 1/10,11/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

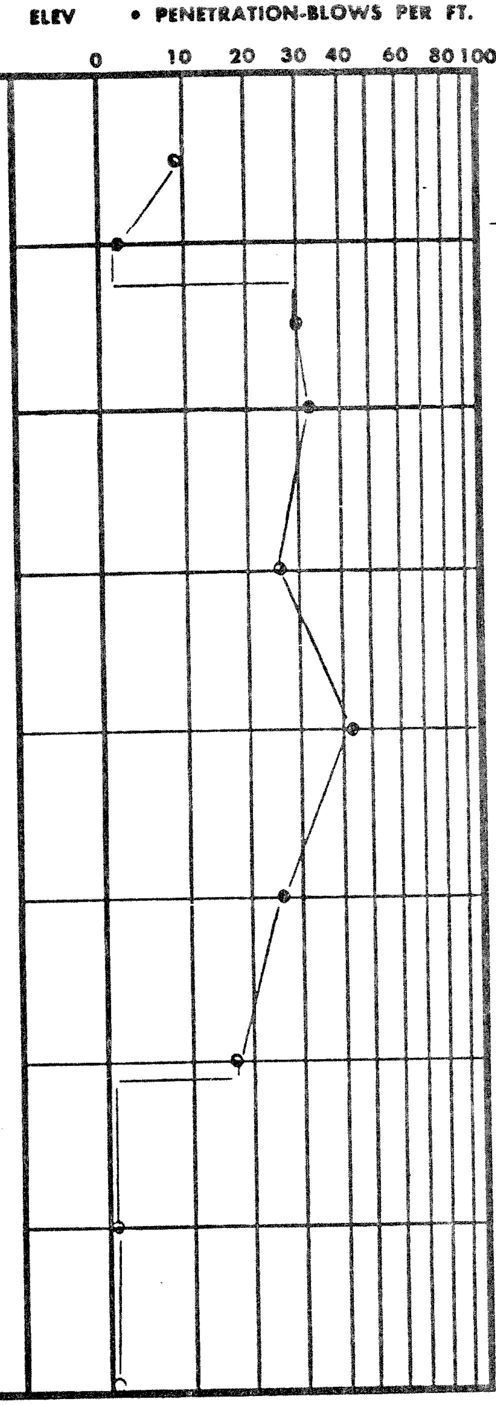
UNDISTURBED SAMPLE — WATER TABLE, 24 HR. — WATER TABLE, 1 HR.

% ROCK CORE RECOVERY — LOSS OF DRILLING WATER

2A-164

DEPTH FT.	DESCRIPTION
0	LOOSE grey slightly silty fine SAND (Fill)
3.5	VERY LOOSE dark brown silty fine SAND with some small roots
6.5	VERY FIRM to DENSE grey fine SAND with shell fragments
11.0	VERY FIRM blue-grey fine SAND
18.5	DENSE blue-grey fine SAND with small shell fragments
22.0	VERY FIRM blue-grey slightly silty fine SAND
29.0	FIRM blue-grey fine SAND & SHELL FRAGMENTS
30.5	VERY SOFT blue-grey slightly organic clayey fine slightly sandy SILT
38.0	VERY LOOSE blue-grey silty clayey fine SAND
40.0	

ELEV



TEST BORING RECORD

(Page 1 of 3 Pages)

BORING NO. B-159 U

DATE DRILLED 1/8/72

JOB NO. J-1540

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

50 % ROCK CORE RECOVERY

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

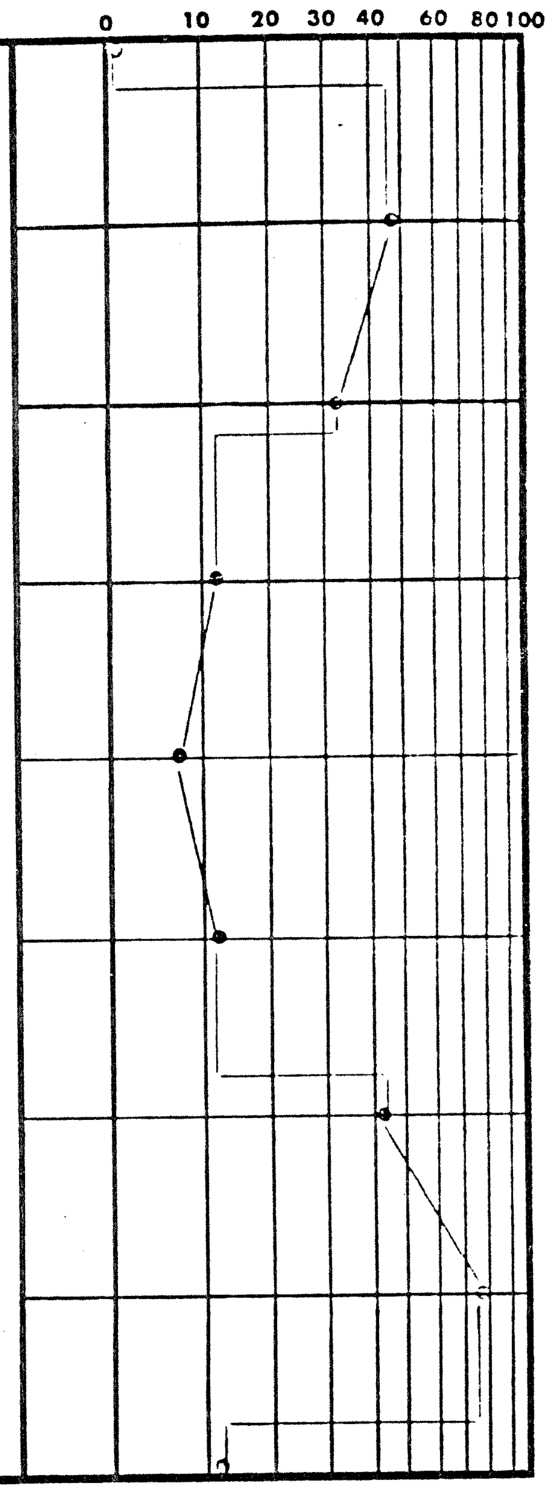
2A-165

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION
40.0	VERY LOOSE blue-grey silty clayey fine SAND
41.5	DENSE blue-grey slightly silty fine SAND with small shell fragments
47.0	DENSE blue-grey slightly silty fine SAND
51.0	FIRM and LOOSE blue-grey slightly silty fine SAND and SHELL FRAGMENTS
69.0	DENSE blue-grey slightly silty fine SAND with shell fragments
73.5	VERY DENSE blue-grey slightly silty fine SAND
78.5	FIRM grey fine SAND and SHELL FRAGMENTS
80.0	

ELEV

• PENETRATION-BLOWS PER FT.



TEST BORING RECORD
(Page 2 of 3 Pages)

BORING NO. B-159

DATE DRILLED 1/8/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

% ROCK CORE RECOVERY

WATER TABLE, 24 HR.

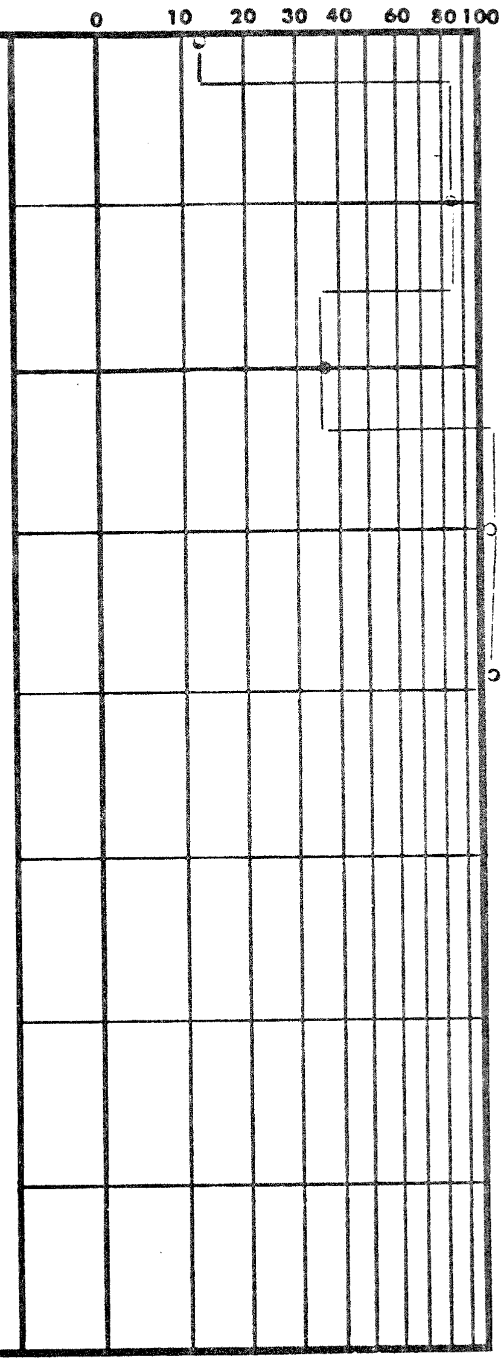
WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

2A-166

DEPTH FT.	DESCRIPTION	ELEV
80.0	FIRM grey fine SAND and SHELL FRAGMENTS	
81.5	VERY DENSE blue-grey slightly silty fine SAND & VERY DENSE blue-grey fine SAND & SHELL FRAGMENT LAYERS	
88.0	DENSE blue-grey slightly silty fine SAND with some small cemented fine sand fragments	
92.5	VERY DENSE blue-grey slightly silty fine SAND with cemented fine sand seams	
95.0	VERY DENSE grey slightly silty fine SAND with fine sand & shell fragment seams	
100.0	BORING TERMINATED	
	(1) slightly silty fine SAND & VERY DENSE blue-grey fine SAND & SHELL FRAGMENT LAYERS	

• PENETRATION-BLOWS PER FT.



TEST BORING RECORD

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BORING NO. B-159 U

DATE DRILLED 1/8/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

50 % ROCK CORE RECOVERY

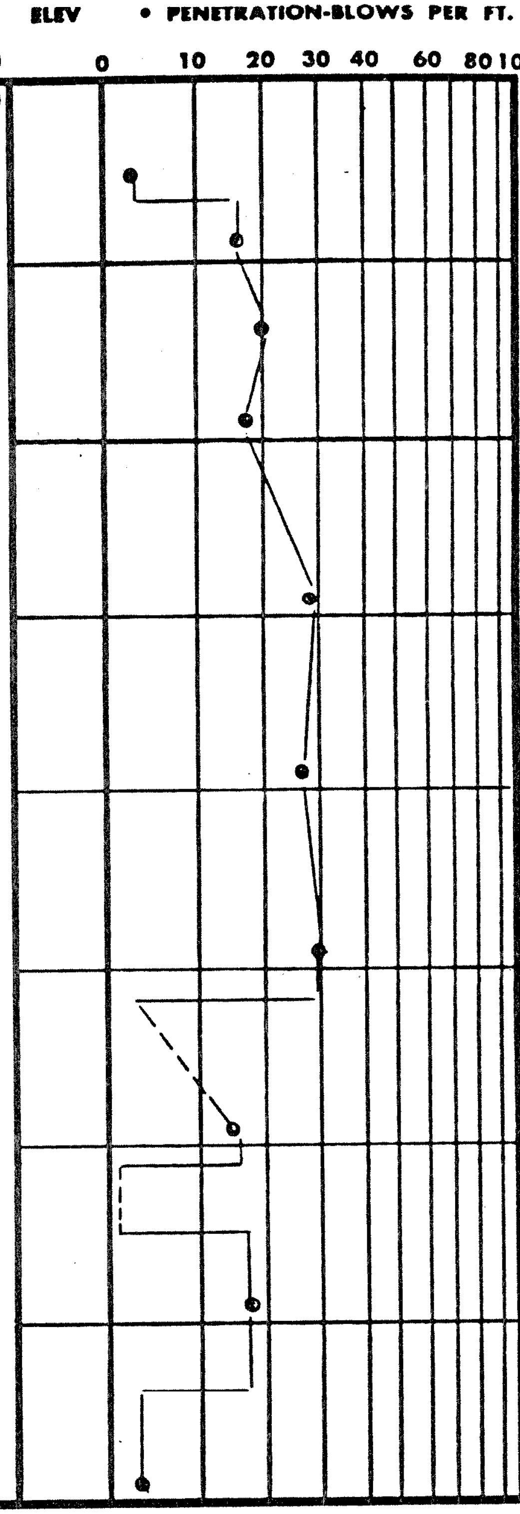
WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

2A-167

DEPTH FT.	DESCRIPTION	ELEV
0		(1)
0.2		(2)
0.5	VERY LOOSE grey slightly silty fine SAND with small roots	
3.5	FIRM grey fine SAND with small shell fragments	
5.5	FIRM blue-grey slightly silty fine SAND with shell fragments	
8.0	FIRM to VERY FIRM blue-grey fine SAND with shell fragments	
17.0	VERY FIRM blue-grey fine SAND & SHELL FRAGMENTS	
26.0	VERY LOOSE to FIRM blue-grey silty clayey to slightly clayey fine SAND	
30.5	VERY SOFT blue-grey clayey fine slightly sandy SILT	
32.5	FIRM blue-grey silty slightly clayey fine SAND	
37.0	VERY LOOSE grey calcareous silty clayey fine SAND with small limestone fragments	
40.0		



TEST BORING RECORD
(Page 1 of 2 Pages)

BORING NO. B-160

DATE DRILLED 1/7,10/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

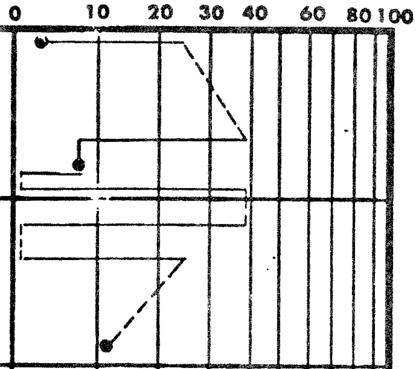
50 % ROCK CORE RECOVERY

LOSS OF DRILLING WATER

2A-168

DEPTH FT.	DESCRIPTION	ELEV
40.0	VERY LOOSE grey calcareous (3)	
40.5	VERY FIRM to DENSE grey fine SAND	
43.5	LOOSE brown fine SAND	
44.5	Decaying WOOD	
45.0	DENSE grey fine SAND	
46.0	VERY SOFT grey clayey SILT	
47.0	VERY FIRM to FIRM grey silty slightly clayey fine SAND	
50.0	BORING TERMINATED	

• PENETRATION-BLOWS PER FT.



(1) VERY SOFT dark brown organic SILT (Peat)
(2) VERY LOOSE dark grey organic slightly silty slightly clayey fine SAND with small roots
(3) silty clayey fine SAND with small limestone fragments

TEST BORING RECORD
(Page 2 of 2 Pages)

BORING NO. B-160H

DATE DRILLED 1/7,10/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

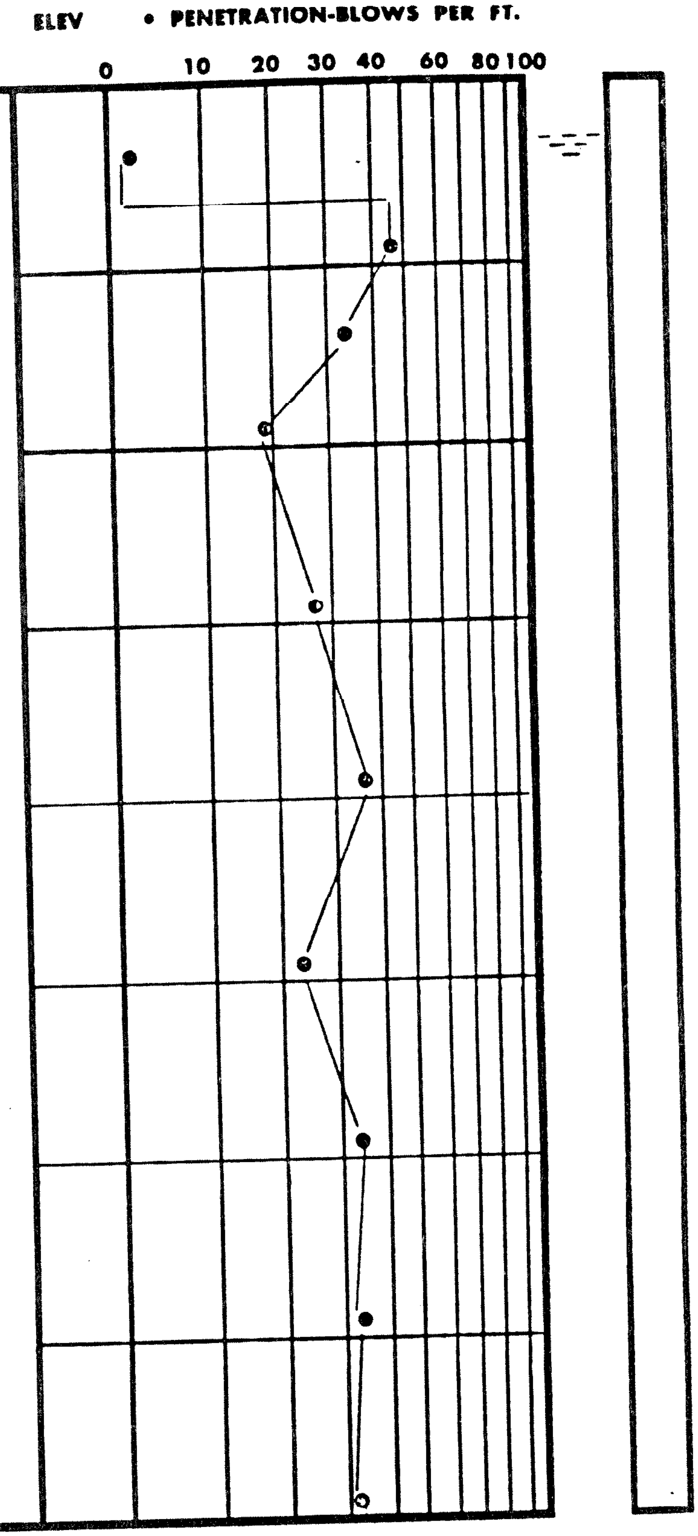
UNDISTURBED SAMPLE — WATER TABLE, 24 HR. — WATER TABLE, 1 HR.

% ROCK CORE RECOVERY — LOSS OF DRILLING WATER

2A-169

DEPTH FT.	DESCRIPTION	
0	VERY SOFT dark brown highly	(1)
1.0	VERY LOOSE grey slightly silty fine SAND with small shell fragments & decayed roots	
3.5	DENSE grey fine SAND with small shell fragments	
8.0	FIRM blue-grey fine SAND	
12.0	VERY FIRM grey-brown fine SAND & SHELL FRAGMENTS	
17.0	DENSE to VERY FIRM grey-brown fine SAND with shell fragments	
27.0	DENSE blue-grey fine SAND with small shell fragments	
37.0	DENSE blue-grey fine SAND	
40.0		

ELEV



TEST BORING RECORD
(Page 1 of 2 Pages)

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER
UNDISTURBED SAMPLE
50 % ROCK CORE RECOVERY

2A-170

BORING NO. B-161 P
DATE DRILLED 1/27/72
JOB NO. J-1540

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION	ELEV	PENETRATION-BLOWS PER FT. (0 10 20 30 40 60 80 100)
.0	DENSE blue-grey fine SAND		
.0	DENSE light grey slightly silty very fine to fine SAND with cemented fine sand fragments		
.0	DENSE blue-grey slightly silty fine SAND with shell & cemented fine sand fragments		
.0	BORING TERMINATED		
	(1) organic SILT (Peat)		

TEST BORING RECORD

(Page 2 of 2 Pages)

BORING NO. B-161 P

DATE DRILLED 1/27/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

% ROCK CORE RECOVERY

WATER - XL, 24 HR.

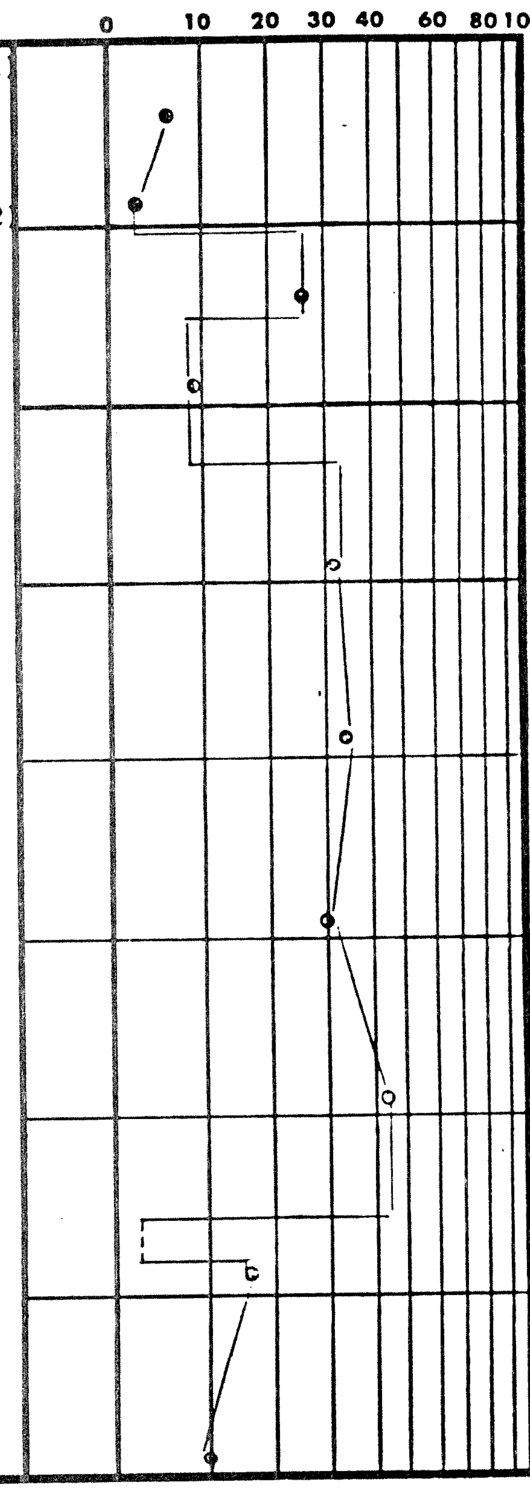
WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

2A-171

DEPTH FT.	DESCRIPTION		ELEV
0			
1.0	LOOSE grey slightly silty fine	(1)	
	LOOSE brown fine SAND with roots		
3.5	VERY LOOSE grey slightly silty fine SAND with small shell	(2)	
5.5	VERY FIRM light grey fine SAND with shell fragments		
8.0	LOOSE brown fine SAND & SHELL FRAGMENTS		
12.0	DENSE & VERY FIRM grey to blue-grey fine SAND with shell fragments		
33.0	VERY LOOSE blue-grey silty clayey fine SAND		
34.0	FIRM blue-grey silty clayey fine SAND with shell fragments		
37.0	LOOSE blue-grey silty clayey fine SAND		
40.0			

• PENETRATION-BLOWS PER FT.



TEST BORING RECORD
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BORING NO. B-162 ᴾ

DATE DRILLED 1/28/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

50 % ROCK CORE RECOVERY

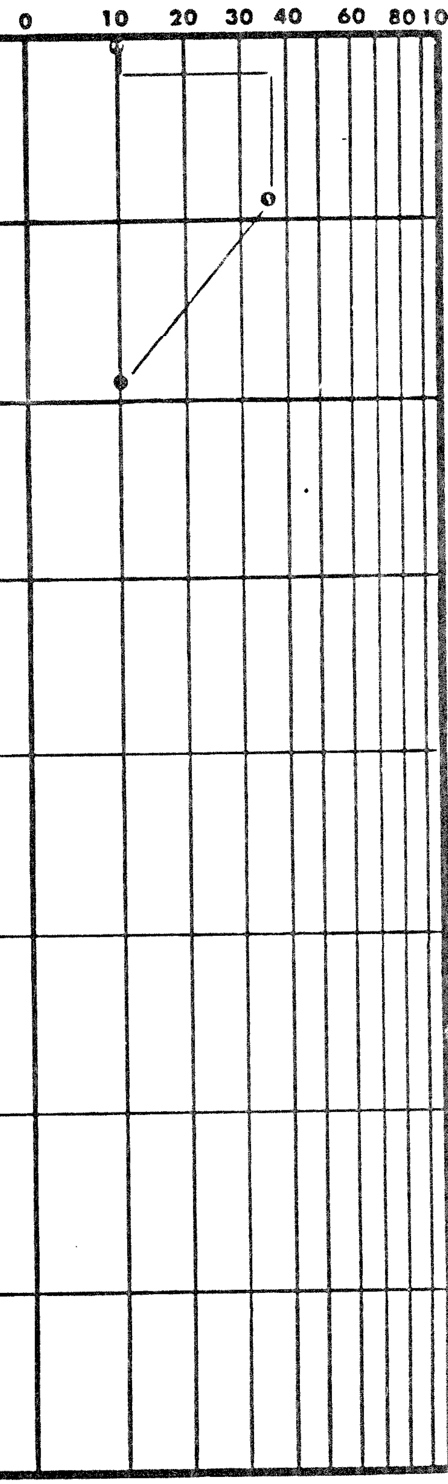
LOSS OF DRILLING WATER

2A-172

DEPTH FT.	DESCRIPTION	ELEV
.0		
.0	LOOSE blue-grey silty clayey	(3
	DENSE light grey slightly silty very fine to fine SAND with cemented fine sand fragments	
.0		
	LOOSE light grey slightly silty very fine to fine SAND with cemented fine sand fragments	
.0	BORING TERMINATED	

(1) SAND with roots (Topsoil)
(2) fragments
(3) fine SAND

• PENETRATION-BLOWS PER FT.



TEST BORING RECORD
(Page 2 of 2 Pages)

BORING NO. B-162 P

DATE DRILLED 1/28/72

JOB NO. J-1540

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

2A-173

% ROCK CORE RECOVERY

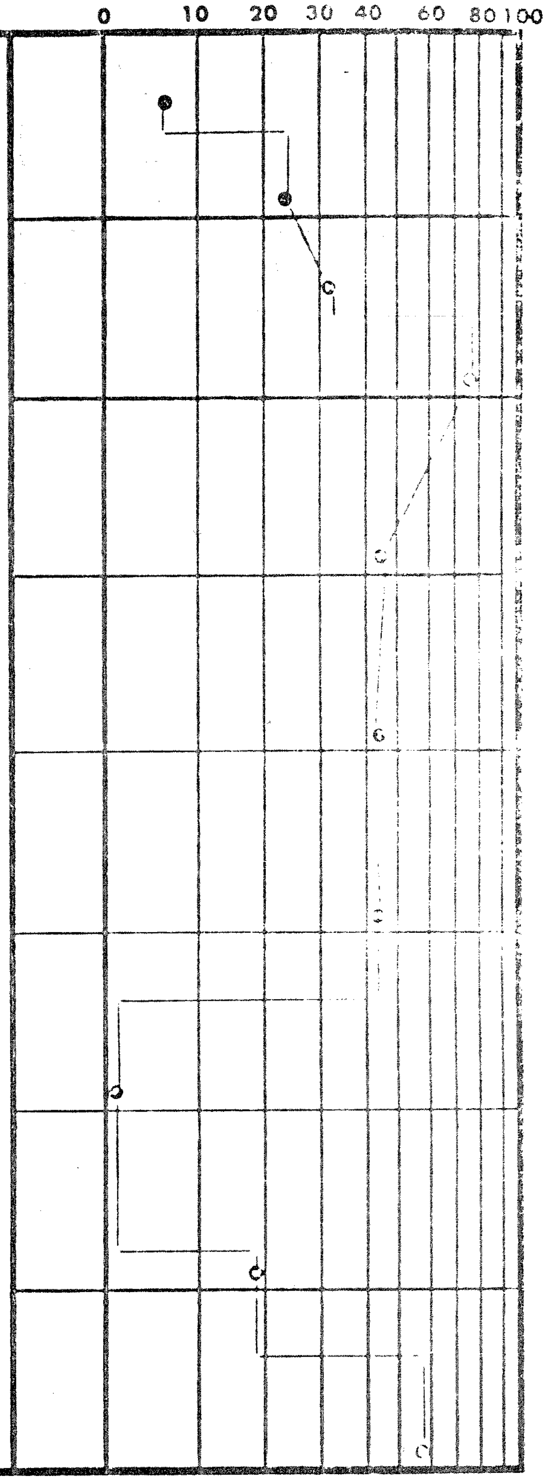
LOSS OF DRILLING WATER

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION
0 – 3.0	VERY LOOSE to LOOSE grey-brown fine SAND & SHELL FRAGMENTS
3.0 – 8.0	VERY FIRM to DENSE brown fine SAND with shell fragments
8.0 – 14.0	VERY DENSE brown fine SAND with shell fragments & cemented fine sand & shell fragments
14.0 – 27.0	DENSE brown fine SAND with some shell fragments
27.0 – 32.0	VERY SOFT blue-grey clayey SILT with thin fine sand seams
32.0 – 34.0	VERY SOFT blue-grey clayey fine sandy SILT
34.0 – 37.0	FIRM blue-grey slightly silty fine SAND
37.0 – 40.0	VERY DENSE blue-grey very slightly silty fine SAND

ELEV

• PENETRATION-BLOWS PER FT.



TEST BORING RECORD

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BORING NO. B-163 P

DATE DRILLED 1/29/72

JOB NO. J-1540

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

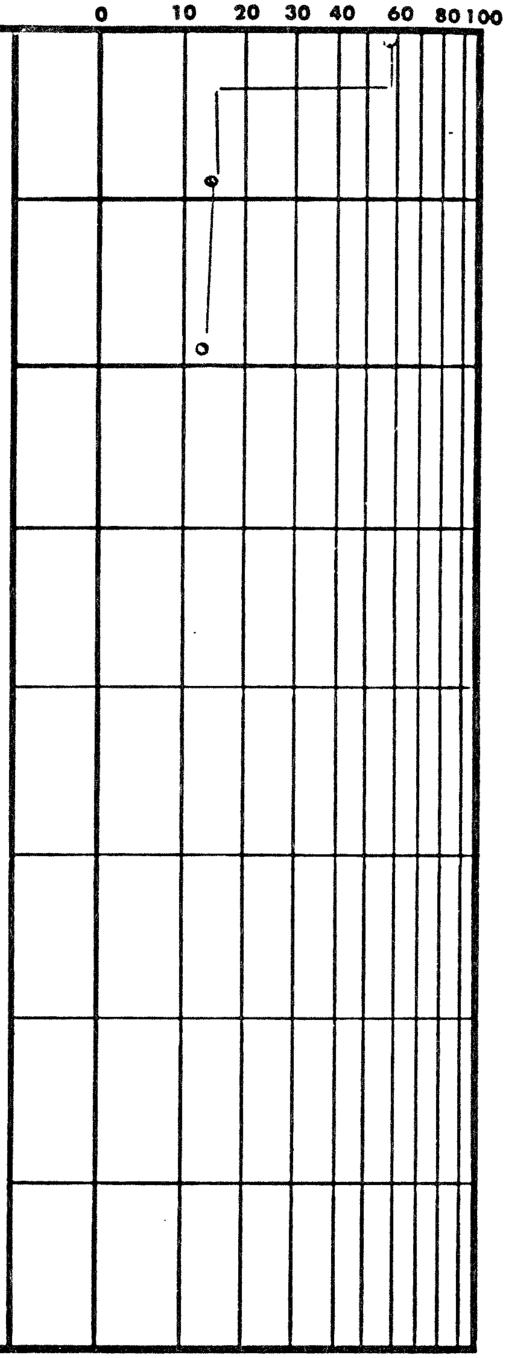
UNDISTURBED SAMPLE

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

DEPTH FT.	DESCRIPTION	ELEV
40.0	VERY DENSE blue-grey very slightly silty fine SAND	
42.0	FIRM grey-green clayey fine SAND with cemented fine sand fragments	
46.0	FIRM light grey silty very fine to fine SAND with cemented fine sand fragments	
50.0	BORING TERMINATED	

• PENETRATION-BLOWS PER FT.



TEST BORING RECORD

(Page 2 of 2 Pages)

BORING NO. B-163 P

DATE DRILLED 1/29/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

50 % ROCK CORE RECOVERY

WATER TABLE, 24 HR.

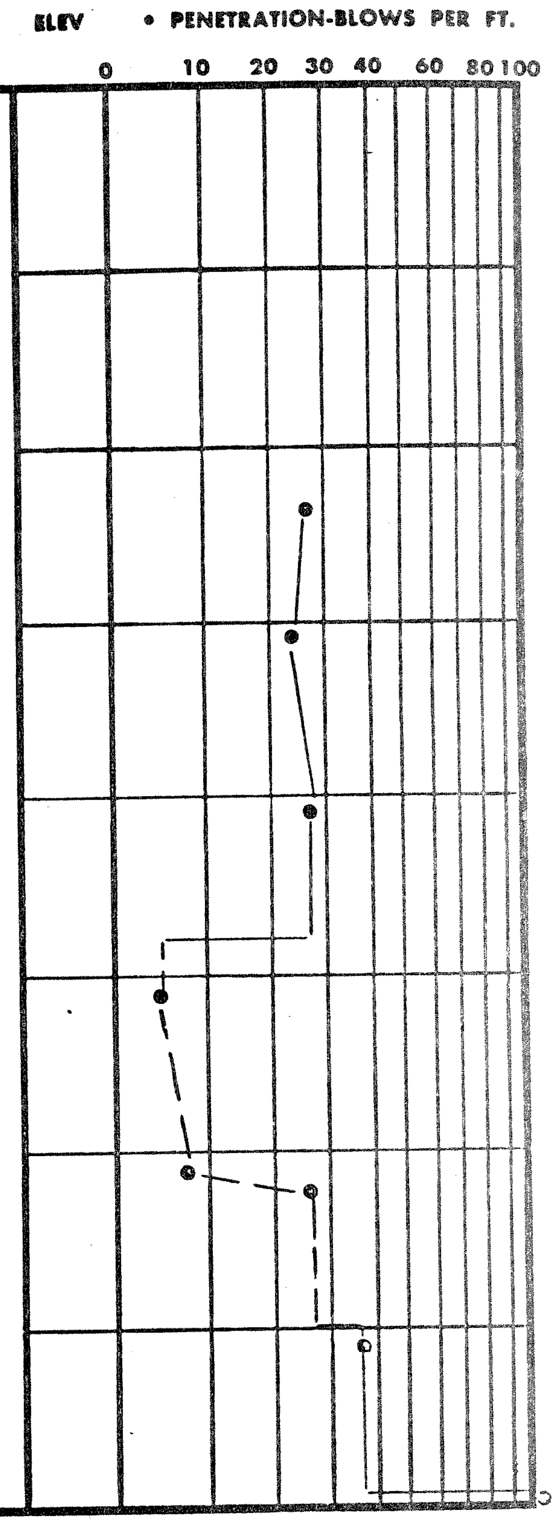
WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

2A-175

DEPTH FT.	DESCRIPTION
0	Water in Atlantic Ocean (Near Low Tide)
11.0	VERY FIRM grey fine SAND
14.0	VERY FIRM grey fine SAND with shell fragments & limestone fragments
18.0	VERY FIRM brown fine SAND & Small SHELL FRAGMENTS
24.0	Alternating Layers of LOOSE blue-grey silty slightly clayey very fine to fine SAND & VERY FIRM slightly silty fine SAND
35.0	DENSE blue-grey slightly silty fine SAND with shell fragments
40.0	

ELEV



TEST BORING RECORD

(Page 1 of 2 Pages)

BORING NO. B-165 P

DATE DRILLED 2/7&15/72

JOB NO. J-1540

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

WATER TABLE, 24 HR.

UNDISTURBED SAMPLE

WATER TABLE, 1 HR.

% ROCK CORE RECOVERY

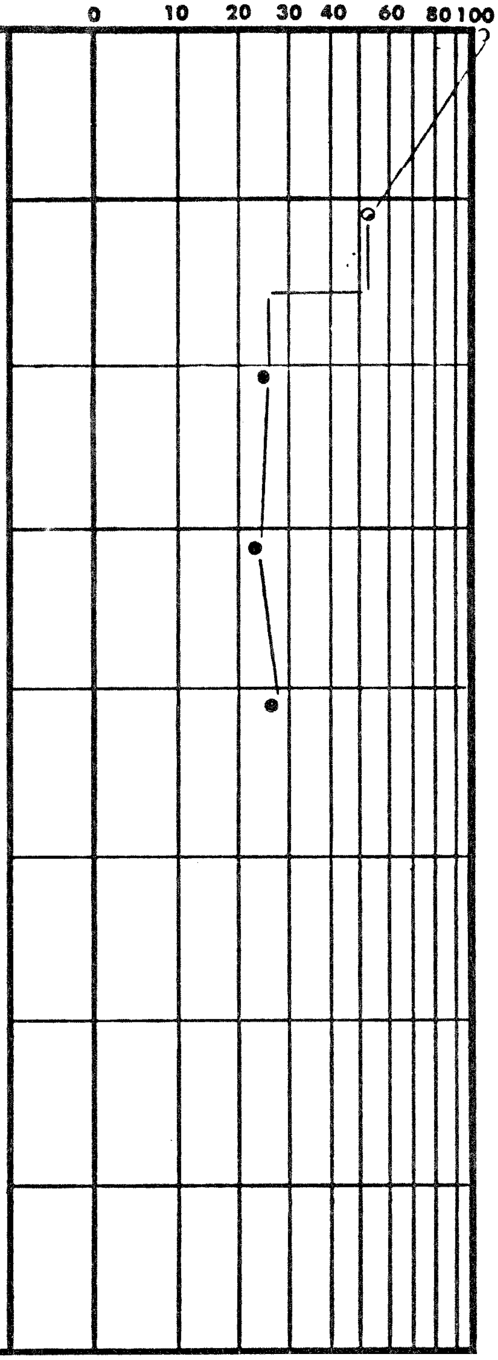
LOSS OF DRILLING WATER

2A-176

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION	ELEV
40.0	VERY DENSE light blue-grey very fine to fine SAND with small shell fragments & cemented fine sand fragments	
43.0	VERY DENSE light blue-grey very fine to fine SAND with shell fragments & cemented fine sand & limestone fragments	
48.0	VERY FIRM light blue-grey slightly silty fine SAND & SHELL FRAGMENTS with cemented fine sand & shell fragments	
53.0	VERY FIRM blue-grey slightly silty fine SAND & SHELL FRAGMENTS	
58.0	VERY FIRM grey fine SAND & SHELL FRAGMENTS	
61.0	BORING TERMINATED	

• PENETRATION-BLOWS PER FT.



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

2A-177

TEST BORING RECORD

(Page 2 of 2 Pages)

BORING NO. B-165P

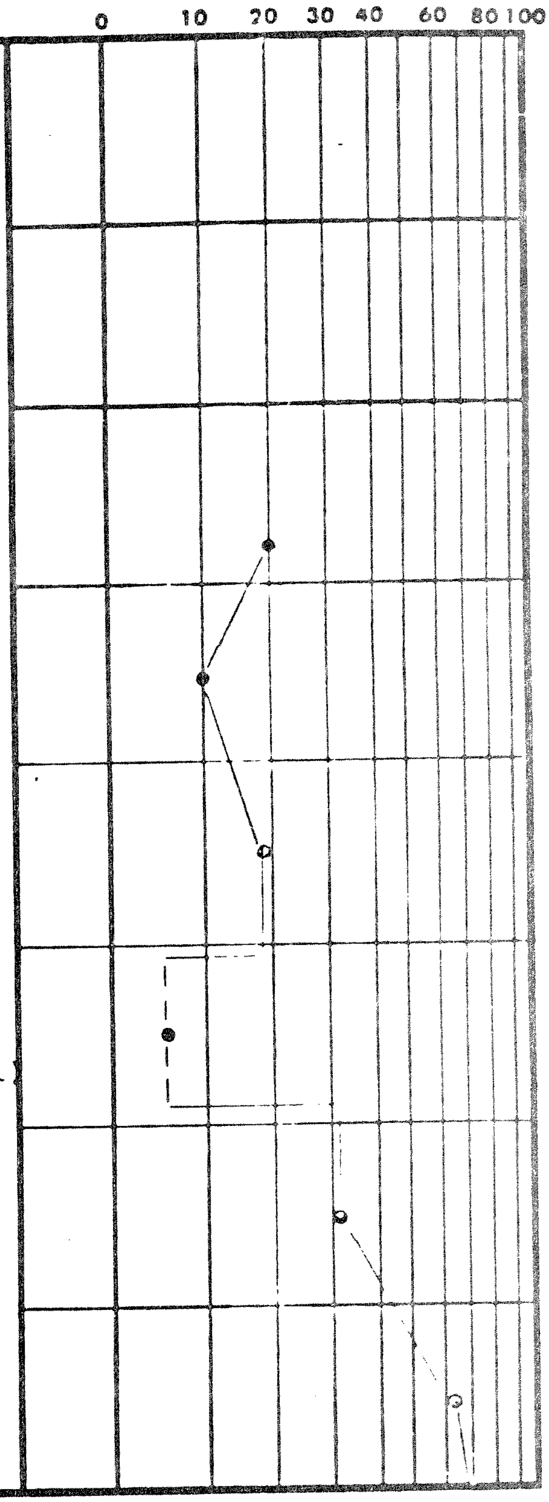
DATE DRILLED 2/7&15/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION	ELEV
0	Water in Atlantic Ocean (Near Low Tide)	
13.0	FIRM grey fine SAND with shell fragments	
15.5	LOOSE grey very slightly silty fine SAND with shell fragments	
20.0	FIRM grey-brown fine SAND and SHELL FRAGMENTS	
25.5	Alternating Layers of LOOSE blue-grey silty clayey fine SAND with shell fragments &	(1
29.5	DENSE to VERY DENSE blue-grey fine SAND	
40.0		

• PENETRATION-BLOWS PER FT.



(1) SOFT clayey SILT

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

2A-178

TEST BORING RECORD

(Page 1 of 2 Pages)

BORING NO. B-166

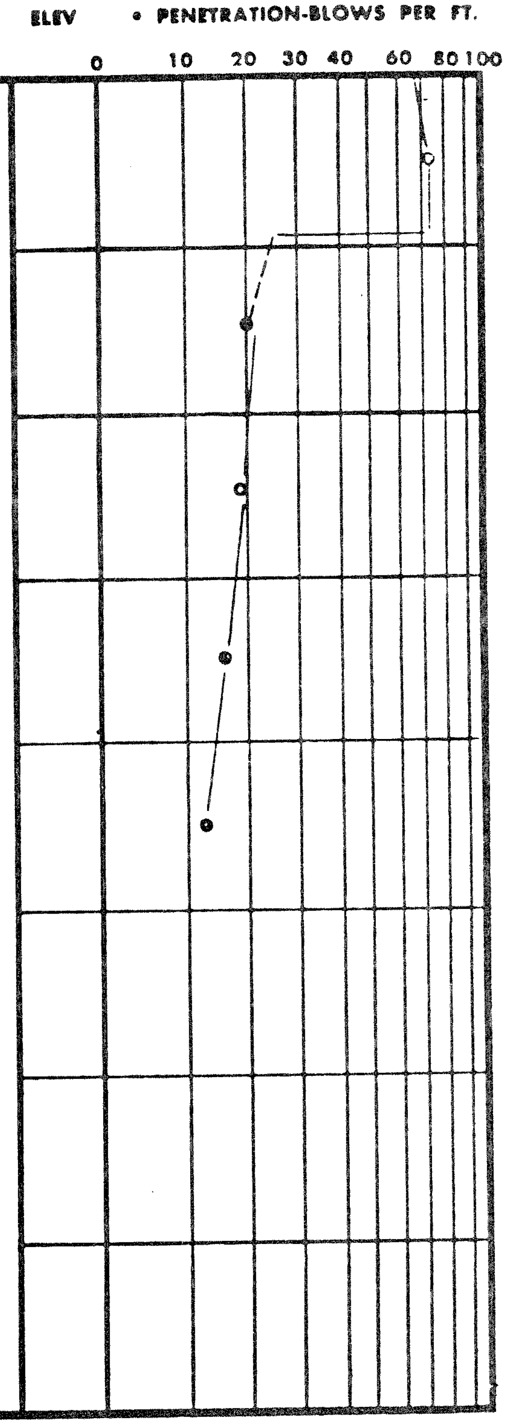
DATE DRILLED 2/1&7/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION
40.0	VERY DENSE light blue-grey very fine to fine SAND with small shell fragments & cemented fine sand seams
45.0	VERY FIRM to FIRM light blue-grey slightly silty very fine to fine SAND with cemented fine sand fragments
49.5	FIRM light blue-grey slightly silty fine SAND & SHELL FRAGMENTS with cemented fine sand & shell fragments
55.0	FIRM grey to blue-grey slightly silty to silty slightly clayey to clayey fine SAND & SHELL FRAGMENTS
63.0	BORING TERMINATED

ELEV



TEST BORING RECORD

(Page 2 of 2 Pages)

BORING NO. B-166 P

DATE DRILLED 2/1&7/72

JOB NO. J-1540

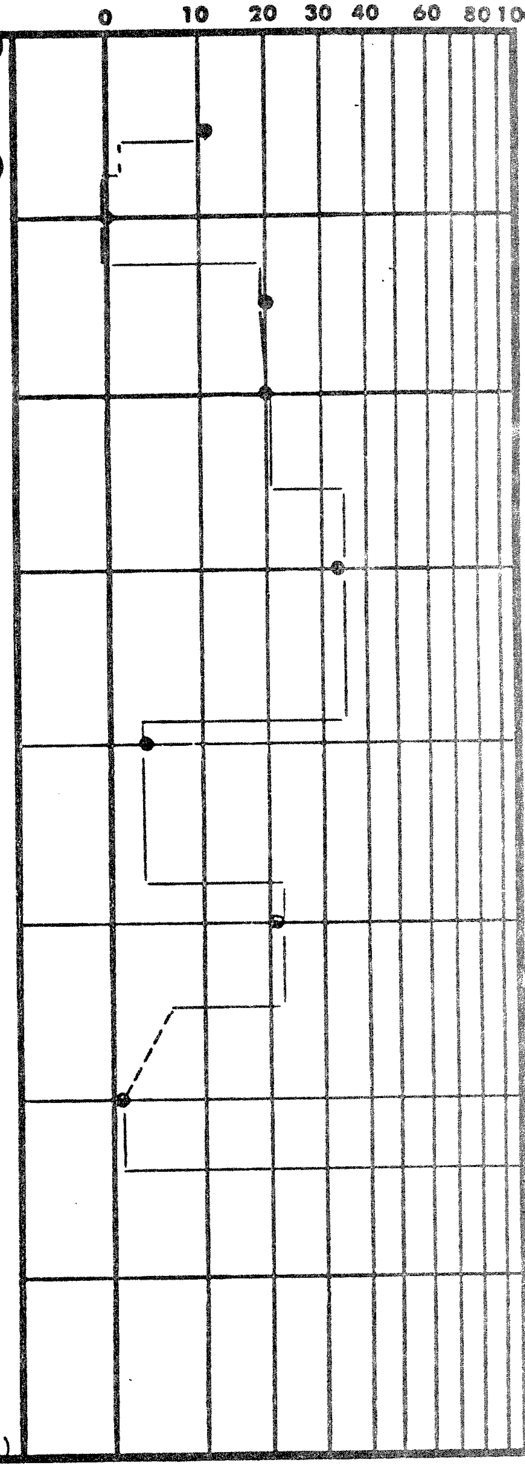
LAW ENGINEERING TESTING CO.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

WATER TABLE, 24 HR.
UNDISTURBED SAMPLE
WATER TABLE, 1 HR.
2A-179
% ROCK CORE RECOVERY
LOSS OF DRILLING WATER

DEPTH FT.	DESCRIPTION	ELEV
0		
0.3	FIRM tan fine SAND with shell fragments & cemented fine sand & shell fragments (Fill)	(1)
3.0	VERY LOOSE blue-grey silty	(2)
4.0	VERY SOFT dark brown organic fine slightly sandy SILT (Peat)	
6.5	FIRM blue-grey fine SAND with small shell fragments	
8.5	FIRM blue-grey fine SAND & SHELL FRAGMENTS	
13.0	DENSE blue-grey slightly silty fine SAND with fine sand & shell seams	
19.5	SOFT blue-grey clayey fine slightly sandy SILT	
24.0	VERY FIRM blue-grey slightly silty slightly clayey fine SAND	
27.5	LOOSE to VERY LOOSE blue-grey silty clayey fine SAND & VERY SOFT fine sandy silty clay LAYERS	
32.0	VERY DENSE grey cemented fine SAND	
39.5	MODERATELY HARD tan sandy	(3)
40.0		

• PENETRATION-BLOWS PER FT.



TEST BORING RECORD
(Page 1 of 3 Pages)

BORING NO. B-169 H

DATE DRILLED 1/13,14/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

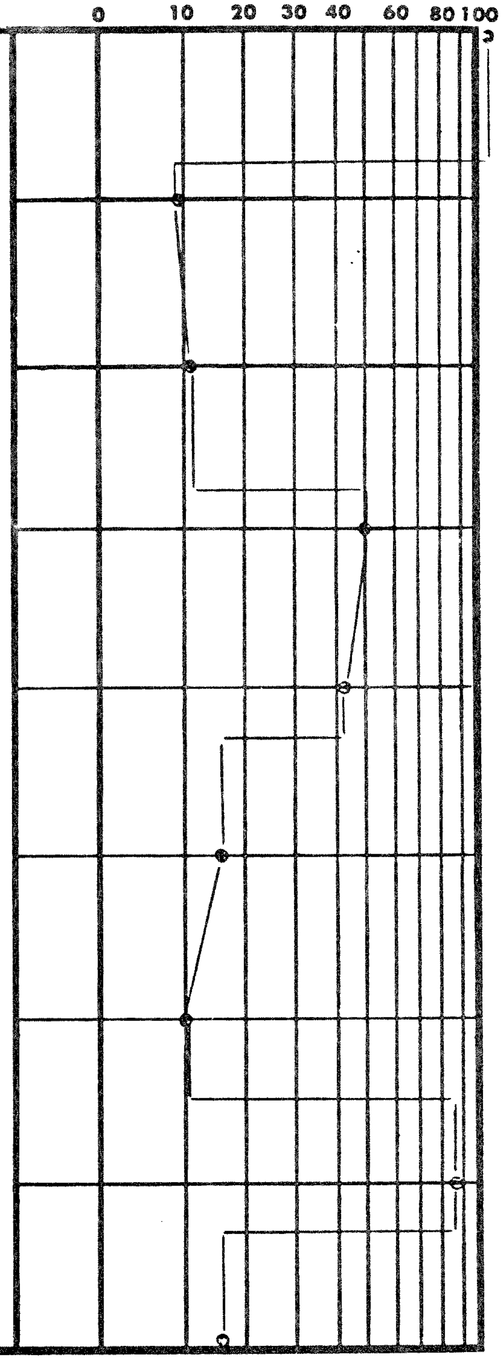
50 % ROCK CORE RECOVERY

LOSS OF DRILLING WATER

2A-180

DEPTH FT.	DESCRIPTION
40.0	MODERATELY HARD tan sandy LIMESTONE
44.0	LOOSE to FIRM tan slightly silty fine SAND with small limestone seams
54.0	DENSE grey cemented fine SAND with slightly silty fine sand seams
61.5	FIRM tan cemented fine SAND & slightly silty fine sand LAYERS
66.0	LOOSE tan slightly silty fine SAND with seams of cemented fine sand & shell fragments
72.5	VERY DENSE blue-grey slightly silty fine SAND
76.5	FIRM blue-grey slightly silty fine SAND with shell fragments
80.0	

ELEV



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

50 % ROCK CORE RECOVERY

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

2A-181

TEST BORING RECORD

(Page 2 of 3 Pages)

BORING NO. B-169 H

DATE DRILLED 1/13,14/72

JOB NO. J-1540

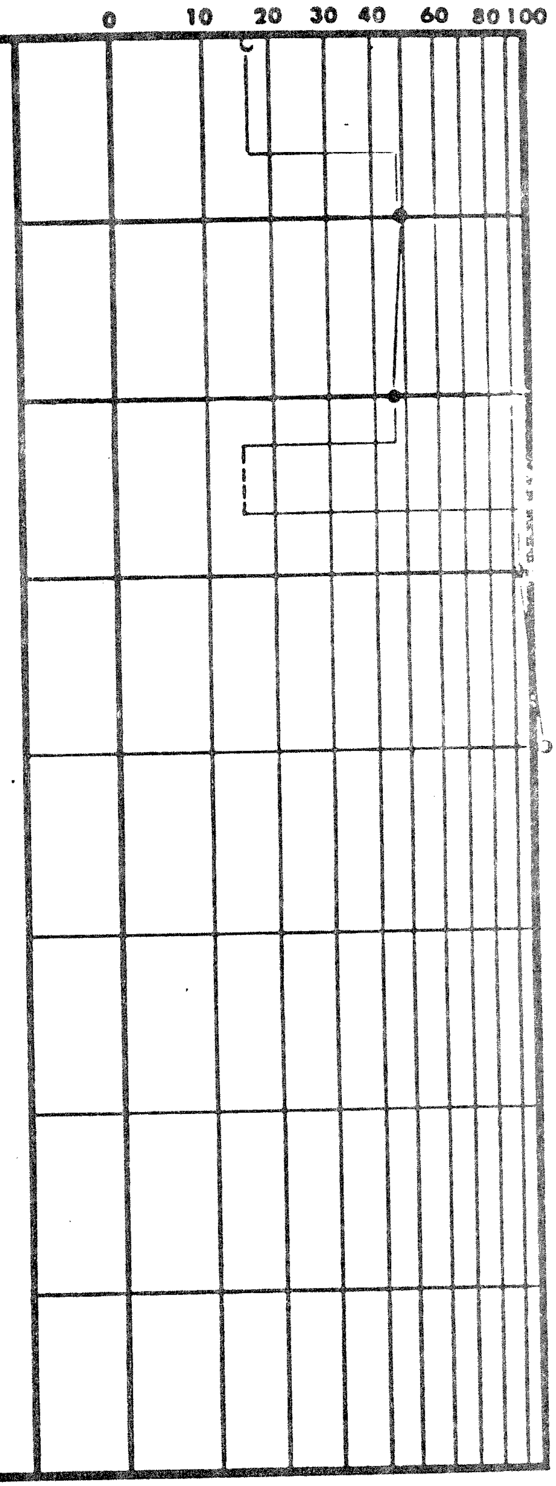
LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION
80.0	FIRM blue-grey slightly silty fine SAND with shell fragments
83.5	DENSE blue-grey slightly silty fine SAND & layers of fine SAND with shell fragments
91.5	FIRM blue-grey slightly silty fine SAND
93.5	VERY DENSE grey slightly silty fine SAND with some shell fragments
100.0	BORING TERMINATED

(1) FIRM grey silty CLAY (Fill)
(2) slightly clayey fine SAND with shell fragments
(3) LIMESTONE

ELEV

• PENETRATION-BLOWS PER FT.



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
% ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

2A-182

TEST BORING RECORD

(Page 3 of 3 ~~Pages~~)

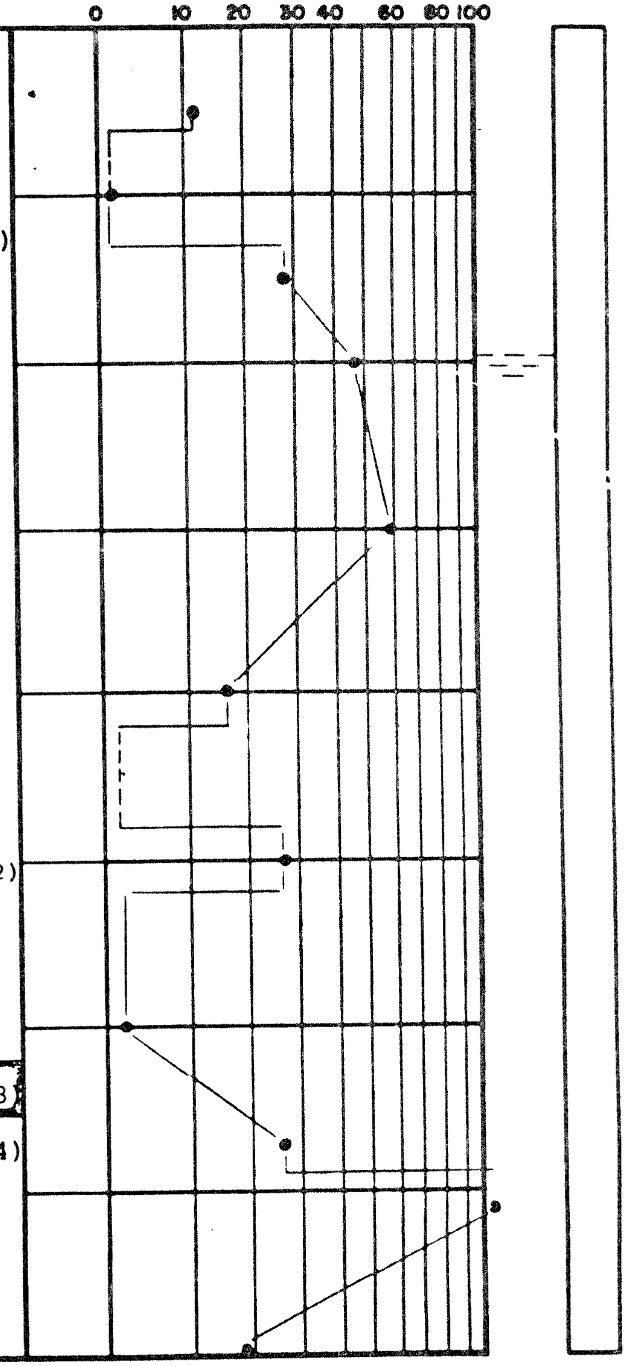
BORING NO. B-169 H

DATE DRILLED 1/13,14/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION	ELEV	● PENETRATION-BLOWS PER FT. 0 10 20 30 40 60 80 100
.0	FIRM tan fine SAND with shell fragments (Fill)		
3.0	VERY SOFT dark brown highly organic SILT (Peat)		
4.5	VERY LOOSE grey silty slightly clayey fine SAND with shell fragments & decayed		
6.5	VERY FIRM to DENSE blue-grey fine SAND with shell fragments	(1)	
14.0	VERY DENSE to FIRM blue-grey fine SAND		
21.0	VERY LOOSE blue-grey silty clayey fine SAND & SOFT silty fine slightly sandy CLAY		
24.0	VERY FIRM blue-grey slightly silty very slightly clayey	(2)	
26.0	VERY LOOSE blue-grey silty clayey fine SAND		
31.0	VERY LOOSE to FIRM dark grey slightly silty very slightly	(3)	
32.5	VERY FIRM grey slightly silty fine SAND with shell &	(4)	
34.5	VERY DENSE to FIRM grey-tan cemented fine SAND with silty fine sand		
40.0			



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

| 50 | % ROCK CORE RECOVERY

LOSS OF DRILLING WATER

TEST BORING RECORD

(Page 1 of 3 Pages)

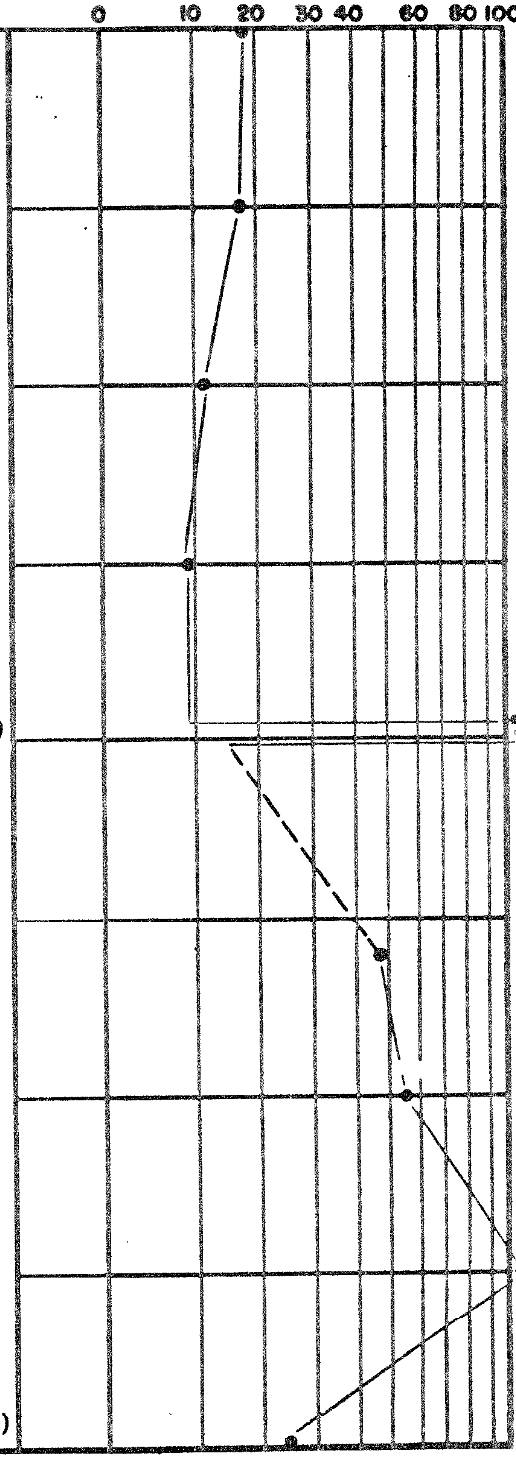
BORING NO. B-170 H

DATE DRILLED 1/21,22/72

JOB NO. J-1540

2A-183

Depth	Description	Sample
40.0	VERY DENSE to FIRM grey-tan	(5)
41.0	LOOSE and FIRM grey-tan silty to slightly silty fine SAND with cemented fine sand shell, quartz & limestone seams	
51.0	LOOSE and FIRM grey-green cemented fine SAND, SHELL, QUARTZ & LIMESTONE & silty slightly clayey fine sand SEAMS	
59.5	VERY DENSE grey cemented	(6)
60.0	FIRM to VERY DENSE blue-grey very slightly silty to slightly silty fine SAND with shell fragments	
72.0	VERY DENSE grey slightly silty fine SAND	
76.5	DENSE to VERY FIRM blue-grey slightly silty fine SAND with shell fragments & slightly silty fine sand	
80.0		(7)



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
| 50 | % ROCK CORE RECOVERY
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
LOSS OF DRILLING WATER

TEST BORING RECORD

(Page 2 of 3 Pages)

BORING NO. B-170 H

DATE DRILLED 1/21,22/72

JOB NO. J-1540

2A-184

FT.

Depth	Description		Penetration (0 10 20 30 40 60 80 100)
80.0	DENSE to VERY FIRM blue-grey slightly silty fine SAND	(8)	
82.0	VERY DENSE blue-grey slightly silty fine SAND & DENSE fine SAND & SHELL FRAGMENTS		
87.0	VERY DENSE blue-grey slightly silty fine SAND with cemented fine sand seams & some shell fragments		
97.0	VERY DENSE grey slightly silty fine SAND		
100.0	BORING TERMINATED		

(1) roots
(2) fine SAND
(3) clayey fine SAND with shell fragments
(4) cemented fine sand fragments
(5) cemented fine SAND with silty fine sand
(6) fine SAND
(7) seams
(8) with shell fragments & slightly silty fine sand seams

BORING AND SAMPLING MEETS ASTM D-1586
ORE DRILLING MEETS ASTM D-2113
ENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE — WATER TABLE, 24 HR. — WATER TABLE, 1 HR.

|30| % ROCK CORE RECOVERY — LOSS OF DRILLING WATER

TEST BORING RECORD
(Page 3 of 3 Pages)

BORING NO. B-170H

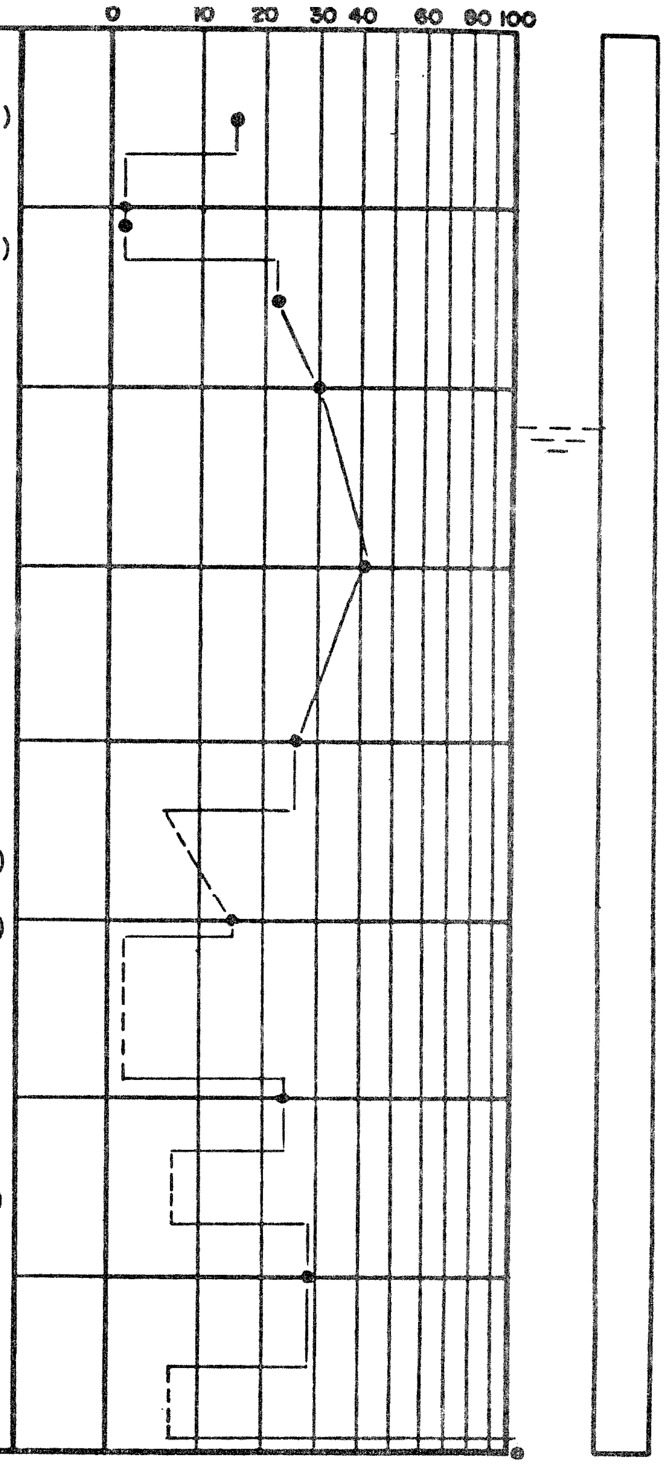
DATE DRILLED 1/21,22/72

JOB NO. J-1540

2A-185

DEPTH FT. 0

Depth	Description	Sample
3.5	FIRM light tan to grey tan fine SAND with shell fragments & cemented fine sand &	(1)
5.0	VERY SOFT dark brown highly organic SILT (Peat)	
6.5	VERY SOFT dark grey organic fine sandy slightly clayey	(2)
8.0	VERY FIRM grey fine SAND	
12.0	VERY FIRM grey fine S^ND & SHELL FRAGMENTS	
22.0	DENSE to VERY FIRM blue-grey fine SAND with some shell fragments	
24.0	LOOSE blue-grey slightly silty slightly clayey fine	(3)
	FIRM blue-grey fine SAND with some small shell	(4)
29.5	VERY SOFT blue-grey clayey fine slightly sandy SILT with shell fragments	
31.5	VERY FIRM dark grey fine SAND	
33.5	LOOSE blue-grey slightly silty slightly clayey fine	(5)
37.5	VERY FIRM blue-grey partially cemented slightly silty fine SAND	
39.5	LOOSE blue-grey slightly silty fine SAND	(6)
40.0		



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

| 50 | % ROCK CORE RECOVERY

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

TEST BORING RECORD
(Page 1 of 3 Pages)

BORING NO. B-171A

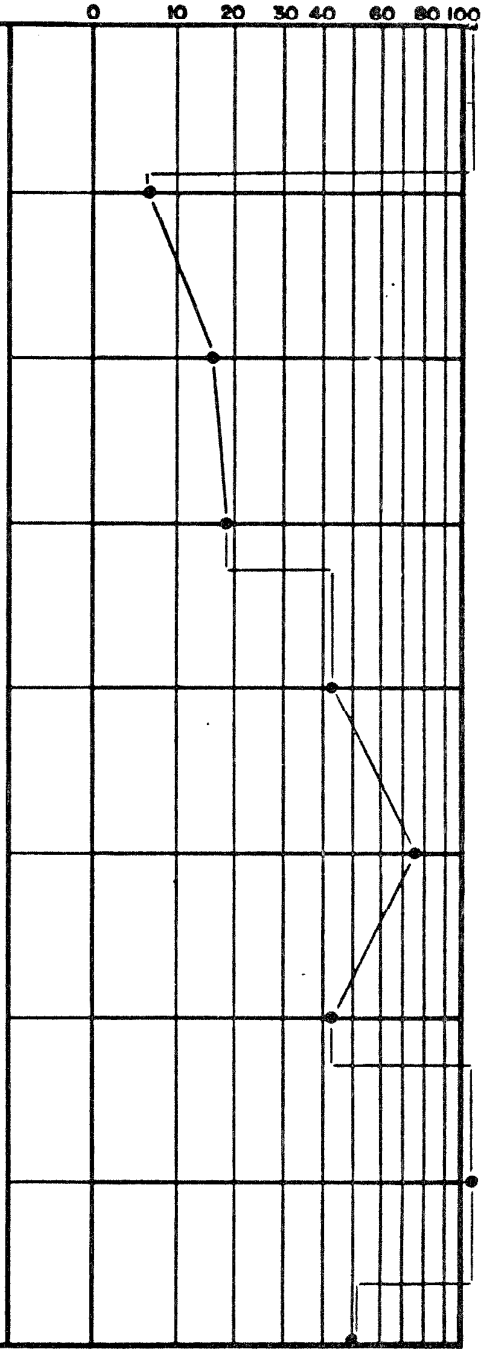
DATE DRILLED 1/15,17/72

JOB NO. J-1540

2A-186

DEPTH FT. 60.0

VERY DENSE blue-grey cemented fine SAND, SHELL FRAGMENTS & QUARTZ	
44.5	
LOOSE to FIRM grey tan slightly silty to silty fine SAND with seams of cemented fine sand & shell fragments	
51.0	
FIRM grey tan cemented fine SAND & SHELL FRAGMENTS	
56.5	
DENSE grey cemented fine SAND & SHELL FRAGMENTS with quartz fragments & silty fine sand lenses	
61.5	
VERY DENSE to DENSE blue-grey fine SAND with shell fragments	
71.5	
VERY DENSE blue-grey slightly silty fine SAND	
78.0	
DENSE blue-grey slightly silty fine SAND & layers of	80.0 (7)



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
50 | % ROCK CORE RECOVERY
LOSS OF DRILLING WATER

TEST BORING RECORD
(Page 2 of 3 Pages)

BORING NO. B-171H
DATE DRILLED 1/15,17/72
JOB NO. J-1540

2A-187

DEPTH FT. 60.0

DENSE blue-grey slightly silty fine SAND & layers of slightly silty fine SAND with shell fragments 83.5

VERY DENSE blue-grey slightly silty fine SAND with shell fragments

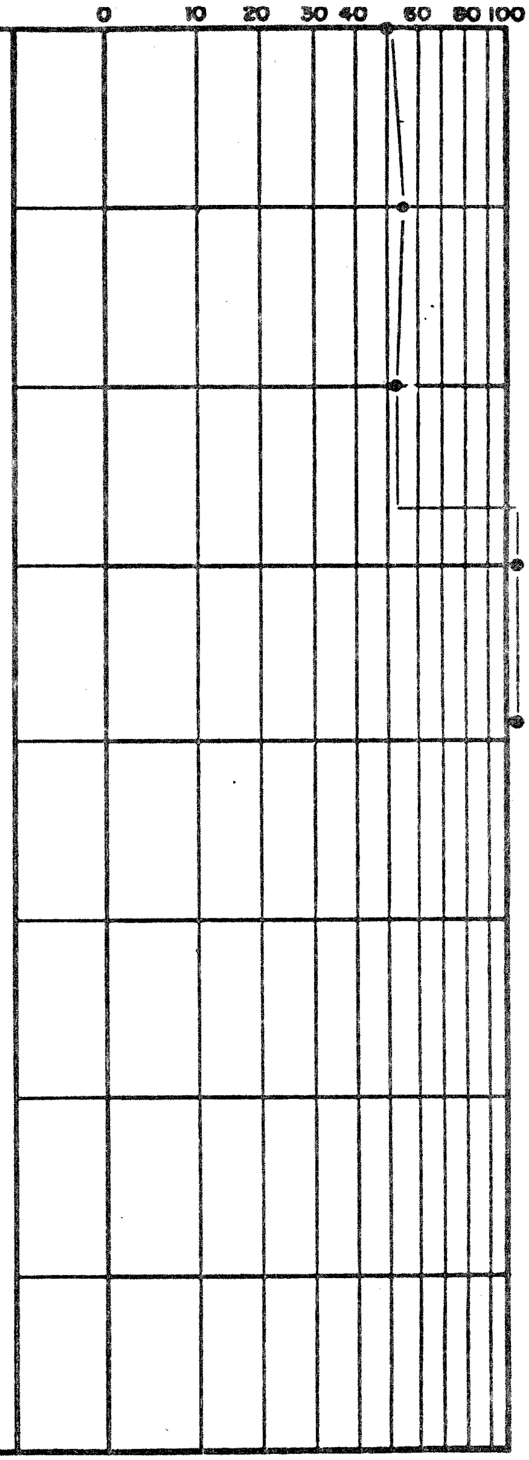
93.5

VERY DENSE blue-grey to grey slightly silty fine SAND

100.0

BORING TERMINATED

(1) shell fragments (Fill)
(2) SILT (Peat)
(3) SAND with shell fragments
(4) fragments
(5) SAND
(6) VERY DENSE blue-grey cemented fine SAND, SHELL FRAGMENTS & QUARTZ
(7) slightly silty fine SAND with shell fragments



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
50 | % ROCK CORE RECOVERY
LOSS OF DRILLING WATER

TEST BORING RECORD
(Page 3 of 3 Pages)

BORING NO. B-171
DATE DRILLED 1/15,17/72
JOB NO. J-1540

2A-188

FT.

0

1.0 LOOSE tan fine SAND with (1)

1.5 VERY LOOSE blue-grey silty (2)

VERY SOFT dark brown highly organic SILT (Peat)

3.5

VERY LOOSE dark grey organic silty fine SAND

5.0

VERY LOOSE to DENSE blue-grey fine SAND with some shell fragments

11.0

VERY DENSE to FIRM blue-grey fine SAND with some small shell fragments

20.5

VERY LOOSE blue-grey silty clayey fine SAND 22.0

VERY FIRM blue-grey fine SAND with small shell fragments

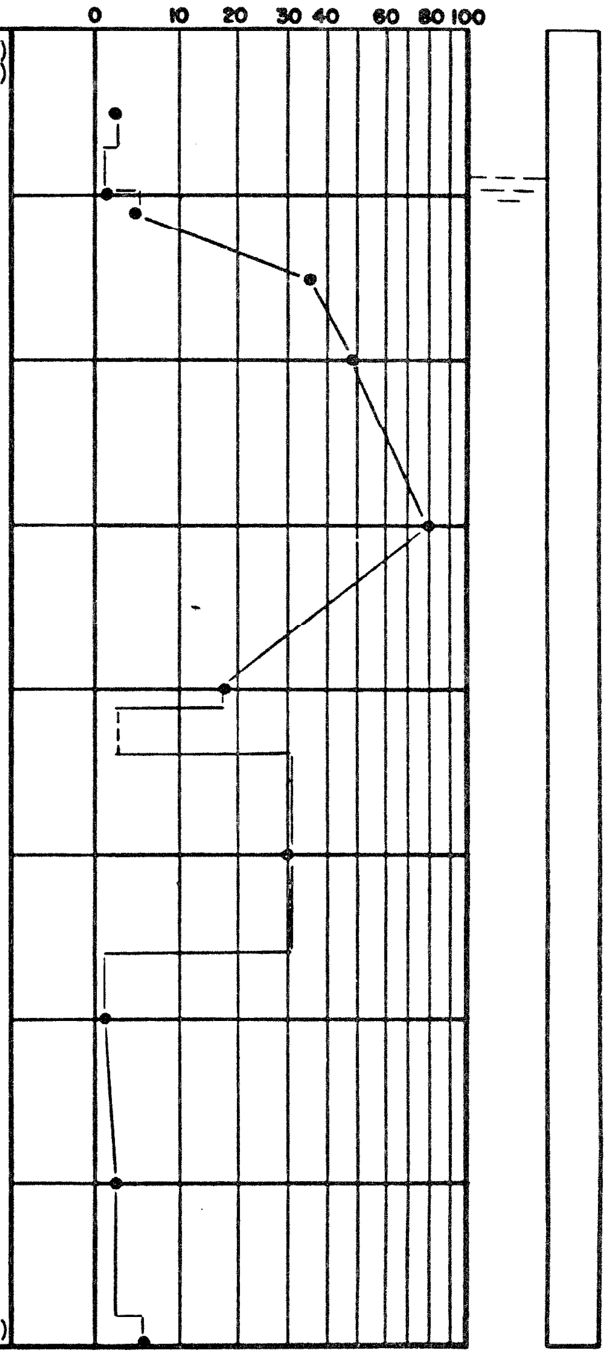
28.0

VERY SOFT blue-grey clayey SILT

39.0

LOOSE blue-grey silty clayey (3)

40.0



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

30 | % ROCK CORE RECOVERY

LOSS OF DRILLING WATER

TEST BORING RECORD
(Page 1 of 3 Pages)

BORING NO. B-172 H

DATE DRILLED 1/18/72

JOB NO. J-1540

FT.

40.0

LOOSE blue-grey silty clayey fine SAND

42.0

FIRM to LOOSE grey to blue-grey slightly silty to silty fine SAND with cemented fine sand fragments

54.0

VERY FIRM tan cemented fine SAND with quartz fragments & silty fine sand lenses

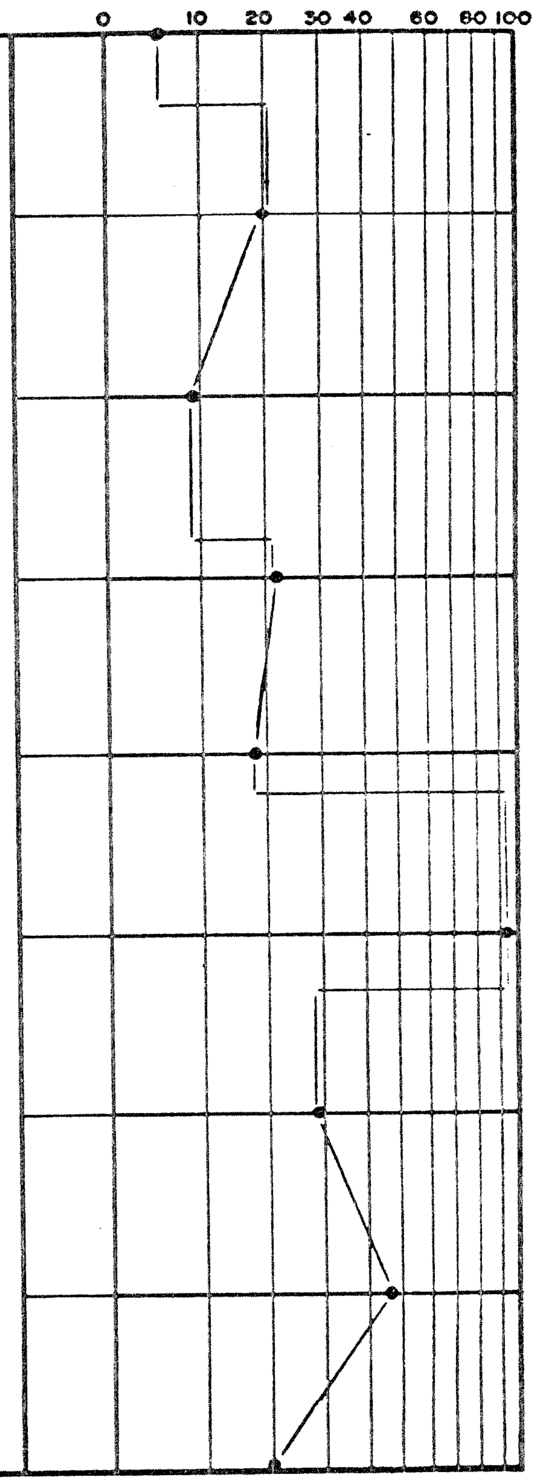
FIRM tan slightly silty fine SAND with shell fragments

61.0

VERY DENSE blue-grey fine SAND with small shell fragments

66.5

FIRM to DENSE blue-grey slightly silty fine SAND & SHELL FRAGMENTS & DENSE to VERY DENSE blue-grey slightly silty fine sand LAYERS



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
ENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER ALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

50 | % ROCK CORE RECOVERY

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

TEST BORING RECORD

(Page 2 of 3 Pages)

BORING NO. B-172 H

DATE DRILLED 1/18/72

JOB NO. I-1540

2A-190

FT.

80.0

FIRM to DENSE blue-grey slightly silty fine SAND & SHELL FRAGMENTS & DENSE to VERY DENSE blue-grey slightly silty fine sand LAYERS

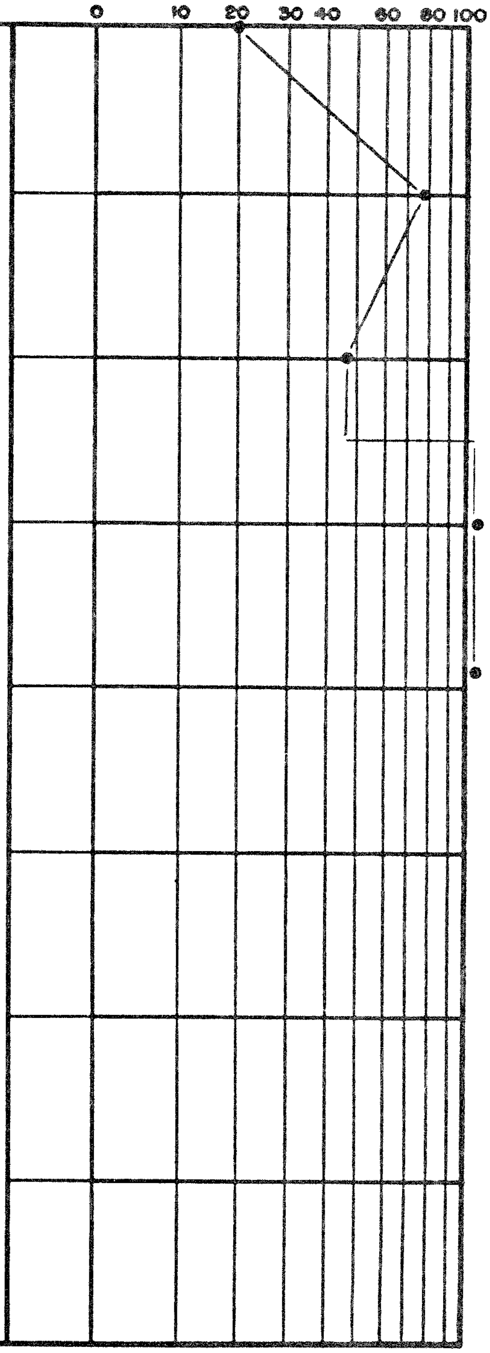
92.5

VERY DENSE grey slightly silty fine SAND

100.0

BORING TERMINATED

(1) shell fragments (Fill)
(2) fine SAND with some shell fragments
(3) fine SAND



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

| 50 | % ROCK CORE RECOVERY

LOSS OF DRILLING WATER

TEST BORING RECORD
(Page 3 of 3 Pages)

BORING NO. B-172 H

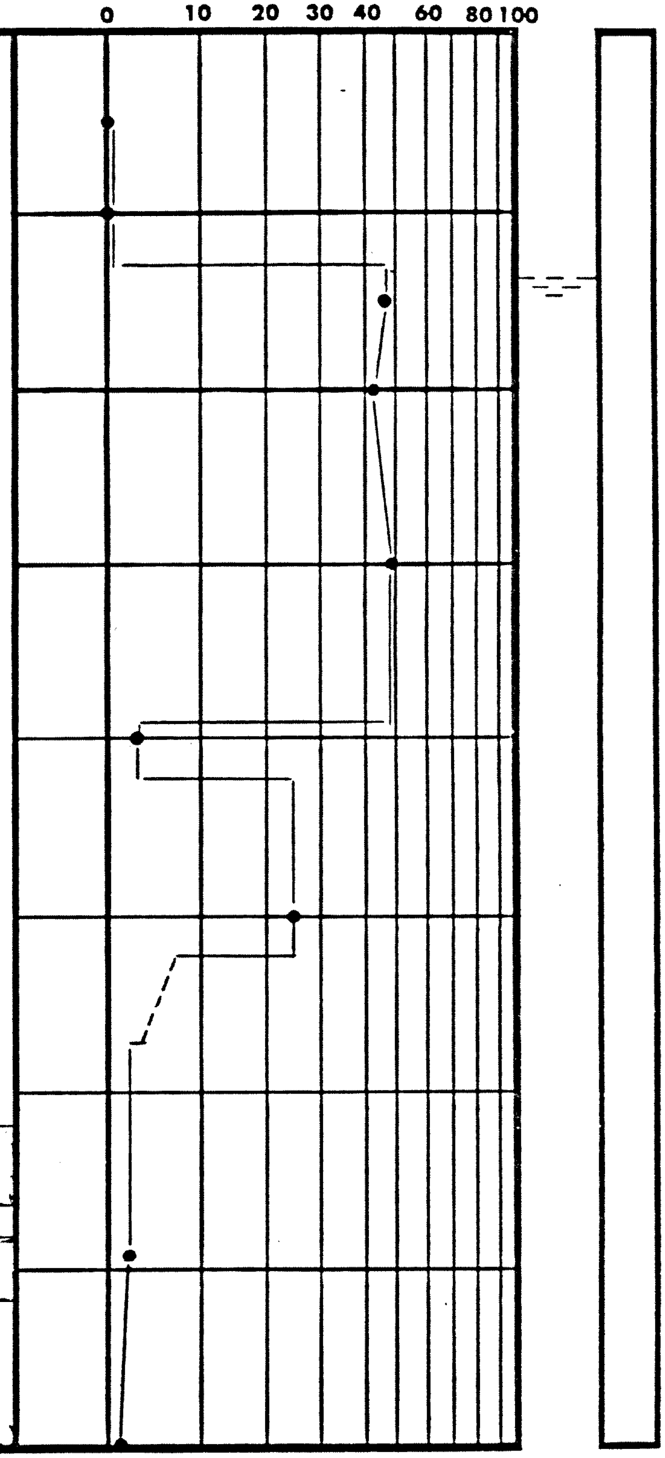
DATE DRILLED 1/18/72

JOB NO. J-1540

2A-191

DEPTH FT.	DESCRIPTION
0	LOOSE tan fine SAND with shell fragments (Fill)
1.5	VERY SOFT dark brown highly organic SILT(Peat)
4.0	VERY LOOSE dark brown silty clayey fine SAND with decayed roots
6.5	DENSE blue-grey fine SAND
19.5	SOFT blue-grey clayey fine slightly sandy SILT
21.0	VERY FIRM blue-grey slightly silty fine SAND
26.0	LOOSE to VERY LOOSE blue-grey silty clayey fine SAND
28.5	VERY SOFT blue-grey clayey SILT
33.5	VERY SOFT blue-grey slightly organic clayey fine sandy SILT with some shell fragments
37.0	SOFT to VERY SOFT dark grey slightly organic clayey SILT
39.5	VERY LOOSE dark grey slightly (1)
40.0	

ELEV • PENETRATION-BLOWS PER FT.



BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
50 % ROCK CORE RECOVERY
LOSS OF DRILLING WATER

2A-192

TEST BORING RECORD
(Page 1 of 3 Pages)

BORING NO. B-173 H

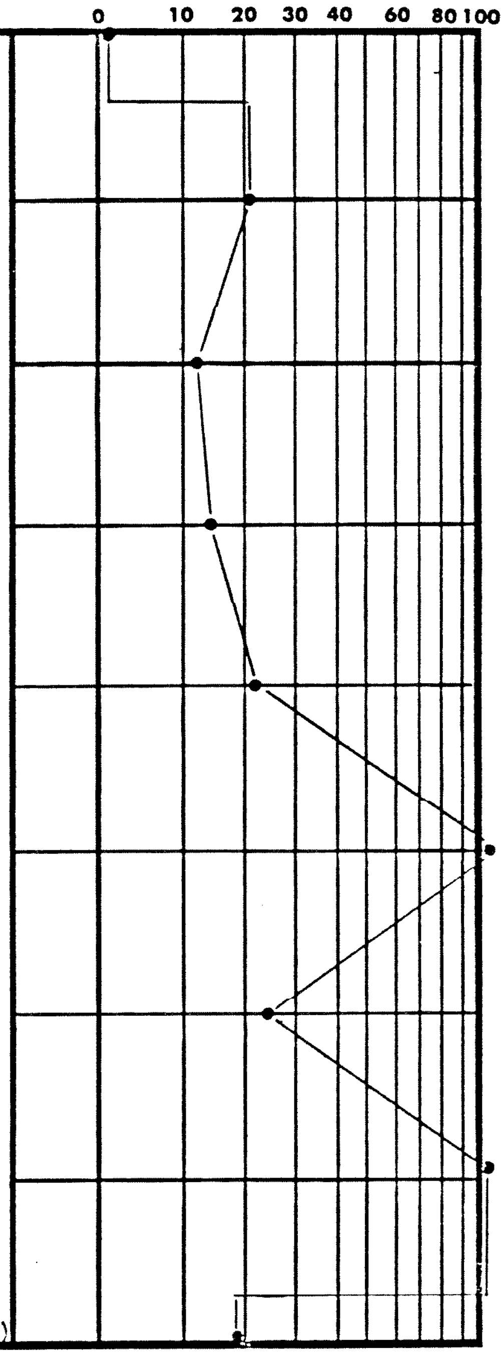
DATE DRILLED 1/22,24/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION	ELEV
40.0	VERY LOOSE dark grey slightly organic silty clayey fine SAND	
42.0	VERY FIRM grey slightly silty fine SAND with small cemented fine sand fragments	
48.5	FIRM blue-grey silty fine SAND & cemented fine sand SEAMS	
58.5	FIRM to VERY DENSE blue-grey slightly silty and very slightly silty fine SAND with shell fragments	
71.0	DENSE to VERY DENSE blue-grey fine SAND	
78.5	FIRM blue-grey slightly silty fine SAND with shell	(2)
80.0		

• PENETRATION-BLOWS PER FT.



TEST BORING RECORD

(Page 2 of 3 Pages)

BORING NO. B-173 H

DATE DRILLED 1/22,24/72

JOB NO. J-1540

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

|50| % ROCK CORE RECOVERY

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

LOSS OF DRILLING WATER

2A-193

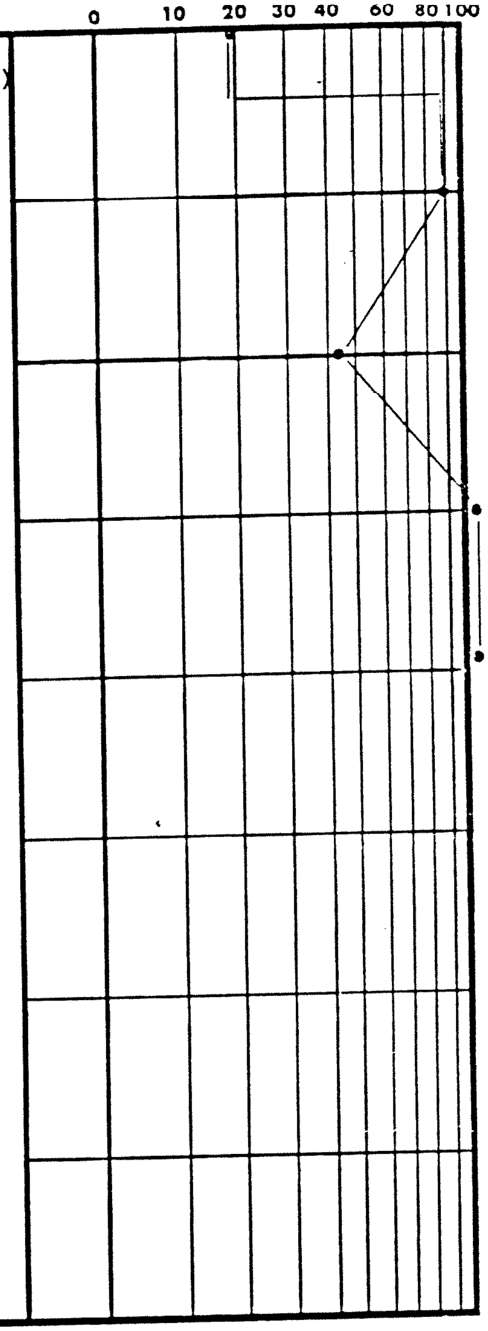
LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION	
80.0	FIRM blue-grey slightly silty fine SAND with shell	(3)
82.0	VERY DENSE blue-grey slightly silty fine SAND	
88.0	DENSE blue-grey slightly silty fine SAND & SHELL FRAGMENTS	
93.0	VERY DENSE grey slightly silty fine SAND	
100.0	BORING TERMINATED	
	(1) organic silty clayey fine SAND (2) fragments & cemented fine sand fragments (3) fragments & cemented fine sand fragments	

ELEV

• PENETRATION-BLOWS PER FT.

0 10 20 30 40 60 80 100



TEST BORING RECORD

(Page 3 of 3 Pages)

BORING NO. B-173 H

DATE DRILLED 1/22,24/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113

PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

WATER TABLE, 24 HR.

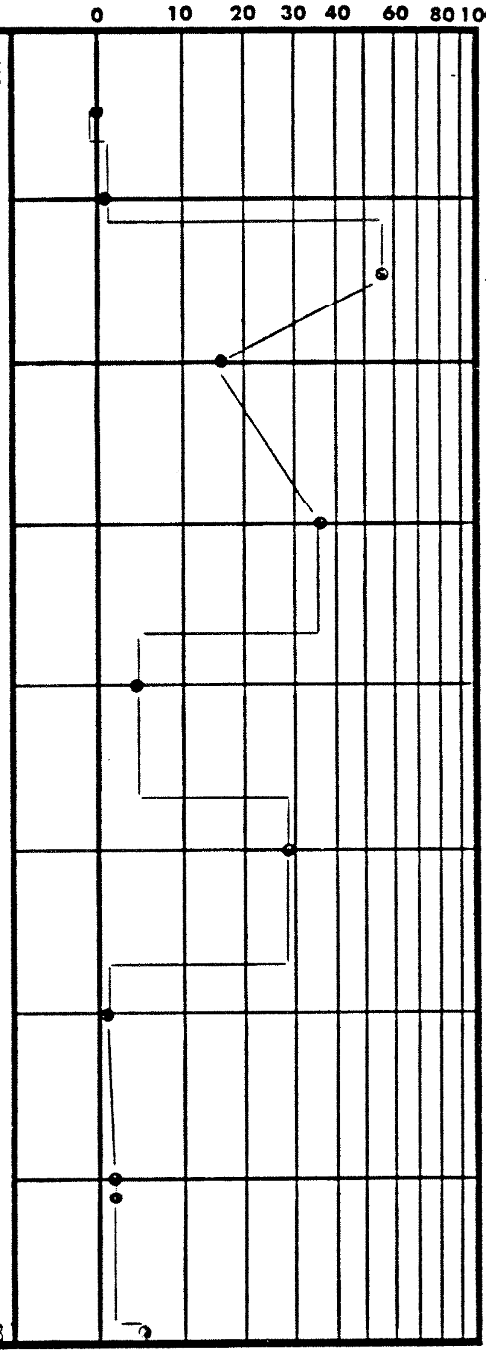
UNDISTURBED SAMPLE — WATER TABLE, 1 HR.

|50| % ROCK CORE RECOVERY — LOSS OF DRILLING WATER

2A-194

DEPTH FT.	DESCRIPTION		ELEV
0			
1.0	VERY LOOSE tan fine SAND with	(1	
1.5	VERY LOOSE blue-grey silty	(2	
	VERY SOFT dark brown highly organic SILT (Peat)		
3.5	VERY LOOSE grey slightly organic silty fine SAND with clayey silt seams		
6.0	FIRM to VERY DENSE blue-grey fine SAND with shell fragments		
18.5	SOFT blue-grey clayey SILT		
23.5	VERY FIRM blue-grey slightly silty fine SAND		
28.5	VERY SOFT blue-grey clayey fine slightly sandy SILT with some shell fragments		
32.0	VERY SOFT blue-grey clayey SILT		
35.0	VERY SOFT blue-grey clayey fine sandy SILT		
39.5	LOOSE blue-grey silty	(3	
40.0			

• PENETRATION-BLOWS PER FT.



TEST BORING RECORD

(Page 1 of 3 Pages)

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

% ROCK CORE RECOVERY

LOSS OF DRILLING WATER

2A-195

BORING NO. B-174 H

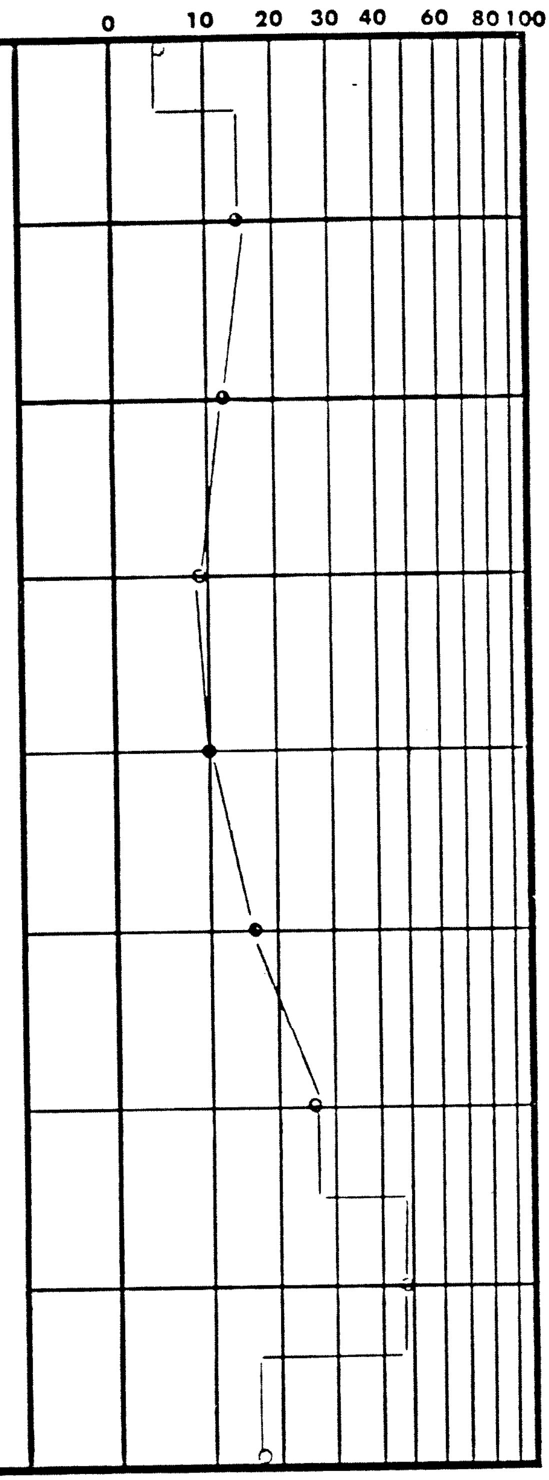
DATE DRILLED 1/18,19/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

DEPTH FT.	DESCRIPTION	ELEV
40.0	LOOSE blue-grey silty slightly clayey fine SAND	
42.0	FIRM grey slightly silty fine SAND with some cemented fine sand fragments	
48.0	FIRM grey cemented fine SAND, SHELL & QUARTZ with silty fine sand lenses	
51.0	LOOSE tan cemented fine SAND with fine sandy silt seams	
63.5	FIRM to VERY FIRM tan partially cemented silty fine SAND	
72.5	DENSE grey slightly silty fine SAND	
77.0	FIRM blue-grey slightly silty fine SAND & SHELL FRAGMENTS	
80.0		

• PENETRATION-BLOWS PER FT.



TEST BORING RECORD
(Page 2 of 3 Pages)

BORING NO. B-174

DATE DRILLED 1/18,1972

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE

WATER TABLE, 24 HR.

WATER TABLE, 1 HR.

50 % ROCK CORE RECOVERY

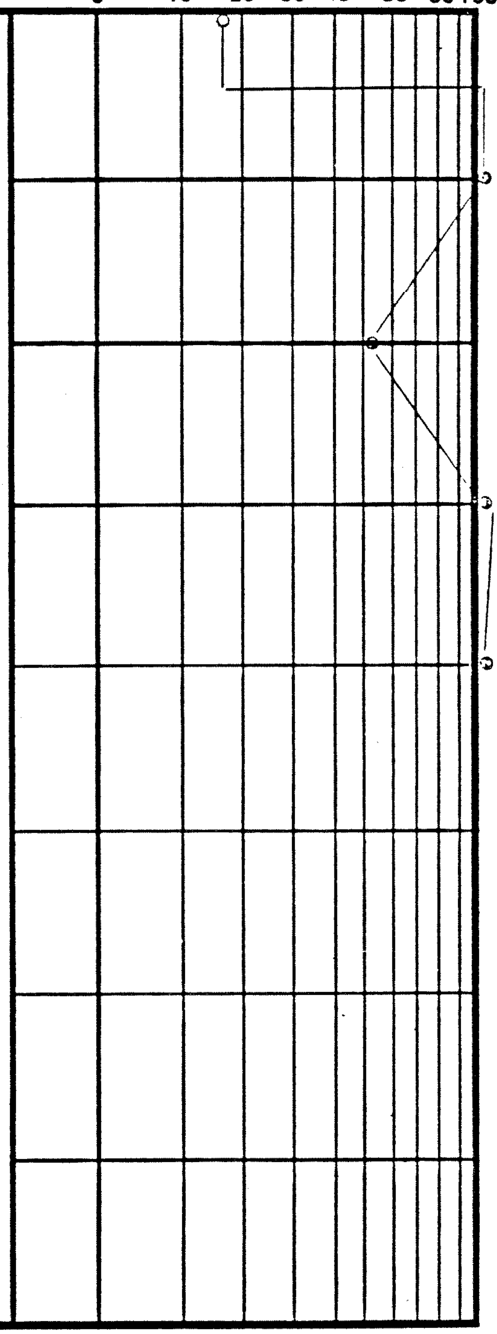
LOSS OF DRILLING WATER

2A-196

DEPTH FT.	DESCRIPTION
80.0	FIRM blue-grey slightly silty fine SAND & SHELL FRAGMENTS
82.5	VERY DENSE blue-grey slightly silty fine SAND
87.0	VERY DENSE blue-grey slightly silty fine SAND with shell fragments
91.5	VERY DENSE grey slightly silty fine SAND
100.0	BORING TERMINATED

ELEV

• PENETRATION-BLOWS PER FT.



(1) shell fragments (Fill)
(2) clayey fine SAND
(3) slightly clayey fine SAND

BORING AND SAMPLING MEETS ASTM D-1586
CORE DRILLING MEETS ASTM D-2113
PENETRATION IS THE NUMBER OF BLOWS OF 140 LB. HAMMER FALLING 30 IN. REQUIRED TO DRIVE 1.4 IN. I.D. SAMPLER 1 FT.

UNDISTURBED SAMPLE
WATER TABLE, 24 HR.
WATER TABLE, 1 HR.
% ROCK CORE RECOVERY
LOSS OF DRILLING WATER

2A-197

TEST BORING RECORD

BORING NO. B-174 H

DATE DRILLED 1/18,19/72

JOB NO. J-1540

LAW ENGINEERING TESTING CO.

APPENDIX 2B

EBASCO SPECIFICATION

FOUNDATION EXCAVATION AND BACKFILL

APPENDIX 2B

Purchaser's Identification:

EBASCO SPECIFICATION
FOUNDATION EXCAVATION AND BACKFILL

No. FLO-8770-471
Issue Date: November 15, 1968
Rev. 1 April 22, 1969

GENERAL

1. All excavation and backfill shall be made in the order of progress required by the Contractor's construction program and to the lines and grades defined on Drawings. All ditches, sumps and drainage necessary to achieve the required excavation and during the backfilling operation to aid in control of surface and rain water shall be established and maintained by the Subcontractor. Removal of surface and rain water will be by others. R1

.1 All trees, brush, roots, other vegetation and muck located within the excavation or stockpile areas shall be removed and disposed of as directed by the Contractor. Disposition of usable material as well as spoil shall be made in a manner which will avoid, insofar as possible, rehandling or other interference with progress in general. Spoil, or earth from excavations which is unsuitable for backfill shall be disposed of in spoil disposal areas. All trees and combustible materials shall be disposed of by burning. All spoil disposal areas will be within the bounds of the property and will be designated by the Contractor. Materials suitable for backfill shall be selected as the excavation progresses and placed in stockpile areas. R1

.2 Finish excavations for the entire foundation area shall be completed to the approximate elevations shown on the Drawings. These finish elevations are to be into dense sand formations at approximate elevation minus 60 ft. below mean low water as determined by the Contractor. R1

.3 This Specification includes references to or requirements for meeting or adhering to certain "Standard Specifications" or "Tentative Specifications" of the American Society for Testing and Materials. In those specifications, the letters "ASTM" or "ASTM Standards" shall mean the latest revision of those Standard Specifications or Tentative Specifications of the American Society for Testing and Materials.

.4 Except as otherwise called for in this Specification, the requirements for and the methods of taking samples and the testing of all constituents shall conform to the pertinent ASTM Standard.

STOCKPILE AREAS

2. Stockpile areas as necessary will be located by Contractor during the dredging of the Indian River and Big Mud Creek. The stockpile areas shall be cleared of all undesirable material such as trees, roots, other vegetation, muck and silt. All rubbish and debris removed from the stockpile areas shall be burned or otherwise disposed of as directed by the Contractor. R1

3. Material to be excavated from the plant area excavation and the stockpile areas that is to be used as compacted backfill shall be a selected sand. It shall be free of muddy material, organic matter, rubbish, debris, or other unsuitable materials. It shall have no more than 12 percent silt content (finer than #200 sieve). The moisture content of the sand shall be within the limits required to obtain 95 percent Modified AASHO Compaction. Dredged borrow material shall be stockpiled so as to facilitate drainage. Any decanting necessary for backfill stockpiles shall be controlled such that the discharge into Big Mud Creek or the Indian River shall not increase the turbidity of the receiving waters by more than 50 Jackson units above the existing turbidity level. No limerock or fragments larger than 6 in. shall be used for the fill. BACKFILL MATERIAL R1

.1 A select sand material, with 3 in. maximum size shall be used in areas where hand compaction is required. R1

.2 Material failing to meet the gradation specification (no more than 12 percent silt) shall be either wasted or washed to remove enough of the silt so that the gradation specification is met. All limerock or material larger than 6 in. or 3 in. depending upon use of material, shall be removed. R1

.3 Material with a higher moisture content than required for the specified compaction shall at the Contractor's option, be either wasted or spread on a dry area and raked and harrowed to reduce the moisture content by evaporation. Material with a lower moisture content than that required for the specified compaction shall be spread and sprinkled with water, then raked and harrowed until the required moisture content is attained. R1

4. The Subcontractor shall prepare the area to be backfilled by compacting the material in place at the bottom of the excavation to a minimum density of 95 percent of the maximum obtained in the Modified AASHO Compaction (ASTM D 1557 - Method D). Compaction shall be by means of the same equipment used for the final compacted fill. BACKFILL R1

.1 The sand material to be used as fill shall be spread and leveled in layers not to exceed 15 in. thick before compaction. Compaction shall be effected by means of a towed vibratory drum roller imparting a minimum dynamic force of 40,000 lbs., or by other means suitable to obtain required results. The speed of the rolling equipment shall not exceed 1.5 miles per hour. The first few lifts of the filling operation shall be carried out as a test section to determine the best possible combination of equipment and compaction procedure therewith to attain required results of uniformly compacted fill of specified density. R1

BACKFILL (Cont'd)

4.2 Any material which fails to meet the specified minimum density shall be recompacted. If the material is too dry to be compacted to minimum required density, it shall be sprinkled with water before recompacting. Material too wet to be compacted to minimum required density shall be removed and replaced with new fill.

.3 Any previously compacted material which has become too wet or in any other way has become unsuitable, as determined by Contractor's tests, shall be removed and replaced with new fill. Any area from which compacted fill has been stripped shall be recompacted before new fill is placed. All layers within the width selected for placing shall be compacted to their full width. No fill shall be placed within 20 ft. of the boundary between material being compacted and the uncompacted material being placed. R1

.4 Compaction shall be uniform within any one layer over the entire area. The surface of each lift shall be kept reasonably smooth and free of ridges or ruts, which might affect the compaction of later lifts. Equipment used for hauling shall follow paths different from each other, to aid compaction, over the entire area. Placement of new fill on fill compacted more than 48 hours previously will be treated as placing fill on the original excavated area. The surface of the area shall be first cleaned of all loose debris. R1 R1

.5 In small areas where it is not possible to reach with large scale mobile compaction equipment, the fill shall not contain material larger than 3 in. in size. It shall be compacted with a mechanical tamper, small vibratory roller, vibratory plate, or other suitable means to attain required compaction. These areas shall be compacted to the same minimum compaction as the rest of the fill and shall be brought up in not greater than 6 in. lifts. Care shall be taken to insure that the fill in these areas is integral with the rest of the fill. R1

.6 Slopes of the sections of fill shall be one vertical to two and one-half horizontal in the excavation proper only. All permanent exposed slopes shall be one vertical to three horizontal. Where two sections of fill join, that fill placed earlier shall be cut back a minimum of three (3) feet to a minimum slope of one vertical to two and one-half horizontal (1:2.5) to expose undisturbed compacted material. R1

IN PLACE DENSITY AND TESTING R1

5. The in-place backfill shall have a relative density of 85 percent. The variation from this degree of compaction shall be a maximum of one standard deviation less than 85 percent relative density. The numerical value of the standard deviation will be established by a series of field tests conducted during the initial compaction operations and will be reported in terms of minimum allowable density required. This minimum considered for the basis of field control at the present time, shall be 95 percent of Modified AASHO.

IN PLACE DENSITY AND TESTING(Cont'd)

5.1 All backfill shall be compacted to a minimum of 95 percent of the maximum density obtained in the Modified AASHO Compaction Test (ASTM D 1557 - Method D). Optimum conditions for moisture and density shall be determined by the Subcontractor for the various sands excavated. Results of tests made on samples are included in the "Report of Sampling and Testing of Proposed Granular Fill Material" which is included in and made a part of these Specifications, and the results of subsequent tests made during construction will be made available to the Subcontractor. R1

.2 Control tests of densities and moisture contents will be made by the Contractor as the work progresses, to assure that required densities and moisture contents are being achieved. R1

.3 The in-place density shall be tested in accordance with ASTM D 1556, ASTM D 2167, ASTM D 2216, and any other method suitable to insure that the backfill has been properly compacted. A test shall be made in each layer for every 10,000 sq. ft. of compacted fill area and one test for every area of less than 10,000 sq. ft. placed in one day. More tests may be run at the discretion of the Contractor. R1

.4 When the weather is of such a nature as to endanger the quality of the fill material being placed, whether this be due to rain, or any other element of the weather, the placement of fill shall be halted until the weather conditions are satisfactory. Under no conditions shall fill be placed during heavy rains.

.5 Any and all questions regarding the borrowing, preparation placement and protection of the compacted fill shall be referred to the Contractor. All decisions by the Contractor regarding the approval of compacted fill, in any aspect, shall be final. R1

.6 During the placing and compacting phases of this Contract the Contractor will have a Soils Engineer on the site to supervise testing of the compacted backfill and based on such tests will have the responsibility of accepting or rejecting the work performed by the Subcontractor. This Soils Engineer will be available to the Subcontractor for such consultation advice as is necessary. R1

APPENDIX 2C

A COMPARATIVE STUDY

OF FLORIDA'S MOST SEVERE TORNADOES

WITH THOSE

IN OTHER PARTS OF THE CONTINENTAL U. S.

Dr. Edward M. Brooks, Professor,
Department of Geology and Geophysics
Boston College, Chestnut Hill, Massachusetts

Harold P. Gerrish, Assistant Professor,

Homer W. Hiser, Professor, and

Harry V. Senn, Associate Professor,

Institute of Marine and Atmospheric Sciences,
University of Miami, Coral Gables, Florida

DOCKET NO. 50-335

FLORIDA POWER & LIGHT COMPANY
MIAMI, FLORIDA

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A COMPARATIVE STUDY OF FLORIDA'S MOST SEVERE TORNADOES WITH THOSE IN OTHER PARTS OF THE CONTINENTAL U.S.

1.0 INTRODUCTION

It has been evident for some time that Florida tornadoes are considerably less intense than the AEC model which was obviously derived from those which have occurred in other parts of the continental U.S. The purpose of this report is to document Florida's most severe tornadoes and to compare these as quantitatively as possible with major tornadoes in other parts of the continental U.S. in order to show that the current AEC model is excessive for Florida.

While thousands of continental tornadoes have been observed in the past, quantitative data on maximum wind speeds and central pressures within the central vortex are practically non-existant. Doppler radar can provide direct measurements of wind speeds along a radial from the radar location; but only a few measurements have been made during the past 10 years due to the lack of an organized doppler network in the tornado areas.

For a few of the intense continental tornadoes, it is possible to derive windspeeds indirectly from their dynamic pressures on structures that failed and/or from static barometric pressure observations inside or near the funnel, as has been suggested by Brooks(1) in a comprehensive survey article. Because of the uncertainties involved in these derivations, the numerical results cannot be taken too literally. Unfortunately, static pressure data do not exist for Florida tornadoes and limited data are available for wind computations based upon dynamic pressure.

The relative severity of tornadoes may also be determined by use of other methods. One such method is the analysis of the micro-and mesoscale synoptic conditions known to be necessary for severe tornadoes. Another is the use of statistics on deaths, damage produced, etc.

The procedure will be to develop the severity of continental U.S. and Florida storms and to compare the maximum wind speeds which might be expected in them.

2.0 DETERMINATION OF TORNADO WINDSPEEDS

2.1 Direct Measurements of Wind Speeds

Observations by wind instruments furnish the most accurate measurements; but unfortunately are limited to minor tornadoes or to the outer portions of major tornadoes. Inside severe tornadoes, the equipment is destroyed before the maximum wind at that point is reached. Remote sensing by the use of Doppler radar has probably provided the only reliable measurements to date of the particle speeds in the core boundary regions.[2]

2.2 Indirect Measurements of Wind Speeds

2.2.1 Determining the speed of the wind from its dynamic pressure

For winds strong enough to produce damage, a minimum dynamic pressure of the wind can be found. It is equal to the computed pressure necessary to produce some particular damage. Engineering estimates can be made of the force required to produce the observed displacement or deformation of selected objects of known mass or internal strength.

Let the kinetic energy of the wind ($1/2\ mv^2$) be used to do the work (Fd) of moving an object a distance of d:

$$1/2\ mv^2 = Fd \qquad (1)$$

m = mass of air
v = speed of wind

Let the object of cross sectional area A sweep out a volume V (equal to Ad) as it is replaced by the same air volume V (equal to $\frac{m}{\rho}$) (where ρ = air density): Divide (1) by the equivalents of V:

$$1/2\ \rho v^2 = \frac{F}{A} \qquad (2)$$

and

$$v = \left(\frac{2F}{\rho A}\right)^{1/2} \qquad (3)$$

This windspeed value applies only to the place and time of the damage and may fall short of the maximum windspeed of the tornado.

2.2.2 Determining the windspeed from its static pressure

From the lowest reading of a barometer, the pressure drop below the ambient pressure outside the tornado is computed. It is assumed that the kinetic energy of the wind is derived from the work done on the air as it moves from the ambient pressure to the minimum pressure in accordance with a simplified Bernoulli equation. However, it is assumed that half of this energy will be lost due to the friction between the accelerating air and its more stagnant environment.

The work of the pressure gradient force (F) is:

$$Fd = (A\ \Delta p)d = (Ad)\ \Delta p = V\Delta p$$

d = distance over which the pressure drop Δp is measured

A = cross-sectional area on which the force is acting

V = volume of air moved the distance d

Equating the kinetic energy per unit volume (left side of eq. (2)) to one half the work per unit volume gives:

$$1/2\ \rho v^2 = 1/2\ (\frac{V\Delta p}{V}).$$

and

$$v = (\frac{\Delta p}{\rho})^{1/2} \qquad (4)$$

In eq. (4), Δp is the pressure drop, measured from an assumed ambient pressure of 30 inches of mercury, at which v is chosen as zero, because the ambient winds are negligible compared to the winds within a tornado. Since the ambient temperature is about 25°C (77°F), the corresponding ambient air density is equal to 1.19×10^{-3} gm cm^{-3}.

For the assumption of incompressible air, a constant ambient density of 1.19×10^{-3} gm cm^{-3} is substituted for ρ in eq. (4). For the assumption of compressible air, ρ is assumed to decrease dry adiabatically as the pressure decreases. The assumption of incompressibility is sufficiently accurate for comparisons of wind pressure for winds of less than 120 mph. At higher speeds the larger values of windspeeds for the compressible cases (with lower densities), may be closer to the true windspeeds than the values for the incompressible cases.

In the absence of a barometer reading, the static pressure drop can be equated to the vertical pressure drop outside the tornado from the height of the base of the funnel cloud to the base of the low clouds.(3) It is assumed that the mixing ratio (or specific humidity) is spatially invariant, such that the lower edge of the funnel and the low cloud base constitute an isobaric surface with a pressure equal to the condensation pressure.

The method of wind determinations from the minimum static pressure is the least reliable of the three methods, because of uncertainties in the applicability of the simplified Bernoulli equation and in the allowance for loss of kinetic energy.

2.2.3 Synoptic Factors Affecting Storm Intensity and Windspeed

Over the years certain combinations of synoptic parameters have been found to be associated with the most vigorous thunderstorms and tornadoes. Generally a subsidence type inversion is involved separating a low-level moisture tongue from dryer air above.

A narrow band of relatively strong wind flow (jet) is required at all levels and it is extremely desirable for the middle-level and low-level jets to intersect. The optimum height of the wet-bulb temperature of zero degrees is about 8,000 ft. The storms do not develop spontaneously but need some sort of lifting mechanism. A detailed survey by Miller (4) of the specific parameters that play a major role in the production of severe thunderstorms and tornadoes is reproduced in Table 1. This table was compiled from a computer study of 328 tornado cases. Here an attempt was made to define limits on each parameter as required to produce storms of various intensities, and to order the parameters according to importance.

Aside from the above comments, there are several other parameters that must be considered in order to complete the picture. Rates of change of surface temperature, pressure and dew point provide information on regions of decreasing stability and areas where low-level convergence, vertical acceleration and divergence aloft are occurring most rapidly. Difluence at 500 mb and 200 mb not only provides a mass evacuation mechanism aloft but signifies the presence of an approaching positive-vorticity center. Experience has shown that the "level of free convection" should occur at a higher pressure than 600 mb.

Of the parameters discussed above, the following are considered to be the most vital for the development of <u>tornadic</u> storms:

a) Middle and upper level jets with shear zones

b) Low level jet

c) 850 mb maximum temperature field

d) 700 mb dry intrusion

e) Low Sfc pressure

f) High Sfc dew points

The ultimate intensity, therefore is related to the degree to which each of these parameters approaches the critical limits shown in Table 1. If all exceed the specified limits, then one would expect a more severe tornado than if only one-half or more of them did. Tornadoes can and do form in the absence of a jet stream, for instance. However, these are not as severe as those that form in conjunction with a jet stream. In South and Central Florida, the limits required for the jets and the dry-air intrusion at the same time are not reached because of the moderating influences of the water on all three sides of the peninsula.

By definition, the most intense tornado produces the highest vortex windspeeds.

2.2.4 Devastation Statistics and Their Relationship to Windspeeds

Quite obviously the number of deaths and amounts of damage produced by various storms can provide an indirect measure of the severity of the storms in terms of windspeed <u>if</u> information on the population

densities and construction in the paths are available. Then one can compare those statistics with the rare windspeed observations of a direct nature, or calculated values using indirect methods, and thereby establish a basis for estimating winds of other storms which occurred during a given historical period.

However, it is extremely difficult to compare the number of deaths or amount of damage caused by a single storm from an earlier period to a recent one because of the problems encountered in normalizing the statistics. It would not be too difficult to normalize population statistics for various years in a given city or state; but the death statistics would still not be valid because of other complicating factors which are far more difficult to assess. For instance, even in 1896, St. Louis was well populated so that if a tornado hit the same sections today, one might expect only a few more deaths from population increases since then; but one would expect fewer deaths after considering the factors of (probably) better warning services and stronger shelter which is not as easily damaged today. The sum total might be fewer deaths from an equal storm today. Despite better building practices, total damages to property would almost certainly be much higher due to the increased value of construction in terms of sophistication, the much greater value due to the shrunken dollar, and the greater number of buildings which might fall in the same path.

It is clear that we should expect roughly the same number of deaths but far greater dollar damage from storms in later years which are actually equal in intensity to their forerunners. The many complicating factors in comparing the statistics of various storms indicate that one must not assign too much weight to small differences in the number of deaths or dollars of damages in assessing the storms' intensities or windspeeds. If, on the other hand, the differences in deaths or damages are an order of magnitude or more, one can be reasonably certain the storms are of significantly different intensities and therefore windspeeds.

TABLE 1

KEY PARAMETERS IN THE PRODUCTION OF SEVERE THUNDERSTORMS AND TORNADOES

(after Miller[4])

RANK	PARAMETER		WEAK	MODERATE	STRONG
1	500 mb Vorticity		Neutral or Negative Vort Advection	Contours Cross Vort Pattern <30°	Contours Cross at more than 30°
2	Stability	Lifted Index	-2	-3 to -5	-6
3	Middle Level Jet	Speed Shear	35 knots 15/90 nm	35-50 knots 15-30/90 nm	50 knots 30/90 nm
4	Upper Level Jet	Speed Shear	55 knots 15/90 nm	55 to 85 knots 15-30/90 nm	85 knots 30/90 nm
5	Low-Level Jet Speed		20 knots	25-34 knots	35 knots
6	Low-Level Moisture Mixing Ratio		8 gm H_2O/kg dry air	8 to 12 gm H_2O/kg dry air	12 gm H_2O/kg dry air
7	850-mb Max-Temp Field		E of Moist Ridge	Over Moist Ridge	W of Moist Ridge
8	Winds Cross 700-mb No-Change Line of Advective Temp.		20°	20° to 40°	40°
9	700-mb Dry-Air Intrusion		Not Available - or Available but weak Wind Field	Winds from Dry to Moist Intrude at an Angle of 10 to 40° are at least 15 knots	Winds Intrude at an Angle of 40° and are at least 25 kn
10	12-hr Sfc Pressure Falls		Zero mb	1 to 5 mb	5 mb
11	500-mb Height Change		30 m	30 to 60 m	60 m
12	Height of Wet-Bulb-Zero above Sfc		Above 11000 ft. Below 5000 ft.	9000 to 11000 ft. 5000 to 7000 ft.	7000 to 9000 ft.
13	Sfc Pressure over Threat Area		1010 mb	1010 to 1005 mb	1005 mb
14	Sfc Dew Point		55°F	55° to 64°F	65°F

nm: nautical miles m: meters
Sfc: surface mb: millibars

3.0 CONTINENTAL U.S. TORNADOES

Table 2 lists nine tornadoes, of which four are tornadoes of maximum intensity (those with speeds above 321 mph). They are arranged in order of their approximate windspeeds, but since these values are crude, they are grouped according to their central pressure drop in the nearest number of inches of mercury. Within each group, the differences between the listed windspeeds of two or more tornadoes are of no significance. Note that "maximum intensity" refers to the rank according to computed windspeeds. Note also, that the nine storms listed were chosen simply on the basis that certain data were available for them; not because they were all among the nine most intense of the past 115 years. Other storms such as the Palm Sunday tornadoes in 1965; the Waco, Texas tornado in 1953; the Dallas storm in 1957; the Tupelo, Miss. storm in 1936 and other tornadoes could have been cited as examples. However, the last two storms in Table 2 stand out with regard to the number of deaths and the amount of damage produced. The last, the Tri-state tornado of 1925 was quite obviously the most intense from all standpoints.

3.1 The Minneapolis, Minnesota Tornadoes of 20 July 1951 and 20 August 1904

In the outer portion of the first tornado (Minneapolis Airport, July 20, 1951) a minimum sea level pressure of 29.15" Hg. was recorded. It is cited as a verification of the third method. Even with half the kinetic energy dropped, the theoretical windspeed for the compressible case (111 miles/hour) still exceeds the observed fastest mile (92 miles/hour). The agreement is good enough to warrant the use of the third method to obtain approximate windspeeds. Even better agreement is found in the outer part of the other Minneapolis tornado listed (Aug. 20, 1904) (6), in which the pressure drop at the City Office of the Weather Bureau was the same. The wind reached an extreme of 110 miles/hour,(6) almost identical to the theoretical value.

3.2 The Brandon, Ohio Tornado of 20 January 1954

The 28" tornado (Brandon, Ohio, January 20, 1854)(7) shows good agreement between winds of 164 and 173 miles/hour from static and dynamic pressures respectively. The lowest static pressure was 28.21" Hg., whereas the dynamic pressure was that required to account for the breaking of an oak tree.(7)

3.3 The St. Louis Tornado of 27 May 1896 and Washington, Kansas Tornado of 4 July 1932

In the 27" group of tornadoes, the static pressure in the St. Louis tornado of May 27, 1896 was measured by an aneroid barometer in Lafayette Park.(8) When corrected to sea level, it yielded a value of 27.30" Hg. In the Washington, Kansas tornado of July 4, 1932, the dynamic pressure which bent the top of a railroad signal was found to be 110 lbs/ft^2. (The coefficient of 1.6, the same as for tall buildings, was used to calculate the windspeed.) (9)

3.4 The Wallingford, Connecticut Tornado of 9 August 1878

The Wallingford, Connecticut tornado of August 9, 1878 falls in the 26" category of tornadoes because of the high wind required to explain a 2 X 2 X 4 ft. cemetary stone blown off its foundation.(10)

3.5 The Minneapolis Tornado of 20 August 1904; Harrison, Ohio Tornado of 14 February 1854; Worcester, Massachusetts Tornado of 9 June 1953; and Tri-State Tornado of 18 March 1925

The four most severe tornadoes belong in the categories of 24" to 22" Hg. mercury. In the Minneapolis tornado of August 20, 1904, a barometer reading of 23" was obtained.(6) If this was station pressure, the tornado belongs in the 24" category (since the sea level pressure would be nearly one inch higher). If the barometer had been set for sea level pressure, then, of course, the tornado belongs in the 23" category. Since this ambiguity was not resolved, the tornado was listed with the appropriate windspeeds (compressible case) in both pressure categories.

The other two 23" tornadoes were so listed because of their wind pressures. In the Harrison, Ohio tornado of February 14, 1854, a scantling was driven 3 1/2 feet into the ground.(11) The wind in the Worcester, Mass. tornado of June 9, 1953 was obtained from the known load resistance on destroyed towers carrying high voltage lines.(12) The Worcester tornado was the strongest New England tornado on record. This tornado formed in conjunction with a pre-cold-frontal squall line that extended from southern Maine to eastern Connecticut on the afternoon of June 9, 1953. The cold front at that time was gently curving from western Maine through western Massachusetts to central Pennsylvania. Earlier it had passed through the Great Lakes area with widespread tornadic activity in advance of the front. The rhythm was such that on the 7th - 9th violent afternoon activity quickly followed relatively quiescent morning situations. The regeneration of tornadoes on the 9th in the New England area was more pronounced than earlier in the history of the system. Of the several that formed, the one in and around Worcester was the most severe. It killed 90 people and produced $52 million of damage.(14)

The synoptic environment aloft was highlighted by a jet stream at high levels and warm air advection ahead of the squall line at low levels. Slight cooling was evident at middle levels. There was positive vorticity advection aloft in association with the closed-low in southeastern Canada. Hence, most if not all of the synoptic requisites for a severe tornado were present. One can only make inferences regarding synoptic situations aloft for the other storms listed in Table 2 because routine data above the surface were not generally available during those years.

The only tornado reaching the 22" category was the Tri-state tornado, which swept through Missouri, Illinois, and Indiana on March 18, 1925.(13) The best data for determining the wind pressure are from a steel water tank and adjacent concrete chimney of Orient Mine No. 2 at West Frankfort, Illinois. The wind pressure was calculated to be 250 lbs/ft^2, or nearly 1/8 of an atmosphere. The Tri-State tornado of 1925 had a path length of 219 mi, a width of 1,000 to 2,000 yds and traversed predominantly rural areas at a speed of 57 to 68 mph. Had it hit more populous areas both the number of deaths and the damage would have soared even higher for the most intense storm observed in over a century.

TABLE 2

APPROXIMATE WINDSPEEDS OF CONTINENTAL U. S. TORNADOES

(ΔP) 30-(P) (Inches HG)	(P) Central P of Tornado (Inches HG)	Windspeed (miles/hour) Theoretical (Static) Incompressible	Compressible	Determined (Refer to Key)	Date	Place & Reference	Deaths	Damage (Megadollars)
1	29	119	121	92w, 111s (Outer Portion)	7/20/ 1951	Minneapolis, Minn. (5)	5	6
2	28	169	174	164s, 173d	1/20/ 1854	Brandon, Ohio (7)	No Data	"Heavy"
3	27	206	214	203s	5/27/ 1896	St. Louis Mo. (8)	306	12.9
				210d	7/4/ 1932	Washington, Kan. (9)	5	.5
4	26	238	251	260d	8/9/ 1878	Wallingford, Conn. (10)	30	.25
5	25	267	285	--	--	--	--	--
6	24	292	316	321s	8/20/ 1904	Minneapolis, Minn. (6)	14	1.5
7	23	316	348	340d	2/14/ 1854	Harrison, Ohio (11)	No Data	No Data
				348s	8/20/ 1904	Minn. Minn. (6)	14	1.5
				343d	6/9/ 1953	Worcester, Mass. (12)	90	52.0
8	22	337	377	363d	3/18/ 1925	Murphysboro, Ill. (Tri-State)	689	165

Key to windspeed determinations:

w means observed windspeeds.

d means calculated from dynamic pressure of wind producing structural failure

s means calculated from observed static pressure drop from ambient to tornado center

Note: All dynamic & observed windspeeds include the effect of translation; static windspeeds do not.

4.0 THE FLORIDA TORNADO

Lists of all known tornadoes in Florida east of the Appalachicola River (excluding the panhandle region) have been compiled. One list includes 114 storms from 1887 to 1949 taken mostly from Flora's <u>Tornadoes of the U.S.</u>[15], and Monthly Weather Reviews[16]. The other includes 315 storms from 1950 to 1968 inclusive, all from the "Storm Data and Unusual Weather Phenomena" from the <u>National Summary of Climatological Data</u>, Dept. of Commerce.[17]

Using all known data on each storm including deaths, injuries, damage produced, path length and width, speed of motion, and type of area affected, each of the tornadoes was graded on an intensity scale which included 1 (minimal), 2 (moderate), and 3 (maximal) categories for Florida tornadoes. The results are shown in Table 3 below.

TABLE 3

INTENSITY RATINGS OF 429 FLORIDA TORNADOES

	1887-1949	1950-1968
Minimal	89	273
Moderate	20	36
Maximal	5	6

The two lists were kept separate because of the obvious population and reporting differences which existed during the two periods. One would expect that many minimal storms would have been unreported during the early period; whereas in the later years every waterspout, "waterspout-tornado", "whirlwind", etc., had found its way firmly into the permanent statistics.

The average tornado in Florida is of minimal intensity, barely able to unroof relatively old wooden farm buildings, packing houses and garages, and/or to defoliate, defruit or blow down trees. The "moderate" category was generally reserved for storms which "demolished" or "destroyed" at least one or two normally constructed, wood or stronger buildings, possibly caused personal injuries to a number of people and/or had significant path widths or damage estimates. The maximal category either did significantly greater damage over a larger area, or it appeared from other facts that it would have had it occurred over a suitable area. To assume the most conservative attitude, the three most intense tornadoes in Florida history (82 years) were chosen for comparison with the AEC standard tornado, as probably embodied in the most intense from Table 2, Section 3 above. The three Florida storms occurred on April 5, 1925, June 17, 1959, and April 4, 1966.

No direct measurement of windspeed has been made in a Florida tornado. Indirect calculations have not been presented herein because speeds on the order of 150 to 200 mph could have produced all the damage that has been photographed and tabulated for Florida tornadoes. Attempts are still in progress to locate evidence of speeds higher than this, however, all efforts to date have been unsuccessful. The fact that the

three most severe tornadoes in Florida occurred in or near populous areas prohibits much higher speeds or they certainly would have been documented by the damage.

4.1 The Hialeah Tornado of April 5, 1925

This storm developed early in the afternoon in advance of a cold front that was pushing down the state from a wave-cyclone centered near Jacksonville, Florida. At the time of the tornado, the front extended off the southwest coast near Ft. Myers, Florida. The tornado developed prior to 1:15 P.M. and its motion was toward the northeast at approximately 12 mph. After 20 minutes of progressive movement the tornado stopped and remained stationary for 5 minutes. During this period it rose and descended twice. It then resumed its northeastward motion causing more damage. The total damage was estimated at $.25 million and there were five deaths. Its diameter increased greatly as it passed north of Miami and became obliterated by heavy rain soon afterward. No serious damage was done after that time.

The tornado was preceded by a heavy fall of hail which was confined primarily along the path and in some areas the ground was completely covered with hailstones as large as a baseball. The path of the tornado itself was 12 miles long and slightly less than 100 yards wide.

Upper-air sounding techniques of today were not available in the twenties. As a result, the upper-air structure is not known with any degree of certainty. Since the tornado moved rather slowly, it can be inferred that the steering current was weak and that no divergence or mass evacuation mechanism such as a jet stream existed aloft.

4.2 The Miami Tornado of June 17, 1959

While all eyes were on Tropical Storm Beulah, centered 100 miles northeast of Tampico, Mexico, a tropical depression formed rather unexpectedly in the eastern Gulf near 25.5°N, 86.5°W on the afternoon of June 17, 1959. During the night it deepened and moved northeastward at 35 mph crossing the west coast of Florida just south of Tampa and exiting the east coast just north of Cape Kennedy.

The tornado occurred in Miami at 9:50 P.M. on the 17th. This position was in the right front quadrant approximately 230 n. miles from the center of the deepening depression. Hiser (18) has summarized the eye-witness accounts of the tornado as it moved from the Coconut Grove area of southern Miami skipping over populous areas near downtown Miami, thence down to the ground again in North Miami. The total damage was estimated to be $3 million and no lives were lost. The state climatologist described the storm as the most intense since the 1925 storm. Despite the improved south Florida building codes, neither the total damage nor the loss of life reached the potential that an intense midwestern storm would have produced over such a populated area.

Although there was no major jet stream over South Florida during this period, there may have been a narrow zone or finger of relatively higher wind speeds on the order of 50 knots from Miami to Grand Bahama. Miami reported a wind at 18,000 ft. of 220°/51 knots. At Grand Bahama the winds

above 40,000 ft. were 50 knots or greater. This may have provided some degree of mass evacuation aloft. The tornado moved toward the northeast at 25-28 miles per hour(18). The Lifted Index was -3.7. The wet-bulb temperature of zero degrees was at 13,000 ft. above M.S.L.

4.3 The Central Florida Tornado of April 4, 1966

A frontal system moved down the state on the 2nd of April and stagnated in South Florida on the 3rd. During the night of the 3rd it washed out and another system moved into the Southeast U. S. trailing a front along the Gulf Coast. The second frontal system moved into the Gulf on the 4th passing through central Florida that night and off the southeast coast during the afternoon of the 5th. Several stable waves formed on that front during its history.

During the morning of the 4th, a tornado formed near Clearwater, Florida and moved east-northeastward across the state to Gibsonia and thence to the Merritt Island area. Another tornado or a family of tornadoes began at Pinellas Point and passed through southern sections of Lakeland, Haines City and thence to Rockledge on a track parallel to the above. Evidence indicates that the northernmost storm maintained continuous contact with the ground from the Gulf to the Atlantic ocean, a distance of 140 miles. The southernmost storm produced an intermittent track as if one tornado lifted and lowered or possibly a family of tornadoes were involved. The northern one was the most severe and produced the most damage of the two. Eleven people were killed, 400 were injured, and property damage was estimated at $11 million. Quite obviously, this was a major storm, probably the largest of Florida record; although no photographs of the funnel(s) have been found because they may have been obscured by precipitation. Considering the nature of this storm and the populated areas traversed by it, it is relatively certain that the damage and deaths were not nearly as great as might have been produced by winds of the order of 300 mph.

This outbreak was associated with a squall line in advance of the frontal system which was located near New Orleans at that time. There was strong convergence aloft at 5,000 ft. with a high-level jet finger of 85 knots oriented WSW-ENE over Tampa. The main jet was over the southeastern U.S. A speed maximum on the order of 60 knots was evident at mid-levels down to at least 10,000 ft. This arrangement of speed maxima aloft is conducive to severe tornadoes, see Table 1. However, there was no indication of strong vorticity advection at 500mb which normally accompanies severe storms. The wet-bulb temperature of zero degrees in this storm was at 13,000 ft. (the same as the '59 tornado), and the Lifted Index on the basis of a partial sounding appeared to be positive.

Proximity soundings showed that while there was a tendency for some drying above 750mb in the 1959 storm, both it and the 1966 tornado soundings were quite moist resembling the Type II Gulf Coast soundings of Fawbush and Miller.(19) Miller(4) states that the Type I sounding is the optimum for severe tornadoes. None of the most severe tornadoes observed in peninsular Florida had Type I air-mass structures. This and certain other key ingredients such as requisite jet maxima, vorticity advection and dry air intrusion have always been missing in varying degrees from even the most severe Florida tornadoes of record.

5.0 COMPARISON OF THE THREE MAXIMAL FLORIDA TORNADOES WITH SEVERE CONTINENTAL U. S. TORNADOES

Flora[14] reports 192 tornadoes in Florida during the years 1916-1949. During this same period, Illinois experienced 190 tornadoes. However, Florida's losses amounted to 31 deaths and 2.4 million dollars property damage while the Illinois losses were 917 deaths and $53.8 million in property damage. The two states are of approximately equal area. The population density per square mile in Illinois was about five times that of Florida. But the Illinois deaths were 30 times larger and damage was more than 22 times greater than Florida's. Flora also listed the outstanding U. S. tornadoes of this period. Several Illinois tornadoes were listed but none for Florida. All of this indicates much less severe tornadoes in Florida than in the midwestern state of Illinois.

Between 1916 and 1958, the average number of people killed per tornado was about nine times higher in the continental U. S. (1.04) than it was in Florida (.12)[13]

In another important tornado publication, Wolford lists the outstanding tornadoes from 1876-1958 for the entire U. S.[13] Again, no Florida storms were included in her listing of 172. Two measures of the intensity of tornadoes used by Wolford include deaths and monetary damage values. Figure 5.1, giving the number of deaths caused by the outstanding storms in Continental U. S., shows clearly that those caused by Florida's most severe would rank her storms in the lowest category of the U. S. outstanding storms. Figure 5.2, showing the damage produced, also tends to confirm the the Florida tornado is not nearly as intense as the most severe storms found elsewhere.

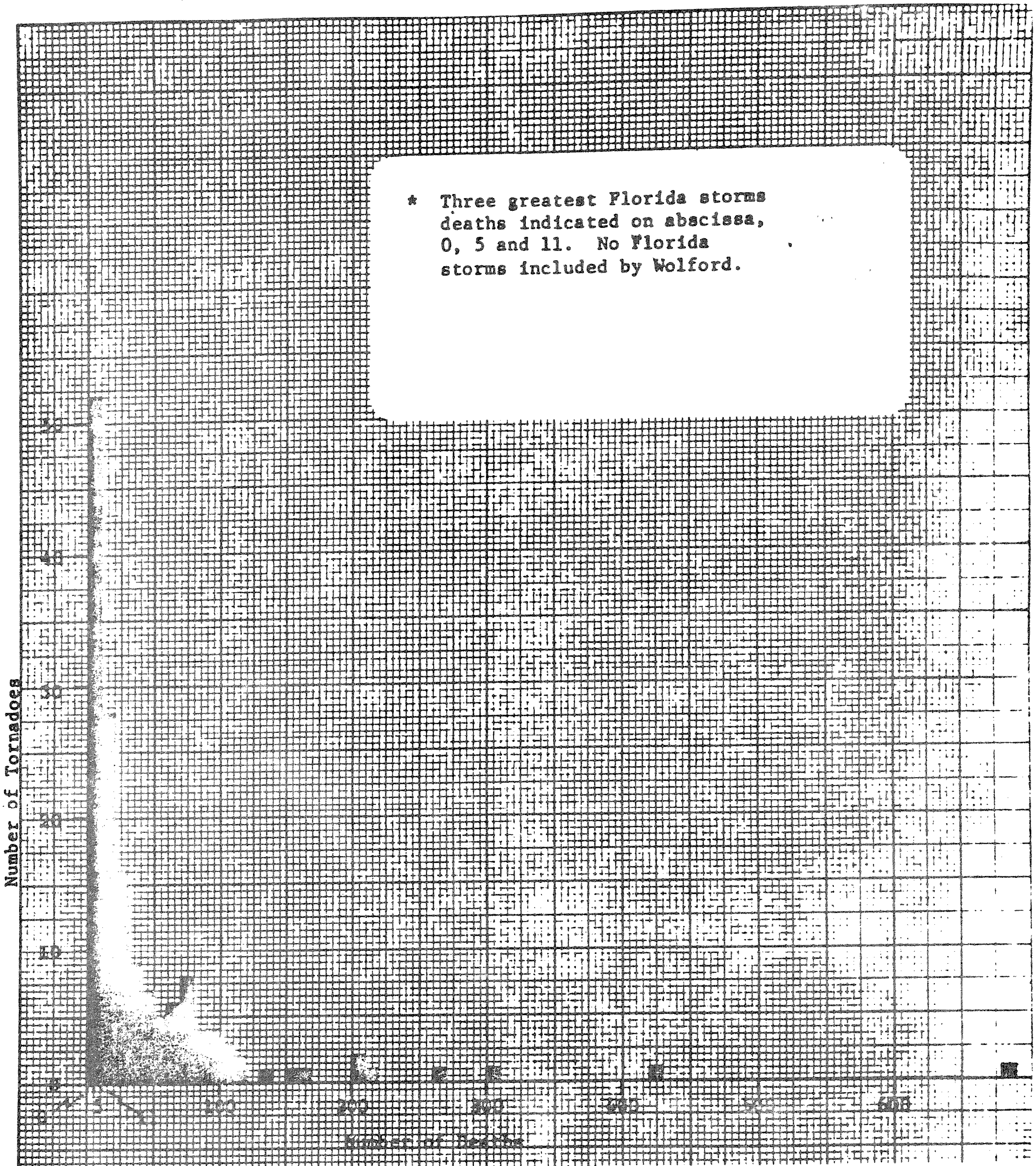
Insofar as tornado intensities and thus windspeeds can be inferred from deaths and damage statistics, it can be shown that continental U. S. storms have considerably higher winds than Florida storms. Evidence presented above shows that there is a greater than order-of-magnitude difference in the statistics for all three of Florida's most severe storms when compared to the most intense of the other continental U. S. tornadoes. Since all three of the severe Florida storms passed over populated areas, in recent years, they had ample opportunity to significantly raise the statistics. The only logical conclusion is that they were not nearly so intense as the most severe continental U. S. tornadoes.

The synoptic environment associated with the three Florida tornadoes was such that several of the key parameters which are generally accepted as being necessary for severe tornadoes, see Section 2.23, were missing. The relatively slow movement of the April 5, 1925 and June 17, 1959 storms indicate that no significant synoptic-scale jet stream existed aloft in those cases. The fast-moving April 4, 1966 tornado did have appropriate wind speeds aloft but they were assocaited with a finger or branch of the main jet which was located in the southeastern United States. Because of the moderating effect of the marine environment around and over peninsular Florida, the dry-air intrusion requirements were not met. Therefore, instability conditions associated with intense storms did not develop. In addition, positive vorticity advection was not indicated in the April 4, 1966 or June 17, 1959 tornadoes. Upper-air charts were not available for the April 5, 1925 storm.

Of the five most severe continental tornadoes for which wind speeds could be determined, Table 2, only one was recent enough to permit both upper-air and surface synoptic analysis. This Worcester, Mass., tornado of June 9, 1953 had the ingredients expected for severe storms as set forth in Table 1 after Miller.

The only reliable direct measurement of windspeed in a tornado was 206 mph in the June 10, 1958 storm at El Dorado, Kansas. This was recorded using Doppler radar. There have been no higher measurements since then. On the basis of indirect measurement, it must be concluded that the most intense continental tornadoes are capable of producing windspeeds on the order of 360 mph which includes effects of translation. The windspeeds calculated upon the basis of static pressures are not as reliable as those calculated from dynamic pressures. The resulting overestimates in the static pressure calculations approximately equal the translation speeds which are included in the dynamic computation. The static value of 377 mph in Table 2 is for a hypothetical 22 inch tornado.

From the standpoint of damage, photographs in Florida do not show buildings being swept clean to the ground and debris carried away as in the most severe continental tornadoes. In no case did the damage substantiate wind speeds exceeding 165 to 200 mph.



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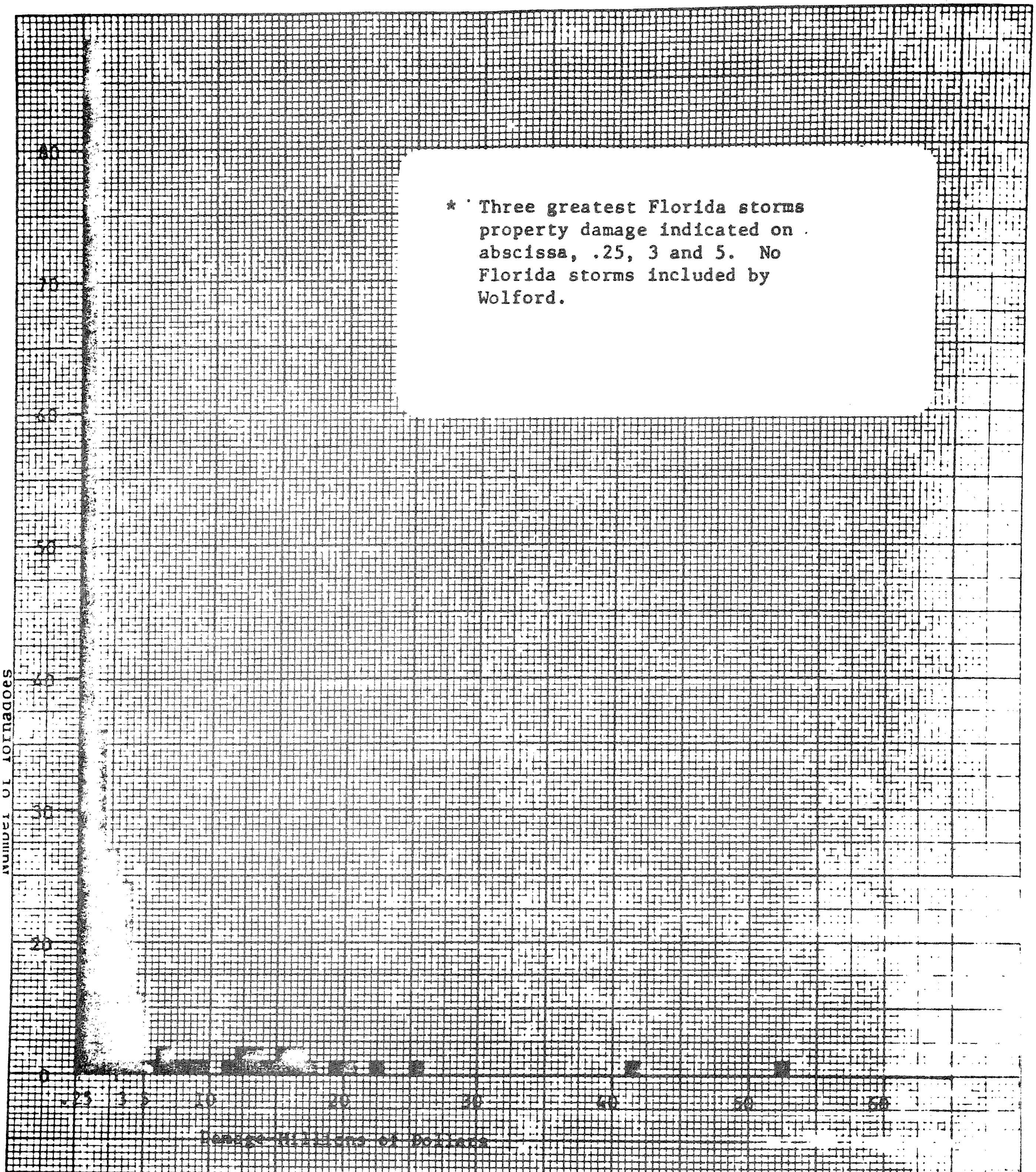
DEATHS FROM "OUTSTANDING" TORNADOES OF THE U. S., 1876-1958 (Wolford[20])
FIG. 5-1



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DAMAGE FROM "OUTSTANDING" TORNADOES OF THE U. S., 1876-1958 (Wolford[20])
FIG. 5-2

6.0 CONCLUSIONS

It has been shown by calculations from dynamic pressures that the most severe U. S. tornado in the past 115 years had a maximum windspeed of 363 mph which included a translation speed of about 60 mph. Unfortunately, limited observations or calculations were possible for Florida's most intense storms. On the basis of deaths or injuries, and property damage suffered, the most severe tornadoes in Florida history produced less severe effects by a full order of magnitude than the most severe storms which occur in other parts of the continental U. S. No photographs or records of damage have been found to substantiate speeds exceeding approximately 200 mph in Florida tornadoes. In addition, other key ingredients for severe tornadoes such as air-mass structure, jet maxima, vorticity advection, and dry air intrusion, have always been missing in varying degrees in Florida tornadoes. This is a result of Florida's southern latitude and its marine environment.

While it may be possible to produce wind speeds greater than 200 mph (including translation effects) in Florida tornadoes, it is highly unlikely that they would ever reach 300 mph. This upper limit is postulated on the basis that all key synoptic parameters will never simultaneously exist in peninsular Florida and that the order of magnitude difference in damage statistics indicates that higher wind speeds do not occur.

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APPENDIX 2D

FLORIDA EARTHQUAKE OF OCTOBER 27, 1973

FLORIDA EARTHQUAKE OF OCTOBER 27, 1973

INTRODUCTION

On October 27, 1973, at 2:21 A.M., E.S.T., about 9,000 square miles of central and eastern Florida experienced an earthquake. The epicenter of this earthquake is estimated to have been between Titusville and Geneva, Florida. Approximately 4,000 square miles was subjected to an intensity of V MM to V/VIMM. No historical earthquake epicenters have been recorded within about 75 miles of this epicenter.

LOCATION

The National Earthquake Information Center (U. S. Geological Survey) lists the earthquake epicenter at 28.7° N, 81.0° W (the edge of Lake Puzzle on the Volusia-Seminole County boundary. The location was estimated based on response to questionnaires submitted to postmasters of cities and towns within a 100 mile radius of the epicentral area. Data from seismic recording stations were insufficient to permit a more exact location of the epicenter.

The listed NEIC epicenter is located near the northern edge of the region of higher intensity. Based on our independent study of intensity responses, the earthquake epicenter could possibly be moved to the south, along the St. Johns River Valley.

Law Engineering learned of the earthquake on the morning of October 27, 1973. A portable, smoke-paper micro-earthquake recorder was installed on property in the boundary of Brevard, Orange, and Osceola Counties by 10:00 p.m. that same date. The recorder was maintained in operation for a period of one week. During this period no identifiable aftershocks were recorded.

MAGNITUDE

The National Earthquake Information Center lists the earthquake as having a magnitude of about 3. Based on preliminary analyses of records from the Atlanta Seismic Station, the earthquake magnitude was estimated as 3.9 ± 0.2.

MICROSEISMIC DATA

Intensity data was collected by Law Engineering from several sources. Most data was obtained by distribution of a modification of the standard questionaire as presented in Richter (1958). Sources of data include:

1) Responses (260) to questionnaires distributed to employees of Florida Power and Light Company and Florida Power Corporation in central and eastern Florida.

2) Responses (126) to the questionnaire published in the Cocoa Today on October 19, 1973.

3) Responses (150) to the Cocoa Today questionnaire collected as a class assignment by physics students at Melbourne High School.

4) Responses collected by Dr. G. A. Bollinger, Virginia Polytechnic Institute, from postcard questionnaires distributed to 36 postmasters in Florida and ten letters in response to an article in the Lakeland Ledger.

Attached to this appendix is a tabulation of all usable responses by location and intensity. See Table 2D-1.

Figure 1 is an Intensity map of the October 27, 1973 earthquake. All sources noted above were used in compiling this map. The boundary between the area not felt and the area felt is approximate. The earthquake occurred at a time period when most responders to the questionnaire were asleep. It is difficult to determine in some cases if the not felt responses were due to low magnitude intensity which did not awake sleepers or, if the intensity of the quake at these locations was minor and not felt by persons awake at that time. The boundary between intensity less than V MM and intensity V MM and greater is more definitive since V MM is severe enough to waken most sleepers.

Twelve persons were interviewed where their questionnaire responses indicated an intencity of V MM or greater. The interviews and observations of earthquake effects at those locations substantiated the general validity of the questionnaire responses.

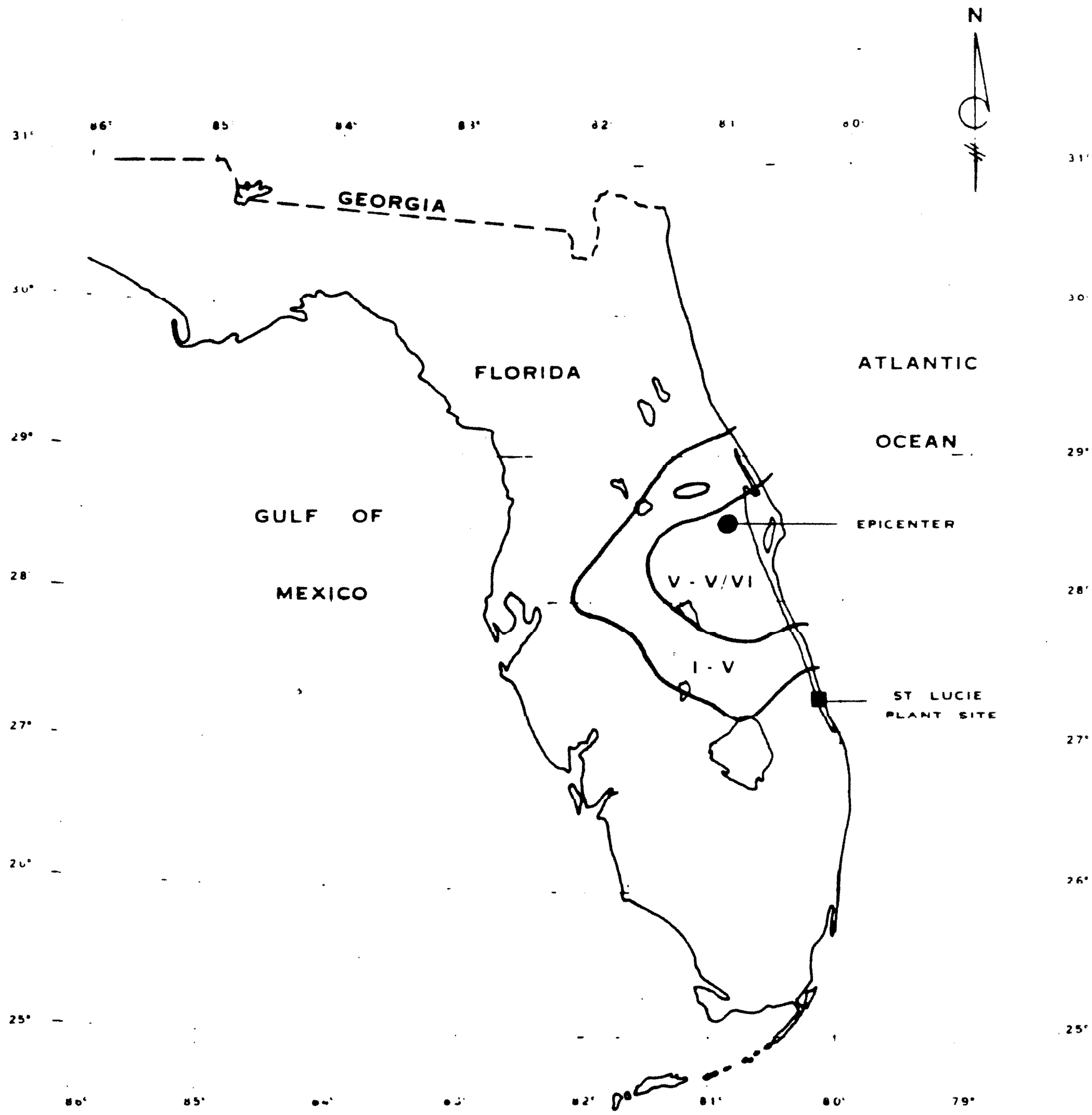
In the higher intensity region (V MM and greater) many people were awakened and a number frightened by the tremor. Small objects were displaced or upset. Hanging objects moved or in some cases fell to the floor. Some pendulum clocks stopped. There were scattered reports of cracks forming or being extended in weak masonry or glass. In inspections made by Law Engineering of several of these reports, the new cracks often occurred along construction joints or previously patched masonry cracks.

Persons responding to questionnaires and/or interviewed who had previously experienced earthquake shocks at other locations almost unanimously agreed that they "knew" immediately that they had felt an earthquake. Some reported a "rolling" sensation similar to that they had previously experienced in earlier earthquakes. Many others accustomed to the sounds and vibrations associated with rocket launches from Cape Canaveral, assumed that an unannounched launch had occurred, and perhaps the launch vehicle had exploded.

TABLE 2D-1

QUESTIONNAIRE RESPONSE

LOCATION	NUMBER OF RESPONSES	INTENSITY (MM) IV/V	<V	V	V/VI	VI
Apopka	1		1			
Christmas	1					1
Cocoa - Rockledge	54	25	18	11		
Daytona Beach - Ormond Beach	12		12			
DeLand	28		28			
DeLeon Springs	1		1			
Deltona	12		11	1		
Geneva	2		2			
Grant	1				1	
Indialantic	38	13	9	11	5	
Indian Harbor Beach	12	5	1	6		
Jacksonville	2		2			
Kissimmee	8			2	6	
Lake Helen	2	1	1			
Lake Mary	5	2	3			
Lake Wales	3		2	1		
Longwood - Altamonte Springs	5	1	4			
Maitland - Fern Park	5	2	2	1		
Melbourne	95	49	3	37	6	
Melbourne Beach	29	13	1	13	2	
Merritt Island - Cocoa Beach	25	13	1	10	1	
Miami	3		3			
Micco	2		1	1		
New Smyrna Beach	2		2			
Orange City - DeBary	34	6	28			
Orlando - Winter Park	11	2	7	2		
Paisley	3		3			
Palatka	3		3			
Palm Bay	15		5	10		
Sanford - Lake Monroe	42		38	4		
Satellite Beach	8	3	1	2	1	1
Sebastian	1	1				
Titusville	70	36	15	17	2	
Winter Gardens	1		1			



NOTES. TOTAL FELT AREA 9000 SQUARE MILES
AREA FELT INTENSITY V-VI 4000 SQUARE MILES

0 25 50
SCALE (MILES)

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INTENSITY MAP OCTOBER 27, 1973
EARTHQUAKE

FIGURE 1 (APPENDIX 2D)

APPENDIX 2E

REFLECTION SURVEY PROCEDURES

APPENDIX 2E

REFLECTION SURVEY PROCEDURES

INTRODUCTION

As part of the geological reconnaissance for a nuclear power plant near Ft. Pierce, Florida, Alpine Geophysical Associates, Inc., has conducted a marine geophysical survey of the area north and south of the plant site (Figure 2.5-92). The primary purpose of the survey was to map the surface of the limestone occurring at about 600 feet below ground level at the plant site and determine the presence or absence of faulting in this surface.

Field work was accomplished between August 22 and 26, 1974. A written report was issued to Law Engineering on September 25, 1974.

Previous Work

There have been no previous efforts to use continuous seismic reflection methods in the survey area to delineate geologic structures. The only previous work consists of a number of wells drilled throughout the county and some supplementary exploratory drill holes nearer the plant site. The elevation of the surface of the limestone was recorded for all these holes and samples taken in some.

Basis for Proposed Faults

A contour map of the limestone surface, based on the drill hole data, showed vertical differences in the limestone surface elevation of up to 270 feet in holes a few miles apart. This difference is greater than could be accounted for on the basis of the regional dip of the limestone surface, which was assumed to be fairly uniform, smooth and dipping slightly to the southeast. Faulting was, therefore, postulated to account for the local vertical offsets. Since some of the traces cross the Intercoastal Waterway, a marine survey was a possible means of checking the presence or absence of the faults.

Limitations of Method

The primary limiting factor of shallow marine seismic surveying is the character of the ocean bottom surface sediments. An unconsolidated gaseous muck sometimes acts as an absorber of the seismic energy being generated on or near the water surface. Insufficient energy is transmitted through such muddy bottoms to allow detection of deeper subbottom layers.

A problem peculiar to high power seismic units used in very shallow water is the loss of resolution of those layers within 50-100 feet of the ocean bottom because of the closely-spaced and high powered return of the bottom multiple signals which completely obliterate the record over this range of depths. Below this depth, the signal returns from the subbottom layers are stronger than the multiple signals of the bottom.

INSTRUMENTATION

Seismic

Continuous Seismic Reflection Profiling

Continuous seismic reflection profiling is a geophysical technique designed to obtain a vertical profile of the ocean floor and to delineate geological stratification of sediments and rock formations as they exist beneath the ocean floor. The seismic reflection technique utilizes an impulsive energy source and the transmission of this energy through the water, sediment and rock. Wave energy reflected from boundaries between geologic materials of contrasting densities and sonic velocities travels back along similar wave paths to the source position where the reflected energy is detected and recorded by sensitive instruments. The seismic reflection technique becomes "continuous profiling" when the energy source is released repeatedly into the water from a vessel underway, and reflected signals are recorded along a continuous traverse.

The concept of continuous seismic reflection profiling is illustrated in Figure 2E-1, which shows a vessel towing the energy source and recording cables, and the various sub-sea level surfaces which reflect wave energy back to the water surface where that energy is detected and recorded.

Acoustic Energy

The seismic reflection method depends upon the transmission and reflection of sound waves at the interface between two media of different acoustic properties. The interface may be an abrupt change in acoustical properties or it may be a zone of very rapid change in these properties. The physical condition causing this change may or may not be related to their lithology, although reflecting interfaces usually are indicative of a change in the sediment types. Changes in sediment densities or water content within a sediment layer may also produce a reflecting interface, although there occurs no variation in the sedimentary material.

The amount of acoustic energy reflected from an interface separating two different sedimentary strata is proportional to their acoustic impedances. The acoustic impedance is expressed mathematically by the following equation:

$$Z = PV$$

Where:

Z is the acoustic impedance,
P is the mass density of the medium,
V is the velocity of the compressional wave through the medium.

The ratio of reflected to incident energy at the interface, normally referred to as the reflection coefficient, is related to the acoustic impedances of the two media for the specific case of normal incidence by the following equation:

$$\frac{E_r}{E_i} = \frac{(P_2V_2 - P_1V_1)^2}{(P_2P_2 + P_1V_1)^2}$$

r and i subscripts refer to the reflected and incident waves
1 and 2 subscripts refer to incident and reflected media, respectively

Thus, the amount of the reflected energy depends upon the contrast in both the densities and velocities of sound in the respective layers.

In an ideal homogeneous medium the sound energy would propagate from a sound source as a spreading spherical wave. The acoustic energy arriving at any point in the medium would, thus, be proportional to the inverse square of the distance from the source. However, attenuation processes, related to the physical characteristic of the transmitting medium, also act to reduce the sound energy level. Thus, such phenomena as absorption, dispersion, scattering and diffraction affect the losses which an acoustic wave encounters while being propagated. In general, attenuation losses are frequency dependent, in that higher frequencies attenuate faster than lower frequencies.

Variations exist in the types of energy sources, in the frequency range of transmitted and reflected energy and in the penetration and resolution capabilities. Thus, several different instruments are generally necessary to achieve both deep penetration and optimum resolution.

3000 Joule Sparker

The acoustical energy source used in continuous seismic profiling for this study was the Alpine 3000 joule Sparker, in which an electric discharge (spark) released in the water near the surface produces a high level, relatively broad-band impulse. The spark is triggered repetitively at a fixed rate by the recorder, the acoustic energy reflected from the ocean bottom and the subbottom horizon is detected by a towed hydrophone array, amplified, filtered, and recorded on a trip chart recorder.

Receiver and Amplifier and Recorder

The detecting hydrophone array (ell) is composed of ten (10) MP-1 pressure sensitive geophones connected in series-parallel, such that the attenuation for an axially propagated wave is maximum at five hundred Hertz (500 Hz). This arrangement effectively attenuates acoustic signals propagated horizontally along the axis of the hydrophone array, such as ship's noise. This enhances the reflected acoustic signals which are propagated vertically and arrive at all the hydrophones simultaneously (in phase). The hydrophones are encased in a pliable plastic tube sealed at both ends and filled with a special acoustic fluid. The long slender shape of the array provides an easy, relatively noise-free tow through the water. The tow

cable is a two-conductor shielded cable taped to both the Sparker cables and a flotation tube for the purpose of confining the array and the spark gap close together and near the surface. Floating the cables also allows the survey vessel to operate in very shallow water at slow speeds.

The recording unit for the transducer-transceiver system is an Alpine/Alden Model Number 469 which was operated at a sweep rate of one-half of a second. At this recording speed, approximately 1500 feet of penetration could be depicted on each sweep.

Navigation

Sextant angles on objects of known position on the mainland were used where possible for navigation offshore. This was supplemented with proximity to to known landmarks. In the inlets and the Intercoastal Water, the frequently placed buoys and channel markers were noted on the record when the boat passed such markers. This notation, together with marking the record every minute during the survey and running at a constant boat speed, allowed interpolation of the one-minute fix marks between known locations. This method was used to generate the detailed maps of the track lines and the fixes taken. The extent of coverage in the survey was limited primarily by the time allotted for the survey.

CORRELATION OF DATA WITH DRILL HOLE DATA

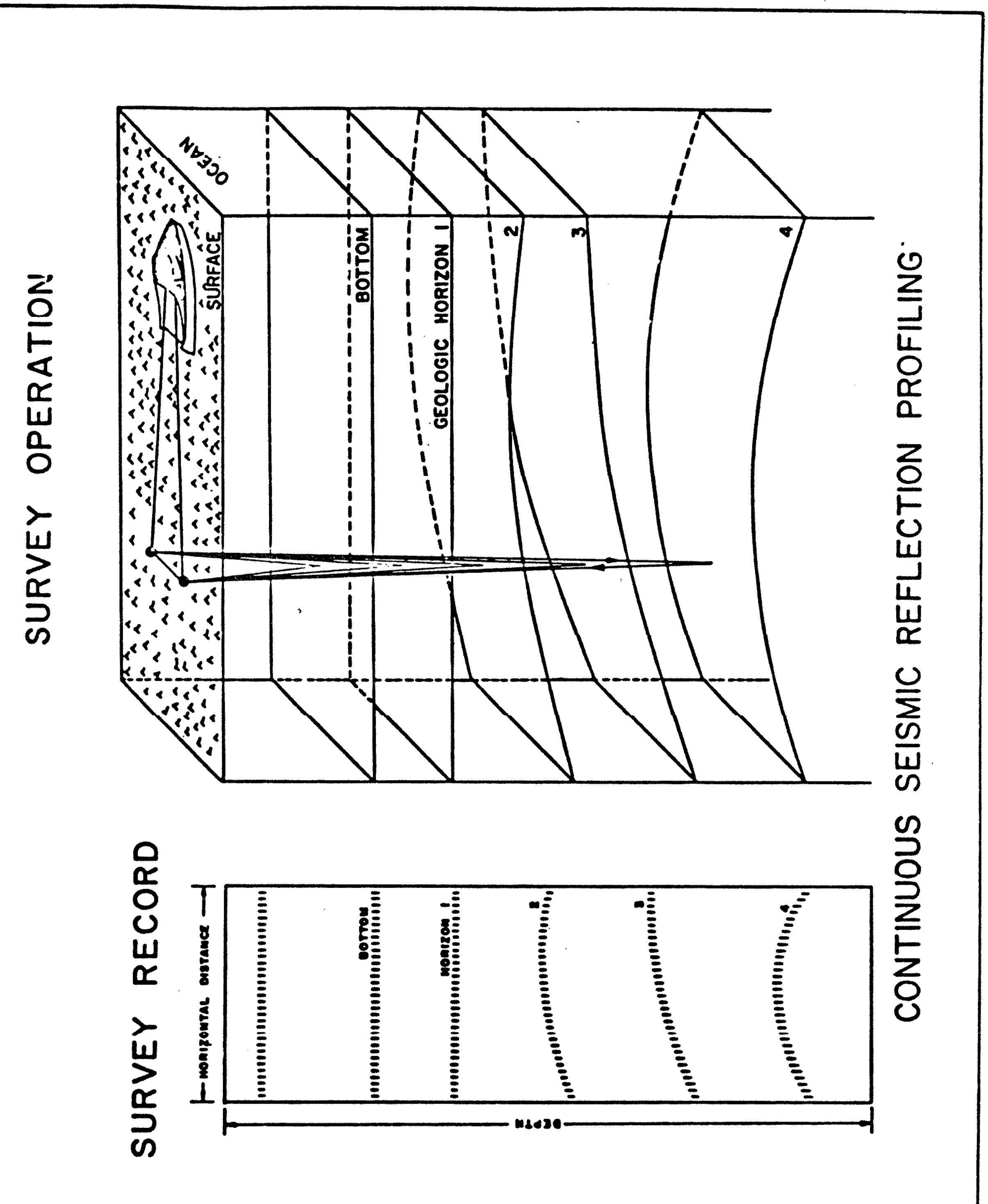
Speed of Sound in Sediments

Seismic field data is printed out in the form of continuous data which is a record of the time of travel of sound through various layers below the bottom. It is preferable to convert this travel time into a depth in feet to each important layer. To do this, some correlation with known various layers must be known. In the present case, both types of data are available.

A drill hole, AG 104, located on land very near fix 173, which is on the west side of Indian River directly across from the power plant, hit top of the Suwannee limestone at a depth of 560 feet. This surface correlates well with a prominent reflector on the seismic records if the speed of sound is about 6000 ft/sec. Some seismic refraction data available in that area showed 6000 to be a reasonable speed of sound in the sediments of the area.

Sediment Depth Scale

On the ten-inch recording chart used, the vertical depth scale corresponding to a speed of sound of 6000 ft/sec would be about 150 feet to the inch. This same vertical scale was used on the final profile sheets, with the horizontal scale controlled to 1:10,000.



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

CONTINUOUS SEISMIC
REFLECTION PROFILING

FIGURE 2E-1

APPENDIX 2F

THE DESIGN BASIS TORNADO FOR THE

ATLANTIC COAST AND FLORIDA'S EAST COAST

APPENDIX 2F

THE DESIGN BASIS TORNADO FOR THE ATLANTIC COAST AND FLORIDA'S EAST COAST

I. TORNADO DESIGN CRITERIA

An analysis of all tornadoes occurring along the Atlantic coast recorded in Storm Data for the period 1950 to 1972 is given in paragraph II. A Design Tornado was developed in parallel with the methodology presented in "Technical Basis for Interim Regional Tornado Criteria."[1]

1.	Maximum speed	=	218 mph
2.	Tangential wind speed (rotational)	=	163 mph
3.	Translational wind speed, maximum	=	55 mph
	minimum	=	5 mph
4.	Pressure drop at center of vortex	=	.944 psi
5.	Maximum rate of pressure drop	=	.508 psi/sec

II. BASIS FOR SELECTION OF DESIGN TORNADO

The development of a "design tornado" follows the probabilistic approach proposed by the AEC which results in the probable 10^{-7} per year wind speed.[1] In this investigation, the 10^{-7} per year Design Tornado is determined for the Atlantic Coastal region. All tornadoes reported in Storm Data (and confirmed by the N.S.S.F.C. logs between 1950 and 1972) which occurred within 4 miles of the Atlantic Coast, or within the Florida Keys were included.

Classification of intensity was made using an objective guide based upon the Dames & Moore Intensity Scale described in Section III given sufficient detail from either Storm Data or the American Red Cross and news clippings. Similarly, the area of each tornado was determined from path length and width data.

The geometric probability is given by

$$P = n(a/A)$$

when P = mean annual probability of a tornado striking a point

n = mean number of tornadoes occurring with the area A per year

a = average path length X path width

The values used for the Atlantic Coast are as follows:

	Atlantic Coast
a	$0.257\ mi^2$
A	$16{,}100\ mi^2$
n	$9.0\ yr^{-1}$
p	$1.44 \times 10^{-4}\ yr^{-1}$
10^{-7} wind speed	

Since Florida had by far the largest sample of tornadoes, the average tornadic area ($P_l \times P_w$) found for Florida was used for the Atlantic coast. The value of "a" from Florida was the highest of the average areas of all the Atlantic States and it is therefore a conservative assumption. The geometric annual probability of any tornado striking a point is coupled with

the probability of a given intensity to yield the probable 10^{-7} (per year) wind speed. The resulting value for the Atlantic coast is 218 mph.

It is felt that the departure of the intensity frequencies from a log-normal distribution is more than adequately compensated for by using the upper bound of the intensity interval. Also, it is likely that a bias exists in the reporting of tornado events. This results primarily in an under-reporting of unseen or less damaging tornadoes which would probably fall into the D&M1 and D&M 2 categories and reduce the slope of the curves shown in Figure 1.

Little recorded data was found on translational wind speed in the regions of interest. Flora[2], for example, states that 45 mph is the average translation wind speed based upon a study of 1,000 tornadoes by J.R. Martin[2] and Wolford [3] states that 40 mph is the average for all tornadoes. A general opinion among severe weather meteorologists is that tornadic intensity is correlated (indirectly) with translational speed. This is due to the fact that intense storms are associated with strong jets in the mid-levels of the troposphere which in turn propels the parent cloud in proportion to the average wind speed of the cloud layer[4]. This is supported by a study of long track ($\geq$ 100 mi. path lengths) tornadoes which are very intense tornadoes and have an average translational speed of 67.8 mph[5].

The Design Basis Tornado parameters are shown below.

Max. Speed (mph)	Rotational Speed (mph)	Translational Speed (mph) Max.	Min.	Total Pressure Drop (Psi) Δp	Maximum Rate of Pressure Drop (psi/sec)
218	163	55	5	.944	.508

The maximum rate of pressure drop occurs at the radius of maximum wind and is determined by:

$$\frac{dp}{dt} = \rho_A \frac{V_m^2 T}{r_m} \quad \text{where}$$

p = pressure

t = time

T = translational speed

r_m = radius of maximum rotational windspeed - 150'

V_m = maximum tangential wind

The total pressure drop, Δp, is determined by:

$$\int_o^r \frac{\partial p}{\partial r} dr = \int_o^r 2 \rho_A \frac{V_m^2}{r_m} dr$$

the application of the cyclostrophic wind equation.

The maximum value of 218 mph is consistent with the previous studies conducted by Florida Power and Light where the maximum tornadic wind is seen from damage estimates to be in the range of 165 to 200 mph[6].

The Design Tornado for the Atlantic Coast is less than that determined by the AEC for Region 1. This results from the following:

(1) The geometric probability is less using the actual path length and width of the region under consideration. The average area of these tornadoes is an order of magnitude less than Iowa tornadoes, .26 mi^2 vs. 2.82 mi^2.

(2) Some of these coastal tornadoes are actually "tornadic waterspouts" or have been induced by hurricanes and are seldom intense due to the lack of strong vertical shear of the horizontal wind through a deep layer of the atmosphere. This shear is essential to the explosive development of large rotating thunderstorms which spawn severe tornadoes.[7,8,9]

(3) The D&M Inteisity Scale is based upon extensive analysis of the effects of wind loadings upon structures such that an objective estimate of wind speed may be made from written summaries of damage accounts. Although a comparative study has yet to be made, it is likely that differences exist between the D&M Intensity classification and the F-Scale classification.

III DAMES & MOORE INTENSITY SCALE

Dames & Moore tornadic wind intensity scale was created in order to evaluate tornado damage and associated causative wind speeds. In development of this scale, it was necessary to calculate the range of wind speeds which could

conceivably give rise to reported structural damage. Probable wind velocities were estimated from observed damage and these velocities were used to classify tornadoes according to intensity. The results of these evaluations permitted a reasonable classification of tornadoes according to wind velocity-damage relationships which are in general agreement with other attempts [10].

Several assumptions were necessary in order to evaluate the wind velocities associated with varying damage of residences and other buildings. Construction variances resulting from differences in local codes and workmanship and quality of construction were accounted for in the calculation of the range of wind velocities associated with particular types and extents of damage. Due to the extent of these variations, a fairly wide range of overlapping wind velocities is given for each type of damage.

Specific assumptions were employed regarding the action of wind forces on the building. A sustained peak wind velocity was considered. Gusting effects, repeated loadings, and racking of structural members and joints were not included. The effects of rapid decrease in air pressure on the structure were disregarded since natural venting through broken windows, damaged siding, etc. minimizes or negates the pressure drop effects and few such cases were observed in the tornado record. The wind pressure coefficients utilized in the structural calculations were selected from the American National Standard Building Code, 1972.[11] Various sizes and numbers of connectors were assumed for the roof to wall connections, thus yielding a range of wind velocity values associated with varying roof damage levels. Calculations were made to verify the wind forces required to inflict

these levels of damage, for partial or total roof removal.

The specific results of the above mentioned calculations are reflected in Table I entitled "Dames & Moore Tornado Intensity Classification." Progressively higher degrees of damage are summarized in the damage description for Dames & Moore Intensity categories 1 through 6. Each category has an associated velocity range which is the sum of the rotational and translational speeds.

The major source of damage description was obtained from the N.S.S.F.C. records and Storm Data. This information was supplemented by data obtained from the American Red Cross and some newspaper articles. In classifying the tornadoes, engineering estimates were based predominately on the highest degree of damage occurring in the description. If the available damage description was inadequate or nonexistent, the tornado path length and width, the dollar damage category and the geographic location were considered in assigning the appropriate damage intensity. However, in some instances, the information available from the Storm Data or American Red Cross was insufficient for classification in accordance with the Dames & Moore Intensity guidelines and, therefore, that tornado was not analyzed.

The application of these guidelines to the Atlantic states is given in Table II.

IV. FLORIDA EAST COAST ANALYSIS

A separate analysis of tornadoes occurring within the four mile inland coastal strip of the Florida Atlantic Coast is provided. All reported tornadoes which originated, terminated or crossed the four mile coastal strip are included in this analysis. However, the computed tornado affected areas (path length and width) are only restricted by an upper path limit of 10 miles. The methodology is identical to the one described in the previous sections except only tornado occurrences indigenous to the Florida east coast are analyzed.

Table III summarizes the climatological tornado data of the Florida east coast by year of occurrence and county. During the period 1950 to 1960, 34 tornadoes were sighted. However, during the period 1960 to 1970, 67 tornadoes were sighted. Comparing the two decades of time, the number of tornado reports increased by 97%. The population increase for the east coast of Florida (documented in Table IV) from 1950 to 1970 was 183%. If climatological tornado trends are discounted, there appears to be a correlation between increasing tornado reports and increasing population densities. Intense tornadoes, however, normally affect a large area[5] and the tornado sightings should be independent of population density in an already populous region. Many low intensity tornadoes may have been undetected or not reported in the earlier portion of the sampling period. The increase in the tornado-affected area ('a' term in the geometric probability equation) associated with unreported low intensity tornadoes is small as compared to more intense reported tornado occurrences.

As previously defined:

$$P = n(a/A)$$

where:

$a = 0.257$ miles^2 (for Florida East Coast)

$A = 2420$ miles^2 (area examined)

$n = 112/23$ (1950 to 1972)

$P = 5.17 \times 10^{-4}\ yr^{-1}$

Table V summarizes the Florida East Coast tornadoes by the Dames & Moore upper class intensity scale. Figure 2 is derived from the data in Table V. To determine the extrapolated percent probability with the associated maximum wind speed for the one in ten million probability:

$$\frac{P \text{ of } 1.0 \times 10^{-7}}{P \text{ for Florida East Coast}} = 0.193 \times 10^{-3} = 0.019\%$$

and the associated maximum wind speed, from Figure 2, is 242 mph for 1.0×10^{-7} probability level.

Detailed information on the tornadoes under study is provided herein. Table VI gives the chronological list of tornadoes for the period of 1950-1972 and the available path lengths, widths and areas. Table VII classifies these tornadoes into the Dames & Moore intensity categories. The tornadoes are tabulated according to damage description in each intensity category. The miscellaneous category includes the damage descriptions listed in Table I which are not separately described in this table.

For purposes of summarizing the Florida East Coast tornado intensity classification, Table VIII has been prepared. It should be pointed out that for tornadoes where the area data is not available the average area was assumed for purposes of this analysis (See note 2, Table VIII).

1. Technical Basis for Interim Regional Tornado Criteria, WASH-1300 (UC-11), U.S.A.E.C. Office of Regulation, May, 1974.

2. Flora, S.D., Tornadoes of the United States, University of Oklahoma Press, Norman, Oklahoma, pp. 194, 1954, 2nd edition

3. Wolford, L.V., Tornado Occurrences in the U.S., USWB Tech Paper No. 20, 1960

4. Wilson, J.W., "Movement and Predictability of Radar Echoes", ERCTM-NSSL-28, Norman, Oklahoma, November, 1966

5. Wilson, J.W. and Morgan, G.N., "Long-Track Tornadoes and Their Significance", Preprints. Seventh Conference on Severe Local Storms, A.M.S., Kansas City, Missouri, October 5-7, 1971

6. Brooks, E.M., Gerrish, H.P., Hiser, H.W. and Senn, H.V., "A Comparative Study of Florida's Most Severe Tornadoes with Those in other Parts of the Continental U.S.", Docket No. 50-335, Fla. Power and Light Company, Miami, Florida

7. Newton, C.W., 1950 "Structure and Mechanism of the Prefrontal Squall Line", J. Meteorology, Vol. 7, No. 3, pp. 210-222

8. Ward, N.B., "Rotational Characteristics of a Tornado Cyclone", Preprints, Sixth Conference on Severe Local Storms, A.M.E., Chicago, Illinois, October, 1969

9. Nicholson, F.H., "The Formation of Severe Local Storms Through the Agency of Random Turbulent Transport", Preprints, Eighth Conference on Severe Local Storms, A.M.S., Denver, Colorado, October 15-17, 1973

10. Fujita, T.T., "Proposed Characterization of Tornadoes and Hurricanes by Area and Intensity", Chicago University, Department of Geophysical Sciences, Satellite and Mesometeorology Research Project, Research Paper No. 91, February 1971.

11. "American National Standard Building Code Requirements for Minimum Design Loads in Buildings and Other Structures", American National Standards Institute, New York, New York, A58.1 - 1972

TABLE I

DAMES AND MOORE

TORNADO INTENSITY CLASSIFICATION

DAMES & MOORE INTENSITY	WIND VELOCITY (mph)	EXPECTED DAMAGE
1	50-90	Partial roof removal of weak rural structures; some trees uprooted and blown
2	80-120	Total roof removal of rural structures; partial roof removal of individual residences; house trailers moved or rolled; more extensive tree uprooting
3	100-150	Rural structures heavily damaged; total roof removal of residences; house trailers destroyed; nonreinforced masonry walls overturned; extensive sign damage and tree uprooting
4	120-180	Rural structures demolished; total roof removal of residences and some walls down; partial roof removal of light steel industrial buildings and wood truss commercial buildings.
5	150-225	Complete homes destroyed; total roof removal of light industrial buildings and wood truss commercial buildings; partial roof removal of heavy industrial buildings
6	200-300	Catastrophic destruction; homes off foundations; substantial commercial and industrial buildings destroyed; large steel framed structures heavily damaged.

TABLE II

TORNADOES AND THEIR INTENSITIES OCCURRING ALONG THE ATLANTIC COAST

FROM 1950 - 1972

Dames & Moore Intensity Class	FLA.E COAST	GA.	SC.	NC.	VA.	MD.	NJ.	MASS.	ME.	Total Classified By Intensity	Adjusted Total For All Tornadoes
6	0	0	0	0	0	0	0	0	0	0	0
5	0	0	0	0	0	0	0	0	0	0	0
4	9	0	1	3	0	1	0	0	0	14	15.8
3	11	1	2	7	1	2	2	0	0	26	29.2
2	47	2	4	7	5	2	1	4	1	73	82.1
1	31	4	11	5	6	6	2	3	3	71	79.9
TOTAL	98	7	18	22	12	11	5	7	4	184	207
All Tors	112	7	20	25	12	11	6	10	4	207	

TABLE III

TORNADO SIGHTING ALONG THE 4 MILE ATLANTIC INLAND SHORELINE FOR EAST COAST FLORIDA COUNTIES BY YEAR

Year	Nassau	Duval	St. Johns	Flagler	Volusia	Brevard	Indian River	St. Lucie	Martin	Palm Beach	Broward	Dade	Totals
1950				1									1
51													0
52										2			2
53						1		2				1	4
54						1		2				1	4
55										1	2		3
56									1	1		1	3
57													0
58			1			1				4			6
1959			1		1	1			1	1	1	1	7
60					2			1		1			4
61	1	1						1		1			4
62			1					1		1		1	4
63					1	1	1			1			4
64			1	1	1	2			1	4		1	11
65										1	2		3
66						3	1					2	6
67						1		1		2	1		5
68					2	7				4		4	17
1969					1						1	2	4
70	1				1	2		1	1	2		1	9
71						3				1	1	1	6
1972		1				6				1	1		9
Totals	2	2	4	2	9	29	2	9	4	28	9	16	116

TABLE IV

POPULATION IN COASTAL FLORIDA COUNTIES

U.S. BUREAU OF CENSUS

	1950	1970
Dade	495,084	1,267,792
Broward	83,933	620,100
Palm Beach	114,688	348,753
Martin	7,807	28,035
St. Lucie	20,180	50,836
Indian River	11,872	35,992
Brevard	23,653	230,006
Volusia	74,229	169,487
Flagler	3,367	4,454
St. John's	24,998	30,727
Duval	304,029	528,865
Nassau	12,811	20,626
Totals	1,176,651	3,335,673

TABLE V

COASTAL EAST FLORIDA TORNADOES

FROM 1950 TO 1972

Dames & Moore Intensity Class	Florida East Coast	Adjusted Total For All Tornadoes	Cumulative Frequency m	Cumulative Percent $\left(1 - \frac{m}{112 + 1}\right)$ Times 100	Dames & Moore Upper Class Wind Speed mph
6	0	0			300
5	0	0			225
4	9	10.3	112.0	0.88	180
3	11	12.6	101.7	10.00	150
2	47	53.7	89.1	21.15	120
1	31	35.4	35.4	68.67	90
Subtotal	98	112.0			
Unknown	14	0			
Total	112	112.0			

TABLE VI

TORNADO STATISTICS FOR THE EAST FLORIDA COAST

Reference: Storm Data; NOAA

Period of Record 1950 to 1972

Number	Year	Month	County	Length (Miles)	Width (Yards)	Width (Miles)	Area (Sq. Miles)
1	1950	March	Flagler		150		
2	1952	February	Palm Beach	1.5	15	.009	.014
3	1952	August	Palm Beach	.17	15	.009	.002
4	1953	April	St. Lucie	2.0			
5	1953	August	St. Lucie				
6	1953	September	Brevard	1.0	200	.114	.114
7	1953	September	Dade				
8	1954	April	Dade				
9	1954	August	St. Lucie				
10	1954	September	St. Lucie				
11	1954	September	Brevard		90		
12	1955	April	Broward				
13	1955	August	Palm Beach				
14	1955	October	Broward				
15	1956	August	Palm Beach				
16	1956	August	Martin				
17	1956	October	Dade				
18	1958	January	Brevard				
19	1958	April	St. John's	3.0	75	.043	.129
20	1958	April	Palm Beach				
21	1958	April	Palm Beach				
22	1958	August	Palm Beach				
23	1958	August	Palm Beach				
24	1959	April	Brevard	1.0	100	.057	.057
25	1959	June	St. John's				
26	1959	June	Dade	10(12.0)	350	.199	1.99
27	1959	June	Palm Beach	7.0	150	.085	.595
28	1959	September	Broward		33		
29	1959	October	Volusia	1.0	70	.040	.040
30	1959	October	Martin				

TABLE VI (Cont'd)

Number	Year	Month	County	Length (Miles)	Width (Yards)	Width (Miles)	Area (Sq. Miles)
31	1960	July	Volusia				
32	1960	July	Volusia				
33	1960	September	Palm Beach				
34	1960	October	St. Lucie				
35	1961	March	Nassau				
36	1961	April	Duval				
37	1961	May	Palm Beach				
38	1961	June	St. Lucie				
39	1962	July	St. Lucie				
40	1962	August	St. John's				
41	1962	September	Palm Beach				
42	1962	November	Dade	1.0			
43	1963	July	Palm Beach				
44	1963	July	Brevard				
45	1963	August	Volusia				
46	1963	November	Indian River				
47	1964	August	Brevard				
48	1964	August	St. John's				
49	1964	August	Flagler				
50	1964	August	Volusia				
51	1964	October	Dade		75		
52	1964	October	Palm Beach				
53	1964	October	Palm Beach				
54	1964	October	Martin				
55	1964	October	Palm Beach				
56	1964	October	Palm Beach				
57	1964	October	Brevard				
58	1965	February	Broward	5.0	60	.034	.170
59	1965	February	Broward	15.0			
60	1965	March	Palm Beach				
61	1966	April	Brevard	10(14.0)	350	.199	1.99

TABLE VI (Cont'd)

Number	Year	Month	County	Length (Miles)	Width (Yards)	Width (Miles)	Area (Sq. Miles)
62	1966	April	Brevard		150		
63	1966	June	Dade	4.0			
64	1966	June	Dade				
65	1966	June	Indian River				
66	1966	September	Brevard				
67	1967	February	Broward				
68	1967	June	Palm Beach				
69	1967	August	St. Lucie				
70	1967	August	Brevard				
71	1967	September	Palm Beach				
72	1968	February	Dade	4.5	100	.057	.257
73	1968	February	Palm Beach				
74	1968	May	Brevard				
75	1968	June	Brevard	0.3			
76	1968	June	Brevard	0.1	15	.009	.001
77	1968	June	Dade				
78	1968	June	Brevard				
79	1968	June	Dade	10.0			
80	1968	July	Brevard				
81	1968	July	Palm Beach				
82	1968	August	Volusia	5.0			
83	1968	August	Volusia	2.0	125	.071	.142
84	1968	September	Brevard	1.5			
85	1968	October	Dade		800		
86	1968	October	Palm Beach				
87	1968	November	Brevard	.25	200	.114	.029
88	1968	November	Palm Beach	8.0	30	.017	.136
89	1969	February	Dade	1.0	125	.071	.017
90	1969	June	Dade				
91	1969	August	Broward				
92	1969	October	Volusia				
93	1970	January	St. Lucie				

TABLE VI (Cont'd)

Number	Year	Month	County	Length (Miles)	Width (Yards)	Width (Miles)	Area (Sq. Miles
94	1970	February	Brevard				
95	1970	March	Brevard	1.9	333	.189	.359
96	1970	March	Dade				
97	1970	March	Palm Beach				
98	1970	June	Martin				
99	1970	July	Nassau				
100	1970	July	Volusia				
101	1970	July	Palm Beach				
102	1971	February	Palm Beach	.057	.10	.006	0(.0005)
103	1971	June	Broward	4.0	50	.028	.112
104	1971	June	Dade	2.0	200	.114	.228
105	1971	August	Brevard	3.0	75	.043	.129
106	1971	August	Brevard	3.0	25	.014	.042
107	1971	September	Brevard	0.25	20	.011	.003
108	1972	February	Brevard				
109	1972	March	Brevard	2	500	.284	.568
110	1972	March	Brevard	1	100	.057	.057
111	1972	March	Brevard	2	50	.028	.056
112	1972	June	Brevard	0.25	50	.028	.007
113	1972	June	Brevard	2	100	.057	.114
114	1972	June	Brevard	4	100	.057	.228
115	1972	June	Brevard	3	100	.057	.171
116	1972	July	Duval	0.25	30	.017	.004

TABLE VII

STORM DATA DAMAGE REPORTS
FOR PERIOD OF RECORD: 1950 TO 1972

Dames & Moore Intensity Categories

Wind Speed Range (mph)	1 50-90	Misc.	2 80-120		Misc.	3 100-150		Misc.	4 120-180		Misc.	5 150-250		Misc.	6 225-300	Misc.
Chronological Listing	Trees Downed and Uprooted		Partial Roof	Small Buildings Damage		Total Roof	Partial Home Damage		Severe Home Damage	Weak Structures Flatten		Homes Destroyed	Substantial Buildings Damaged		Substantial Buildings Destroyed	
1					X											
2					X											
3				X												
4		X														
5		X														
6		X														
7		X														
8		X														
9		X														
10				X												
11					X											
12					X											
13																
14				X												

TABLE VII (Con't)

STORM DATA DAMAGE REPORTS
FOR PERIOD OF RECORD: 1950 TO 1972

Dames & Moore Intensity Categories

Wind Speed Range (mph)	1 50-90 Misc.	2 80-120	Misc.	3 100-150	Misc.	4 120-180	Misc.	5 150-250	Misc.	6 225-300 Misc.
Chronological Listing	Trees Downed and Uprooted	Partial Roof	Small Buildings Damage	Total Roof	Partial Home Damage	Severe Home Damage	Weak Structures Flatten	Homes Destroyed	Substantial Buildings Damaged	Substantial Buildings Destroyed
15	X									
16	X									
17			X							
18	X									
19						X	X			
20	X									
21	X									
22	X									
23	X									
24						X	X			
25	X									
26							X			
27		X	X							
28	X									

TABLE VII (Con't)

STORM DATA DAMAGE REPORTS
FOR PERIOD OF RECORD: 1950 TO 1972

Dames & Moore Intensity Categories

Wind Speed Range (mph)	1 50-90 Misc.	2 80-120 Misc.		3 100-150 Misc.		4 120-180 Misc.		5 150-250 Misc.		6 225-300 Misc.
Chronological Listing	Trees Downed and Uprooted	Partial Roof	Small Buildings Damage	Total Roof	Partial Home Damage	Severe Home Damage	Weak Structures Flatten	Homes Destroyed	Substantial Buildings Damaged	Substantial Buildings Destroyed
29			X							
30										
31	X									
32										
33	X									
34			X							
35					X					
36			X							
37	X									
38										
39			X							
40			X							
41										
42			X							

TABLE VII (Con't)

STORM DATA DAMAGE REPORTS
FOR PERIOD OF RECORD: 1950 TO 1972

Dames & Moore Intensity Categories

Wind Speed Range (mph) / Chronological Listing	1 50-90 Misc. Trees Downed and Uprooted	2 80-120 Misc. Partial Roof	2 Small Buildings Damage	3 100-150 Misc. Total Roof	3 Partial Home Damage	4 120-180 Misc. Severe Home Damage	4 Weak Structures Flatten	5 150-250 Misc. Homes Destroyed	5 Substantial Buildings Damaged	6 225-300 Misc. Substantial Buildings Destroyed
43			X							
44	X									
45	X									
46			X							
47	X									
48										
49			X							
50										
51	X									
52	X									
53			X							
54	X									
55	X									
56			X							

TABLE VII (Con't)

STORM DATA DAMAGE REPORTS
FOR PERIOD OF RECORD: 1950 TO 1972

Dames & Moore Intensity Categories

Wind Speed Range (mph)	1 50-90 Misc.	2 80-120 Misc.		3 100-150 Misc.		4 120-180 Misc.		5 150-250 Misc.		6 225-300 Misc.
Chronological Listing	Trees Downed and Uprooted	Partial Roof	Small Buildings Damage	Total Roof	Partial Home Damage	Severe Home Damage	Weak Structures Flatten	Homes Destroyed	Substantial Buildings Damaged	Substantial Buildings Destroyed
57					X					
58					X					
59				X	X					
60			X							
61					X					
62				X	X					
63			X							
64										
65										
66										
67	X									
68			X							
69			X							
70			X							

TABLE VII (Con't)

STORM DATA DAMAGE REPORTS
FOR PERIOD OF RECORD: 1950 TO 1972

Dames & Moore Intensity Categories

Wind Speed Range (mph)	1 50-90	Misc.	2 80-120		Misc.	3 100-150		Misc.	4 120-180		Misc.	5 150-250		Misc.	6 225-300	Misc.
Chronological Listing	Trees Downed and Uprooted		Partial Roof	Small Buildings Damage		Total Roof	Partial Home Damage		Severe Home Damage	Weak Structures Flatten		Homes Destroyed	Substantial Buildings Damaged		Substantial Buildings Destroyed	
71								X								
72									X	X	X					
73					X											
74																
75					X											
76																
77					X											
78					X											
79					X											
80					X											
81					X											
82					X											
83							X	X								
84			X	X	X											

TABLE VII (Con't)

STORM DATA DAMAGE REPORTS
FOR PERIOD OF RECORD: 1950 TO 1972

Dames & Moore Intensity Categories

Wind Speed Range (mph)	1 50-90	Misc.	2 80-120		Misc.	3 100-150		Misc.	4 120-180		Misc.	5 150-250		Misc.	6 225-300	Misc.
Chronological Listing	Trees Downed and Uprooted		Partial Roof	Small Buildings Damage		Total Roof	Partial Home Damage		Severe Home Damage	Weak Structures Flatten		Homes Destroyed	Substantial Buildings Damaged		Substantial Buildings Destroyed	
85		X														
86						X		X								
87				X	X											
88								X								
89					X											
90																
91				X												
92		X														
93				X												
94				X												
95									X	X						
96				X												
)7	X	X														
)8				X												

TABLE VII (Con't)

STORM DATA DAMAGE REPORTS
FOR PERIOD OF RECORD: 1950 TO 1972

Dames & Moore Intensity Categories

Wind Speed Range (mph)	1 50-90		2 80-120			3 100-150			4 120-180			5 150-250			6 225-300	
Chronological Listing	Trees Downed and Uprooted	Misc.	Partial Roof	Small Buildings Damage	Misc.	Total Roof	Partial Home Damage	Misc.	Severe Home Damage	Weak Structures Flatten	Misc.	Homes Destroyed	Substantial Buildings Damaged	Misc.	Substantial Buildings Destroyed	Misc.
99		X														
100		X														
101				X												
102																
103			X	X	X											
104				X	X											
105								X								
106					X											
107					X											
108		X														
109									X							
110								X								
111							X	X								
112		X														
113									X	X						
114										X	X					
115									X	X						
116		X														

TABLE VIII

TORNADO INTENSITY CLASSIFICATION

Dames & Moore Intensity Class	Wind Speed (MPH)[1]	Number of Classified Tornadoes	Number After Adjustment[2]
1	50-90	34	38.7
2	80-120	46	52.3
3	100-150	13	14.8
4	120-180	9	10.2
5	150-225	0	0
6	200-300	0	0

(1) Each tornado is assumed to have the maximum possible wind speed for the Dames and Moore intensity class to which it has been assigned.

(2) Since fourteen (14) tornadoes were of unknown intensity although reported, the last column reflects those 14 distributed proportionately to those which were classified by intensity.

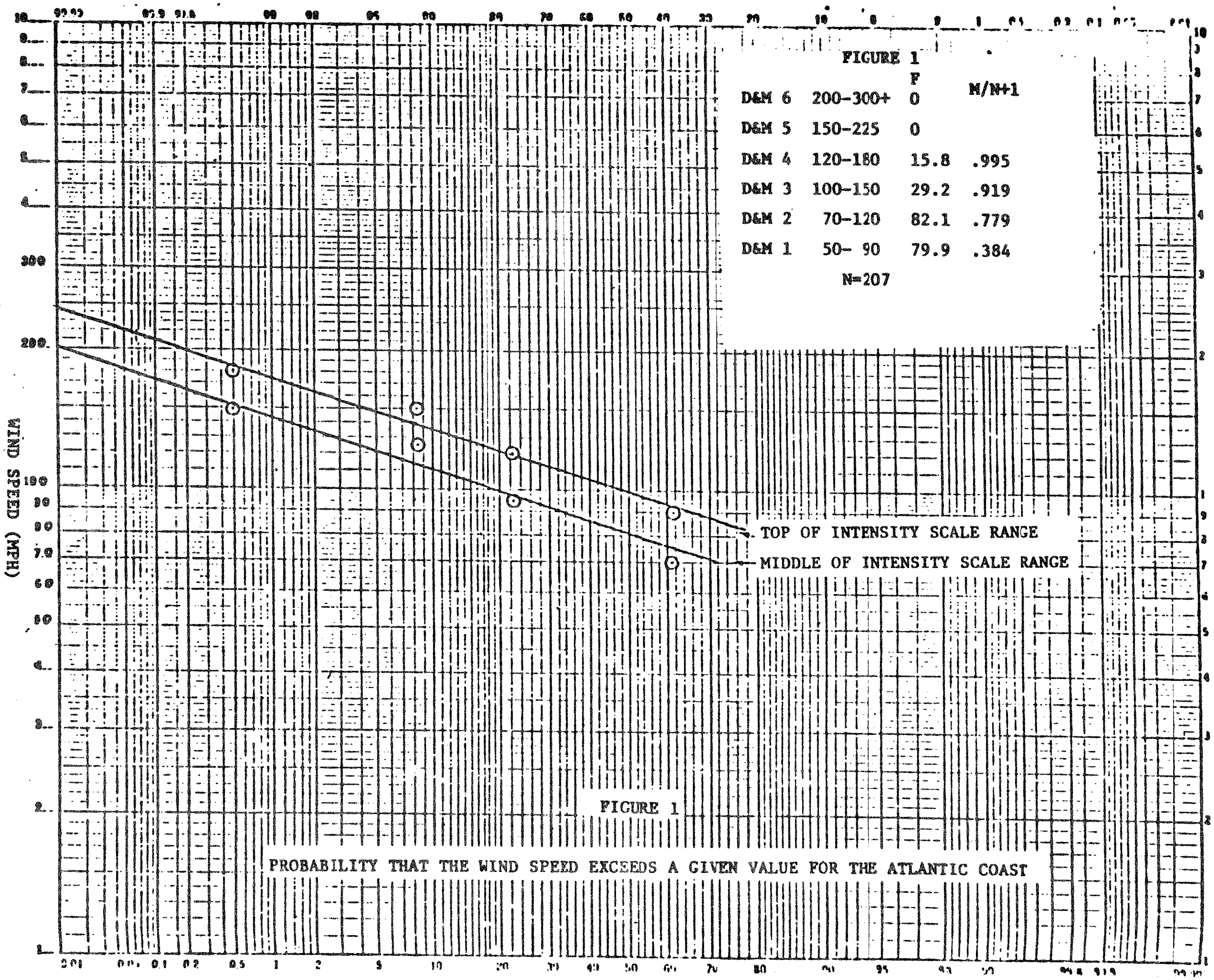


FIGURE 1

PROBABILITY THAT THE WIND SPEED EXCEEDS A GIVEN VALUE FOR THE ATLANTIC COAST

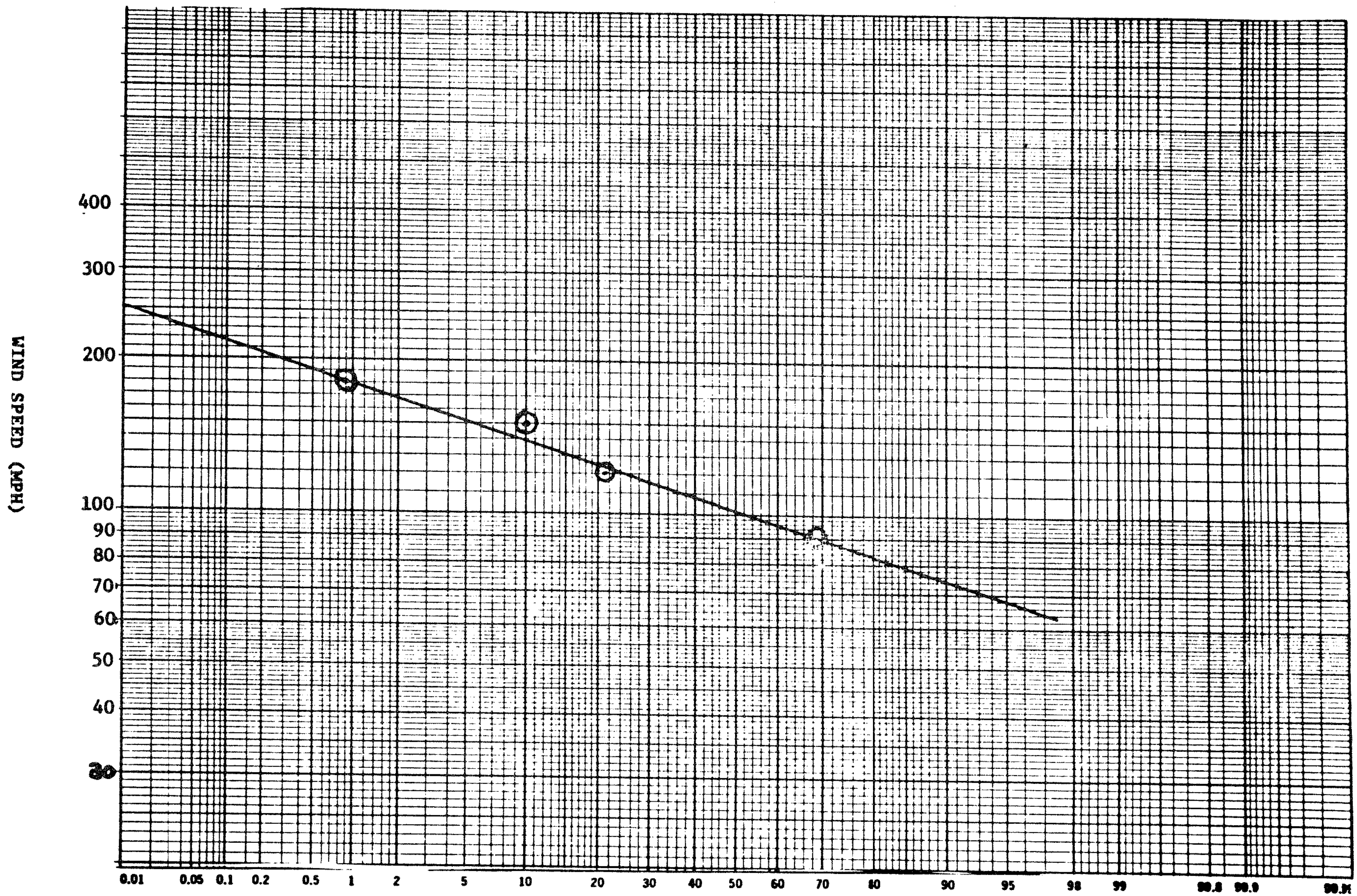


FIGURE 2

PERCENT PROBABILITY THAT THE WIND SPEED EXCEEDS A GIVEN VALUE FOR THE EAST COAST OF FLORIDA

BEFORE THE

UNITED STATES ATOMIC ENERGY COMMISSION

Docket No. 50-335

In the Matter of

Florida Power & Light Company

APPENDIX 2G

SWITCHYARD & CANAL INVESTIGATION & ANALYSIS

FOR

ST. LUCIE PLANT UNIT NO. 1

ST LUCIE

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ST LUCIE

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Appendix 2G (Cont'd)

ST. LUCIE

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APPENDIX 2G

SWITCHYARD & CANAL INVESTIGATION & ANALYSIS

2G1.0 INTRODUCTION

Since the first submittal of the St. Lucie No. 1 (then Hutchinson Island) PSAR in 1969, it was recognized that the general site area, in particular, the upper 50 feet of materials were considered to have some degree of liquefaction potential. For this reason the plant island proper was excavated to EL-60 and replaced with a highly controlled and engineered backfill. In particular, slopes of the original intake canal extending to Big Mud Creek, now the ultimate heat sink (emergency) canal, were considered to be potentially unstable. The primary reason for this instability was due to liquefaction.

Various stability analyses were performed at that time, namely conventional wedge and slip-circle types of failures were considered. They indicated only shallow sloughing or local sliding would result. However, the overpowering consideration of liquefaction based on the state of the art and the soils information available could conceivably result in a flow type of sliding and failure. Thus, a literature and personnel communication research study of liquefaction experience during earthquakes was conducted as described in PSAR Section 2.5.5 and the resulting conservative 20:1 final liquefaction induced slope type of analysis was utilized in the canal design. This design, as described in the PSAR Section 2.5.5 envisions partial liquefaction flow slides resulting in large conservative lateral movements of thousands of yards of sandy materials and still provides a 100 fold cross-section for water requirements. The conservative nature of this design flow slide condition, which can only really occur in a large magnitude earthquake of long duration, should be recognized. Florida is one of the lowest seismic regions of the United States.

The AEC staff could not conclude that this flow slide approach yielded a sufficiently conservative canal design; that all possible modes of failure were considered; and that its required degree of confidence was achieved. Thus, the program discussed herein was conducted to precisely define soil conditions in the emergency canal area, switchyard, and in intake forebay area in order that detailed analyses based on in situ soil characteristics could be performed. The additional analyses provide a detailed evaluation of liquefaction potential and slope stability considerations.

The analytical work described hereinafter is based on in situ soil properties. The results reaffirm the conservatism in assuming massive flow slides with 20:1 final slopes for canal design purposes. The study indicates minor reorientation of soils is possible. Specifically it indicates a potential for straining or sloughing of submerged canal surfaces of one to two feet. The conclusions of the field, laboratory, and analytical program are that the slopes are acceptably stable; that the UHS barrier wall can be designed to remain upright during and subsequent to DBE; and that massive flow slides do not occur, thus the Unit 1 PSAR design basis criterion of slides with a 20:1 final slope was excessively conservative.

2G2.0 FIELD INVESTIGATIONS

2G2.1 General

In order to establish the materials within the slopes of the intake cooling water and emergency cooling water canals Ebasco undertook an extensive field investigation program in October and November, 1974. The

drilling and sampling were performed by Law Engineering Testing Company and by Girdler Engineering under supervision of Law Engineering Testing Company. In addition, an Ebasco soils engineer for Quality Assurance was present at all times.

The program entailed a series of 26 initial borings followed by 25 detailed borings for undisturbed sampling in the switchyard area and along a cross-section of the emergency cooling water canal, north and east of the switchyard (See Figure 2G1). Refer to Figures 2GA1 through 2GA50 for detailed boring logs.

All the initial holes were drilled to elevation - 60 and ranged approximately 65' to 80' in depth. Standard penetration tests were performed in all initial borings, at two foot intervals, for the entire depth of the hole. Five drill rigs with hydraulic feed drill heads were used. As the program continued, the additional 25 bore holes were added to obtain undisturbed samples. These additional holes were added at 5 ft offsets to the initial holes where areas of particular interest were encountered. They are indicated on the boring location plan as borings with the suffix letter A, B, C, etc. as shown on Figure 2G1.

2G2.2 Drilling Equipment Utilized

The drilling equipment included one truck-mounted Failing 1500 rig, three failing 250 rigs, and one skid-mounted Failing 250 rig mounted on a trailer bed. With the exception of one soil test boring for which N rod was used, AW rod and side discharge drag bits were used for all standard penetration testing. Both AW and N rod were used in undisturbed sampling.

2G2.3 Procedures

2G2.3.1 Standard Penetration Test

Due to the importance of this investigation program special precautions were taken to insure the validity of results. Measurements were continually made to assure the proper hammer drop, and to accurately monitor the movement of the split spoon sampler. In addition, the depth of the drill during washing was cross-checked with the depth to which the spoon would settle before the standard penetration test. Measurements were taken to insure that the sampler came to rest within at least 0.1 feet of the wash depth. Thus it could be determined if material from the sides of the hole had dropped to the bottom. Each initial boring was made in accordance with ASTM Designation D1586 except that the penetration test values were two foot intervals. The test borings were advanced by washing with drilling fluid to the top of the sample interval, driving an 18 inch standard split-spoon and again washing to the top of the next two foot interval. A side discharge rotary drilling technique was used to advance the hole in order to minimize disturbance to the bottom of the hole. Side and top discharge drag bits were used; these bits cause the stream of drilling fluid to discharge in a direction other than downward through the bit.

A thick bentonite slurry was used as the drilling fluid. The purpose of drilling fluid is (1) to minimize the differential hydrostatic head on the sides and bottom of the bore hole due to groundwater - a condition which could cause collapse of the hole and damage to soils at the points of sampling - and (2) to flush, to the surface, in

suspension, soil particles which have been loosened by the action of the bit. The drilling fluid was sufficiently thick to prevent its penetration into the soil during drilling.

When the drilling rods were removed from the bore hole, care was taken to remove the rods slowly and minimize possible "quick" disturbance at the bottom of the hole. Additionally, care was taken to maintain the fluid level at or above the groundwater level.

During the drilling operation notes of all water losses, changes in strata indicated by erratic drilling, and other special drilling conditions were noted on the Field Log. AW-rod was utilized in connection with the split-barrel sampler to obtain all SPT values, with the exception of one boring where N-rod was used. This variation in rod size has been noted on the appropriate Test Boring Record. In all supplemental holes where SPT values were obtained in conjunction with undisturbed samples, AW rod was used to obtain the SPT values.

Material removed from the split spoon sampler was logged and placed in jars by a Law Engineering Soils Engineer. Jar samples were carefully labeled to indicate bore hole, depth of sampling, blow count and description of material. Upon completion of a hole, jar samples were taken to the on-site Ebasco soils lab for further analysis. At this time, also, the hole was washed and slotted PVC pipe was installed so that the water level could be monitored.

2G2.3.2 Undisturbed Sampling

Osterberg and Piston samplers were used to obtain undisturbed sand and clay samples. Again special precautions were taken to insure the proper depth of sampling and that "good" undisturbed samples were obtained. Once pushed into the soil, the sampling tube was left in the hole for fifteen minutes to allow the sample to stabilize. The tube was then raised a few feet and left for approximately five minutes. As the sample was raised to the surface the drilling mud was kept at a constant high level to insure constant pressure in the hole.

Once removed from the hole, the soil in the tube was carefully trimmed. Measurements were taken from the "edge of tube to soil" and were marked on the tube. This measurement was utilized in two ways:

1) As a check on sample disturbance, i.e., sample movement would be indicated if the "edge of tube to soil" measurement changed in transport to the laboratory.

2) A volume calculation was made to determine the field density of the sample. Subtracting the known weight of tube (taken before sampling) from the gross weight of tube and sample, all information for a field wet density value was available.

Upon completion of measurements and examination of the materials at the ends of the tube, each sample was carefully handled and sealed in the tube with wax. Each tube was labeled to indicate bore hole number, depth of sampling, length of recovery, measurements from edge

of sample tube to soil, bulk densities and any significant detail encountered during sampling. Samples were transported in specially designed wooden boxes lined with styrofoam and placed in a wooden container specially suspended in the back of the Law Engineering Truck. This method of soil transport has been especially designed and approved by the Corps of Engineers on another similar project site study.

Some undisturbed samples were brought to the on-site Ebasco lab for testing.

2G2.3.2.1 Special Sampling Procedures

In numerous supplemental bore holes it was desired to obtain special undisturbed soil samples for laboratory testing. The soils sampled in this manner generally consisted of loose to medium sands. Because of the nature of the soils it was necessary to obtain these undisturbed samples with methods and apparatus more sophisticated than the conventional open-ended, thin walled tube method, to insure better retention of the soil sample. For this sampling, the Osterberg sampler was used.

The Osterberg Piston Sampler consists of an outer pressure cylinder, a thin-walled sampling tube fixed to an upper movable piston, and an inner hollow piston rod to which is attached a fixed lower piston. After the thin-walled sampling tube is seated in the sampler the device is lowered into the drill hole until it comes to rest at the desired sampling interval. The other pressure cylinder remains at a fixed level at the bottom of the drilled borehole during the sampling process.

Water pressure is applied to the upper side of the top piston (attached to the upper end of the movable thin-walled sampling tube) through the drill rods. This fluid pressure forces the thin-walled sampling tube out of the pressure cylinder and down into the soil beneath the bottom of the hole. Air in the sample tube is vented upward through the fixed piston, hollow piston rod, and a ball check valve.

When the thin-walled sampling tube reaches its full stroke (penetrates its full length into the soil) the fixed piston is at or near the top of the sample. Water pressure within the pressure cylinder and above the piston attached to the sampling tube is relieved by permitting circulation through a port in the hollow piston rod and through the ball check valve.

The sampler is then rotated one revolution to shear off the sample at the bottom (the sampling tube is held to the outer pressure cylinder by means of a friction clutch) then allowed to rest and stabilize for 15 minutes before removal. The sample was then carefully handled, trimmed and measured as described in undisturbed sampling above.

2G2.3.3 Additional Field Testing

In order to establish standard penetration test blow counts of the undisturbed samples being taken, the two procedures were alternated in the same hole. An undisturbed sample was taken (i.e. depth 10' - 12'); it was followed by a standard penetration test (depth 12' - 13.5'). The hole would then be washed for six inches and the procedure was repeated for the entire depth of interest. This procedure was performed on six borings. This procedure ensured that the undisturbed material was in fact the lower blow count materials of interest.

In order to gain additional insight to soil conditions in the switchyard area, an extensive soil probing program was undertaken to better define conditions between borings. A twenty foot hollow steel pipe with a jet of water flowing through it was inserted into the ground. The effort necessary to advance the probe was noted. The soil was probed on twenty foot centers (three rows) along the south end of the switchyard. The result of this probing verified the erratic and variable nature of the various materials and thickness.

Two test pits were excavated so that visual inspection of the in-situ soil conditions could be conducted. The test pits were excavated using a back hoe to a depth of approximately twelve feet. One test pit was located along the slope of the intake cooling water canal east of the switchyard while the other was excavated along the proposed emergency cooling water canal north of the switchyard. Both test pits were excavated through lower blow count zones. The sides of the holes, which were approximately twenty feet long, twelve feet deep and three feet wide, were stable and remained nearly vertical throughout the excavation. Only after 15 minutes when water began to flow into the test pit, did the lower portion of the pits begin to slough. This gave an additional "feel" for the quality of the materials within the lower blow count zones being investigated.

2G3.0 LABORATORY TESTING PROCEDURES

2G3.1 General

Various tests were performed on selected samples to evaluate the engineering and physical properties of the subsurface materials. The testing was performed at the Ebasco onsite soils laboratory or at the Law Engineering Testing Companies offices in Atlanta, Georgia; Birmingham, Alabama or Jacksonville, Florida.

2G3.2 Laboratory Tests

Standard classification tests, grain size analyses, hydrometer analyses, moisture contents, atterberg limits, percent organics, unit weights, proctor tests, maximum and minimum density tests were performed. In addition strength tests, such as, unconfined compression tests, consolidated undrained triaxial tests with pore pressure measurements were performed. All test procedures where applicable were performed in accordance with ASTM standards and are summarized on Table 2G3. Figures 2GB1 through 2GB16 present the stress strain plots for individual triaxial tests.

Cyclic Triaxial Shear tests were performed on representative undisturbed samples to determine the cyclic strain potential. The samples were saturated and allowed to consolidate under an isotropic confining pressure equal to approximately the effective overburden pressure. After consolidation was complete, drainage lines were disconnected and a pore pressure transducer was connected to the bottom drainage outlet and into a recorder.

A specified controlled deviator stress was applied cyclically to the specimen at the rate of 1 cycle per second. Each cycle consisted of an alternate axial stress increase and decrease at the constant load amplitude. For each cycle, graphical recordings were made of the axial cyclic load, the pore pressure and sample deformation. Refer to Figures 2GD1 through 2GD10. The samples were cyclically loaded until a double amplitude strain of about 15% or more was reached.

2G3.3 Sample Selection

The undisturbed samples were selected for testing from undisturbed thin wall tube samples from adjacent borings to the main borings, designated by letters A, B, C, etc. Samples were obtained in lower blow count zones of both clay and sand materials. In some cases a standard penetration test was alternated with undisturbed sampling to try to define the looser zones outlied by the main borings and assure that the materials in the thin wall were from the desired zone.

The Osterberg sampler was used to obtain special undisturbed samples of the lower blow count sandy zones outline by the standard penetration test. Unit weight determinations were performed on the undisturbed sand samples in the field since a visual observation of the firm quality of the sample did not correlate with the low blow counts obtained from the standard penetration test or with the observations within the test pits. As shown on Figures 2G19, 2G20 and 2G21 the relative density

obtained from laboratory testing and field densities were considerably higher than that which would be obtained from correlations using Gibbs and Holtz data[2] since the silty and clayey nature of the samples seriously lowers the penetration resistance. The studies and correlation by Gibbs and Holtz were made for clean sands with less than 14% fines and are not directly applicable to these silty and clayey zones. Standard penetration blow counts were therefore discounted in these silty clayey sands and densities were based on the more reliable in-place density determinations, laboratory strength determinations on undisturbed samples, and visual observations.

2G3.4 Undisturbed Sample Tube Opening

The sample tubes were carefully handled. At first an electric continuous band saw was used to attempt to cut the undisturbed samples open. Observations indicated that this method, ordinarily acceptable, was not acceptable for the fine silty sand samples obtained from low blowout zones. A single blade tube cutter was attempted but this tended to make the tube oval. A four blade tube cutter was obtained which evenly cut the tube. A special stand was built to permit careful easy cutting of the tubes with the four blade cutter, thereby minimizing sample disturbance.

2G4.0 Design Parameters

2G4.1 General

The selection of static shear strength and dynamic shear strength design parameters are strongly influenced by the complexity of the soil profile and how samples are selected for testing. The graphical representation of shear strength along with comparisons with literature values are studied along with conservative assumptions made in selecting the design parameters.

2G4.2 Soil Profile

Figures 2G1 and 2G2 present a graphical representation of the complex subsurface profile underlying the switchyard and emergency cooling water canal. From elevation +18 to approximately +2 clean sand materials were encountered. From elevation +2 to approximately elevation -20 silty and clayey sands consisting of medium to fine sand with up to 50% silt and clay. Typical grain size analyses are attached for each boring. From approximately elevation -20 to -30 clay materials were encountered. In some instances the clay was several feet thick while in other places it appeared as 6 inch layers of silty clayey sand to sandy clayey silt intermingled with clay. Beneath the clay layer a silty sand material was encountered to the bottom of the borings at elevation -60, similar in grain size to the sandy material described above. The grain size analyses for each boring wherever possible are plotted on one sheet so that the range of materials encountered may be seen. Refer to Figures 2GC1 to 2GC28.

2G4.3 Sample Selection

The approximate location of most of the 99 undisturbed samples obtained for testing are also shown on Figure 2G1 and 2G2 along with a plot

of standard penetration resistance. Particularly for borings AE 5A,B and C and AE 27 A,B,C,D and E, it is obvious that the undisturbed sandy samples were selected from the lower blow material zones. Refer to Section 2G3.3 for a discussion of blowcounts. The static and dynamic tests on sand materials were performed from these seven borings AE-27A, B,C,D,E and AE-5A,B,C. Similarly for boring AE-1 A and B, AE-2 A and B and AE-4 A the undisturbed clay materials were selected from the lower blow material zones. The tests on clay materials were performed from these four borings.

The selection of design strength parameters therefore incorporates the conservative use of data from the above mentioned lower bound penetration resistance samples.

2G4.4 Shear Strength

2G4.4.1 Static Properties

Figures 2G3 and 2G4 present the Mohr's envelopes for the effective and total stress plots for all the sand samples tested in triaxial shear. All the data is plotted on a single figure for effective stress and again on a single figure for the total stress so that Mohr's envelopes could be selected to cover the range of sand materials. The sieve analyses of all of these 22 tests are presented on Figure 2G5 to show the range of grain size characteristics of sand material tested from medium to fine sand with less than 10% silt to medium to fine sand with up to 50% silt and clay. It was necessary to select typical values; that is, a maximum value representing sand, a minimum value representing silty clayey sand, and an average value representing all ranges of gradations because of the complex subsurface profile as discussed in Section 2G4.2.

Similarly Figures 2G6 and 2G7 present the Mohr's envelopes for the effective and total stress plots for all the clay samples tested in triaxial shear. All the data is plotted on a single figure for effective stress and again on a single figure for the total stress so that Mohr's envelopes could be selected to cover the range of clay materials. The sieve and hydrometer analyses of all of these 16 to 21 tests are presented on Figure 2G8 to show the range of clay material tested from sandy silty clay to sandy clayey silt. As with the sand, it was necessary to select typical values; that is, a maximum value representing clay, a minimum value representing sandy clayey silt and an average value representing all ranges of gradations because of the complex subsurface profile as discussed in Section 2G4.2.

The average values from Figures 2G3, 2G4, 2G6 and 2G7 are selected as the most reasonable values to use in the stability analyses because of the complex nature of sand, silt and clay fractions as discussed above; however, parametric studies were performed using all values: minimum, average and maximum with results presented in Section 2G6.

Mohr's envelopes on each figure were drawn using standard procedures discussed in the literature;[14,15,16] that is, for the sand cases some cohesion is presented for the total stress plot because of the clay fraction

present. The clay Mohr's envelope for the total stress were drawn horizontal. The envelope lines are also appropriately drawn for the effective stress plots. The slight deviation from a straight line of the envelope presented in Figure 2G4, shown as dotted lines, is usually attributed to cohesion and is in part caused by resistance to volume increase. Volume increases which are taking place at failure cause somewhat greater values of shearing strength along the curved portion of the envelopes, whereas volume decreases cause a lowering of the shearing strength along the straight line portions of these envelopes. The minimum value curved portion is drawn similarly to the average value curved portion on Figure 2G4 based on the data from sample AE5B, T-1-B and AE-5B, T-5-A which are samples having high percentages passing the number 200 sieve.

The results presented here-in are further conservative since triaxial tests were used rather than plane strain tests, Mohr's envelope of failure was used rather than the relationship between the shear stress on the failure plane at failure, and the normal stress on the failure plane at the time of consolidation as proposed by Lowe and Karahath and isotropic tests were used rather than anisotropic tests.

2G4.4.2 Dynamic Properties

The data from stress controlled cyclic triaxial tests performed on undisturbed samples of sand from the switchyard area is presented in Table 2G2 and Figure 2GD1 to 2GD10. The field dry density as well as the laboratory dry density prior to saturating the sample are presented in this table. Sieve analyses for all samples are presented on Figure 2G26. The slight variation in densities is due to measurements made for a section of the tube versus the actual tested sample and allowing some of the samples to drain slightly prior to testing to minimize disturbance while setting up the sample. The uncorrected deviator stress is listed in this table.

In order to correctly normalize the deviator stresses to the average and average-minus-one standard deviation relative density, the relative densities for field sample density were determined. Rather than make this normalization based on the test data for the liquefaction samples alone, a study from the total data from as many undisturbed samples as possible was utilized. Because of the limited amount of material available from each tested sample, static or dynamic, for maximum/minimum density determinations and for modified proctor tests, families of all available density characteristics of site soils in the switchyard and canal slopes areas were determined. Eight samples as shown on Figure 2G9 were processed using the entire undisturbed sample from the specified tubes. Field densities, dry minimum densities, dry maximum densities, wet maximum densities and maximum modified proctor densities were determined. As indicated on Figure 2G9, tests for dry and wet maximum densities were performed for several samples. The wet maximum densities were tried on the undisturbed tubes, set in a special holding device on the vibrating table, to try to obtain results on the stratified sandy soils in the tubes since it was felt that mixing the samples and then performing wet maximum density determinations would give misleading results. As can be noticed on Figure 2G9, results for dry and wet maximum vibrated

density are presented for information only since the high percentage of fines invalidates the test procedures, in accordance with the ASTM D2049. Sieves were performed on each sample and are presented in Figure 2G10. Once all this data was available and correlated it was only necessary to obtain unit weights, minimum densities and grain size analyses in order to fit other undisturbed sample densities into a family to establish the maximum density.

As can be seen on Figure 2G9, samples with more than 15% fines have maximum densities governed by the modified proctor test. As can be seen from the majority of grain size analyses presented, the sand material in the switchyard and canal consists mainly of this type of material that is more than 15% silt. Graphs were prepared (Figures 2G11 through 2G18) which can be used to calculate relative density once a material was placed into a proper family. Tables 2G1 and 2G3 presents the data available for the relative density study. As an example: sample AE-27A,T11 with a minimum density of 64.5 pcf and 33 percent passing the number 200 sieve was placed in the AE-27E,T6 family with a minimum density of 64 pcf and 32 percent passing the number 200 sieve and a maximum density of 109.5 pcf. The relative density for sample AE-2A, T11 was then calculated using the 109.5 pcf as the maximum density and a value of 85 percent was obtained. In order to verify the accuracy of this procedure of using grain size shapes, percent passing the 200 sieve and minimum density to fit a sample into a family and select a maximum density, a check was made using combined material obtained from samples AE-27A,T2, AE-27A, T11 and AE-27B, T7. According to the procedure described above, the maximum proctor value should have been 109.5. As a result of performing a modified proctor, the maximum value was 111.2. Since these types of checks involved very little error, the procedure of using grain size shapes, percent passing the 200 sieve and minimum density to fit a sample into a family to calculate relative density was used.

A further verification and validity of the method can be seen if the St. Lucie data is compared with Lee & Fitton's data[8] as shown on Figures 2G19 and 2G20. On Figure 2G19 the data is plotted for the eight family curves while the data on Figure 2G20 is available from the 26 samples as listed in Table 2G1.

For example, sample AE-27E,T6 with a calculated relative density of 89 percent plots on Figure 2G19 at about 83 percent on Lee and Fitton's data.

The data from the field densities of twenty-six samples (Table 2G1 and Figure 2G20) were statistically analyzed to obtain a mean relative density of 73.3 percent and a mean-minus-one standard deviation of 60.8 percent.

Figures 2G22 through 2G25 present the conservative normalized shear stress ratio test data versus number of cycles for 10 and 15% double amplitude strain for the low 60.8% and average 73.3% relative densities. The data was not plotted for 20% strain or used in subsequent liquefaction analyses to be conservative since 20% is often defined as full liquefaction, in lieu of complete strength loss. The strength line was

drawn through the lower bound of the data as a further conservatism. It should also be pointed out that in Table 2G2 for up to 15% double amplitude strain for all the samples tested, pore pressure never equalled confining pressure. This is an indication of the cyclic mobility of silty and clayey sands as opposed to complete pore pressure response of clean loose to medium sands.

The 15% double amplitude strain and 73.3 percent relative density cyclic strength curve, Figure 2G25, was selected as the average value and as a reasonable value to use in the liquefaction analysis; however, parametric studies were performed using all values, 10% and 15% strain and 60.8 and 73.3 percent relative density, with results presented in Section 2G5.

In order to correlate the cyclic test results with others, the St. Lucie data for 73.3% relative density and 15% double amplitude strain is plotted on Lee and Fittons data (Figure 2G27).[8] Plotted on Figure 2G28, which is Seed and Idrisses data,[10] is the St. Lucie data corrected to 50% relative density for 15% double amplitude strain. Excellent agreement is seen on both figures and therefore it can be concluded that the data for this study compares favorably with published literature values and are valid tests results.

2G4.5 Design Parameter Conclusions

All of the above field, laboratory and observational information was utilized in the selection of design parameters. Both the realistic and conservative nature of the parameter selection must be kept in mind for a complete understanding of the analyses presented in the following sections for the switchyard and ultimate heat sink UHS canal. Where appropriate other field or observational information as well as other conservative assumptions are presented for each analysis.

2G5.0 LIQUEFACTION ANALYSES

2G5.1 General

The subsurface conditions underlying the switchyard and canal slopes are presented on Figures 2G1 and 2G2. Since significant saturated sand deposits occur beneath the slopes, this section assesses the potential for liquefaction of the granular soils as a result of shear stresses induced by the DBE.

2G5.2 Analyses

The stress ratio causing cyclic strain (liquefaction) was determined from stress controlled, cyclic loaded triaxial tests conducted on representative samples of the weaker materials encountered at the site. As shown on Figures 2G1 and 2G2 the undisturbed samples were obtained from the lower blowcount zones. Samples from borings AE-27A and B and AE-2A and B were conservatively selected as being representative of lower bound data. As an example Sample AE-27A, T2 was from an area where the standard penetration resistance was 3 blows per foot.

Further conservatism is added by reducing the shear stress ratios from the tests to the mean relative density and the mean relative density minus one standard deviation. The mean value was determined by studying the data on 26 undisturbed samples obtained from lower blowcount zones which is of itself conservative. This data is presented on Figures 2G 21 and Table 2G1. Figures 2G22, 2G23, 2G24 and 2G25 present the plots of the cyclic shear stress ratios versus number of cycles. Lower bound lines were conservatively drawn through the data.[9] Shear stress ratios corresponding to 10 cycles of strong motion were conservatively used in this analysis since no more than 2 to 3 cycles of strong motion can be expected for a DBE of Intensity VI MM.[1] If consideration were given to 3 significant cycles of strong motion, rather than 10 cycles, an increase of 15% to 20% in safety factor would result, even considering the higher average shear stress of the significant strong motion.

The stress ratio at various depths causing 10% and 15% cyclic strain for each case was computed from the following relationship:[13]

$$\tau_1 = \frac{\sigma dp}{2\sigma_a} \left(\sigma_o'\right) \quad Cr$$

where:

τ_1 = In-situ effective shear stress causing cyclic strain

$\frac{\sigma dp}{2\sigma_a}$ = Stress ratio from test program causing cyclic strain

σ_{dp} = Cyclic confining stress

σ_a = Triaxial deviator stress

σ_o' = In-situ effective overburden stress

Cr = A correction factor applied to laboratory triaxial test data to obtain stress conditions causing liquefaction in the field. The "Cr" correction accurately corrects for the effects from laboratory to field conditions by reducing the effective in-situ shear strength.

To assess the liquefaction potential of the site soils, the shear stresses causing cyclic strain were compared to average shear stresses induced by the DBE. The shear stresses[7] induced by the DBE were calculated for site soil column by utilizing the SHAKE Computer Program [11] developed by the University of California at Berkeley. The program utilized the site dependent soil modulus [3,4,5] and damping properties as established from both field shear wave velocities and laboratory tests as given in PSAR Section 2.5.4.4. The elastic properties were iterated to strain compatibility and the peak shear stresses were established at ten foot intervals. [12] Sixty-five percent of the peak shear stresses [6] were utilized as average stresses at 10 equivalent uniform cycles of strong motion.

The average shear stresses were plotted as if the full DBE occurred at the ground surface of the 3 locations presented on Figure 2 -29. This is the case for the ground surface at EL +18 but is conservative for the localized case in the canal at EL -30. The average uniform shear stress induced by the DBE and the shear stress causing liquefaction are shown on Figure 2G29 . The safety factor was calculated by dividing the shear stresses required to cause 10% and 15% cyclic strain by the shear stresses developed during the postulated seismic event.

2G5.3 Liquefaction Analysis Conclusions

The safety factor against cyclic strain (liquefaction) thus calculated assuming average strength condition and 15% strain for a_{max} = 0.1g and 10 equivalent cycles of strong motion are shown on Figure 2G29. Also shown on Figure 2G29 are safety factors for the mean relative density minus one standard deviation and 10% strain. These safety factors again demonstrate that liquefaction would not occur at the switchyard or canal areas under the postulated DBE conditions.

Figure 2G29 outlines the areas analyzed and the respective safety factors for each area; namely, the switchyard, midslope of the canal and the submerged portion of the canal. The analysis in the switchyard and canal slope, even utilizing 10% strain criteria and the mean relative density minus one standard deviation, yields safety factors of 1.3. The bottom canal condition is evaluated at the mean relative density cyclic stress resistance from the lower blow count zones, when in fact the zone below EL-30 is a dense cemented sand zone; additionally the canal condition analyzed uses the surface acceleration of the site at 0.1g when in fact it would be somewhat lower as a local depressed condition on the site.

An evaluation of the actual induced stresses and the cyclic strain potential at 10 cycles could cause local straining or sloughing of one to two feet along the submerged canal slopes. However, there would be no massive sliding or flowing of materials at the stresses induced by the DBE. In fact, a realistic appraisal of the cyclic strain at 2 or 3 cycles, compatible with the Florida area, would limit the actual strains to within 1 to 2 percent, and this results in very local and limited movements on the order of a few inches. This condition would not induce suspended sandy materials or even high turbidity as is conservatively considered in PSAR Section 2.5.5 with the earlier flow slide analysis.

Figure 2G31 outlines the subsurface conditions, stability and liquefaction potential along the canal in the vicinity of the Ultimate Heat Sink Canal Barrier. As can be seen on this figure the barrier construction will involve excavation and compacted backfilling in the vicinity of the barrier wall foundation to EL-14. The minimum safety factor against 10% cyclic strain in the vicinity of the barrier is 1.4. The compacted backfilling of the barrier area has the beneficial effect of confining the foundation in high strength materials, thereby reducing the cyclic mobility of the foundation materials. The evaluation of the actual resisting stresses at the barrier location (soils underlying the barrier) limits the strains induced by the DBE and thereby, establishes a stable foundation for the barrier wall. Safety factors for soils underlying the barrier wall are in excess of those provided on Figure 2G31.

2G6.0 STABILITY ANALYSES

2G6.1 GENERAL

Two representative soil profiles covering all possible soil strata conditions typical of what exists in the switchyard area were analyzed to determine slope stability characteristics. The two typical cross sections were selected from the results of the extensive subsurface investigation program. The location and description of the different soil strata for each section were determined by the use of blow count records, field descriptions and laboratory results.

The two sections studied run east-west through the switchyard and across the intake canal to the intake structure (see Figure 2G30). East-west sections through the intake canal were selected as the critical direction to analyze due to the greatest change in elevation along the slope (+18 to -30). The sections basically consist of a sandy material with a narrow clayey layer at EL - 20'. The difference between the two sections selected is that in one section the clayey layer is continuous across the entire profile as opposed to an assumed discontinuous clayey layer in the second profile.

Two sets of soil shear strength data were used in the analyses: drained soil parameters (Figures 2G4 and 2G7) for the static analyses and undrained soil parameters (Figures 2G3 and 2G6) for the dynamic analyses. A drained state of soil properties would be characteristic of a long term static condition in which any buildup of pore pressures in the soil due to construction is considered to be dissipated. The laboratory tests yielding drained soil strength properties were therefore established to simulate this field condition of normal water level pore pressures. An undrained soil condition is one whereby the pore pressure in the soil has been built up as a result of a quick load application as characterized by the design seismic event. A study of the pore pressures during undrained triaxial testing indicates that the excess pore pressures during static testing average 39% of confining pressure. The excess pore pressures after ten cycles of dynamic loading averaged 35% of confining pressure. This study is summarized on Table 2G4 and established a sound basis for utilizing the total strength parameters for stability at pore pressures compatible with 10 cycles of motion.

Minimum, average and maximum soil strength properties were considered for the two cases in the analyses. The basis for the selection of the minimum, average and maximum soil strength properties in the drained and undrained condition is discussed in Section 2G4 and can be seen from the Mohr-circle plots for the different soil strata (Figures 2G3, 2G4, 2G6 & 2G7). Unit weights used in the analysis for the various soil layers are:

Sandy Material - γSat - 125 PCF
γMoist - 115 PCF
Clayey Material - γSat - 115 PCF

Two methods of analysis, the U.S. Army Corps of Engineers sliding wedge method and the M.I.T. ICES - LEASE slip circle computer program, were used to investigate the stability of the slopes.

2G6.2 SLIDING WEDGE METHOD

The sliding wedge method consists of an active soil wedge being mobilized against a neutral horizontal block and a passive resisting wedge. The factor of safety is calculated as the ratio of the sum of the resisting forces in the horizontal direction to the sum of the driving forces in the horizontal direction. In applying the sliding wedge method to the site conditions, the possible failure planes to be analyzed were selected in a manner to have no passive resisting wedge. This would yield more conservative results than if a passive resisting wedge were considered. The method also includes a seismic loading in the analyses. This was done by including the product of the weights of the active wedge and the neutral block with the horizontal acceleration factor of 0.1g. This force was then considered to act in the direction of the postulated slide as a driving force. The vertical component of the seismic loading is also incorporated into the solution tending to reduce frictional resistance between the sliding wedges. This vertical seismic force is computed as the product of the weights of the neutral block and the active wedge with the vertical acceleration factor of 0.067 g. As can be seen from Figure 2G30 the lowest factors of safety were obtained by the smallest and near surface slope wedges considered.

The method of wedges was used to evaluate the case where the clayey layer appears continuous across the entire profile. This kind of layering system lends itself more to the sliding wedge type failure. Also to be noted is that computations were made on the horizontal neutral block for two cases: one case where the clayey soil strength properties controlled, and a second case where the sandy strength properties controlled. This was done since the clayey zone is not entirely continuous throughout the area.

Computed safety factors are shown in Figure 2G30. The UHS canal section shown on Figure 2G31 was not considered with respect to wedge type of failure due to the minor slope (elevation +5 to elevation -14). This can also be realized on Figure 2G31 by recognizing the high factors of safety for the slip circle analyses.

2G6.3 SLIP-CIRCLE ANALYSES

In performing the slip-circle method of stability analyses, the M.I.T. ICES - LEASE I computer program was used. The method employed by the program, the simplified Bishop approach, is one in which a circular failure surface is assumed to form about its center of rotation. The circle through the slope is then divided into vertical slices and the tangential resisting and driving forces along the circular surface are computed for each slice. The factor of safety against sliding is computed as the ratio of the sum of the resisitng moments taken about the center of rotation to the sum of the driving moments about the same center of rotation.

To use the program the slope geometry must be fully defined on a coordinate grid system along with changes in soil layers. The soil encountered on the slope being analyzed must be fully defined with respect to its saturated unit weight and shear strength. Water level along the slope must also be defined, whether it be in the form of free standing water, groundwater, or pore pressure built up within the soil. Finally, if applicable, the horizontal and vertical components of the design basis earthquake are input.

To find the worst possible radius and center of rotation yielding the circle with the lowest factor of safety, a search routine is built into the program by which a trial center of rotation is selected. The program will investigate different radii from that center of rotation computing and recording the safety factor for each radius. It then moves the center of rotation at a prescribed increment to a different trial location and the above process is repeated until the lowest safety factor is reached.

The simplified Bishop solution yields results that are conservative in that shear resistance between slices, which would tend to raise the factor of safety against sliding, is neglected. When the simplified Bishop solution is used to compute a factor of safety under dynamic loading additional conservatism is built into the program in that the computed safety factor is calculated assuming the components of the design earthquake acceleration act only in one direction, neglecting any back and forth motion, and the magnitude of the acceleration of the design earthquake is taken to be a constant over the entire slope for an infinite length of time.

This approach was used to analyze both sections as well as the UHS canal in the vicinity of the barrier wall. The analysis indicates factors of safety in excess of 1.5 for all cases. See Figure 2G30 and 2G31 for the computed safety factors and the location of the circles with the lowest factors of safety.

2G7.0 SUMMARY AND CONCLUSIONS

The above described detailed investigation has accurately established the soil conditions in the vicinity of the switchyard and ultimate heat sink (emergency) canal. The continuous split-spoon sampling in the initial 26 borings and the careful undisturbed sampling of 99 thin wall tubes in the additional 25 borings in the weaker zones establishes a sound basis for the selection of lower bound strength samples. The detail and care of undisturbed sampling and in-situ density determinations assures valid density correlations for the dynamic testing. The selection of design strength parameters incorporates the use of the average strength properties from the above mentioned lower bound samples selected and tested. Additionally, a parametric study of higher and lower strength parameters is presented, which indicates stable conditions even with the lowest strength parameter of an already lower bound biased sampling field. The liquefaction analysis is also considered and presented in a lower bound approach in that the actual measured cyclic strengths have been reduced (normalized) to represent the strength at a relative density of one standard deviation below the already lower bound average obtained from the samples selected in the field. This analysis indicates that the switchyard area and canal slopes are stable even for 10 cycles of motion associated with the DBE at the lower bound cyclic strength.

In conclusion, the study indicates that the switchyard area and emergency canal area slopes are stable with respect to large scale movements during the DBE The analysis indicates that potential for local sloughing of the slopes exist as evidenced by the strains associated with the cyclic mobility during testing. It is concluded that under actual DBE conditions and 10 equivalent cycles of strong motion, the near surface of the submerged slopes in the areas studied could slough as much as 1 to 2 feet;

however, massive sliding will not occur or hinder the operation of the intake forebay or canals. A more realistic appraisal of an actual earthquake for the Florida area would limit the cycles of strong motion to 2 to 3 cycles, [1] and thereby limit the resulting strains to yeild movement within a few inches. The UHS barrier wall foundation and compacted backfill in the area are stable under DBE conditions.

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TABLE 2G-1

ST. LUCIE
RELATIVE DENSITY STUDY

Sample & Hole	Field Unit Wgt (Dry. PCF)	Void Ratio	D_{50} (mm)	% Passing 200	Minimum Density (PCF)	Family	Relative Density (%)
AE 1B T11	103.5	.60	.13	30	70.3	AE 1B T11	74
AE 5C T1	91.2	.81	.10	14.5	70.6	AE 5C T1	74
AE 5C T3	91.8	.80	.085	29.1	60.2	AE 5C T3	75
AE 27D T2	88.2	.88	.085	21.8	71.0	AE 27D T2	55
AE 27D T6	102.7	.61	.10	25.8	70.2	AE 27D T6	87
AE 27E T1	83.8	.97	.105	22.8	71.5	AE 27E T1	42
AE 27E T5	97.1	.70	.080	26.5	70.6	AE 27E T5	97
AE 27E T6	101.3	.63	.074	32	64.0	AE 27E T6	89
AE 27A T2	89.8	.84	.09	35	62.2	AE 27E T6	70
AE 27A T2	89.8	.84	.09	35	62.2	AE 27E T6	70
AE 27A T2	87.7	.88	.087	20	61.7	AE 27E T6	65
AE 27A T8	95.7	.73	.10	22	70.4	AE 27E T5	77
AE 27A T11	99.3	.66	.074	33	64.5	AE 27E T6	85
AE 27A T8	99.1	.67	.13	23	70.9	AE 27E T5	85
AE 27B T5	97.6	.69	.10	19	69.8	AE 5C T1	91
AE 27B T7	97	.70	.09	40	66.0	AE 27E T6	82
AE 2B UD3	93	.78	.075	33	60.2	AE 5C T3	82
AE 2C UD2	92.7	.78	.13	12	68.3	AE 5C T1	79
AE 2C UD1	91.5	.81	.10	16	71.5	AE 5C T1	75
AE 27C T8	90.1	.83	.09	36	61.1	AE 27E T6	70
AE 5B T1	90.8	.82	.10	24	69.0	AE 1B T11	53
AE 5B T3	94.6	.75	.08	31	64.4	AE 27E T6	77
AE 5B T4	93.4	.77	.074	38	61.5	AE 27E T6	75
AE 5B T5	93.0	.78	.12	12	73.1	AE 5C T1	79
AE 5B T6	90.4	.82	.095	14	71.2	AE 5C T1	72
AE 5B T8	112.8	.47	.01	31	66.3	AE 27E T6	90

TABLE 2G-2

CYCLIC TRIAXIAL TEST DATA
FOR ST. LUCIE

Test No.	Boring No.	Sample No.	Field Dry Unit Wt. (PCF)	Lab. Dry Unit Wt. (PCF)	Moisture Content Field (%)	Moisture Content Lab (%)	Minimum Density (PCF)	Percent Passing #200 Sieve	D_{50} (MM)
1	AE-27A	T-2	89.8	87.3	32.8	34.4	62.2	35	0.09
2	AE-27A	T-2	87.7	86.1	34.2	31.3	61.7	20	0.087
3	AE-27A	T-8	95.7	93.8	28.0	24.9	70.4	22	0.10
4	AE-27A	T-11	99.3	98.0	26.7	26.7	64.5	33	0.074
5	AE-27A	T-8	99.1	93.9	26.2	27.1	70.9	23	0.13
6	AE-27B	T-5	97.6	95.2	27.6	28.0	69.8	19	0.10
7	AE-27B	T-7	97.0	97.7	28.9	27.8	66.0	40	0.09
8	AE-2B	UD-3	93.0	90.8	28.1	30.2	60.2	33	0.074
9	AE-2C	UD-1	91.5	91.3	32.6	29.5	71.5	16	0.10
10	AE-27C	T-8	90.1	88.3	–	31.7	61.1	36	0.074

Test No.	Deviator Stress (PCF)	Confining Pressure (PSI)	Back Pressure (PSI)	"B" Value (PSI)	No. of Cycles to Double Amplitude Strain 2%	5%	10%	15%	Max Pore Pressure
1	1029	2500	–	–	75	80	88	96	10.0
2	1548	2500	60	.99	2	3	5	8	14.0
3	1520	2500	60	.98	4	7	16	30	14.0
4	1244	2500	60	.99	1	4	12	28	17.4 @ 40 cycles
5	1714	2500	65	.99	3	8	18	47	15.2
6	2046	2500	40	.98	2	7	20	56	9.6
7	1770	2500	40	.98	1	3	8	14	9.0
8	1123	2500	60	.98	1	3	6	10	12.5
9	2336	2500	30	.97	2	7	22	48	12.3
10	2295	2500	61	.98	–	–	–	1	10.0

Table 2G-3

SUMMARY OF PHYSICAL PROPERTIES TESTS

Boring No.	Sample No.	Depth	Sample Type	Sample Description	Dry Unit Weight	Moisture Content - %	% Passing 200 Sieve	D_{50} (mm)	Loss on Ignition	Atterburg Limits LL	PL	PI	Min Density	Max Density Undisturbed	Max Density Disturbed Dry	Disturbed Wet	Proctor Density	Proctor Moisture	In Place Dd
AE-1A	UD-1	32/32.9	P																
	UD-2	34/36	P	A) Silt and Clay, Little Fine Sand	65.4	61.0	85		5.33	98	29	69							
				B) Silt, Some Clay, Some Fine Sand	74.5	53.0	73		3.95	75	25	50							
				C) Some Silt, Some Clay, Some Fine Sand	69.5	65.5	63		3.43	65	23	42							
AE-1B	T-2	12/13	O	Fine Sand, Little Silt	105.0		17.8												
	T-4	16/17	O																
	T-7	36/37.5	P	Sample Disturbed			52.9												
	T-9	40/42	O		94.9		52.0												
	T-11	44/46	O	Fine Sand, Some Silt	103.5		30	0.13					70.3		92.9		123.7		74
AE-2A	UD-1	31.5/32.5	P	Drilling Mud															
	UD-2	34/36	P	A) Silty Clay, Little Fine Sand	62.0	65.0	82.0		13.4	71	27	44							
				B) Silty Clay, Little Fine Sand	73.2	46.6	80.0		16.0	52	20	32							
				C) Silty Clay, Little Fine Sand	60.6	59.3	90.0		15.8	74	29	45							
	UD-3	36/36.5	P																
AE-2B	UD-2	12.5/14.5	P	Fine Sand and Silt	81.5	42.8	44	0.075					-						
	UD-3	14.5/15.5	P	Fine Sand and Silt	93.0	28.1	33	0.075					60.2						82
AE-2C	UD-1	9.3/10.3	P	Fine Sand, Little Silt, Shell Fragments	91.5	32.6	16	0.10					71.5						75
	UD-2	11.3/12.1	P	A) Fine Sand, Little Silt, Shell Fragments	92.7	28.7	13	0.12					68.3						79
				B) Fine Sand, Trace Silt, Shell Fragments	92.7	28.2	4	0.19					70.2						
	UD-3	13.1/14.5	P																
	UD-4	32.5/34.5	P	A) Clayey Silt, Trace Fine Sand	62.4	60.0	93		16.5	40	16	24							
				B) Clayey Silt, Some Fine Sand, Shell Fragments	62.7	55.0	78		15.7	74	35	39							
				C) Clay, Little Sand, Shell Fragments	71.4	48.1	86		13.4	51	21	30							
	UD-5	34.8/36.8	P	A) Clayey Silt, and Fine Sand	78.2	41.7	55		10.8	48	21	27							
				B) Silty Clay, Some Fine Sand, Shell Fragments	75.7	41.0	64		12.6	47	22	25							
				C) Silty Clay, and Medium Fine Sand, Shell Fragments	-	44.3	53			45	15	30							
AE-3A	T-1	34/36	P																
	T-2	36/38	P																
	T-3	38/40	P	A) Medium to Fine Sand, Some Clayey Silt, Shell Fragments	99.2	24.6	25	0.2	1.42	23	21	2							
				B) Medium to Fine Sand, Some Clayey Silt	101.2	24.4	20	0.2	1.45	21	19	2							
				C) Medium to Fine Sand, Some Clayey Silt	108.5	21.7	26	0.2	1.16	20	17	3							
AE-4A	T-1	12/14	P	Sample Disturbed															
	T-2	16/18	P	Sample Disturbed															
	T-3	24/26	P	Fine Sand, and Silt	101.6		38.4						73.6		90.4	90.5	114.5		
	T-4	32/34	P	Sample Disturbed															
	T-6	38/40	P	A) Silty Clay, Little Fine Sand	71.9	50	89.0	-	4.5	93	29	64							
				B) Silt and Clay, Some Fine Sand	63.9	77	67.0	-	3.8	83	28	55							
				C) Silt and Clay, Some Fine Sand	72.4	60	75.0	-	3.8	92	29	63							
	T-7	40/42	P	Sample Disturbed															
	T-8	42/44	P	Sample Disturbed															
	T-9	58/60	P	Sample Disturbed															
	T-10	64/66	P		110.8		13.6						89.5		112.5	112.5	123.1		
AE-5A	UD-1	27.4/28.4	P	A) Medium to Fine Sand, Little Silt, Trace Gravel	85.2	32.6	16.9	0.10											
				B) Silt, Some Fine Sand, Some Clay	85.9	41.4	74.9	0.0022		28	21	7							
				C) Silt, Clay and Sand	85.2	32.8	59.6	0.03		27	23	4							
	UD-2	30/32.4	P	A) Medium to Fine Sand, Trace Silt, Trace Clay, Shell Fragments	91.5	26.7	15.7	0.10											
				B) Medium to Fine Sand, Some Silt, Trace Gravel, Shell Fragments	88.0	26.8	32.6	0.10											
				C) Coarse to Fine Sand, Trace Gravel, Little Clayey Silt, Shell Fragments	87.1	27.9	37.6	0.10											
	UD-3	35/37.4	P	A) Medium to Fine Sand, Some Silt		25.2	25	0.15	1.01	24	21	3							
				B) Medium to Fine Sand, Some Silt		23.7	22	0.25	0.80	24	23	1							
				C) Medium to Fine Sand, Some Silt		26.6	25	0.09	0.81	36	19	17							

2G-23

Table 2G-3 (Cont'd)

SUMMARY OF PHYSICAL PROPERTIES TESTS

Boring No.	Sample No.	Depth	Sample Type	Sample Description	Dry Unit Weight	Moisture Content - %	% Passing 200 Sieve	D50 (mm)	Loss on Ignition	Atterburg Limits LL	PL	PI	Min Density	Max Density Undisturbed	Max Density Disturbed Dry	Max Density Disturbed Wet	Proctor Density	Proctor Moisture	In Place Dd
AE-5B	T-1	10/12	O	A) Medium to Fine Sand, Little Silt	90.8	28.9	15.8	0.12											53
				B) Fine Sand, Some Silt	90.8	43.2	32.2	0.08											
	T-2	12/14	O	A) Medium to Fine Sand, Little Silt, Trace Gravel	92.8	33.2	19.5	0.10											
				B) Fine Sand, Trace Silt	92.8	24.5	9.0	0.16											
	T-3	14/16	O	A) Fine Sand, Some Silt	94.6	33.8	28.6	0.08											77
				B) Fine Sand, Some Silt	94.6	35.0	32.5	0.08											
	T-4	16/18	O	A) Fine Sand and Silt	93.4	37.3	40.1	0.075											75
				B) Fine Sand and Silt	93.4	37.6	35.6	0.08											
	T-5	28/30	O	A) Medium to Fine Sand, Little Silt	93.0	29.6	11.8	0.12											79
				B) Medium to Fine Sand, Little Silt, Trace Gravel	93.0	29.8	11.0	0.10											
	T-6	30/32	O	A) Medium to Fine Sand, Little Silt	90.4	29.7	13.1	0.11											72
				B) Medium to Fine Sand, Little Silt, Trace Gravel	90.4	35.1	15.4	0.084											
	T-7	32/34	O	A) Medium to Fine Sand, Trace Silt	103.6	28.4	4.9	0.15											
				B) Medium to Fine Sand, Trace Silt	103.6	29.5	4.6	0.15											
	T-8	34/36	O	A) Medium to Fine Sand, Some Silt	112.8	23.8	27.3	0.09											
				B) Medium to Fine Sand, Some Silt	112.8	22.9	34.1	0.10											90
AE-5C	T-1	10/12	O	Fine Sand, Little Silt	91.2		14.5	0.10					70.6		92.7	79.8	101.3		74
	T-3	14/15	O	Fine Sand, Some Silt	91.8		29.1	0.85					60.2		78.8	95.8	111.4		75
AE-16A	UD-1	20/21.5	P																
	UD-2	26/25.5	P																
	UD-3	28/29	P																
AE-16B	UD-3	26/27.9	P																
AE-22A	UD-1	38/39	P	Sample Disturbed															
	UD-2	41/42	P	Sample Disturbed															
AE-27A	T-2	12/14	O	Fine Sand, Some Silt, Shell Fragments	89.8	32.8	35	0.09					62.2						70
				Fine Sand, Little Silt, Shell Fragments	87.7	34.2	20	0.087					61.7						65
	T-8	20/22	O	Fine Sand, Some Silt, Shell Fragments	95.7	28	22	0.10					70.4						77
				Fine Sand, Some Silt	99.1	26.2	23	0.13					70.9						85
	T-11	24/26	O	Fine Sand, Some Silt	99.3	26.7	33	0.074					64.5						85
AE-27B	T-1	11.5/13.5	O	Sample Disturbed															
	T-3	15/17	O																
	T-5	19/21	O	Fine Sand, Little Silt	97.6	27.8	19	0.10					69.8						91
	T-7	23/25	O	Fine Sand, and Silt	97.0	28.9	40	0.09					66.0						82
AE-27C	T-1	10/12	O	Sample Disturbed															
	T-2	12/14	O	Sample Disturbed															
	T-3	14/16	O	Sample Disturbed															
	T-6	20/22	O	Sample Disturbed															
	T-7	22/24	O	Sample Disturbed															
	T-8	24/26	O	Fine Sand, and Silt	90.1		36.0	0.09					61.1						70
	T-11	40/42	O	Sample Disturbed															
	T-12	42/44	O	Sample Disturbed															
AE-27D	T-2	12/13	O	Fine Sand, Some Silt	88.2		21.8	0.085											55
	T-4	16/1[illegible]	O	Sample Disturbed			8.9												
	T-6	20/21	O	Fine Sand, Some Silt	102.7		25.8	0.10					70.2						87
	T-8	24/26	O				30.9												
AE-27E	T-1	10/11.3	O	Fine Sand, Some Silt	83.8		22.8	0.105					71.5						42
	T-3	14/16	O	Silt, Some Fine Sand	80.0		66.7												
	T-5	18/20	O	Fine Sand, Some Silt	97.1		26.5	0.080					70.6		91.6	102.2	106.6		97
	T-6	22/24	O	Fine Sand, Some Silt	101.3		12	.074					64.0		81.5		109.5		89
	T-8	26/2[illegible]	O																

TABLE 2G-4

ST LUCIE

SILTY SANDY MATERIAL

STATIC AND DYNAMIC TEST PORE PRESSURES

STATIC TESTS

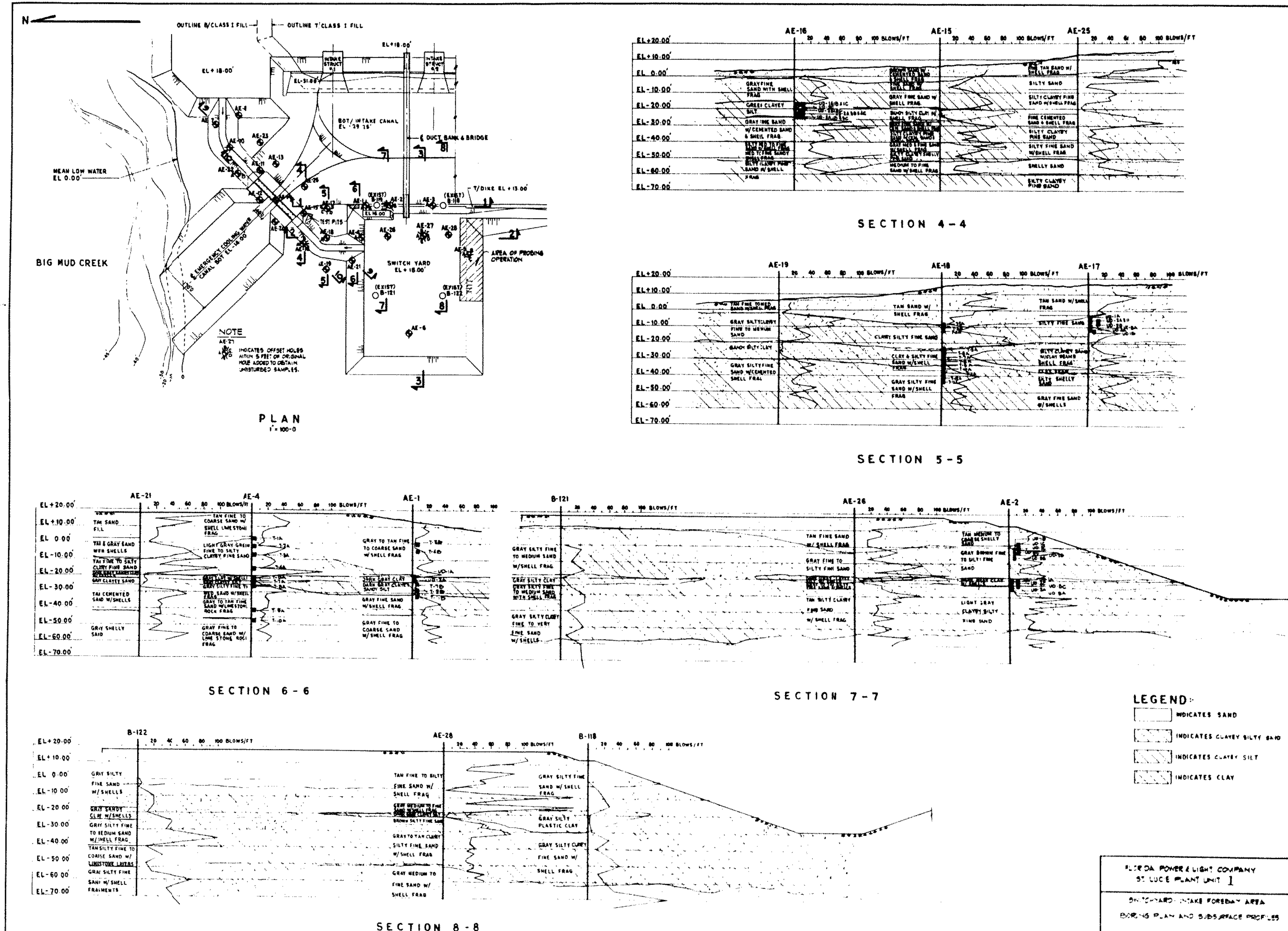
BORING	SAMPLE	DEPTH	CONFINING PRESSURE $\sigma 3$ - (KSF)	PORE PRESSURE U - (KSF)	$\frac{U}{\sigma 3}$ (%)
AE - 5A	UD-1A	27.4' - 29.9'	2.0	1.0	50
AE - 5A	UD-1B	27.4' - 29.9'	3.0	1.3	43
AE - 5B	T-2A	12.0' - 14.0'	1.25	0.25	20
AE - 5B	T-4A	16.0' - 18.0'	2.25	1.0	44
AE - 5B	T-4B	16.0' - 18.0'	2.5	1.2	48
AE - 5B	T-5A	28.0' - 30.0'	2.5	1.0	40
AE - 5B	T-6A	30.0' - 32.0'	3.25	0.1	3
AE - 5B	T-7A	32.0' - 34.0'	3.5	1.7	49
AE - 5B	T-7B	32.0' - 34.0'	4.0	1.6	40
AE - 5B	T-8A	34.0' - 36.0'	4.25	1.85	44
AE - 5B	T-8B	34.0 - 36.0'	4.5	2.3	51
				Ave = 1.21 KSF	Ave = 39.3%

DYNAMIC TESTS

BORING	SAMPLE	DEPTH	CONFINING PRESSURE $\sigma 3$ - (KSF)	PORE PRESSURE - U (KSF) EXT.	COMP.	AVE.	$\frac{U \text{ ave.}}{\sigma 3}$ (%)
AE - 2B	UD-3	14.5' - 15.5'	2.5	+0.65	+1.80	1.23	49.0
AE - 2C	UD-1	9.5' - 11.5'	2.5	-0.79	+1.53	0.37	14.8
AE - 27A	T-2B	12.0' - 14.0'	2.5	+0.72	+2.07	1.40	55.9
AE - 27A	T-8A	20.0' - 22.0'	2.5	+0.29	+2.04	1.17	46.7
AE - 27A	T-8B	20.0' - 22.0'	2.5	+0.29	+2.01	1.15	46.1
AE - 27B	T-5A	19.0' - 21.0'	2.5	-0.82	+1.37	0.27	11.0
AE - 27B	T-7	23.0' - 25.0'	2.5	0	+1.15	0.58	23.0
						Ave = 0.88 KSF	Ave = 35.2%

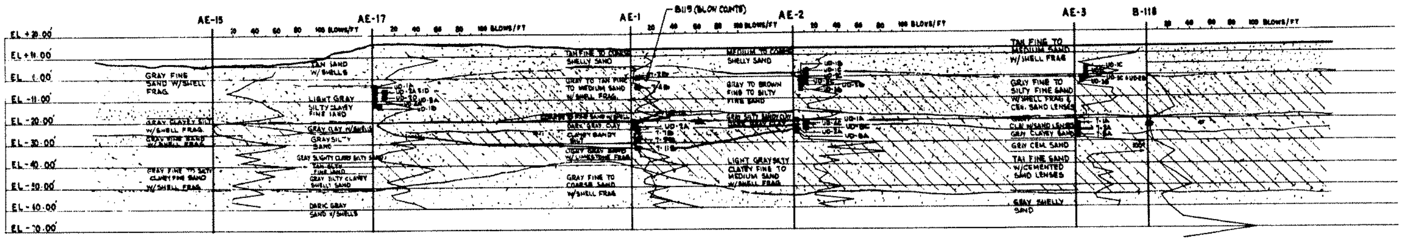
Average excess pore pressure (%) static = 39.3%

Average excess pore pressure (%) dynamic = 35.2%

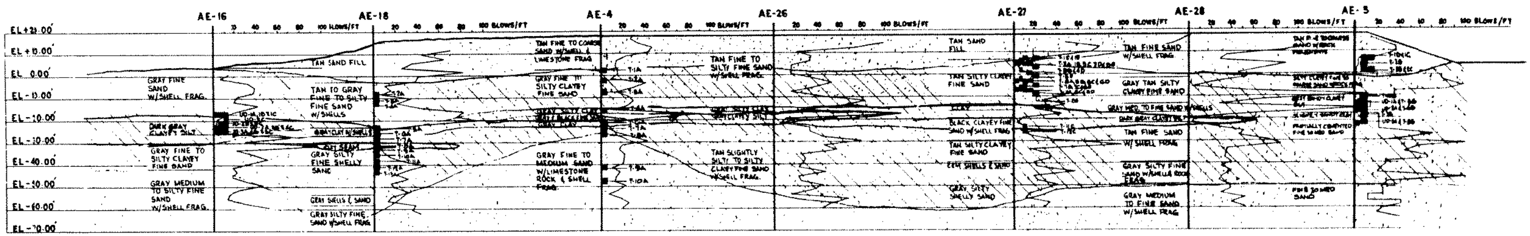


N
OUTLINE B/CLASS I FILL
OUTLINE T/CLASS I FILL
EL + 18.00'
INTAKE STRUCT
BOT/ INTAKE CANAL
DUCT BANK & BRIDGE
MEAN LOW WATER EL 0.00'
T/DIKE EL + 13.00
TEST PITS
SWITCH YARD EL + 18.00'
AREA OF PROBING OPERATION
BIG MUD CREEK
EMERGENCY COOLING WATER CANAL
NOTE
INDICATES OFFSET HOLES WITHIN 5 FEET OF ORIGINAL HOLE ADDED TO OBTAIN UNDISTURBED SAMPLES.
PLAN
1" = 100-0
SECTION 4-4
SECTION 5-5
SECTION 6-6
SECTION 7-7
SECTION 8-8
LEGEND:
INDICATES SAND
INDICATES CLAYEY SILTY SAND
INDICATES CLAYEY SILT
INDICATES CLAY
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1
SWITCHYARD - INTAKE FOREBAY AREA
BORING PLAN AND SUBSURFACE PROFILES

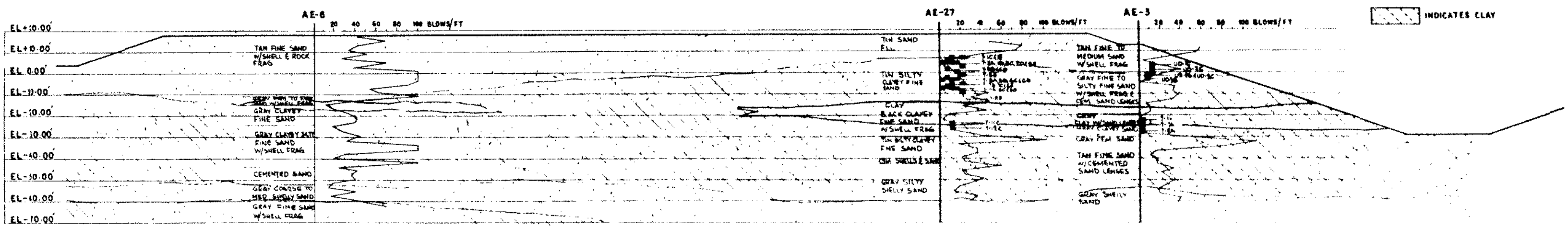
FIGURE 2G-1



SECTION 1-1



SECTION 2-2



SECTION 3-3

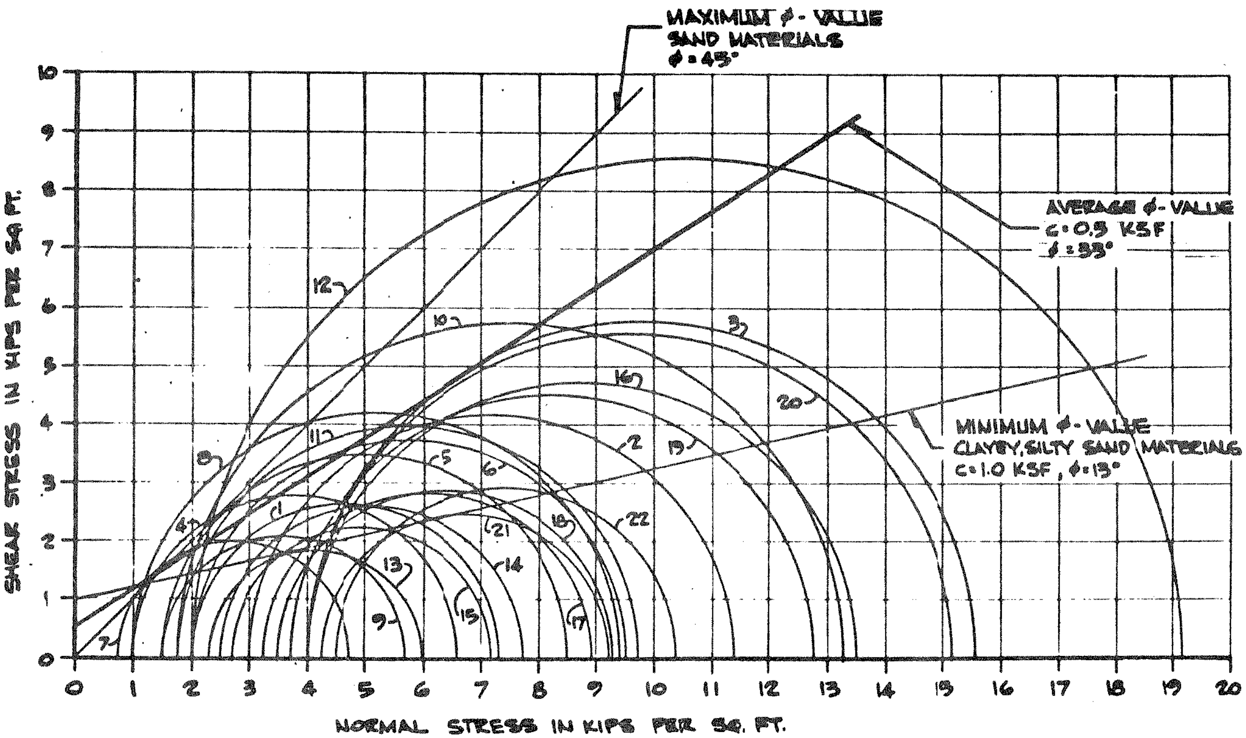
LEGEND:

- INDICATES SAND
- INDICATES CLAYEY SILTY SAND
- INDICATES CLAYEY SILT
- INDICATES CLAY

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SWITCHYARD-INTAKE FOREBAY AREA
SUBSURFACE PROFILES

FIGURE 2G-2



NO	BORING NO	SAMPLE NO	DEPTH	TEST
1	AE-5A	UD-1-A	27.4-29.9	CU/PP
2	AE-5A	UD-1-B	27.4-29.9	CU/PP
3	AE-5A	UD-1-C	27.4-29.9	CU/PP
4*	AE-5A	UD-2-A	30.0-32.4	CU/PP
5*	AE-5A	UD-2-B	30.0-32.4	CU/PP
6*	AE-5A	UD-2-C	30.0-32.4	CU/PP
7*	AE-5B	T-1-A	10.0-12.0	CU/PP
8*	AE-5B	T-1-B	10.0-12.0	CU/PP
9	AE-5B	T-2-A	12.0-14.0	CU/PP
10*	AE-5B	T-2-B	12.0-14.0	CU/PP
11*	AE-5B	T-3-A	14.0-16.0	CU/PP
12*	AE-5B	T-3-B	14.0-16.0	CU/PP
13	AE-5B	T-4-A	16.0-18.0	CU/PP
14	AE-5B	T-4-B	16.0-18.0	CU/PP
15	AE-5B	T-5-A	28.0-30.0	CU/PP
16*	AE-5B	T-5-B	28.0-30.0	CU/PP
17	AE-5B	T-6-A	30.0-32.0	CU/PP
18*	AE-5B	T-6-B	30.0-32.0	CU/PP
19	AE-5B	T-7-A	32.0-34.0	CU/PP
20	AE-5B	T-7-B	32.0-34.0	CU/PP
21	AE-5B	T-8-A	34.0-36.0	CU/PP
22	AE-5B	T-8-B	34.0-36.0	CU/PP

* SAMPLES DILATED DURING TEST.

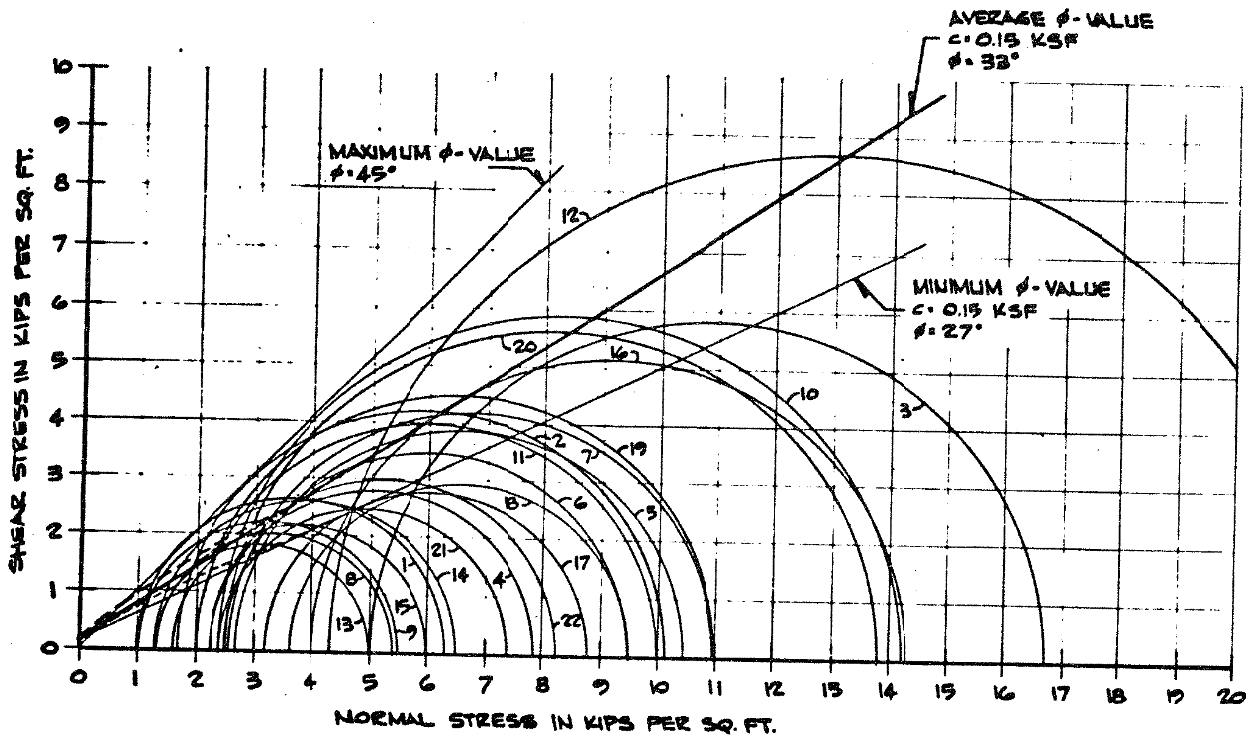
TOTAL STRESS ENVELOPES

SANDY ZONE MATERIALS

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

STRENGTH PARAMETERS
SANDY MATERIALS
TOTAL STRESS

FIGURE 2G-3



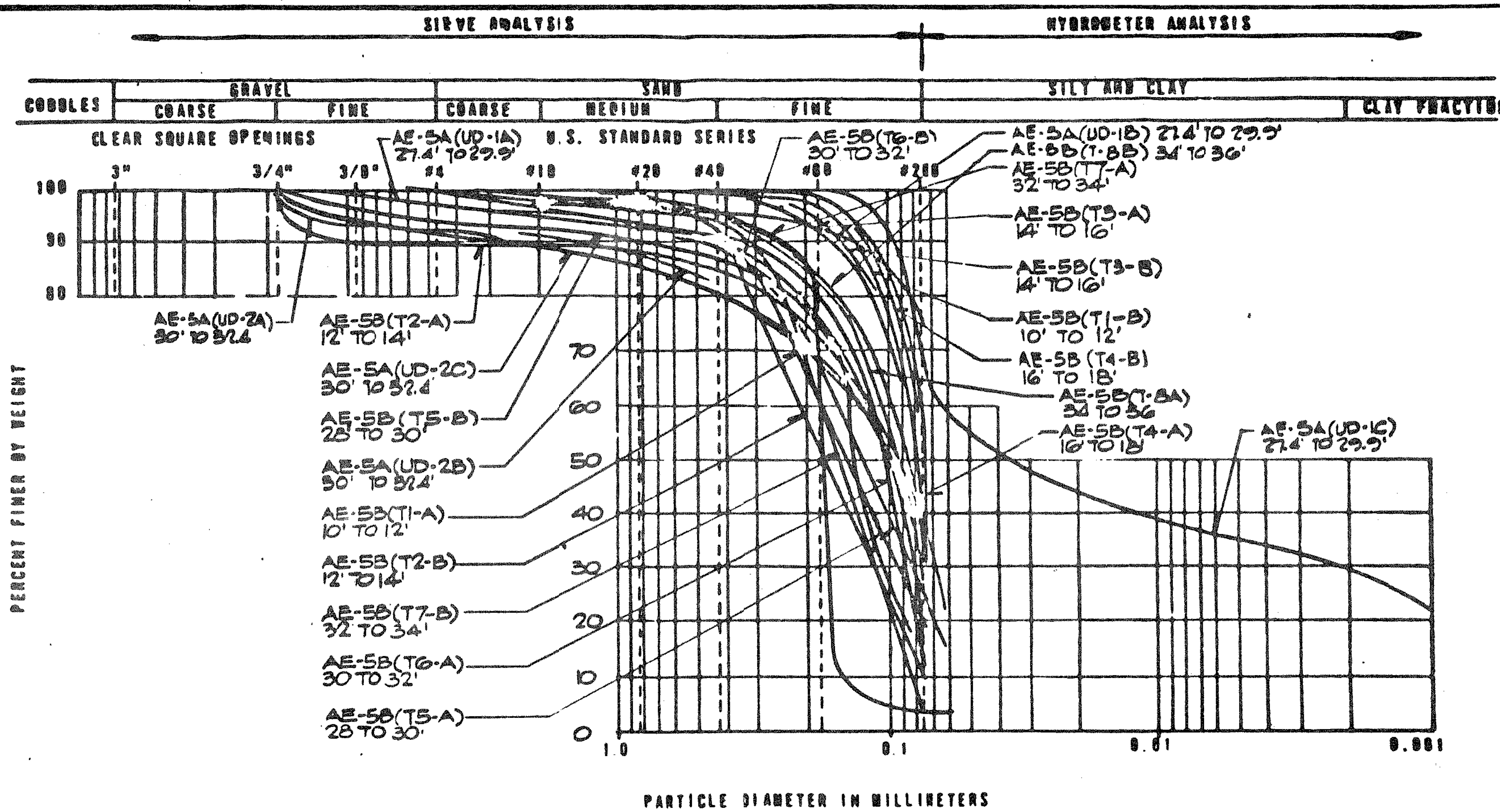
NO	BORING NO	SAMPLE NO	DEPTH	TEST
1	AE-5A	UD-1-A	27.4-29.9	CU/PP
2	AE-5A	UD-1-B	27.4-29.9	CU/PP
3	AE-5A	UD-1-C	27.4-29.9	CU/PP
4*	AE-5A	UD-2-A	30.0-32.4	CU/PP
5*	AE-5A	UD-2-B	30.0-32.4	CU/PP
6*	AE-5A	UD-2-C	30.0-32.4	CU/PP
7*	AE-5B	T-1-A	10.0-12.0	CU/PP
8*	AE-5B	T-1-B	10.0-12.0	CU/PP
9	AE-5B	T-2-A	12.0-14.0	CU/PP
10*	AE-5B	T-2-B	12.0-14.0	CU/PP
11*	AE-5B	T-3-A	14.0-16.0	CU/PP
12*	AE-5B	T-3-B	14.0-16.0	CU/PP
13	AE-5B	T-4-A	16.0-18.0	CU/PP
14	AE-5B	T-4-B	16.0-18.0	CU/PP
15	AE-5B	T-5-A	28.0-30.0	CU/PP
16*	AE-5B	T-5-B	28.0-30.0	CU/PP
17	AE-5B	T-6-A	30.0-32.0	CU/PP
18*	AE-5B	T-6-B	30.0-32.0	CU/PP
19	AE-5B	T-7-A	32.0-34.0	CU/PP
20	AE-5B	T-7-B	32.0-34.0	CU/PP
21	AE-5B	T-8-A	34.0-36.0	CU/PP
22	AE-5B	T-8-B	34.0-36.0	CU/PP

* SAMPLES DILATED DURING TEST.

EFFECTIVE STRESS ENVELOPES
SANDY ZONE MATERIALS

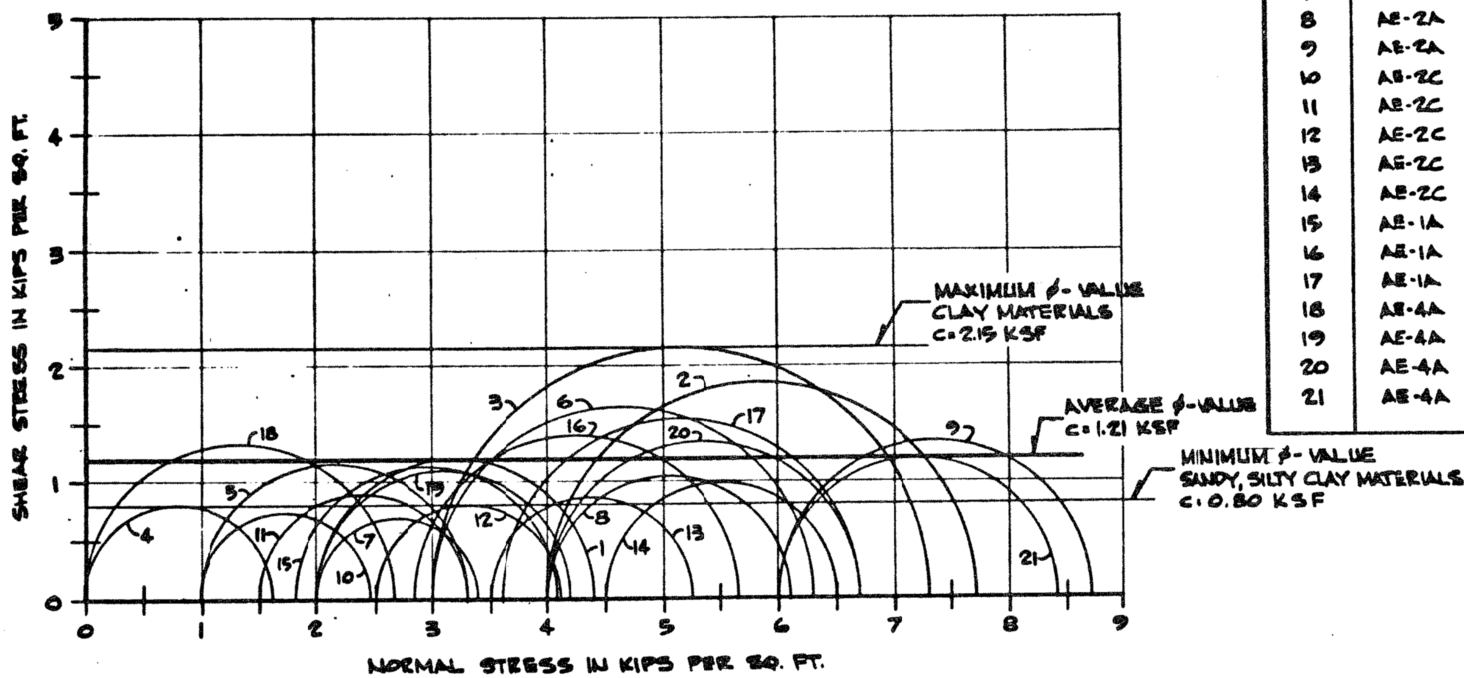
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

STRENGTH PARAMETERS
SANDY MATERIALS
EFFECTIVE STRESS
FIGURE 2G-4



SANDY ZONE MATERIALS

FLORIDA POWER & LIGHT COMPANY ST. LUCIE PLANT UNIT 1
GRAIN SIZE ANALYSES FOR SAND MATERIALS TESTED IN TRIAXIAL SHEAR FIGURE 2G-5



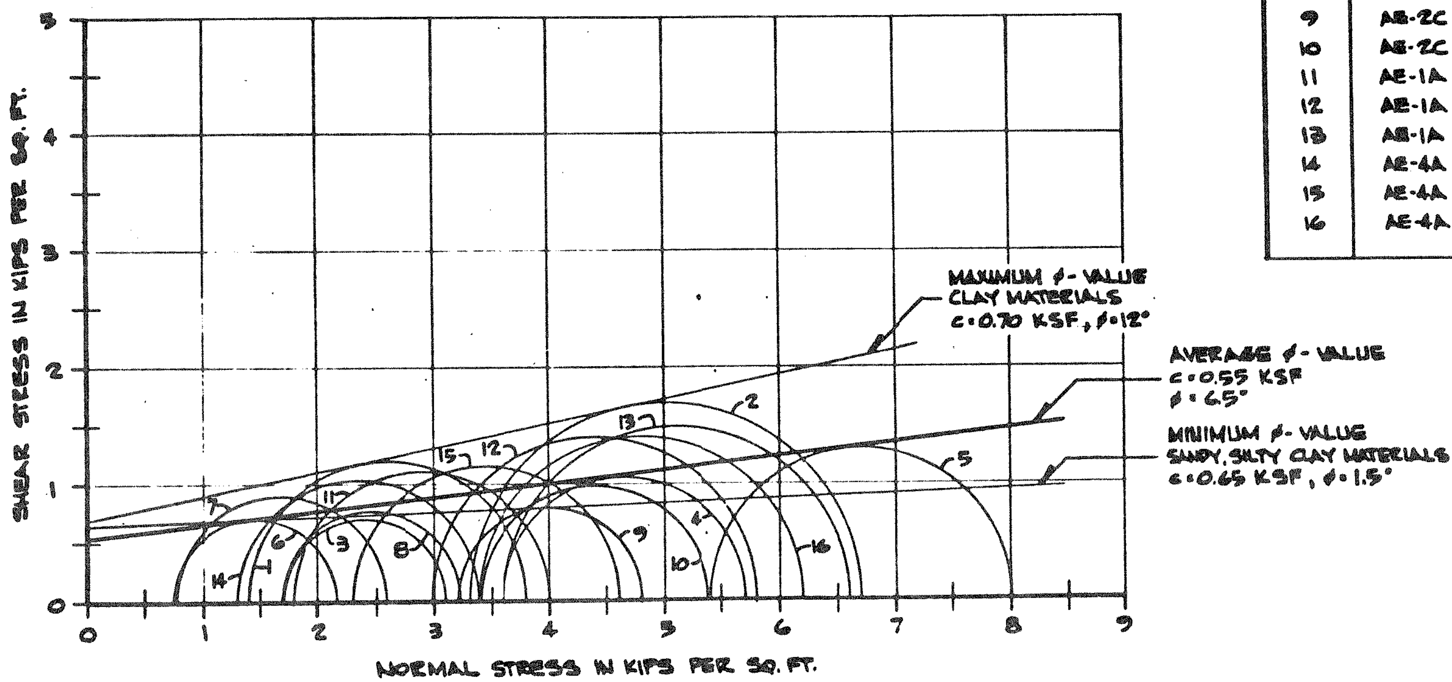
NO	BORING NO	SAMPLE NO	DEPTH	TEST
1	B-109	UD-A	37.0	CU/PP
2	B-109	UD-B	37.0	CU/PP
3	B-106	UD	31.5	UU
4	B-109	UD-A	37.0	UC
5	B-109	UD-B	37.0	UU
6	B-109	UD-C	37.0	UU
7	AE-2A	UD-2-A	34.0-36.0	CU/PP
8	AE-2A	UD-2-B	34.0-36.0	CU/PP
9	AE-2A	UD-2-C	34.0-36.0	CU/PP
10	AE-2C	UD-5-A	34.8-36.8	CU/PP
11	AE-2C	UD-5-B	34.8-36.8	CU/PP
12	AE-2C	UD-4-A	32.5-34.5	CU/PP
13	AE-2C	UD-4-B	32.5-34.5	CU/PP
14	AE-2C	UD-4-C	32.5-34.5	CU/PP
15	AE-1A	UD-2-A	34.0-36.0	CU/PP
16	AE-1A	UD-2-B	34.0-36.0	CU/PP
17	AE-1A	UD-2-C	34.0-36.0	CU/PP
18	AE-4A	T6	38.0-40.0	UC
19	AE-4A	T-6-A	38.0-40.0	CU/PP
20	AE-4A	T-6-B	38.0-40.0	CU/PP
21	AE-4A	T-6-C	38.0-40.0	CU/PP

TOTAL STRESS ENVELOPES
CLAYEY ZONE MATERIALS

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

STRENGTH PARAMETERS
CLAYEY MATERIALS
TOTAL STRESS
FIGURE 2G-6

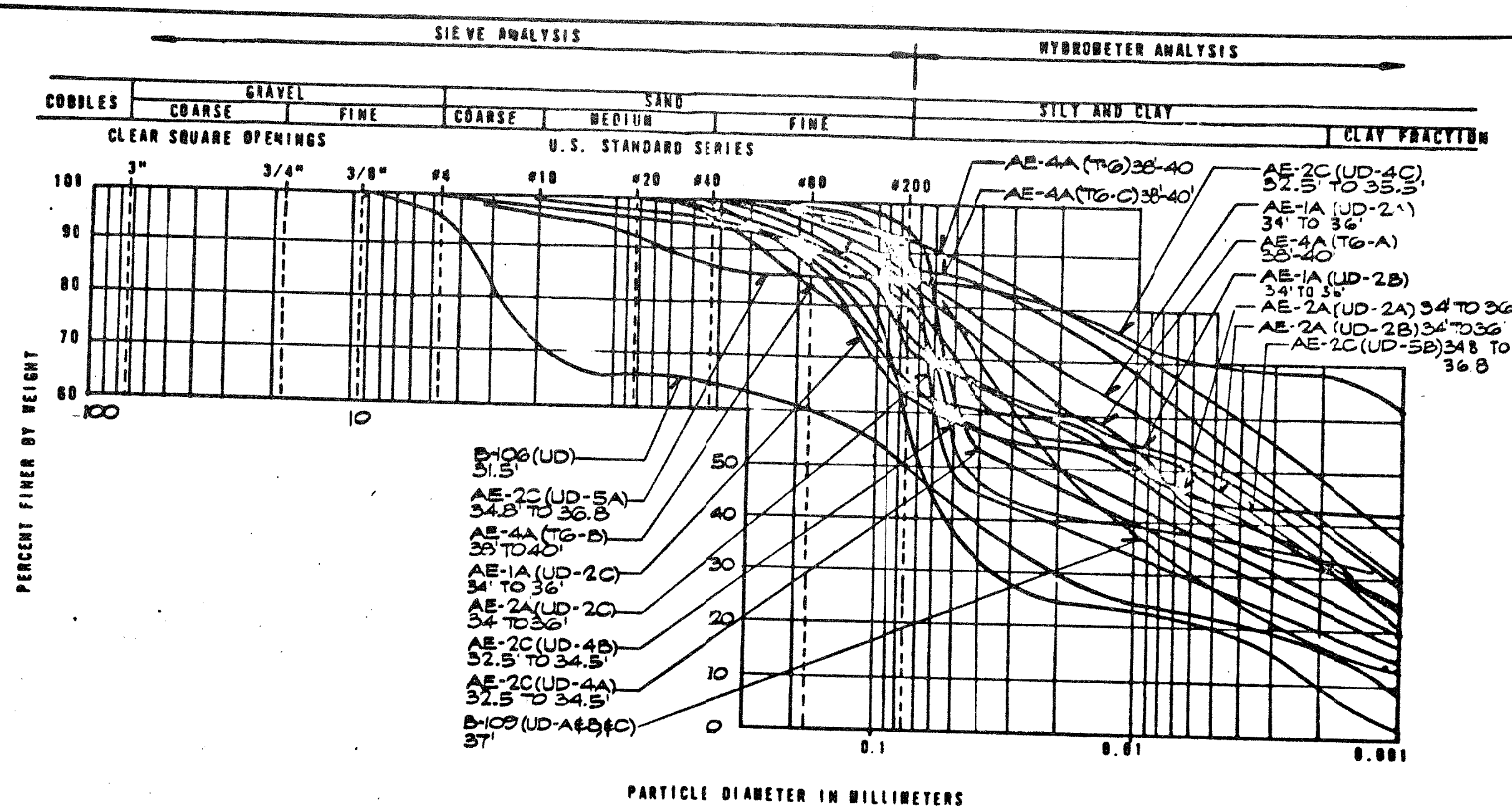
NO	BORING NO	SAMPLE NO	DEPTH	TEST
1	B-109	UD-A	37.0	CU/PP
2	B-109	UD-B	37.0	CU/PP
3	AE-2A	UD-2-A	34.0-36.0	CU/PP
4	AE-2A	UD-2-B	34.0-36.0	CU/PP
5	AE-2A	UD-2-C	34.0-36.0	CU/PP
6	AE-2C	UD-3-A	34.8-36.8	CU/PP
7	AE-2C	UD-3-B	34.8-36.8	CU/PP
8	AE-2C	UD-4-A	32.5-34.5	CU/PP
9	AE-2C	UD-4-B	32.5-34.5	CU/PP
10	AE-2C	UD-4-C	32.5-34.5	CU/PP
11	AE-1A	UD-2-A	34.0-36.0	CU/PP
12	AE-1A	UD-2-B	34.0-36.0	CU/PP
13	AE-1A	UD-2-C	34.0-36.0	CU/PP
14	AE-4A	T-6-A	38.0-40.0	CU/PP
15	AE-4A	T-6-B	38.0-40.0	CU/PP
16	AE-4A	T-6-C	38.0-40.0	CU/PP



EFFECTIVE STRESS ENVELOPES
CLAYEY ZONE MATERIALS

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

STRENGTH PARAMETERS
CLAYEY MATERIALS
EFFECTIVE STRESS
FIGURE 2G-7



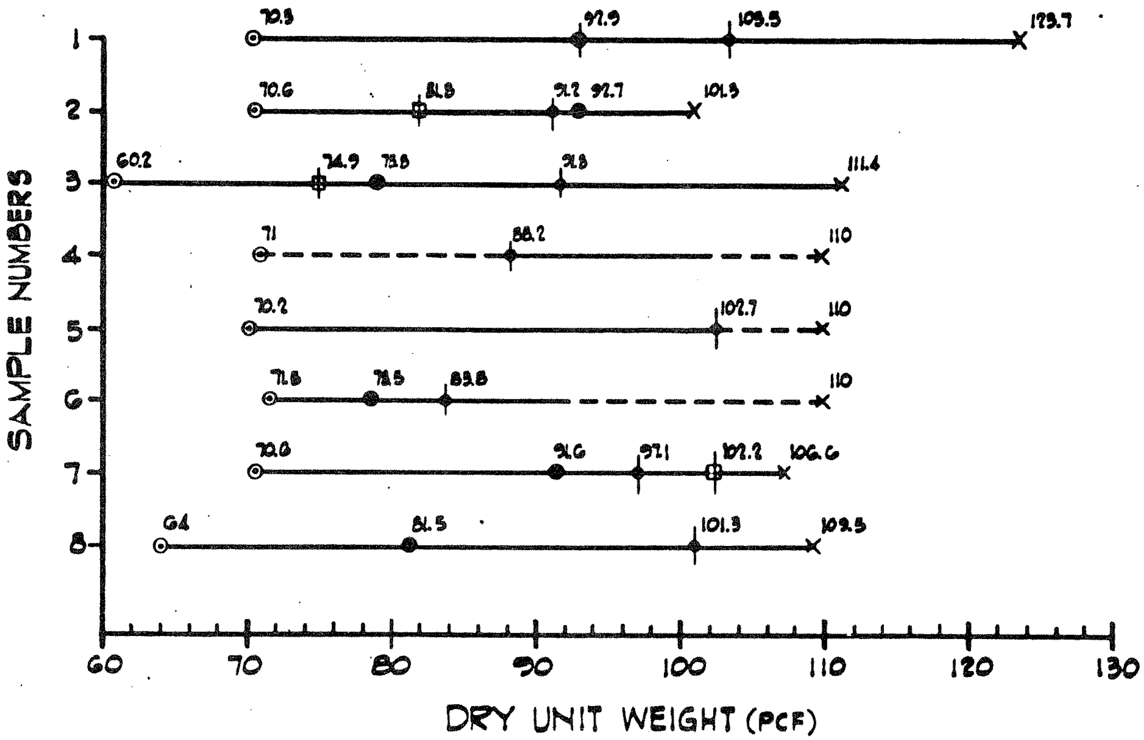
CLAYEY ZONE MATERIALS

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE ANALYSES OF CLAY MATERIALS TESTED IN TRIAXIAL SHEAR

FIGURE 2G-8

Nº	SAMPLE	%<#200
1	AE 1B-T11	30
2	AE 5C-T1	14.5
3	AE 5C-T3	29.1
4	AE 27D-T2	21.8
5	AE 27D-T6	25.8
6	AE 27E-T1	22.8
7	AE 27E-T5	26.5
8	AE 27E-T6	37



SANDY ZONE MATERIALS DENSITY CHARACTERISTICS

NOTES

DRY AND WET MAXIMUM AND VIBRATED DENSITY DATA PRESENTED FOR INFORMATION ONLY SINCE HIGH PRECENT FINES INVALIDATES TEST PROCEDURES.

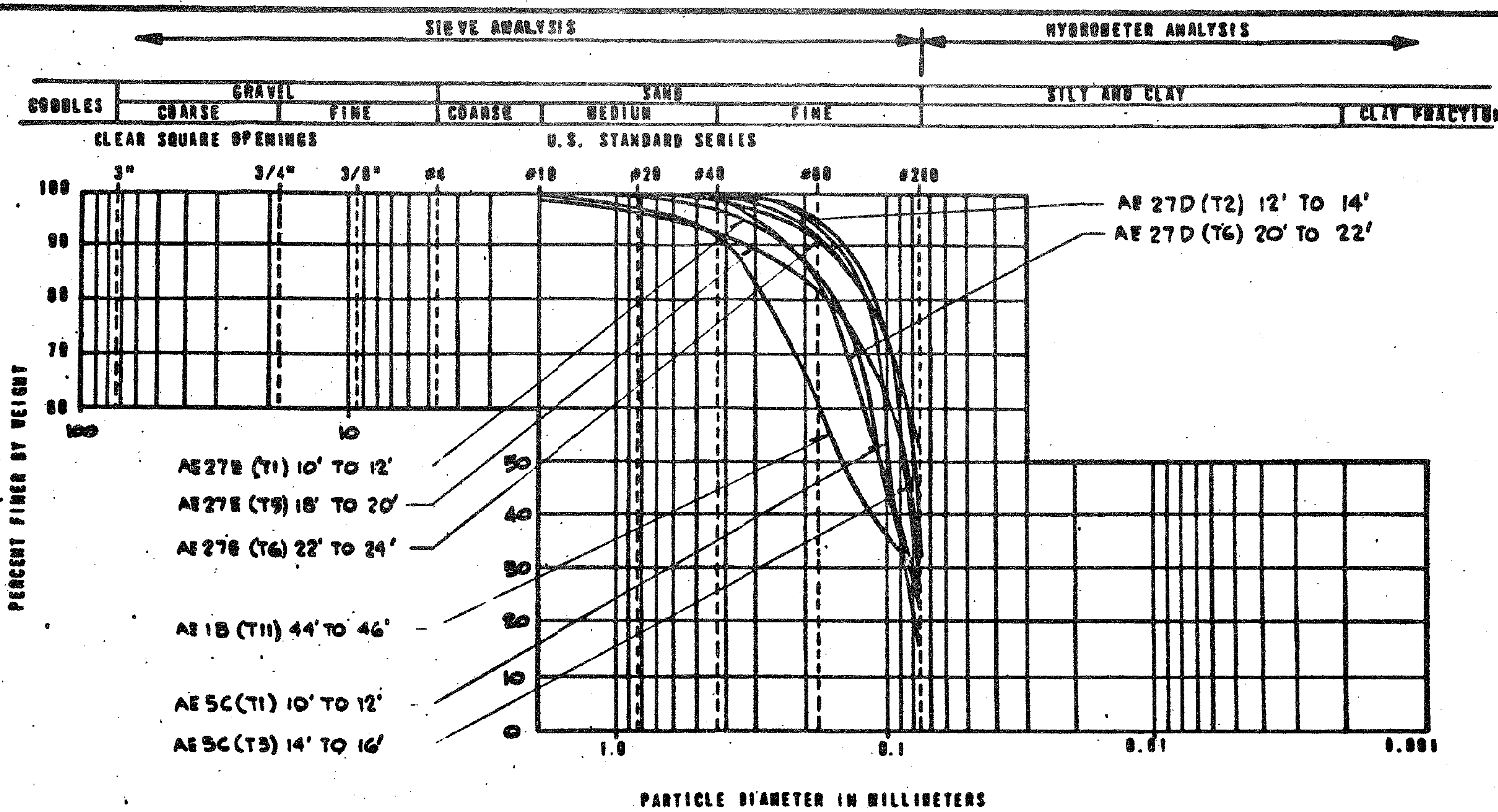
LEGEND

- ♦ = FIELD DENSITY
- ⊙ = DRY MINIMUM DENSITY
- ● = DRY MAXIMUM DENSITY
- ⊞ = WET MAXIMUM DENSITY
- X = MAXIMUM MODIFIED PROCTOR DENSITY
- --- ESTIMATED FROM AVAILABLE DATA

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

DENSITY CHARACTERISTICS
OF SITE SOILS

FIGURE 2G-9

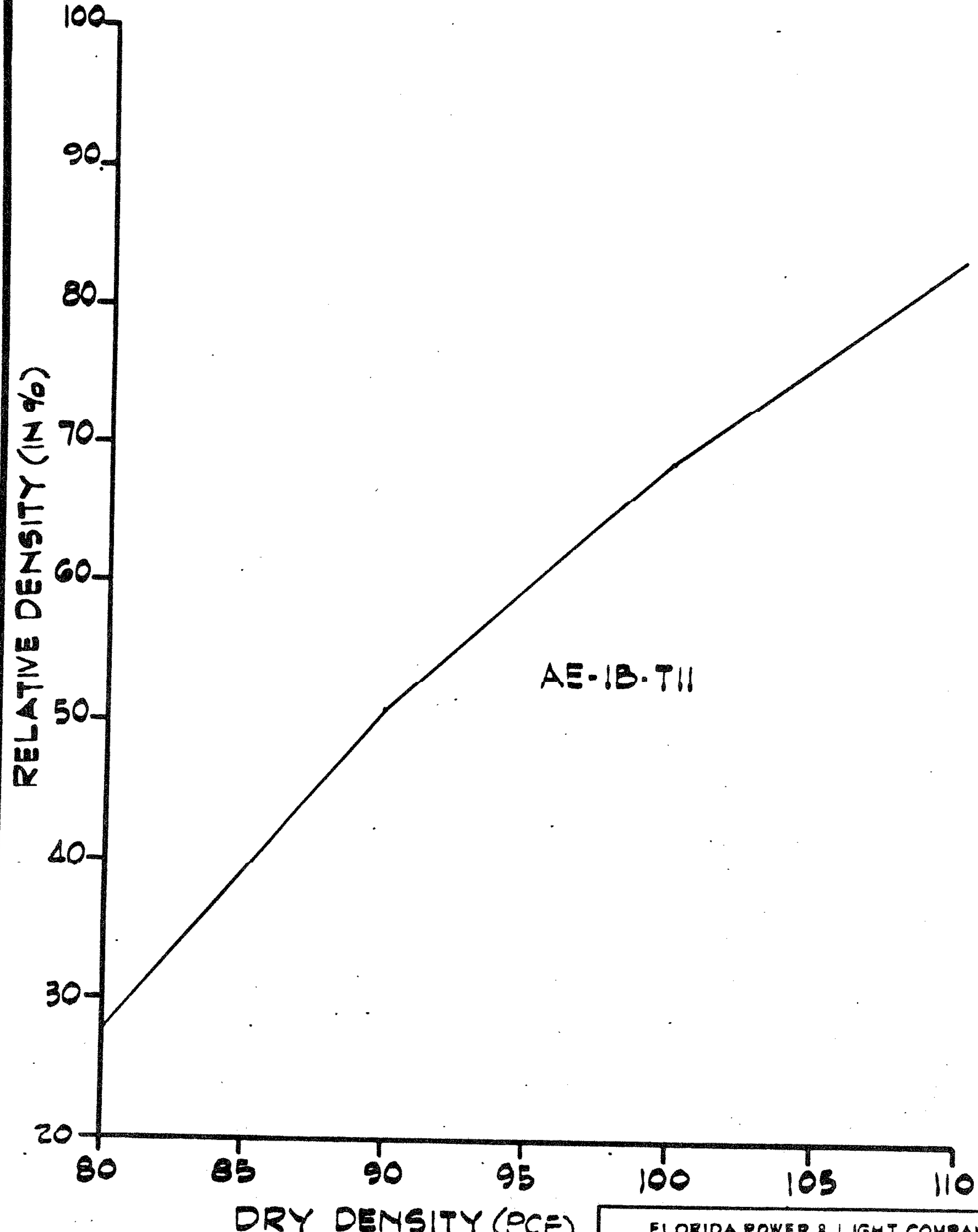


SANDY ZONE MATERIALS

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1.

GRAIN SIZE ANALYSES OF DENSITY STUDY SAMPLES

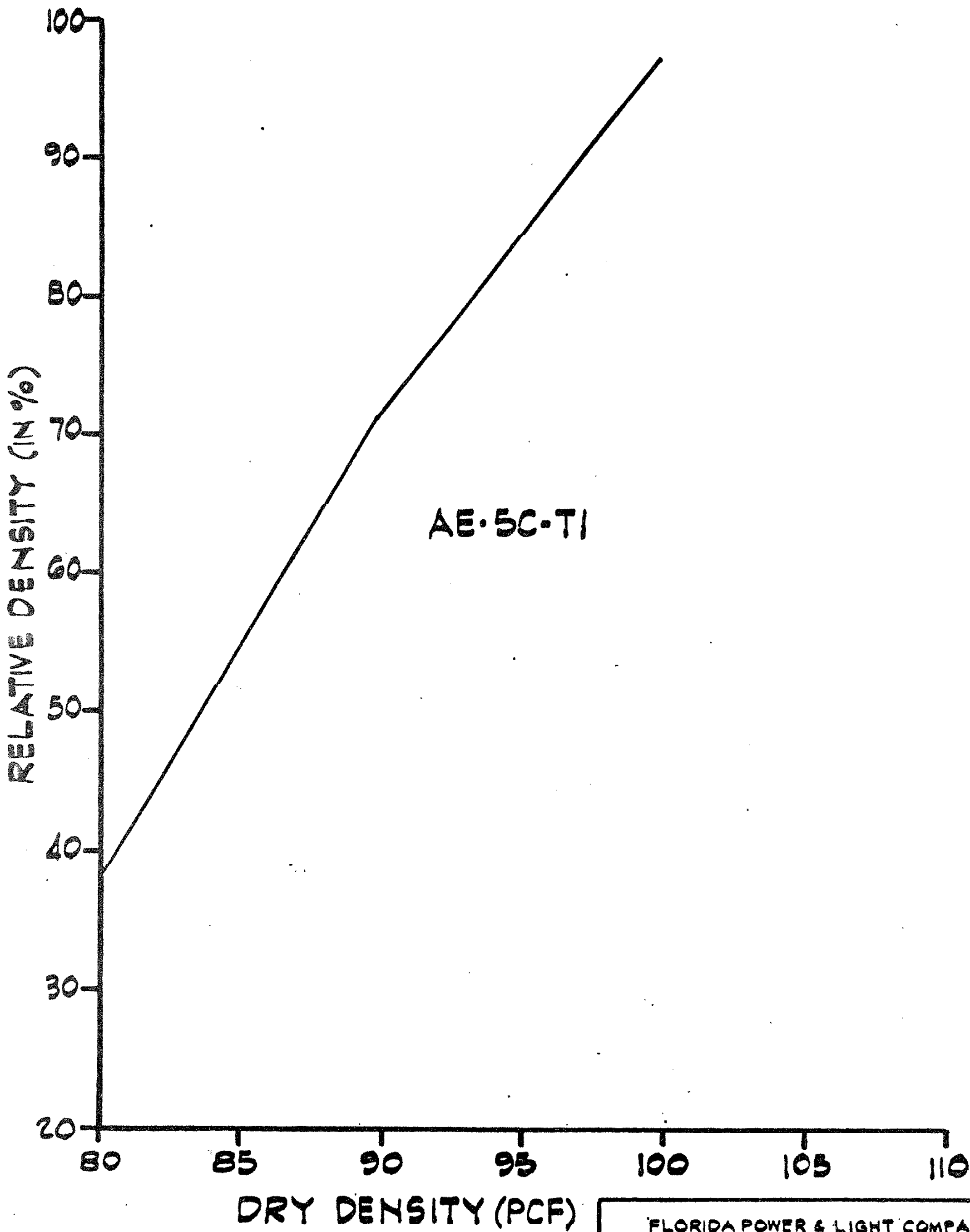
FIGURE 2G-10



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

DRY DENSITY VS. RELATIVE DENSITY

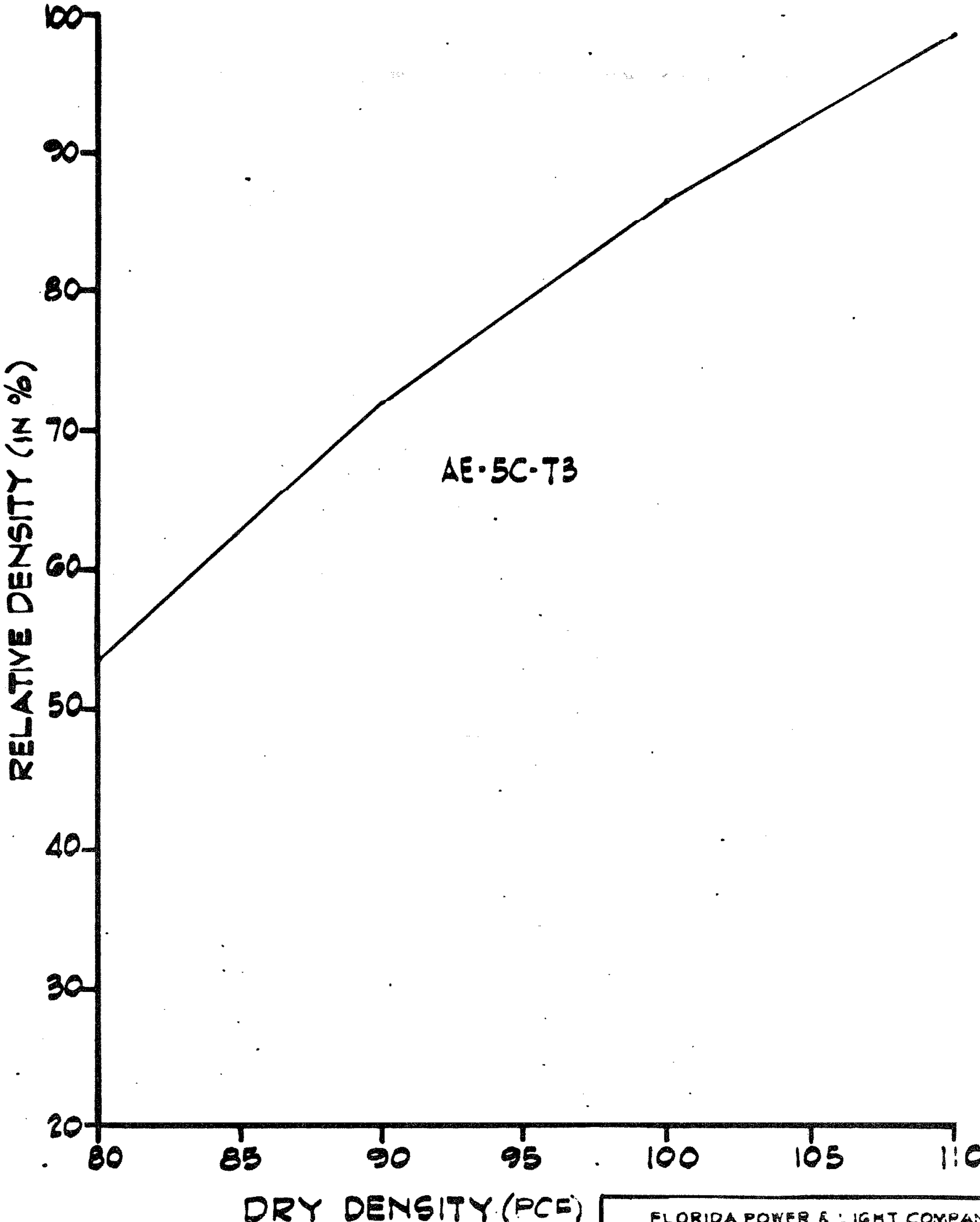
FIGURE 2G 11



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

DRY DENSITY VS. RELATIVE DENSITY

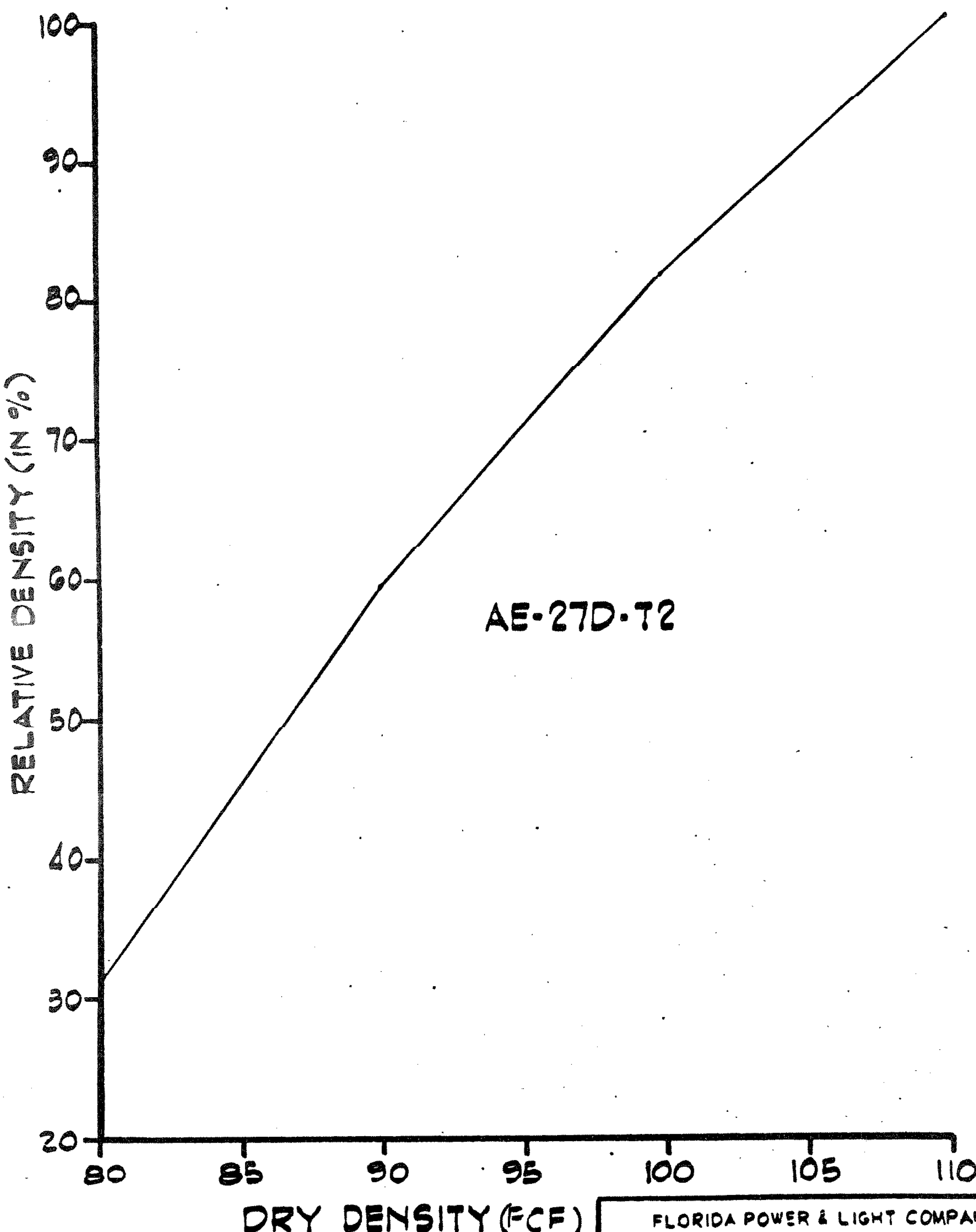
FIGURE 2G12



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

DRY DENSITY VS. RELATIVE DENSITY

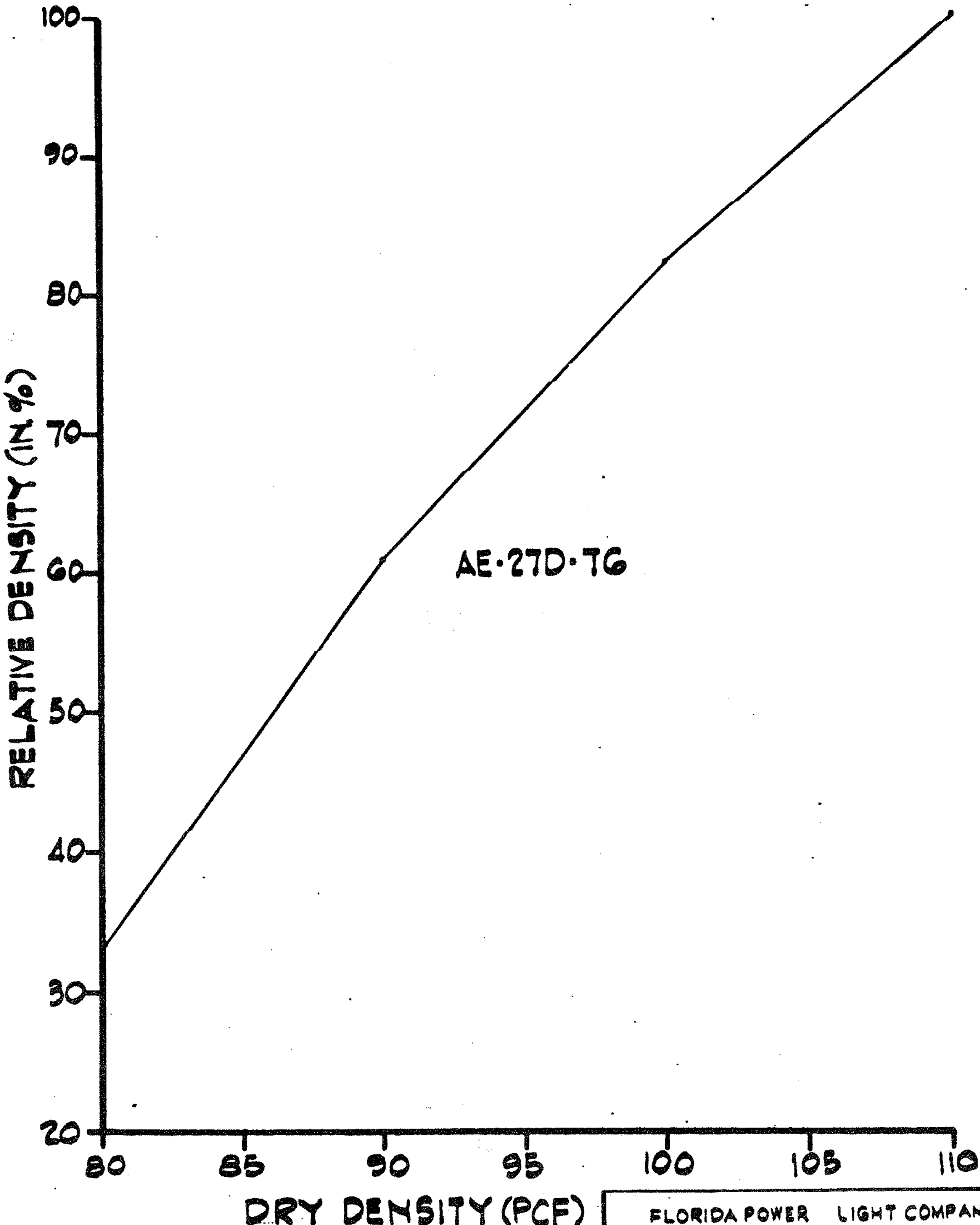
FIGURE 2G-13



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

DRY DENSITY VS. RELATIVE DENSITY

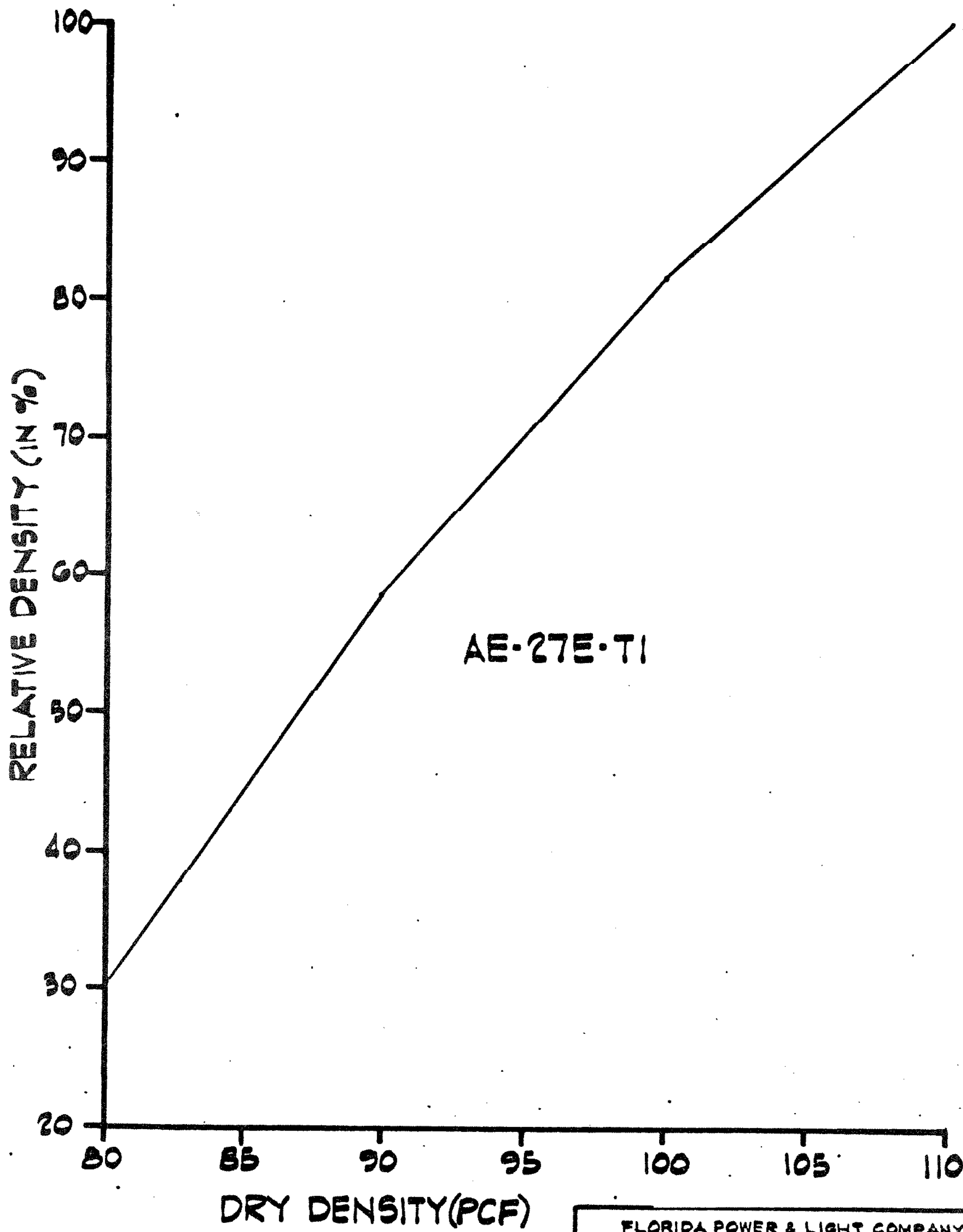
FIGURE 2G.14



FLORIDA POWER LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

DRY DENSITY VS. RELATIVE DENSITY

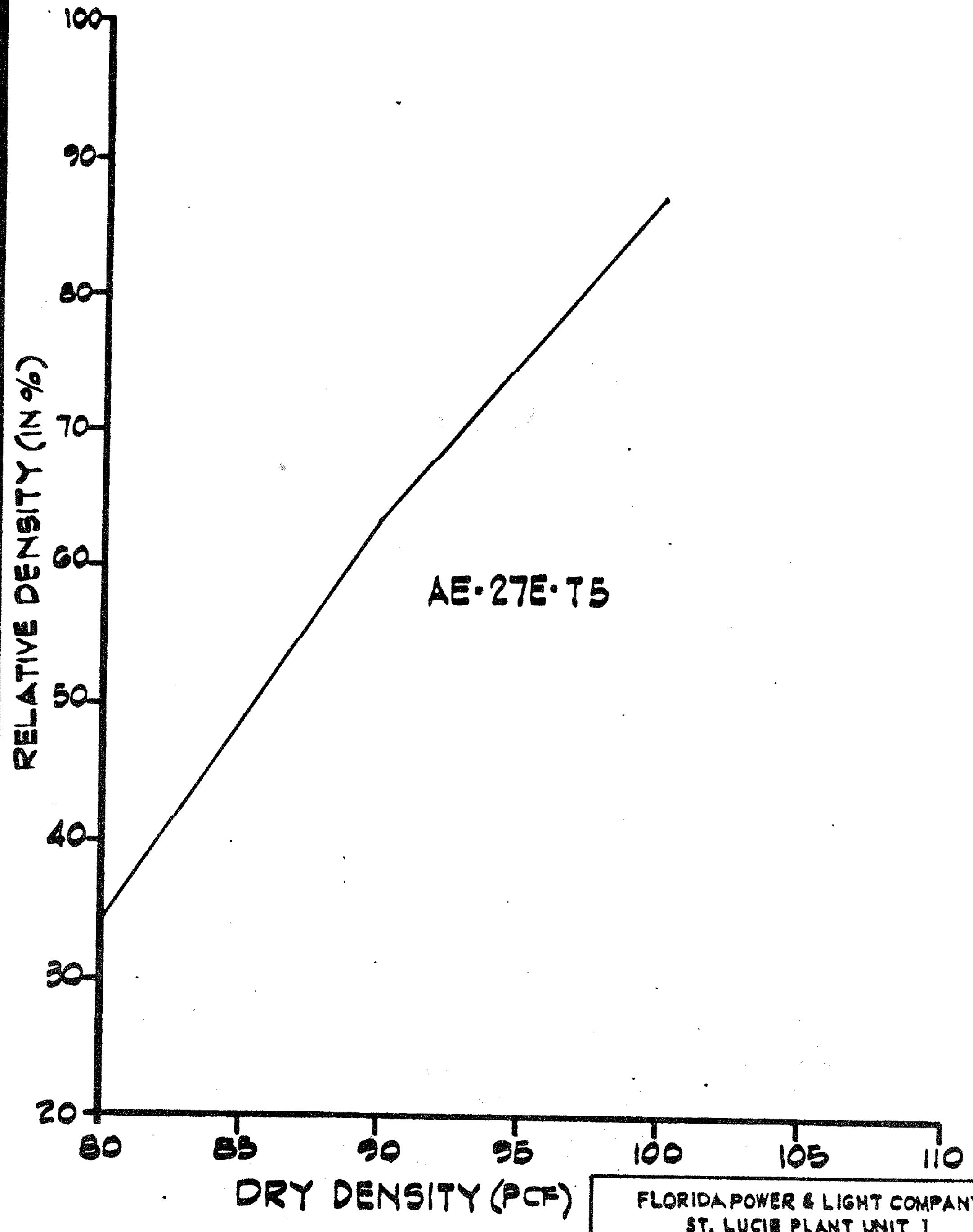
FIGURE 2G15



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

DRY DENSITY VS. RELATIVE DENSITY

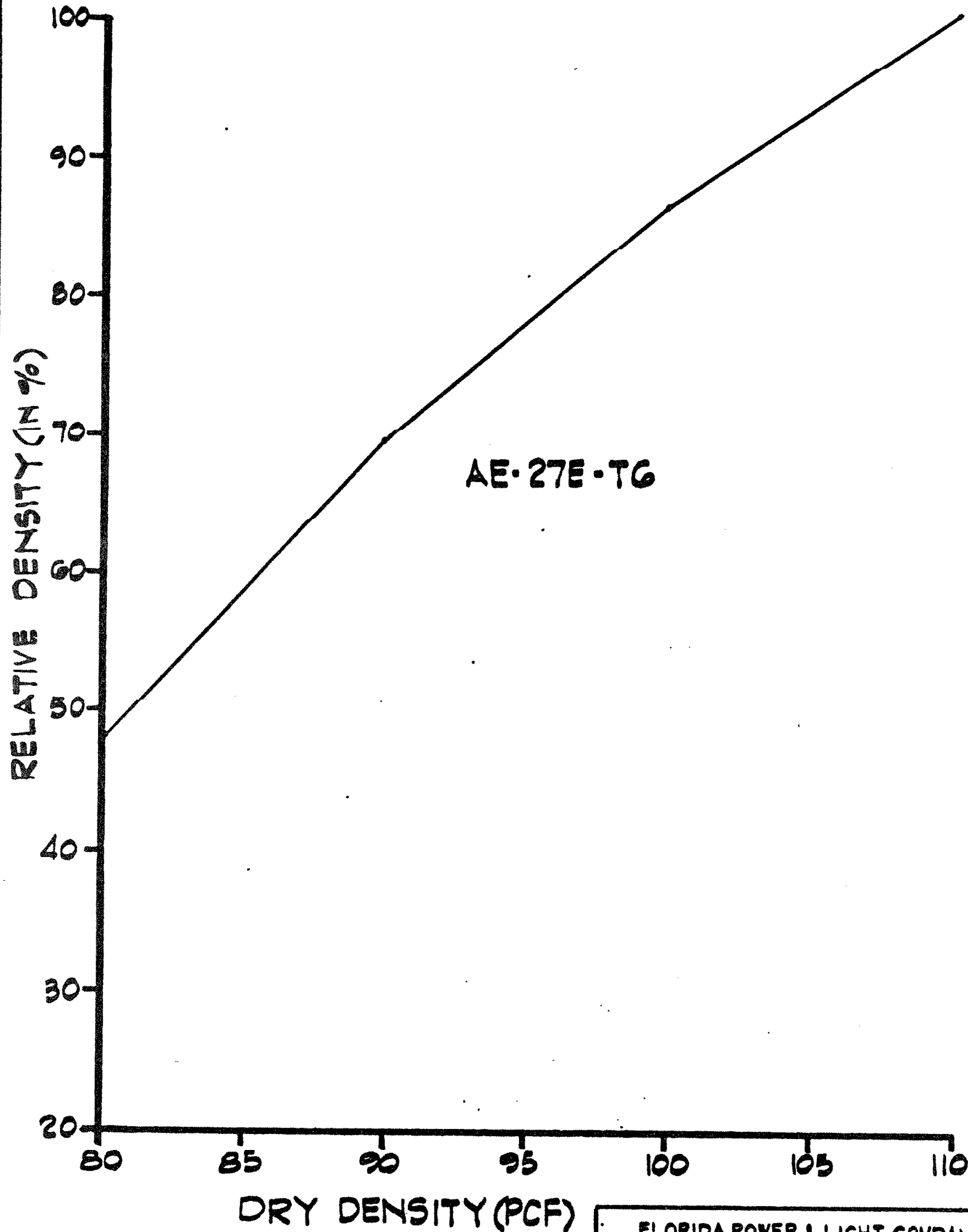
FIGURE 2G16



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

DRY DENSITY VS. RELATIVE DENSITY

FIGURE 2G 17



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

DRY DENSITY VS. RELATIVE DENSITY

FIGURE 2G-18

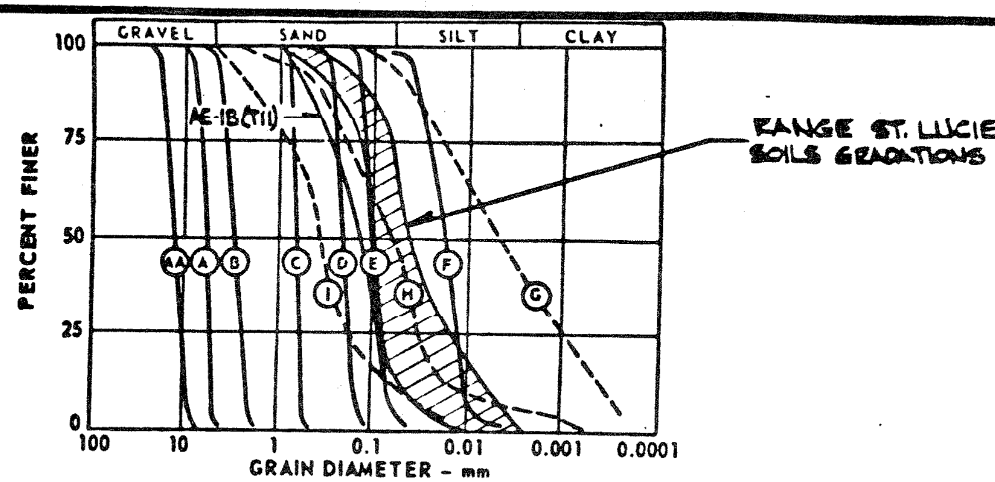


FIG. 1 - GRAIN-SIZE DISTRIBUTIONS FOR SOILS TESTED

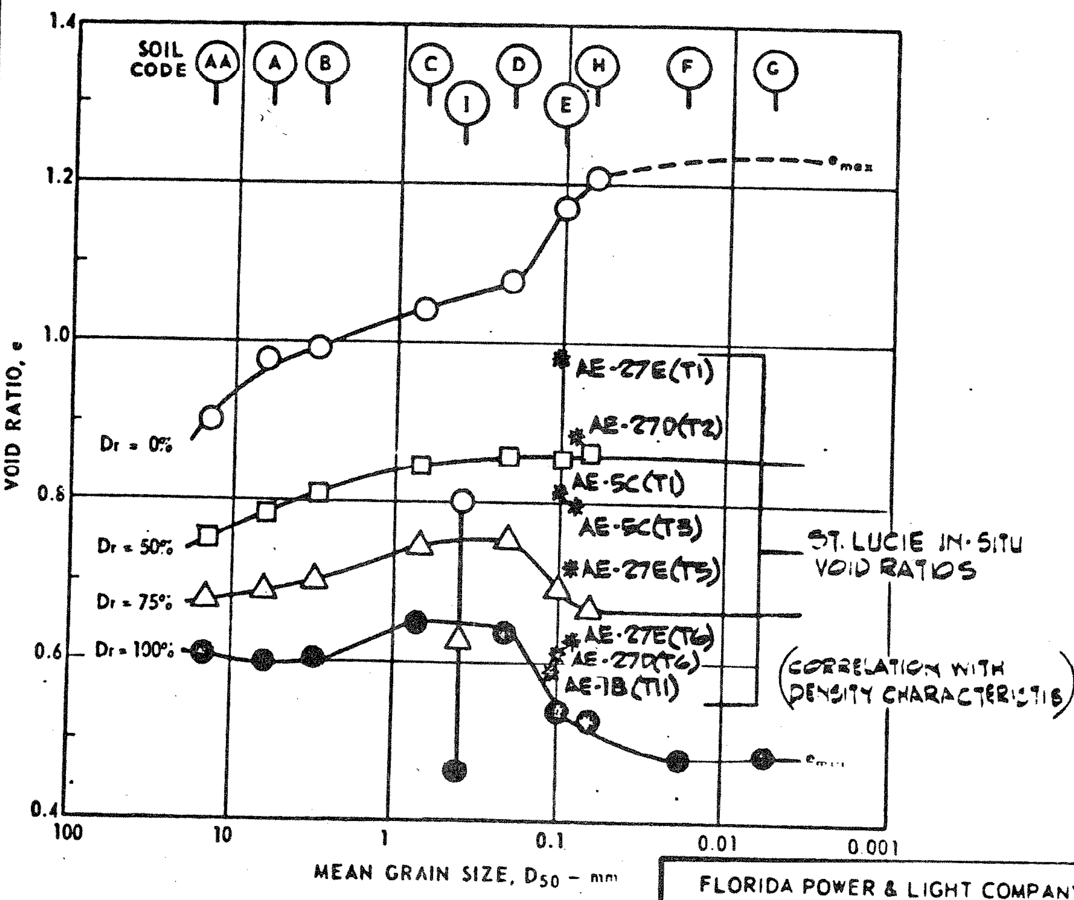


FIG. 2 - RELATIVE DENSITIES OF SOILS TESTED

REFERENCE
LEE & FITTON - ASTM STP 450 (1969)

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE & RELATIVE DENSITY COMPARISONS

FIGURE 2G-19

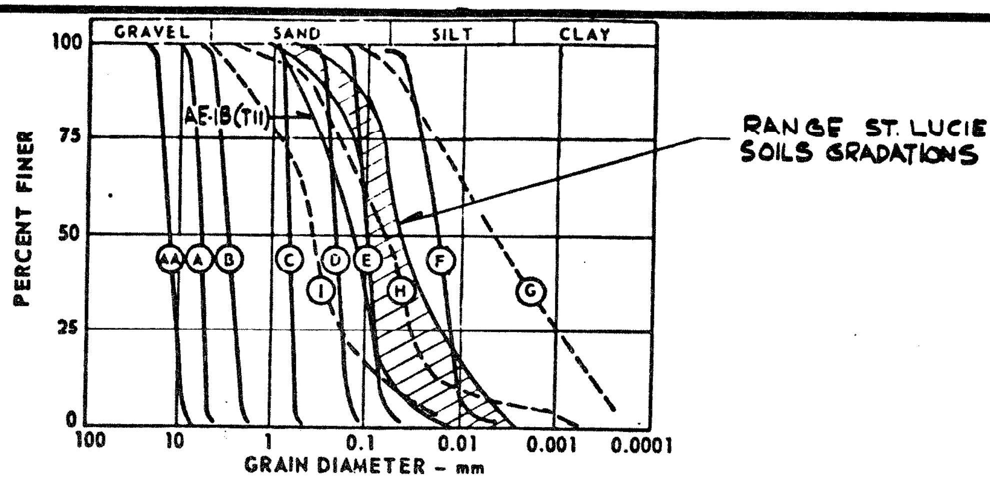


FIG. 1 – GRAIN-SIZE DISTRIBUTIONS FOR SOILS TESTED

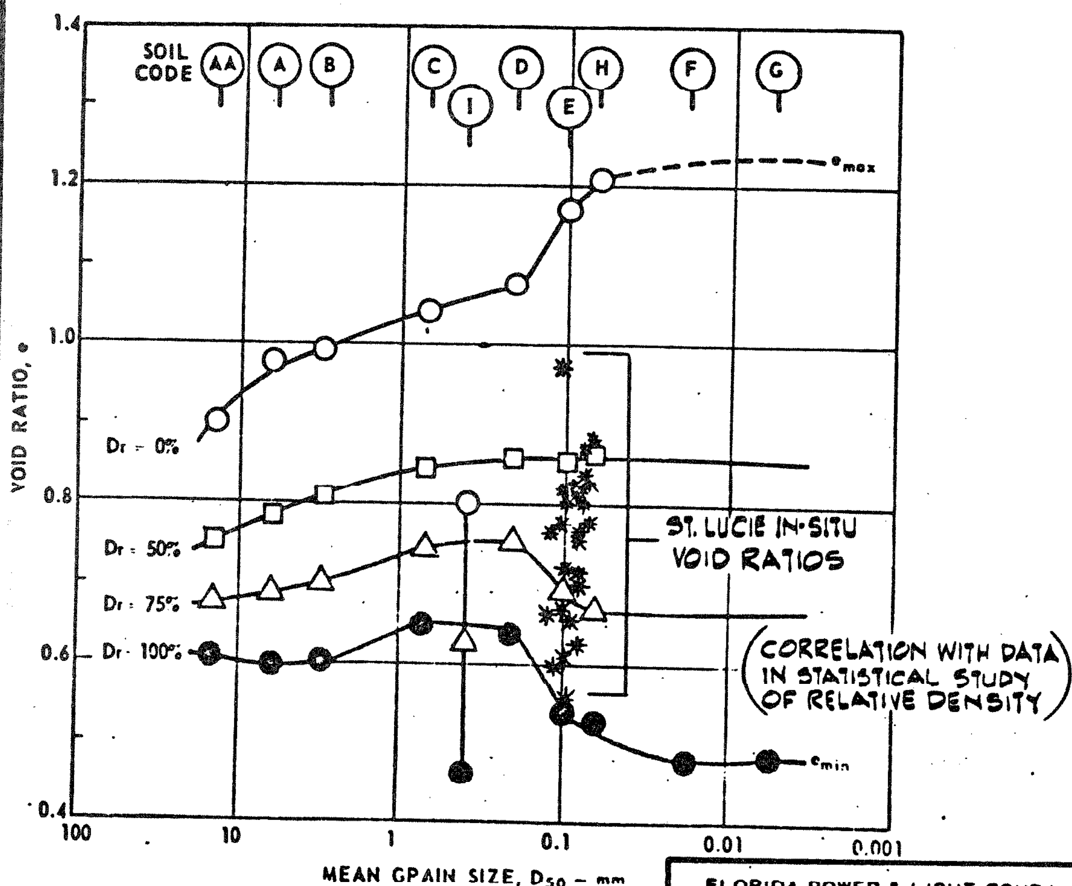


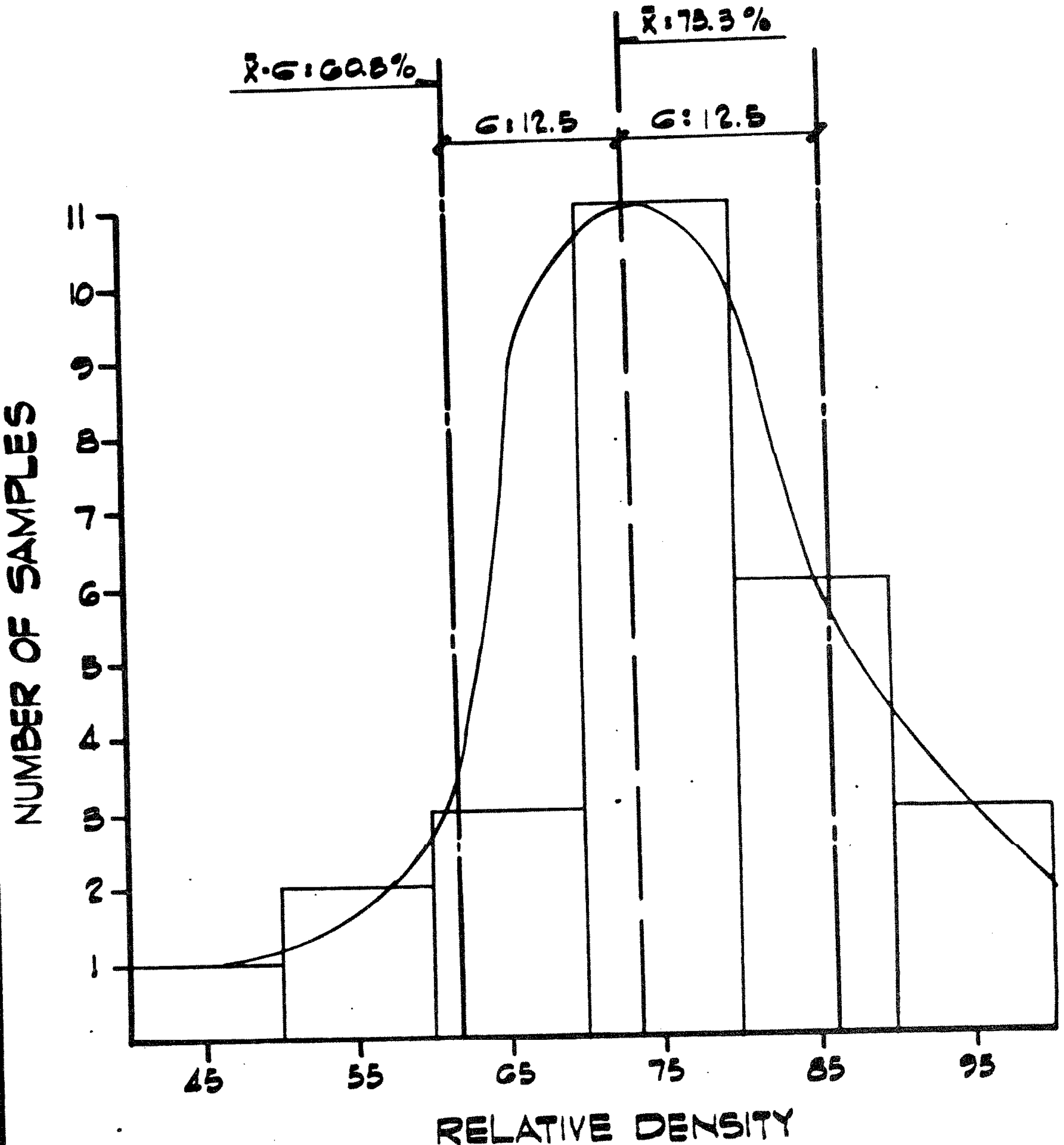
FIG. 2 – RELATIVE DENSITIES OF SOILS TESTED

REFERENCE:
LEE & FITTON - ASTM STP 450 (1969)

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

CORRELATION OF IN SITU VOID RATIOS WITH RELATIVE DENSITY

FIGURE 2G-20

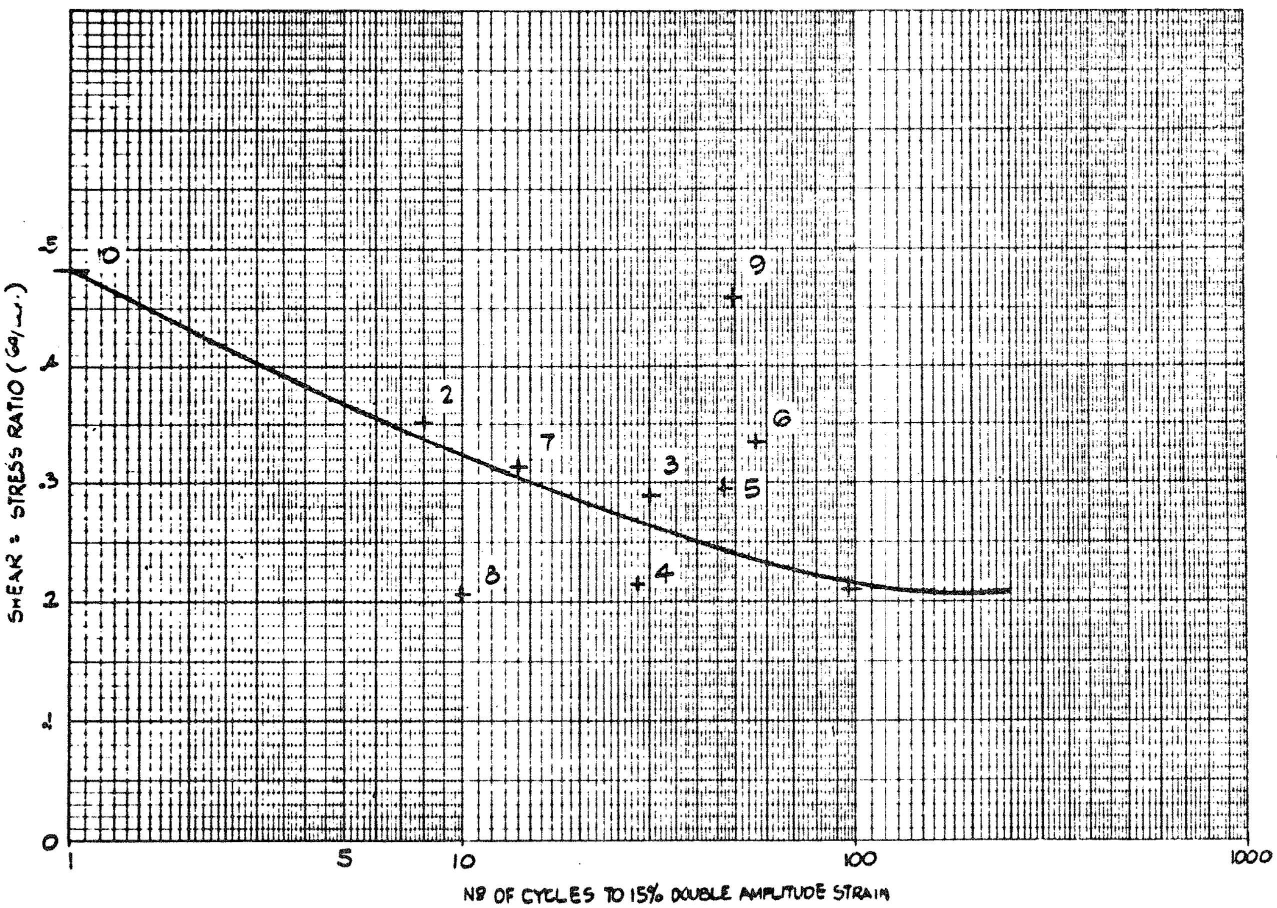


26 SAMPLES (U.D. TUBES)

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

STATISTICAL ANALYSIS
OF RELATIVE DENSITY

FIGURE 2G-21

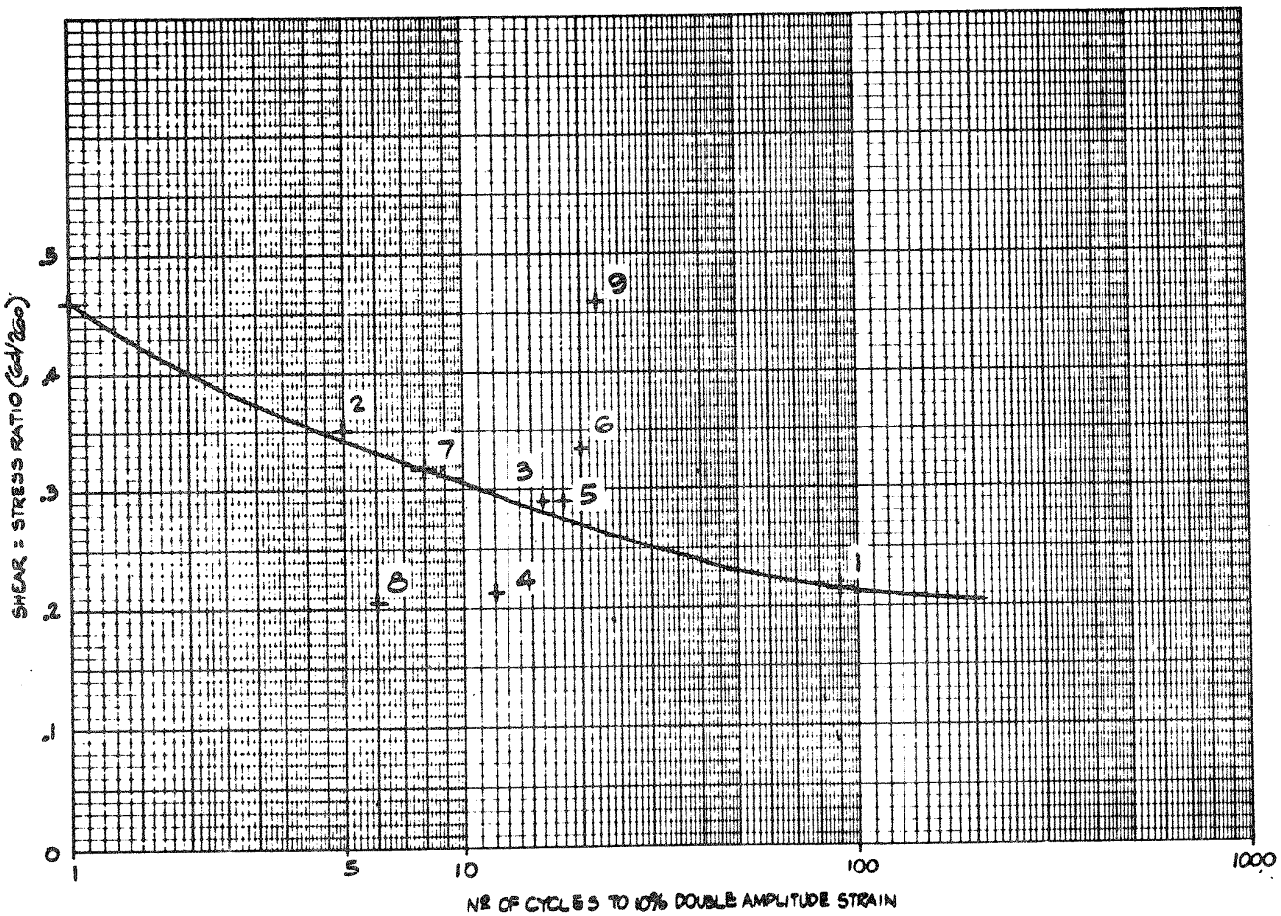


NOTE:
ALL DATA NORMALIZED TO
73.3% RELATIVE DENSITY

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

CYCLIC-STRENGTH AT 15% STRAIN

FIGURE 2G-22

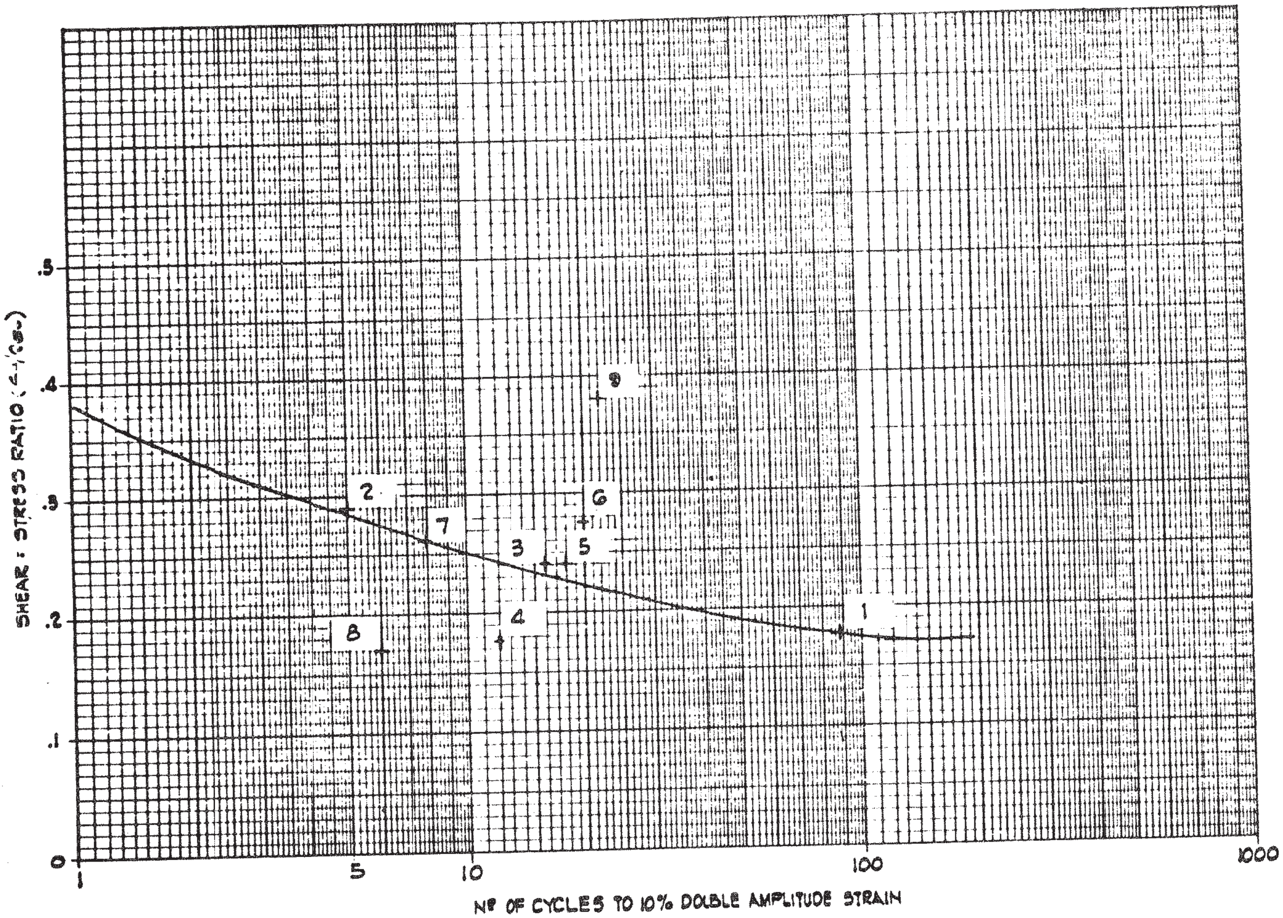


NOTE
ALL DATA NORMALIZED TO
73.3% RELATIVE DENSITY

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

CYCLIC STRENGTH AT 10% STRAIN

FIGURE 2G-23

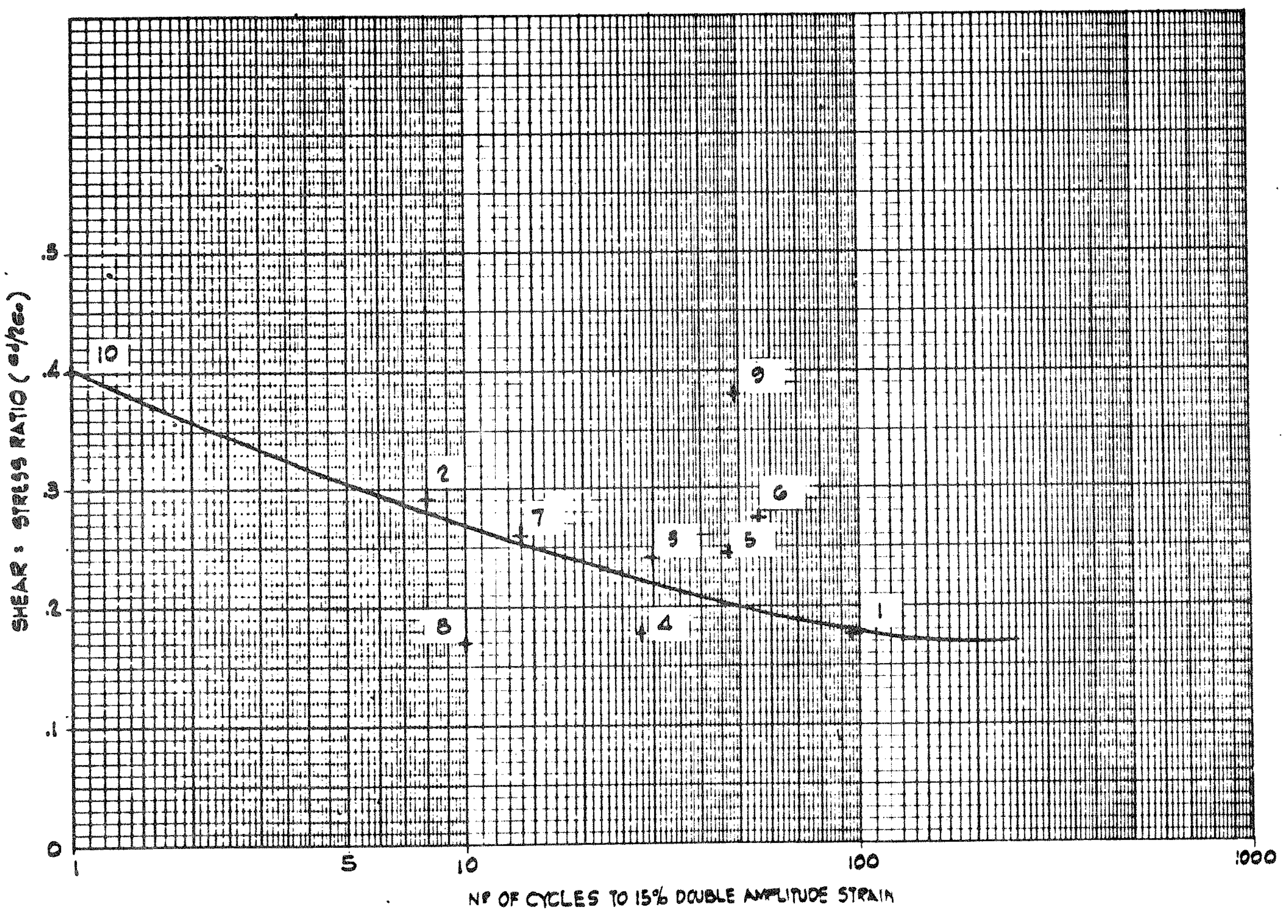


NOTE:
ALL DATA NORMALIZED TO
60.8% RELATIVE DENSITY

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

CYCLIC STRENGTH AT 10% STRAIN

FIGURE 2G-24

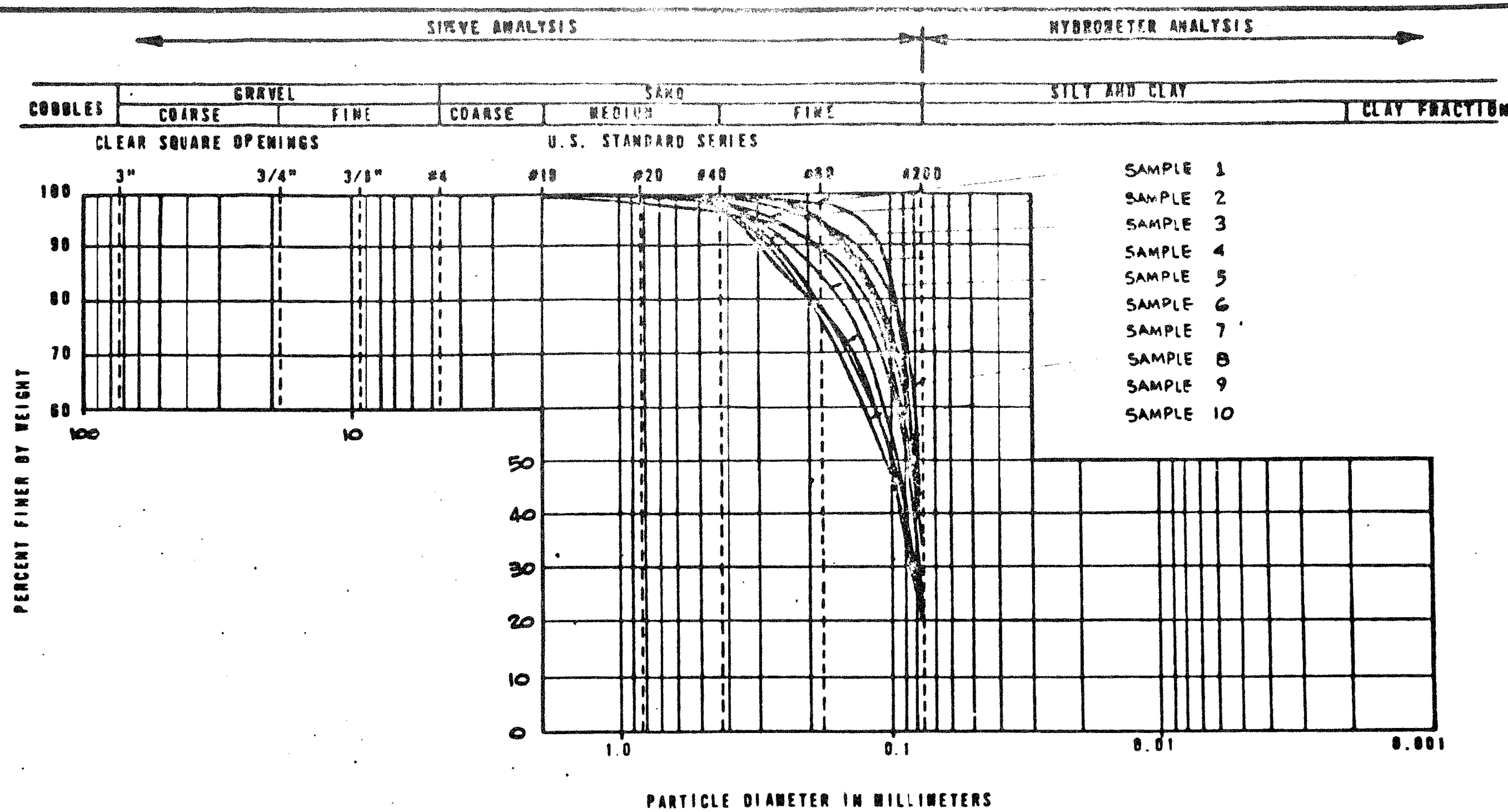


NOTE:
ALL DATA NORMALIZED TO 60.8% RELATIVE DENSITY

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

CYCLIC STRENGTH AT 15% STRAIN

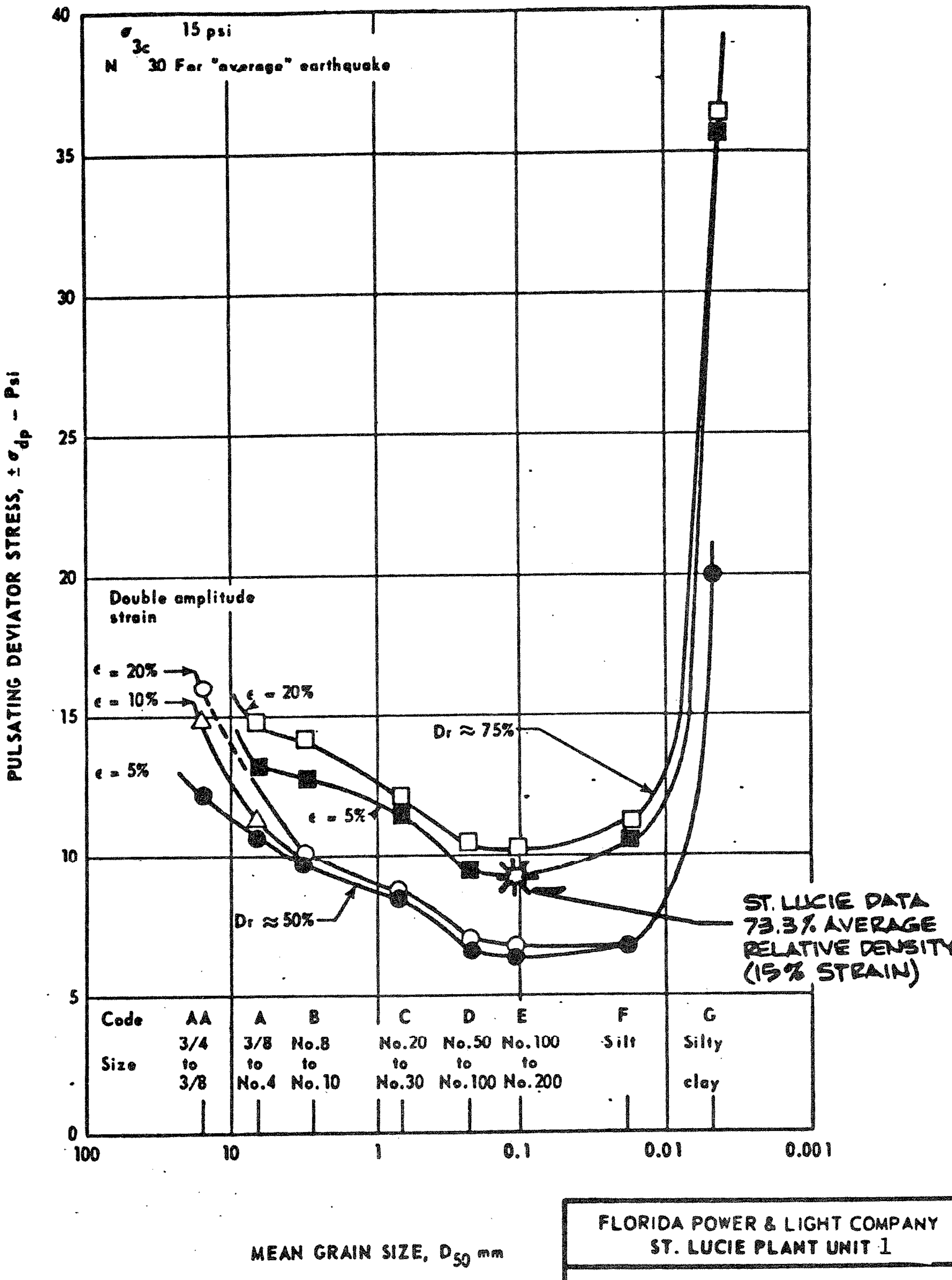
FIGURE 2G-25



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE ANALYSIS OF CYCLIC TRIAXIAL SAMPLES

FIGURE 2G-26

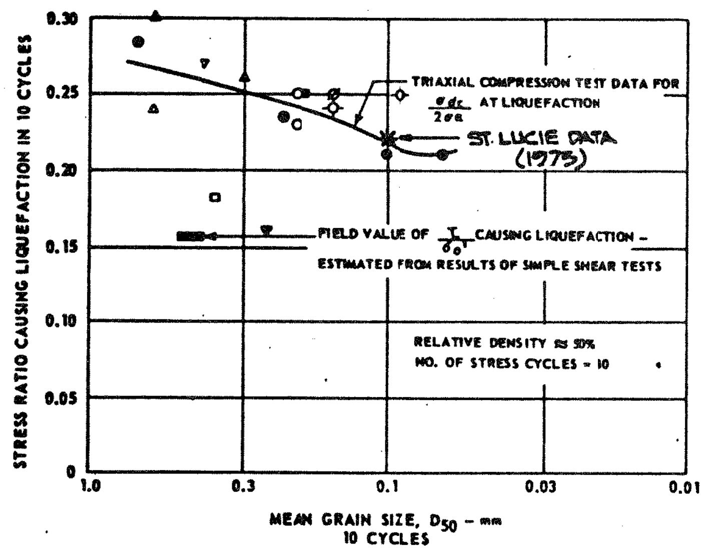


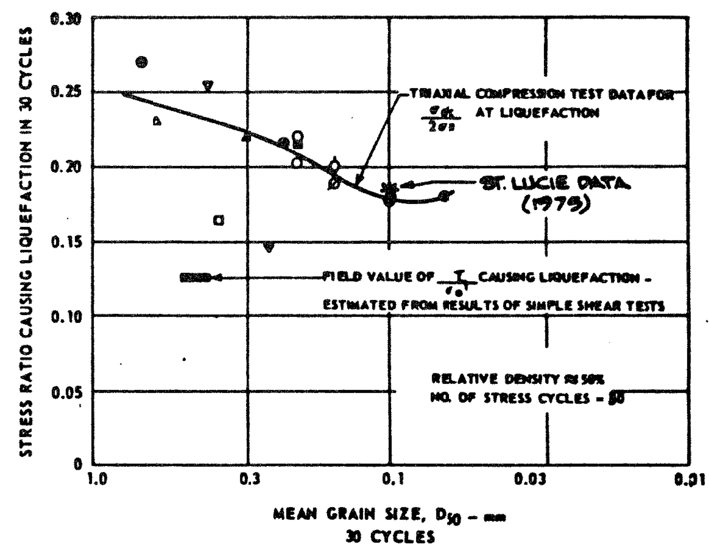
REFERENCE:
LEE & FITTON - ASTM STP 450 (1969)

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

CYCLIC STRENGTH COMPARISON

FIGURE 2G-27

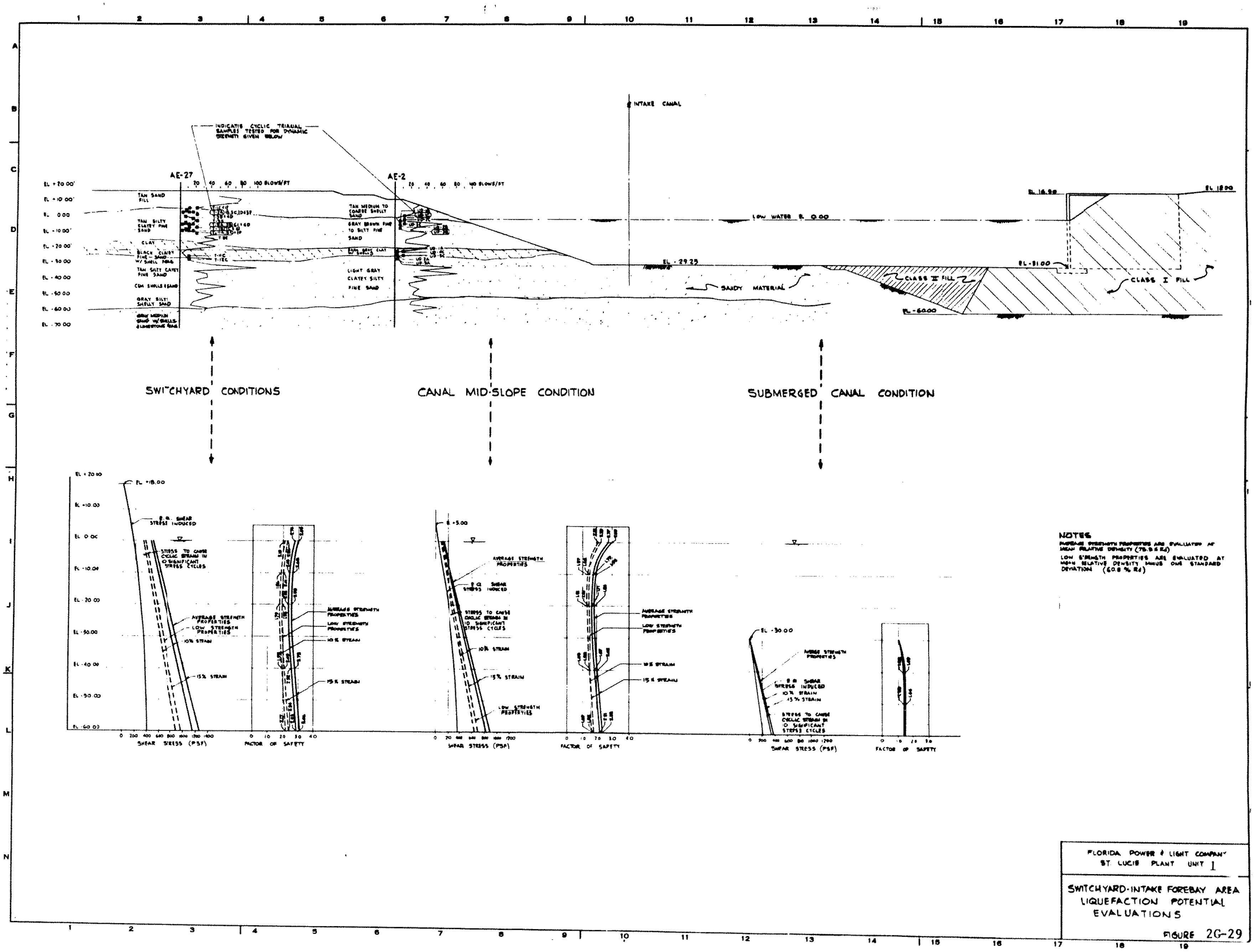




FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

CYCLIC STRENGTH COMPARISONS

FIGURE 2G-28



NOTES

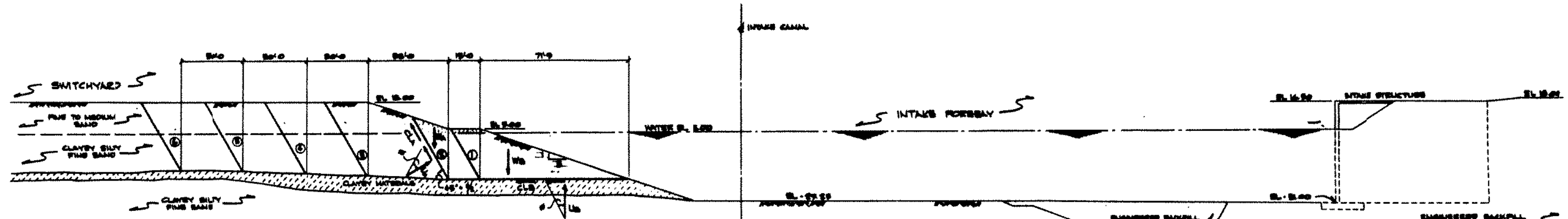
AVERAGE STRENGTH PROPERTIES ARE EVALUATED AT MEAN RELATIVE DENSITY (75.5 % Rd)

LOW STRENGTH PROPERTIES ARE EVALUATED AT MEAN RELATIVE DENSITY MINUS ONE STANDARD DEVIATION (60.8 % Rd)

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

SWITCHYARD-INTAKE FOREBAY AREA LIQUEFACTION POTENTIAL EVALUATIONS

FIGURE 2G-29



CIRCLE	COORDINATES X	COORDINATES Y	RADIUS
A-A	220	190	116.17
E-E	289	240	179.10

SLIDING WEDGE ANALYSIS
1"=20'

SAFETY FACTORS FOR STATIC ANALYSIS

CASE CONDITION	SOIL PROPERTIES AT ACTIVE WEDGE c (KSF)	ϕ	AT NEUTRAL BLOCK c (KSF)	ϕ	PLANE NUMBER 1	2	3	4	5	6
MIN	0.15	27°	0.15	27°	1.6	1.8	2.1	2.8	3.5	4.2
			[illegible]	15°	1.7	1.7	1.7	1.9	2.1	2.4
AVG	0.15	33°	0.15	33°	1.8	2.0	2.1	2.1	3.9	4.7
			[illegible]	[illegible]	1.7	1.7	1.6	1.8	2.6	2.9
MAX	0	45°	0	45°	1.9	2.5	3.0	4.2	5.9	6.8
			0.70	15°	2.1	2.5	2.5	2.9	3.4	4.0

NOTE:
[0.00] INDICATES FAILURE IN SAND ALONG BASE OF NEUTRAL BLOCK.
[0.00] INDICATES FAILURE IN CLAY ALONG BASE OF NEUTRAL BLOCK.

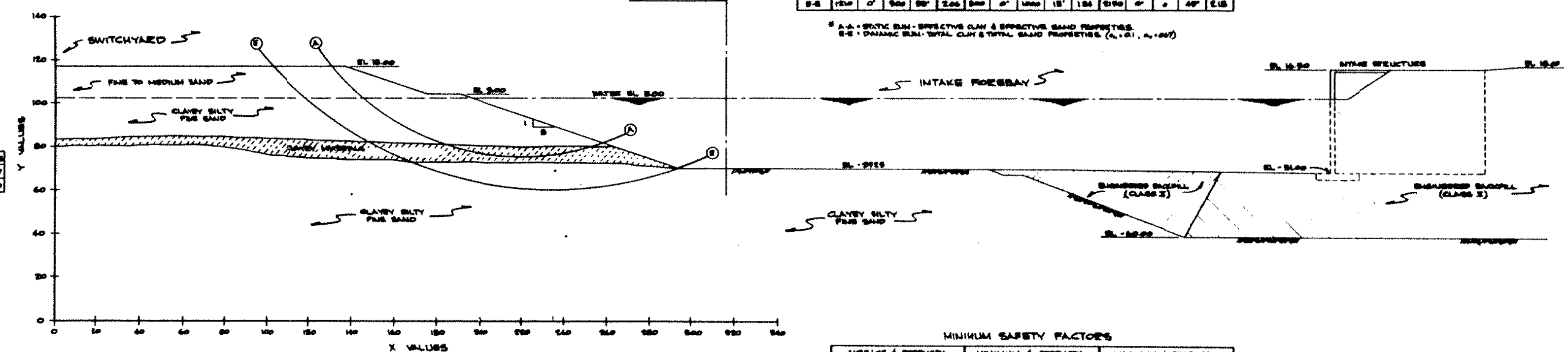
SAFETY FACTORS FOR DYNAMIC ANALYSIS

CASE CONDITION	SOIL PROPERTIES AT ACTIVE WEDGE c (KSF)	ϕ	AT NEUTRAL BLOCK c (KSF)	ϕ	PLANE NUMBER 1	2	3	4	5	6
MIN	1.00	15°	1.00	15°	2.3	2.1	2.0	2.3	2.5	2.7
			0.60	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
AVG	0.30	33°	0.30	33°	2.1	2.2	2.1	2.5	2.9	3.2
			[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
MAX	0	45°	0	45°	1.2	1.4	1.6	2.3	2.7	3.0
			[illegible]	[illegible]	[illegible]	[illegible]	3.1	[illegible]	[illegible]	3.6

MINIMUM SAFETY FACTORS

CASE*	AVERAGE ϕ-STRENGTH CLAYEY c	ϕ	SANDY c	ϕ	FACTOR OF SAFETY	MINIMUM ϕ-STRENGTH CLAYEY c	ϕ	SANDY c	ϕ	FACTOR OF SAFETY	MAXIMUM ϕ-STRENGTH CLAYEY c	ϕ	SANDY c	ϕ	FACTOR OF SAFETY
A-A	550	6.5°	150	33°	2.58	650	1.9°	150	27°	2.14	700	12°	0	45°	3.07
E-E	1210	0°	500	33°	2.06	800	0°	1000	15°	1.84	2150	0°	0	45°	2.18

* A-A = STATIC RUN - EFFECTIVE CLAY & EFFECTIVE SAND PROPERTIES.
E-E = DYNAMIC RUN - TOTAL CLAY & TOTAL SAND PROPERTIES. (a_h = 0.1, a_v = .067)

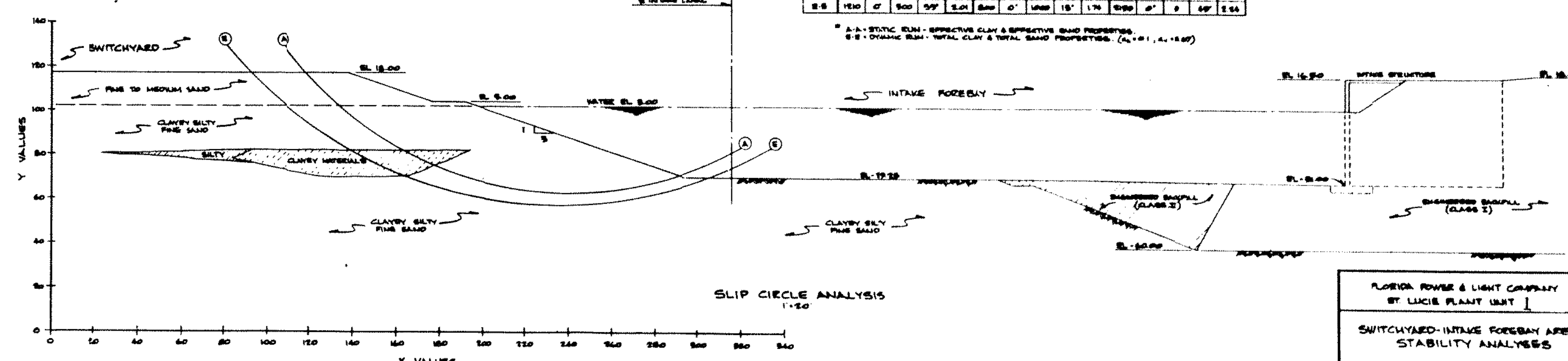


CIRCLE	COORDINATES X	COORDINATES Y	RADIUS
A-A	240	229	142.66
E-E	295	295	197.18

MINIMUM SAFETY FACTORS

CASE*	AVERAGE ϕ-STRENGTH CLAYEY c	ϕ	SANDY c	ϕ	FACTOR OF SAFETY	MINIMUM ϕ-STRENGTH CLAYEY c	ϕ	SANDY c	ϕ	FACTOR OF SAFETY	MAXIMUM ϕ-STRENGTH CLAYEY c	ϕ	SANDY c	ϕ	FACTOR OF SAFETY
A-A	550	6.5°	150	36°	2.45	650	15°	150	27°	2.01	700	12°	0	45°	3.23
E-E	1210	0°	500	33°	2.01	800	0°	1000	15°	1.74	2150	0°	0	45°	2.54

* A-A = STATIC RUN - EFFECTIVE CLAY & EFFECTIVE SAND PROPERTIES.
E-E = DYNAMIC RUN - TOTAL CLAY & TOTAL SAND PROPERTIES. (a_h = 0.1, a_v = .067)

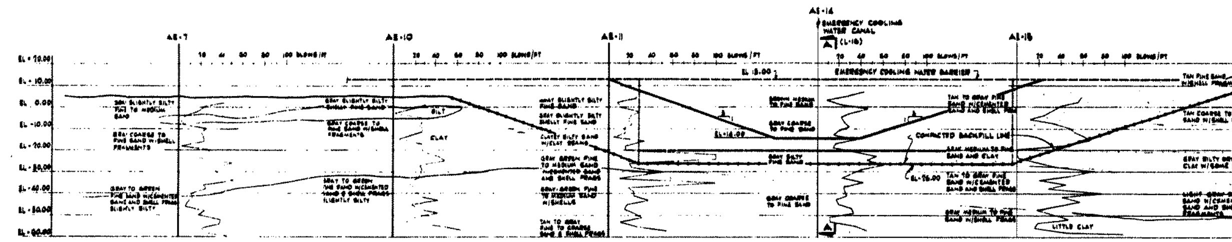


SLIP CIRCLE ANALYSIS
1"=20'

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

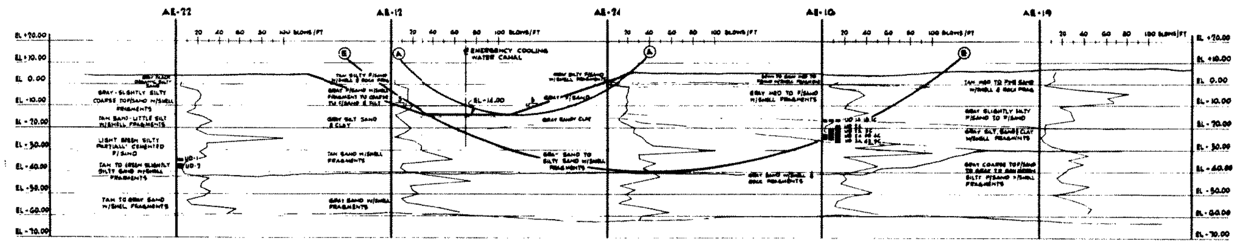
SWITCHYARD-INTAKE FOREBAY AREA
STABILITY ANALYSES

FIGURE 2G-30



SECTION 9-9

CROSS-SECTIONAL PROFILE EMERGENCY COOLING CANAL

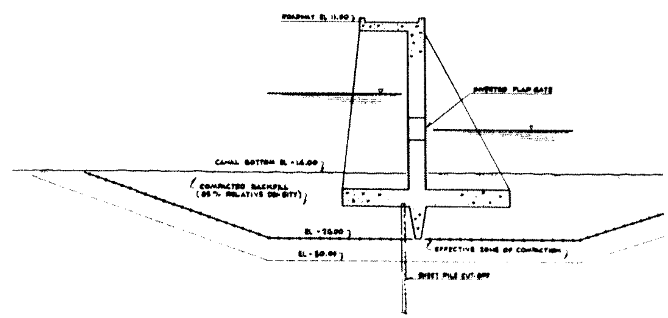


SECTION 10-10

MINIMUM SAFETY FACTORS

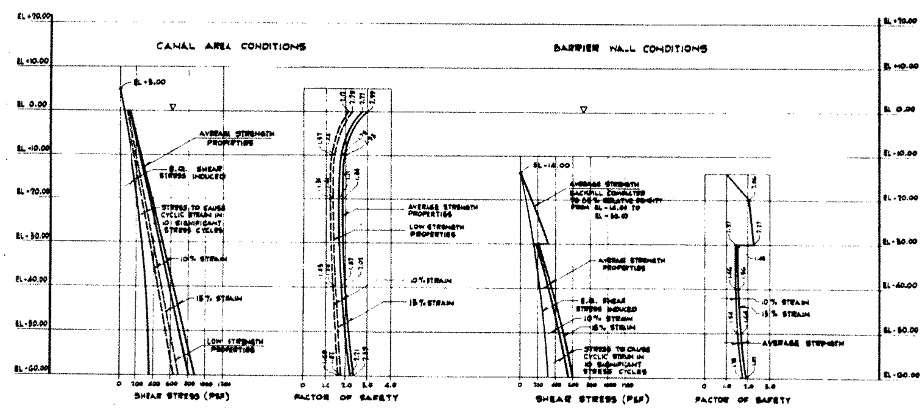
CASE*	AVERAGE φ-STRENGTH					MINIMUM φ-STRENGTH					MAXIMUM φ-STRENGTH				
	CLAYEY		SANDY		FACTOR OF SAFETY	CLAYEY		SANDY		FACTOR OF SAFETY	CLAYEY		SANDY		FACTOR OF SAFETY
	C	φ	C	φ		C	φ	C	φ		C	φ	C	φ	
A-A	850	6.5°	150	35°	3.67	650	1.5°	150	27°	3.27	700	12°	0	45°	3.34
B-B	1210	0°	500	33°	2.79	800	0°	1000	18°	2.89	2150	0°	0	45°	3.17

* A-A: STATIC RUN - EFFECTIVE CLAY & EFFECTIVE SAND PROPERTIES.
B-B: DYNAMIC RUN - TOTAL CLAY & TOTAL SAND PROPERTIES [illegible]



SECTION A-A (A-10)

EMERGENCY COOLING WATER BARRIER



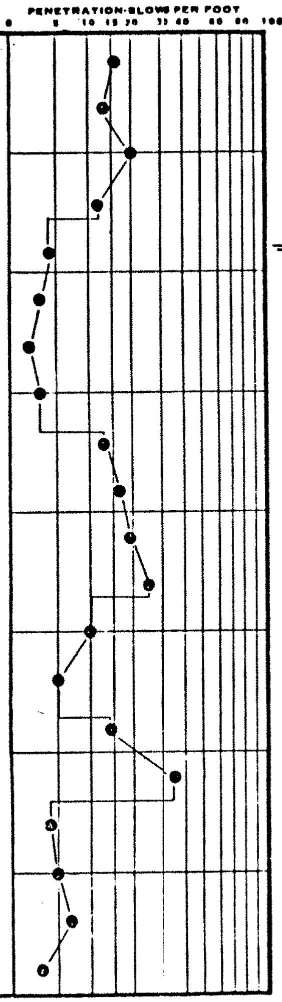
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

STABILITY AND LIQUEFACTION POTENTIAL EVALUATION AT UHS BARRIER

FIGURE 2C-31

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
+12.9		TAN FINE TO COARSE SHELLY SAND WITH ROCK FRAGMENTS (FILL)
+ 7.9		
	7.5	TAN FINE SLIGHTLY SHELLY SAND (FILL)
+ 2.9	9.5	TAN - GRAY MEDIUM TO FINE SAND WITH SHELL FRAGMENTS AND NODULES
- 2.1		
	16.5	GRAY SLIGHTLY SHELLY SAND WITH SOME CEMENTED ROCK FRAGMENTS
- 7.1		
	23.5	GRAY MEDIUM TO FINE SAND WITH SOME SHELL FRAGMENTS
-12.1		
	28.5	GRAY FINE TO COARSE SHELLY SAND WITH CEMENTED ROCK FRAGMENTS
-17.1		
	32.0	DARK GRAY-GREEN CLAY WITH THIN SANDY SEAMS
-22.1		
	35.8	DARK GRAY-GREEN SILTY, SLIGHTLY CLAYEY SAND WITH FINE SAND, AND SILTY SEAMS
-27.1		



REMARKS:
WATER TABLE ON 10/29/74

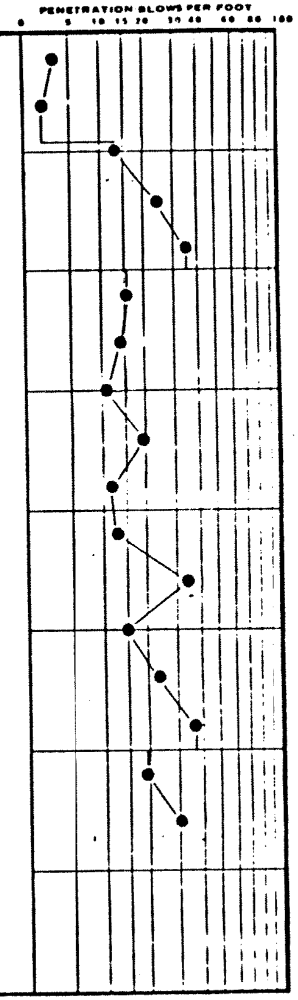
DRILLED BY	LITCO/ F-1500	BORING NUMBER	AE-1
LOGGED BY	GAW	DATE STARTED	10/24/74
CHECKED BY	GAW	DATE COMPLETED	10/25/74
		JOB NUMBER	SA-737
		COORDINATES	N 824.5 E 602.6

DRILLED WITH AW ROD AND 3 7/8" SIDE DISCHARGE DRAG BIT
PAGE 1 OF 2 PAGES

FIG 2G-A1

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
-27.1		GRAY MEDIUM TO FINE SAND WITH SHELL FRAGMENTS, SOME CLAY
-32.1	44.5	GRAY COARSE TO FINE SAND AND SHELL FRAGMENTS AND NODULES
-37.1	50.0	GRAY COARSE TO FINE SAND WITH SHELL FRAGMENTS AND NODULES
-42.1		
-47.1		
-52.1		
-57.1	70.0	GRAY FINE SAND
	73.5	BORING TERMINATED
-62.1		



REMARKS:

DRILLED WITH AW ROD AND 3 7/8" SIDE DISCHARGE DRAG BIT

DRILLED BY	LITCO/F-1500	BORING NUMBER	AE-1
LOGGED BY	GAW	DATE STARTED	[illegible]
CHECKED BY	GAW	DATE COMPLETED	10 25 74
			SA-737

FIG 2G-A1 Cont.

PAGE 2 OF 2 PAGES

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	S#	R	PENETRATION BLOWS PER FOOT (0 5 10 15 20 30 40 50 60 80 100)
+13.2	0				
+ 8.2					
+ 3.2		WASH DRILL TO 32.0 FEET			
- 1.8					
- 6.8					
-11.8					
-16.8					
	32.0				
	32.9	DARK GRAY-GREEN CLAY (1)	UD1	0.9	
	34.0	WASH DRILL TO 34.0 FEET			
-21.8	36.0	DARK GRAY-GREEN CLAY INTO FINE SAND	UD2	2.0	
		BORING TERMINATED			
-26.8		(1) WITH SANDY SHELL SEAMS			

REMARKS:

S# = UNDISTURBED SAMPLE NUMBER
R = RECOVERY (FEET)
PISTON SAMPLE

DRILLED BY LETCO/F-1500
LOGGED BY G. A. W.
CHECKED BY G. A. W.

BORING NUMBER AE-1A
DATE STARTED 10/26/74
DATE COMPLETED 10/26/74
JOB NUMBER SA-737
COORDINATES N 836.9
E 603.3

DRILLED WITH AW ROD AND 3 7/8" SIDE DISCHARGE DRAG BIT

FIG 2G-A2

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	S#	R		PENETRATION BLOWS PER FOOT (0 5 10 15 20 30 40 50 60 80 100)
+12.8		WASH DRILL TO 8.0 FEET				
+ 7.8						
	8.0					
	9.0	TAN FINE SAND WITH ROCK FRAGMENTS	0	1.0	JS	
+ 2.8	10.0	WASH DRILL TO 10.0 FEET				
	12.0	NO RECOVERY				
	14.0	TAN VERY FINE SANDY SILT	T-2	1.0		
- 2.2	16.0	TAN SILTY FINE SAND				
	18.0	TAN SILTY FINE SAND	T-4	1.0		
		WASH DRILL TO 32.0 FEET				
-17.2						
	32.0					
		GRAY FINE SAND INTO DARK GRAY FINE SANDY SILTY CLAY	0	0.3	JS	
-22.2	36.0					
	38.0	GRAY SILT AND FINE SAND	T-7	1.5		
-27.2	40.0	GRAY SILTY SHELLY FINE SAND				
	42.0	GRAY SILT AND FINE SAND WITH SHELL FRAGMENTS	T-9	2.0		
	44.0	GRAY CLAYEY FINE SAND WITH TRACE OF SHELL FRAGMENTS				
-32.2	46.0	GRAY FINE SAND WITH SHELL PARTICLES, SOME CLAY	T-11	2.0		
		BORING TERMINATED				

REMARKS: 0 = OSTERBERG SAMPLE ATTEMPT
γ_t = BULK DENSITY (lb ft^3)
S# = UNDISTURBED SAMPLE NUMBER
R = RECOVERY (FEET)
JS = JAR SAMPLE RETAINED

DRILLED BY GIRDLER / F-1500
LOGGED BY CJR
CHECKED BY GAW

BORING NUMBER AE-1B
DATE STARTED 12/5/74
DATE COMPLETED 12/6/74
SA-737

PISTON SAMPLE
OSTERBERG SAMPLE

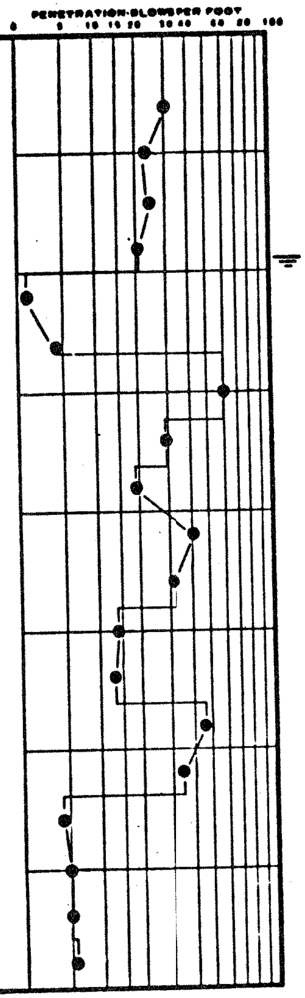
DRILLED WITH AW ROD AND 3 7/8" TRICONE ROLLER BIT. BLOW COUNTS OBTAINED USING AW ROD

FIG. 2G-A2 Cont.

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
+12.8		
+ 7.8		LIGHT BROWN TO TAN FINE TO COARSE SHELLY SAND WITH PARTIALLY CEMENTED SAND
+ 2.8	10.0	
		TAN FINE SAND WITH SHELL FRAGMENTS, SOME SILT
	13.3	
- 2.2		GRAY-BROWN FINE SAND
	16.0	
		BROWN TO GRAY COARSE TO MEDIUM SAND WITH SHELL PARTICLES
	18.0	
- 7.2		GRAY-BROWN FINE SAND WITH SILT
	24.0	
-12.2		TAN MEDIUM TO FINE SAND WITH SHELL FRAGMENTS AND NODULES, SOME SILT
	28.0	
-17.2		TAN MEDIUM TO FINE SAND WITH SHELL FRAGMENTS, LITTLE SILT
	32.0	
-22.2		GRAY CLAY, SOME FINE SAND AND SILT
	38.0	
-27.2		GRAY SILTY CLAY AND FINE SAND

PENETRATION-BLOWS PER FOOT



REMARKS:

WATER TABLE ON 11/15/74

DRILLED WITH AW ROD AND 2-15/16" SIDE DISCHARGE DRAG BIT

DRILLED BY	GIRDLER/F-250	BORING NUMBER	AE-2
LOGGED BY	M.C.M.	DATE STARTED	11/3/74
CHECKED BY	G.A.W.	DATE COMPLETED	11/4/74
		JOB NUMBER	SA-737
		COORDINATES	N 755.1 E 597.8

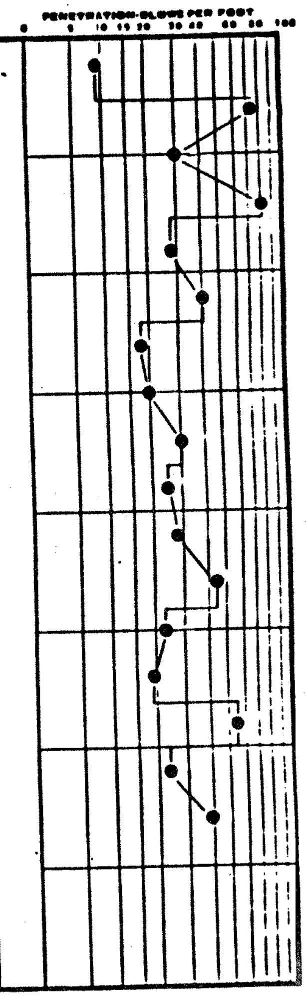
PAGE 1 OF 2 PAGES

FIG. 2G-A3

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
-27.2		GRAY MEDIUM TO FINE SAND AND SHELL FRAGMENTS, LITTLE CLAY (1)
	42.5	
-32.2		TAN MEDIUM TO FINE SAND WITH SHELL FRAGMENTS AND NODULES
	47.5	
-37.2		GRAY TO TAN SLIGHTLY SILTY PARTIALLY CEMENTED SAND
	52.0	
-42.2		TAN TO GRAY PARTIALLY CEMENTED FINE SAND WITH SOME SHELL FRAGMENTS
	58.0	
-47.2		GRAY MEDIUM TO FINE SAND WITH SHELL FRAGMENTS
	64.0	
-52.2		GRAY AND BLACK FINE SANDY MEDIUM TO COARSE SHELL FRAGMENTS
	68.0	
		GRAY AND BLACK FINE TO COARSE SHELLY SAND
-57.2	70.0	
		GRAY AND BLACK FINE TO COARSE SHELLY SAND
	73.5	
-62.2		BORING TERMINATED

PENETRATION-BLOWS PER FOOT



REMARKS: (1) OR SILT

DRILLED WITH AW ROD AND 2-15/16" SIDE DISCHARGE DRAG BIT

DRILLED BY	GIRDLER/F-250	BORING NUMBER	AE-2
LOGGED BY	M. C. M.	DATE STARTED	11/3/74
CHECKED BY	G. A. W.	DATE COMPLETED	11/4/74
		--	SA-737

PAGE 2 OF 2 PAGES

FIG 2G-A3 Cont.

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	S#	R
+12.8		WASH DRILL TO 9.0 FEET		
+ 7.8				
	9.0			
+ 2.8	11.0	NO RECOVERY	P	
	12.0	WASH DRILL TO 12.0 FEET		
	14.0	NO RECOVERY	P	
- 2.2		WASH DRILL TO 31.5 FEET		
-17.2	31.5			
	33.5	LIGHT BROWN FINE SHELLY SAND INTO DARK GRAY SANDY CLAY	UD1	1.0
	34.0	WASH DRILL TO 34.0 FEET		
-22.2	36.0	DARK GRAY SILTY TO SANDY CLAY	UD2	2.0
	38.0	DARK GRAY SILTY TO SANDY CLAY INTO CLAYEY SAND	UD3	0.5
-27.2	40.0	NO RECOVERY	P	0.3
	42.0	NO RECOVERY	P	0.2
-32.2		BORING TERMINATED		

PENETRATION-BLOWS PER FOOT: 0 5 10 15 20 30 40 60 80 100

REMARKS:
S# = UNDISTURBED SAMPLE NUMBER
P = PISTON SAMPLE ATTEMPT
R = RECOVERY (FEET)

PISTON SAMPLE

DRILLED WITH AW ROD AND 3 7/8" TRICONE ROLLER BIT

DRILLED BY GIRDLER/F-250
LOGGED BY M.C.M.
CHECKED BY G.A.W.

BORING NUMBER AE-2A
DATE STARTED 11/14/74
DATE COMPLETED 11/15/74
JOB NUMBER SA-737
COORDINATES N 760.3 E 597.6

FIG 2G-A4

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	S#	R	
+12.8					
+ 7.8		WASH DRILL TO 10.0 FEET			
+ 2.8	10.0				
	12.0	LIGHT GRAY-GREEN SILTY SHELLY FINE SAND	P		JS
	12.5	WASH DRILL TO 12.5 FEET			
	14.5	GRAY-TAN SHELLY FINE SAND INTO FINE SAND	UD2	2.0	
- 2.2	15.5	LIGHT GRAY FINE SAND INTO (1)	UD3	1.0	
		BORING TERMINATED			
- 7.2					
		(1) SHELLY SAND			

PENETRATION-BLOWS PER FOOT: 0 5 10 15 20 30 40 60 80 100

REMARKS:
S# = UNDISTURBED SAMPLE NUMBER
R = RECOVERY (FEET)
P = PISTON SAMPLE ATTEMPT

PISTON SAMPLE

JS = JAR SAMPLE RETAINED

DRILLED BY GIRDLER/F-250
LOGGED BY M.C.M.
CHECKED BY G.A.W.

BORING NUMBER AE-2B
DATE STARTED 11/15/74
DATE COMPLETED 11/16/74
SA-737
COORDINATES N 766.4 E 597.5

DRILLED WITH AW ROD AND 3-7/8" TRICONE ROLLER BIT

FIG 2G-A5

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	S#	R
+12.9				
+ 7.9		WASH DRILL TO 9.3 FEET		
+ 2.9	9.3	LIGHT GRAY SHELLY FINE SAND	UD1	0 1.0
	11.3	LIGHT GRAY SHELLY FINE SAND	UD2	0 1.0
	13.1	LIGHT GRAY-TAN SHELLY FINE SAND	UD3	1.5
- 2.1	14.5			
- 7.1		WASH DRILL		
-12.1				
-17.1				
	32.5	DARK GRAY SANDY CLAY WITH SHELL FRAGMENTS	UD4	2.0
-22.1	34.5 / 34.8	(1)		
		DARK GRAY CLAYEY FINE SAND	UD5	2.0
	36.8	BORING TERMINATED AT 36.8 FEET		
-27.1		(1) WASH DRILL TO 34.8 FEET		

PENETRATION-BLOWS PER FOOT: 0 5 10 15 20 30 40 60 80 100

REMARKS:
S# = UNDISTURBED SAMPLE NUMBER
R = RECOVERY (FEET)

DRILLED BY GIRDLER/F-250
LOGGED BY M.C.M.
CHECKED BY G.A.W.

BORING NUMBER AE-2C
DATE STARTED 11/18/74
DATE COMPLETED 11/18/74
JOB NUMBER SA-737
COORDINATES N 771.9 E 598.1

PISTON SAMPLE

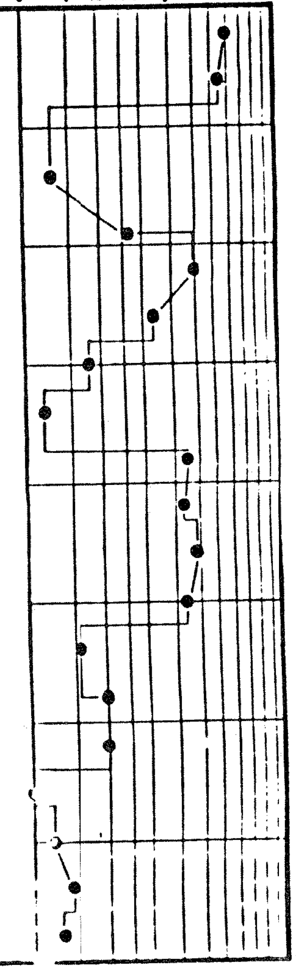
DRILLED WITH AW ROD AND 3 7/8" TRICONE ROLLER BIT

FIG. 2G-A6

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
+12.0		TAN SLIGHTLY SHELLY PARTIALLY CEMENTED FINE SAND
+ 7.0	4.0	TAN TO GRAY SILT, SOME FINE SAND
+ 2.0	9.5	TAN COARSE TO FINE SAND, SOME SHELL FRAGMENTS AND NODULES
- 3.0	14.0	TAN TO GRAY FINE SAND WITH SHELL FRAGMENTS, SOME SILT
	16.0	TAN TO GRAY MEDIUM TO FINE SAND, WITH SHELL FRAGMENTS, SOME SILT
- 8.0	18.5	TAN TO GRAY MEDIUM TO FINE SAND WITH SHELL FRAGMENTS, LITTLE SILT
-13.0	21.5	BROWN TO GRAY FINE SAND WITH PARTIALLY CEMENTED SAND AND SHELL PARTICLES
-18.0	26.0	GRAY-BROWN TO GRAY-BLACK SILTY PARTIALLY CEMENTED SLIGHTLY SHELLY TO SHELLY SAND
	32.0	GRAY ORGANIC CLAY WITH SHELL PARTICLES AND FINE SAND
-23.0	33.5	GRAY-GREEN ORGANIC CLAY
-28.0	38.0	GRAY FINE SAND WITH CLAY AND SHELL FRAGMENTS

PENETRATION-BLOWS PER FOOT: 0 5 10 15 20 30 40 60 80 100



REMARKS:
WOR = WEIGHT OF ROD
WOH = WEIGHT OF HAMMER

DRILLED WITH AW ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT

DRILLED BY GIRDLER/F-1500
LOGGED BY J. L.P.
CHECKED BY G.A.W.

BORING NUMBER AE-3
DATE STARTED 11/11/74
DATE COMPLETED 11/12/74
SA-737

COORDINATES NA

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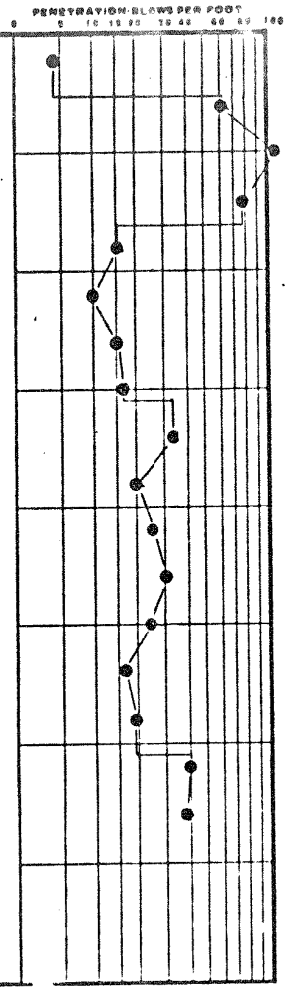
FIG. 2G-A7

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
-28.0		GRAY FINE SAND WITH SHELL FRAGMENTS
	42.5	
-33.0		GRAY MEDIUM TO FINE SAND WITH CEMENTED SAND AND SHELL FRAGMENTS
	48.0	
-38.0		TAN FINE SAND WITH CEMENTED SAND AND SHELL FRAGMENTS
-43.0	55.5	
-48.0		TAN COARSE TO FINE SAND WITH SHELL FRAGMENTS AND NODULES
-53.0	66.0	
		GRAY COARSE TO FINE SAND WITH SHELL FRAGMENTS
-58.0	70.5	
		GRAY COARSE TO FINE SAND WITH SHELL FRAGMENTS, LITTLE SILT
	73.5	
-63.0		BORING TERMINATED



REMARKS:

DRILLED WITH AW ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT

DRILLED BY GIRDLER/F-1500
LOGGED BY J.L.P.
CHECKED BY G.A.W.

BORING NUMBER AE-3
DATE STARTED 11/11/74
DATE COMPLETED 11/12/74
JOB NUMBER SA-737
PAGE 2 OF 2 PAGES

FIG 2G-A7 Cont.

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	S#
+12.0			
+ 7.0			
+ 2.0			
- 3.0		WASH DRILL TO 32.0 FEET	
- 8.0			
-13.0			
-18.0			
	32.0		
		NO RECOVERY	P
	34.0		
-23.0		DARK GRAY ORGANIC SILTY CLAY	T-1
	36.0		
		DARK GRAY ORGANIC SILTY CLAY	T-2
	38.0		
		DARK GRAY ORGANIC SILTY CLAY	T-3
-28.0	40.0		

PENETRATION-BLOWS PER FOOT

REMARKS: BORING TERMINATED AT 42.0 FEET

WATER TABLE ON 11/15/74

S# = UNDISTURBED SAMPLE NUMBER
P = PISTON SAMPLE ATTEMPT

PISTON SAMPLE

DRILLED WITH AW ROD AND 3 7/8" SIDE DISCHARGE DRAG BIT

DRILLED BY GIRDLER/F-1500
LOGGED BY J.L.P.
CHECKED BY G.A.W.

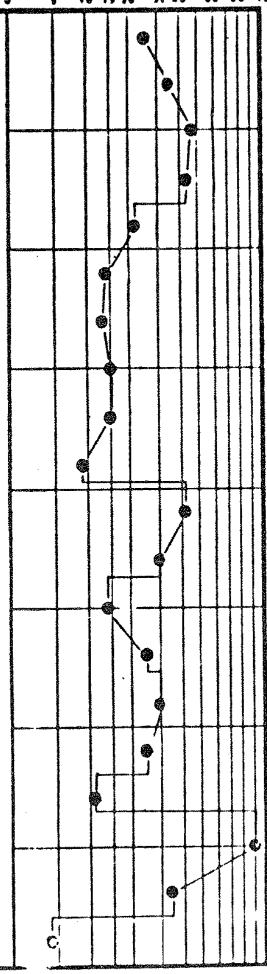
BORING NUMBER AE-3A
DATE STARTED 11/12/74
DATE COMPLETED 11/12/74
JOB NUMBER SA-737

FIG 2G-A8

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
+18.0		TAN FINE TO COARSE SLIGHTLY SHELLY SAND WITH ROCK FRAGMENTS
+13.0		
	8.0	TAN FINE SAND, LITTLE SILT
+ 8.0		
	14.0	GRAY FINE SAND, SOME SILT
+ 3.0		
	19.8	GRAY FINE SAND, LITTLE SILT
- 2.0		
	23.8	GRAY-GREEN SLIGHTLY SILTY FINE SAND
- 7.0		
	27.8	LIGHT GRAY-GREEN FINE PARTIALLY CEMENTED FINE SAND
-12.0		
	32.0	LIGHT GRAY-GREEN SILTY CLAY
	33.5	GRAY-BLACK FINE SAND
-17.0		
	38.0	DARK GRAY-GREEN CLAY WITH SILTY SHELLY SAND SEAMS
-22.0		

PENETRATION-BLOWS PER FOOT



REMARKS:

WATER TABLE ON 11/7/74

DRILLED BY LETCO/F-1500
LOGGED BY G.A.W.
CHECKED BY G.A.W.

BORING NUMBER AE-4
DATE STARTED 10/26/74
DATE COMPLETED 10/29/74
JOB NUMBER SA-737
COORDINATES N 828.2 E 509.9

DRILLED WITH AW ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT

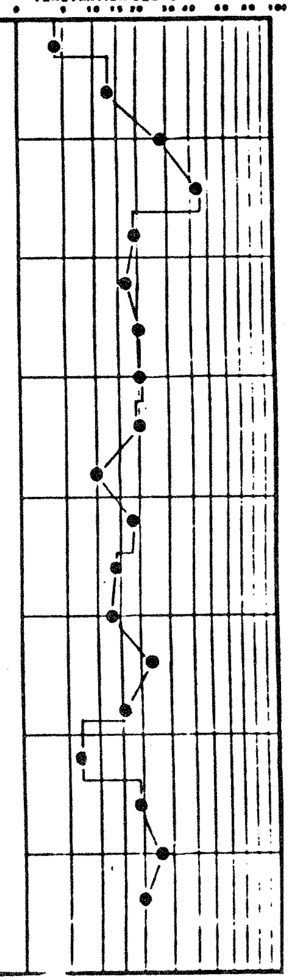
PAGE 1 OF 2 PAGES

FIG 2G-A9

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
-22.0	40.2	GRAY-GREEN SLIGHTLY SANDY CLAYEY SILT
	41.5	GRAY SLIGHTLY SILTY SLIGHTLY SHELLY FINE TO MEDIUM SAND
-27.0	44.5	GRAY-BLACK SHELLY FINE TO MEDIUM SAND
	48.0	6" CEMENTED LIMESTONE LENSE
-32.0		GRAY COARSE TO FINE SAND AND SHELL FRAGMENTS, LITTLE SILT
	52.0	TAN-GRAY PARTIALLY CEMENTED FINE SAND
-37.0	56.0	3" CEMENTED LIMESTONE LENSE
-42.0		TAN PARTIALLY CEMENTED FINE SAND
	62.5	GRAY COARSE TO FINE SAND AND SHELL FRAGMENTS WITH NODULES, LITTLE SILT
-47.0		
	69.5	GRAY COARSE TO FINE SAND WITH SHELL FRAGMENTS AND NODULES
-52.0	72.0	GRAY-GREEN SHELLY FINE TO M MEDIUM SAND
-57.0		
	77.5	BORING TERMINATED
-62.0		

PENETRATION-BLOWS PER FOOT



REMARKS:

DRILLED WITH AW ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT

DRILLED BY LETCO/F-1500
LOGGED BY G.A.W.
CHECKED BY G.A.W.

BORING NUMBER AE-4
DATE STARTED 10/26/74
DATE COMPLETED 10/29/74
JOB NUMBER SA-737
PAGE 2 OF 2 PAGES

FIG 2G-A9 Cont.

TEST BORING RECORD

Elevation scale (ELEV.): +18.0, + 8.0, + 3.0, - 2.0, - 7.0, -12.0, -17.0, -22.0, -27.0

PENETRATION-BLOWS PER FOOT: 0, 5, 10, 15, 20, 30, 40, 60, 80, 100

DEPTH FEET	DESCRIPTION	S#
0 – 12.0	WASH DRILL TO 12.0 FEET	
12.0 – 14.0	TAN FINE SAND	T-1
14.0 – 16.0	WASH DRILL TO 16.0 FEET	
16.0 – 18.0	LIGHT GRAY-BROWN FINE SAND	T-2
18.0 – 24.0	WASH DRILL TO 24.0 FEET	
24.0 – 26.0	TAN FINE SAND, AND SILT	T-3
26.0 – 32.0	WASH DRILL TO 32.0 FEET	
32.0 – 34.0	GRAY FINE SAND	T-4
34.0 – 36.0	NO RECOVERY	P
36.0 – 38.0	WASH DRILL TO 38.0 FEET	
38.0 – 40.0	DARK GRAY SLIGHTLY SANDY CLAY WITH SHELL FRAGMENTS	T-6
40.0 – 42.0	DARK GRAY SANDY CLAY WITH SHELL FRAGMENTS	T-7
42.0 – 44.0	DARK GRAY-BLACK SLIGHTLY SANDY CLAY WITH SHELL FRAGMENTS	T-8

REMARKS:

S# = UNDISTURBED SAMPLE NUMBER
P = PISTON SAMPLE ATTEMPTED

PISTON SAMPLE

DRILLED WITH AW ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT

DRILLED BY GIRDLER/F-1500
LOGGED BY J.L.P.
CHECKED BY G.A.W.

BORING NUMBER AE-4A
DATE STARTED 11/8/74
DATE COMPLETED 11/8/74
JOB NUMBER SA-737
COORDINATES N 828.1
E 499.4

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FIG 2C–A10

TEST BORING RECORD

Elevation scale (ELEV.): -27.0, -32.0, -37.0, -42.0, -47.0, -52.0, -57.0

PENETRATION-BLOWS PER FOOT

DEPTH FEET	DESCRIPTION	S#
– 58.0	WASH DRILLED TO 58.0 FEET	
58.0 – 60.0	LIGHT TAN SLIGHTLY SILTY SAND	T-9
60.0 – 64.0	WASH DRILL TO 64.0 FEET	
64.0 – 66.0	GRAY COARSE TO FINE SAND WITH SHELL PARTICLES	T-10
66.0 – 68.0	WASH DRILL TO 68.0 FEET	
68.0 – 70.0	NO RECOVERY	P
70.0 – 72.0	WASH DRILL TO 72.0 FEET	
72.0 – 74.0	NO RECOVERY	P
74.0	BORING TERMINATED	

REMARKS:

S# = UNDISTURBED SAMPLE NUMBER
P = PISTON SAMPLE ATTEMPTED

PISTON SAMPLE

DRILLED BY GIRDLER/F-1500
LOGGED BY J.L.P.
CHECKED BY G.A.W.

BORING NUMBER AE-4A
DATE STARTED 11/8/74
DATE COMPLETED 11/8/74
JOB NUMBER SA-737

DRILLED WITH AW ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT

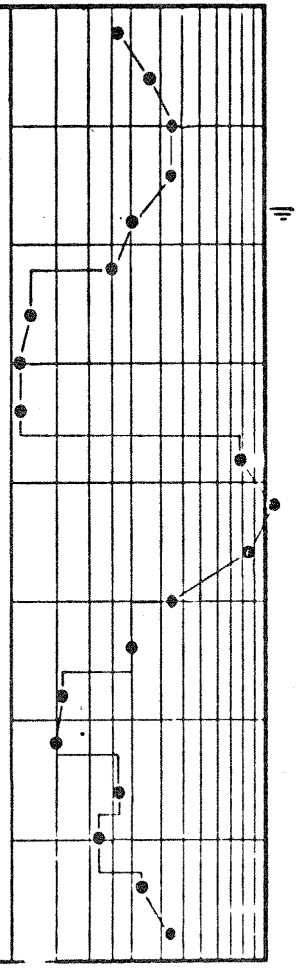
PAGE 2 OF 2 PAGES

FIG 2C–A10 Cont.

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
+18.0		
+13.0		TAN FINE TO COARSE SAND WITH ROCK FRAGMENTS
+ 8.0	9.5	
		LIGHT GRAY-GREEN FINE SAND
	11.0	
+ 3.0		GRAY-GREEN SILT, SOME FINE SAND WITH SHELL FRAGMENTS
	18.0	
- 2.0		GRAY-GREEN COARSE TO FINE SAND, WITH SHELL AND ROCK FRAGMENTS
- 7.0	25.0	
		GRAY-GREEN SANDY SILT
	28.0	
-12.0		GRAY-GREEN CLAYEY SILT AND CLAY WITH SAND SEAMS
	31.5	
		DARK GRAY-GREEN FINE SAND
	34.0	
-17.0		DARK GRAY-GREEN SLIGHTLY SANDY CLAY
	36.5	
		GRAY-GREEN SHELLY FINE SAND
	38.5	
-22.0		

PENETRATION-BLOWS PER FOOT: 0, 5, 10, 15, 20, 30, 40, 60, 80, 100



REMARKS:

WATER TABLE ON 11/11/74

DRILLED WITH N ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT

DRILLED BY	GIRDLER/F-1500
LOGGED BY	G.A.W.
CHECKED BY	G.A.W.
BORING NUMBER	AE-5
DATE STARTED	11/5/74
DATE COMPLETED	11/7/74
JOB NUMBER	SA-737
COORDINATES	N 516.5 E 347.0

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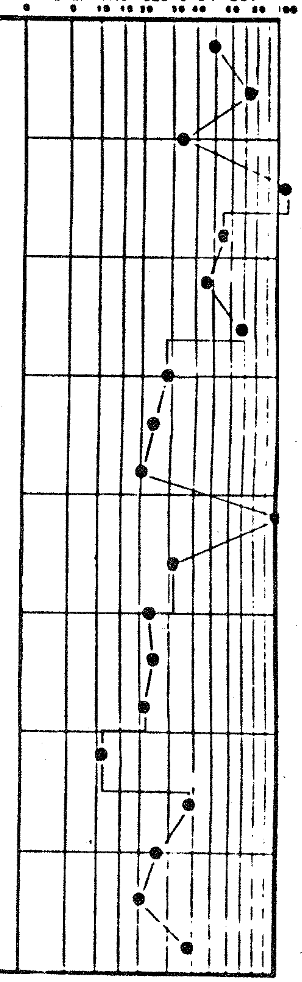
FIG 2G-A11

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
-22.0		GRAY-GREEN SHELLY PARTIALLY CEMENTED FINE TO MEDIUM SAND
-27.0		
	48.0	3" CEMENTED LIMESTONE LENSE
-32.0		GRAY-GREEN PARTIALLY CEMENTED FINE SAND
	53.5	
-37.0		
-42.0		LIGHT TAN-GREEN PARTIALLY CEMENTED FINE SAND
-47.0	65.5	
		GREEN-BLACK PARTIALLY CEMENTED FINE TO COARSE SHELLY SAND
-52.0	70.0	
		GRAY SLIGHTLY CLAYEY SLIGHTLY SHELLY PARTIALLY CEMENTED FINE (1)
	72.5	
-57.0		BLACK SPECKLED MEDIUM TO FINE SAND WITH SHELL FRAGMENTS
-62.0		

PENETRATION-BLOWS PER FOOT: 0, 5, 10, 15, 20, 30, 40, 60, 80, 100



REMARKS: (1) TO COARSE SAND

DRILLED WITH N ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT

DRILLED BY	GIRDLER/F-1500
LOGGED BY	G.A.W.
CHECKED BY	G.A.W.
BORING NUMBER	AE-5
DATE STARTED	11/5/74
DATE COMPLETED	11/7/74
JOB NUMBER	SA-737

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FIG 2G-A11 Cont.

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	PENETRATION-BLOWS PER FOOT
-62.0	81.5	GRAY SHELLY FINE TO MEDIUM SAND	
		BORING TERMINATED	
-67.0			

REMARKS:

DRILLED BY GIRDLER/F-1500
LOGGED BY G.A.W.
CHECKED BY G.A.W.

BORING NUMBER AE-5
DATE STARTED 11/5/74
DATE COMPLETED 11/7/74
JOB NUMBER SA-737

DRILLED WITH N ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT

PAGE 3 OF 3 PAGES

FIG 2G-A11 Cont.

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	S#	R	PENETRATION-BLOWS PER FOOT
+18.0					
+13.0					
+ 8.0		WASH DRILL TO 25.0 FEET			
+ 3.0					
- 2.0					
- 7.0	25.0	NO RECOVERY	P		
	27.4	GREEN SILT WITH SAND SEAMS	UD-1		
-12.0	29.9 / 30.0	GRAY-GREEN FINE SAND WITH SILT SEAMS	UD-2		
	32.4	NO RECOVERY	P		
-17.0	34.9 / 35.0	GRAY-GREEN SANDY SILTY AND VERY SANDY CLAY	UD-3		
	37.4	BORING TERMINATED			
-22.0					

REMARKS:
S# = UNDISTURBED SAMPLE NUMBER
R = RECOVERY (FEET)
P = PISTON SAMPLE ATTEMPTED
▨ PISTON SAMPLE

DRILLED BY GIRDLER
LOGGED BY GAW
CHECKED BY GAW

BORING NUMBER AE-5A
DATE STARTED 11/6/74
DATE COMPLETED 11/6/74
JOB NUMBER SA-737
COORDINATES N 516.5
E 343.0

DRILLED WITH N ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT

FIG 2G-A12

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	S#	R
+18.0				
+13.0		WASH DRILL TO 10.0 FEET		
+ 8.0	10.0	TAN SILTY FINE SAND	T-1	2.0
	12.0	TAN SILTY FINE SAND	T-2	2.0
+ 3.0	14.0	TAN SILTY FINE SAND WITH SHELL FRAGMENTS	T-3	2.0
	16.0	TAN SLIGHTLY SILTY FINE SAND	T-4	2.0
	18.0			
- 2.0		WASH DRILL TO 28 FEET		
- 7.0				
	28.0	GRAY FINE SAND WITH SHELL FRAGMENTS	T-5	2.0
-12.0	30.0	GRAY FINE SAND WITH SHELL FRAGMENTS	T-6	2.0
	32.0	GRAY SLIGHTLY SILTY SLIGHTLY SHELLY FINE SAND INTO SLIGHTLY CLAYEY (1)	T-7	2.0
-17.0	34.0	GRAY SLIGHTLY CLAYEY FINE SAND WITH SHELL FRAGMENTS	T-8	2.0
	36.0	BORING TERMINATED		

PENETRATION-BLOWS PER FOOT: 0 5 10 15 20 30 40 60 80 100

REMARKS:
γ_t = BULK DENSITY (lb/ft^3)
S# = UNDISTURBED SAMPLE NUMBER
R = RECOVERY (FEET)

DRILLED BY GIRDLER/F-1500
LOGGED BY CJR
CHECKED BY GAW

BORING NUMBER AE-5B
DATE STARTED 11/26/74
DATE COMPLETED 12/2/74
JOB NUMBER SA-737
COORDINATES NA

DRILLED WITH N ROD AND 3-7/8" TRICONE ROLLER BIT

OSTERBERG SAMPLE

FIG 2G-A13

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	S#	R
+18.0				
+13.0		WASH DRILL TO 10 FEET		
+ 8.0	10.0	TAN FINE SAND, LITTLE SILT	T-1	2.0
	12.0	TAN SILTY FINE SAND		
+ 3.0	14.0	TAN-GRAY FINE SAND WITH SHELLS, SOME SILT	T-3	2.0
	16.0	TAN SILTY FINE SAND		
	18.0	TAN SILTY FINE SAND INTO TAN AND GRAY FINE SAND PARTIALLY CEMENTED		
- 2.0	20.0	BORING TERMINATED		

PENETRATION-BLOWS PER FOOT: 0 5 10 15 20 30 40 60 80 100

REMARKS:
γ_t = BULK DENSITY (lb/ft^3)
S# = UNDISTURBED SAMPLE NUMBER
R = RECOVERY (FEET)

DRILLED BY GIRDLER/F-1500
LOGGED BY CJR
CHECKED BY GAW

BORING NUMBER AE-5C
DATE STARTED 12/2/74
DATE COMPLETED 12/2/74
JOB NUMBER SA-737
COORDINATES N 515.5
E 353.0

DRILLED WITH AW ROD AND 3-7/8" TRICONE ROLLER BIT
BLOW COUNTS OBTAINED USING AW ROD

OSTERBERG SAMPLE

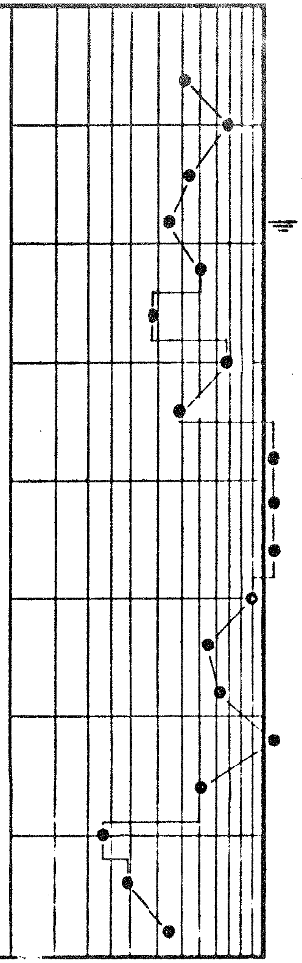
WOR = WEIGHT OF ROD

FIG. 2G-A14

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
NA		TAN TO BROWN SLIGHTLY SHELLY TO SHELLY FINE SAND
	12.0	LIGHT BROWN SLIGHTLY SHELLY FINE SAND
	14.0	GRAY-TAN FINE SAND
	17.5	GRAY-BROWN FINE SAND WITH SHELL FRAGMENTS
	24.0	GRAY MEDIUM TO FINE SAND WITH SHELL FRAGMENTS
	34.5	DARK GRAY CLAY, SOME FINE SAND WITH SHELL FRAGMENTS
	36.0	GRAY MEDIUM TO FINE SAND WITH SHELL FRAGMENTS, SOME CLAY
	40.0	

PENETRATION-BLOWS PER FOOT



REMARKS:

WATER TABLE ON 11/15/74

DRILLED WITH AW ROD, 0 TO 43.5'-3 7/8" SIDE DISCHARGE DRAG BIT
43.5 TO 81.5'-2 15/16" SIDE DISCHARGE DRAG BIT

DRILLED BY GIEDLER/F-250
LOGGED BY MEK
CHECKED BY GAW

BORING NUMBER AE-6
DATE STARTED 11/8/74
DATE COMPLETED 11/13/74
JOB NUMBER SA-737
COORDINATES NA

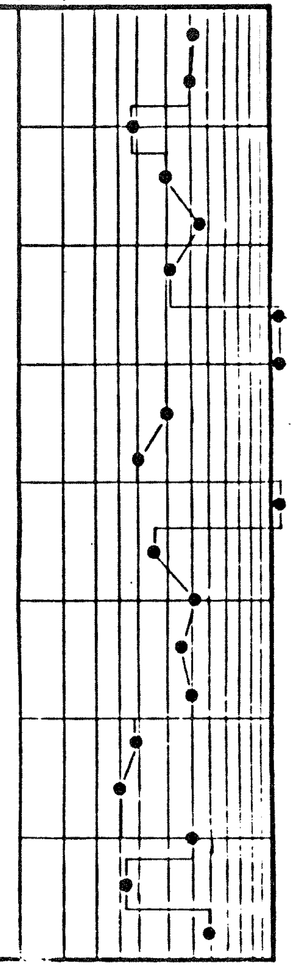
PAGE 1 OF 3 PAGES

FIG 2G-A15

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
	40.0	GRAY SLIGHTLY CLAYEY TO CLAYEY FINE SAND
NA	44.0	TAN FINE SAND WITH SHELL FRAGMENTS AND NODULES, SOME SILT
	46.0	GRAY FINE SAND WITH CEMENTED ROCK AND SHELL FRAGMENTS, SOME SILT
	52.5	GRAY BROWN CEMENTED SHELLY SAND
	55.0	GRAY-GREEN PARTIALLY CEMENTED SILTY FINE SAND
	60.0	GRAY-BROWN PARTIALLY CEMENTED SAND WITH QUARTZ FRAGMENTS
	62.0	GRAY-BROWN PARTIALLY CEMENTED SHELLY SAND, SOME SILT
	66.0	GRAY-BROWN PARTIALLY CEMENTED SHELLY SILTY FINE SAND
	70.0	GRAY COARSE TO FINE SAND WITH SHELL FRAGMENTS AND NODULES
	75.0	[illegible] CLAY SILTY SHELLY FINE SAND
	76.0	GRAY MEDIUM TO FINE SAND WITH SHELL FRAGMENTS
	78.0	GRAY FINE SAND WITH SHELL FRAGMENTS

PENETRATION-BLOWS PER FOOT



REMARKS:

DRILLED WITH AW ROD:
0 TO 43.5'-3 7/8" SIDE DISCHARGE DRAG BIT
43.5' TO 81.5'-2 15/16" SIDE DISCHARGE DRAG BIT

DRILLED BY GIEDLER/F-250
LOGGED BY MEK
CHECKED BY GAW

BORING NUMBER AE-6
DATE STARTED 11/8/74
DATE COMPLETED 11/13/74
JOB NUMBER SA-737

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FIG 2G-A15 Cont.

TEST BORING RECORD

ELEV	DEPTH FEET	DESCRIPTION
NA	81.5	GRAY SLIGHTLY SHELLY FINE SAND
		BORING TERMINATED

REMARKS:

DRILLED WITH AW ROD:
0 TO 43.5'-3 7/8" SIDE DISCHARGE DRAG BIT
43.5' TO 81.5'-2 15/16" SIDE DISCHARGE DRAG BIT

DRILLED BY GIEDLER/F-150
LOGGED BY MEK
CHECKED BY GAW

BORING NUMBER AE-6
DATE STARTED 11/8/74
DATE COMPLETED 11/13/74
JOB NUMBER SA-737

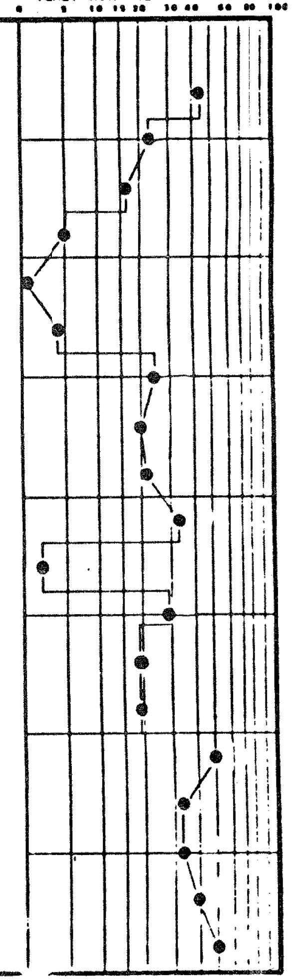
PAGE 3 OF 3 PAGES

FIG 2G-A15 Cont.

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
NA		LIGHT GRAY SLIGHTLY SILTY FINE TO MEDIUM SAND
	4.0	GRAY SLIGHTLY SHELLY SLIGHTLY SILTY FINE TO MEDIUM SAND
	8.0	GRAY SHELLY SLIGHTLY SILTY COARSE TO FINE SAND
	14.0	GRAY COARSE TO FINE SAND WITH SHELL FRAGMENTS
	22.0	DARK GRAY SHELLY FINE TO MEDIUM SAND
	24.0	TAN COARSE TO FINE SAND WITH SHELL FRAGMENTS AND NODULES
	25.5	LIGHT GREEN SILTY CLAYEY FINE SAND WITH CEMENTED SAND FRAGMENTS
	30.0	LIGHT GREEN FINE SILTY SAND WITH CEMENTED SAND FRAGMENTS
	40.0	

PENETRATION-BLOWS PER FOOT



REMARKS:

WOH = WEIGHT OF HAMMER

DRILLED WITH AW ROD AND 3½" TOP DISCHARGE DRAG BIT

DRILLED BY GIEDLER/F-250
LOGGED BY ER
CHECKED BY GAW

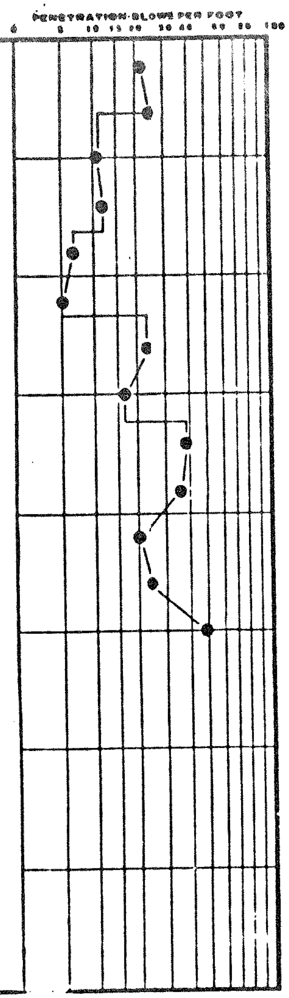
BORING NUMBER AE-7
DATE STARTED 12/2/74
DATE COMPLETED 12/3/74
JOB NUMBER SA-737
COORDINATES NA

PAGE 1 OF 2 PAGES

FIG 2G-A16

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
NA		GREEN MEDIUM TO FINE SILTY SAND WITH CEMENTED SAND FRAGMENTS
	43.0	LIGHT GREEN FINE PARTIALLY CEMENTED SAND
	48.0	TAN COARSE TO FINE SAND, WITH SHELL FRAGMENTS
	51.5	GRAY COARSE TO FINE SAND, SOME CEMENTED SAND AND SHELL FRAGMENTS, SOME SILT
	56.0	GRAY-GREEN SHELLY PARTIALLY CEMENTED FINE SAND
	65.5	BORING TERMINATED



REMARKS:

DRILLED BY GIRDLER/F-250
LOGGED BY ER
CHECKED BY GAW

BORING NUMBER AE-7
DATE STARTED 12/2/74
DATE COMPLETED 12/3/74
JOB NUMBER SA-737

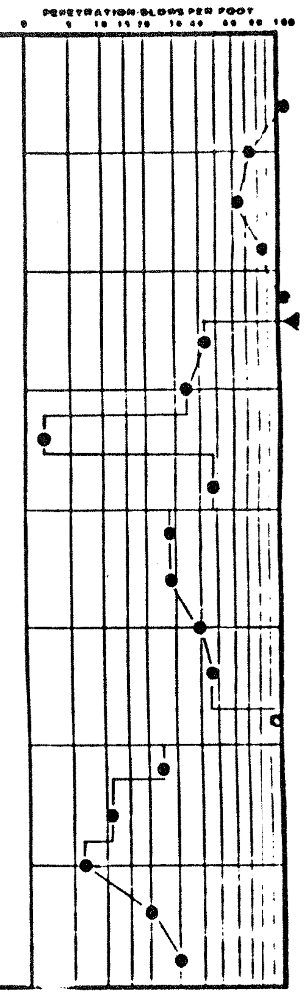
DRILLED WITH AW ROD AND 3½" TOP DISCHARGE DRAG BIT

PAGE 2 OF 2 PAGES

FIG 2G-A16 Cont.

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
NA		TAN TO GRAY FINE SAND
	12.0	GRAY SHELLY FINE SAND WITH TRACE OF ORGANIC MATTER
	16.0	GRAY SHELLY SAND
	17.5	GRAY FINE SHELLY SAND WITH ORGANIC MATTER
	20.0	GRAY COARSE TO FINE SHELLY SAND
	28.5	GRAY FINE SAND
	30.0	GRAY COARSE TO MEDIUM SAND WITH SHELL FRAGMENTS
	31.5	GRAY COARSE TO FINE SAND WITH SHELL FRAGMENTS, TRACE CLAY
	34.0	GRAY MEDIUM TO FINE SAND AND CLAY
	38.0	



REMARKS:

◀ LOSS OF DRILLING FLUID

DRILLED BY GIRDLER/F-250
LOGGED BY CJR-GAW
CHECKED BY GAW

BORING NUMBER [illegible]
DATE STARTED [illegible]
DATE COMPLETED [illegible]
JOB NUMBER SA-737
COORDINATES NA

DRILLED WITH AW ROD AND 3 1/2" TOP DISCHARGE DRAG BIT

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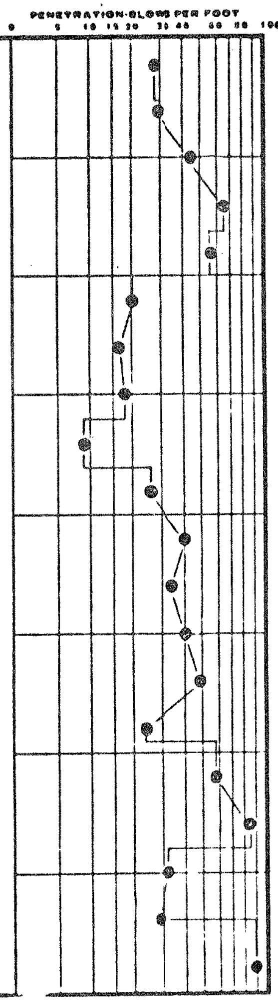
FIG 2G-A17

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
	36.0	GRAY-GREEN SILTY CLAY WITH SHELLY SAND SEAMS
NA	42.5	GRAY-BROWN TO GRAY-GREEN SLIGHTLY CLAYEY FINE SAND
	48.0	TAN SILTY PARTIALLY CEMENTED FINE SAND
	50.0	TAN PARTIALLY CEMENTED FINE SAND
	56.0	TAN FINE SAND, SOME SILT
	58.0	TAN CEMENTED SAND AND SHELL FRAGMENTS, LITTLE FINE SAND, LITTLE SILT OR CLAY
	68.0	GREEN COARSE PARTIALLY CEMENTED SHELLY SAND
	69.5	GRAY FINE SHELLY SAND
	74.0	GRAY COARSE SHELLY SAND
	77.	GRAY FINE TO MEDIUM SHELLY SAND



REMARKS:

DRILLED WITH AN ROD AND 3-1/2" TOP DISCHARGE DRAG BIT

DRILLED BY GIDDLER/F-250
LOGGED BY CJP-GAW
CHECKED BY GAW

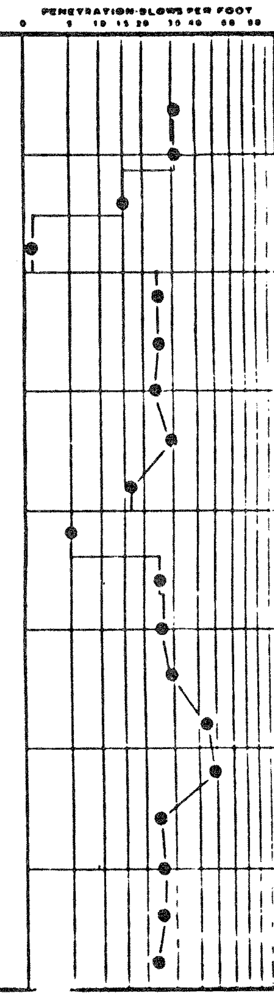
BORING NUMBER AE-8
DATE STARTED 11/26/74
DATE COMPLETED 12/3/74
JOB NUMBER SA-7??

PAGE 2 OF 2 PAGES

FIG 2G-A17 Cont.

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
+ 5.1		TAN SLIGHTLY SILTY SHELLY FINE SAND WITH ROCK FRAGMENTS (FILL)
+ 0.1		
	5.5	GRAY SLIGHTLY SHELLY FINE SAND WITH SHELL FRAGMENTS
	7.5	GRAY SILT, SOME FINE SAND
- 4.9	10.0	GRAY COARSE TO FINE SAND WITH SHELL FRAGMENTS
- 9.9		
-14.9	20.0	GRAY MEDIUM TO FINE SAND AND SHELLS, SOME CLAY
	22.0	TAN TO GRAY COARSE TO FINE SAND WITH SHELL FRAGMENTS
	23.5	GRAY TO GREEN FINE SILTY CLAYEY SAND WITH SHELL FRAGMENTS
-19.9		
-24.9		
	31.5	LIGHT GRAY-GREEN SLIGHTLY SILTY FINE SAND
-29.9		
	36.0	LIGHT GRAY TO GRAY-GREEN PARTIALLY CEMENTED SHELLY SLIGHTLY SILTY FINE SAND
-34.9	40.0	



REMARKS:

WOH = WEIGHT OF HAMMER

DRILLED WITH AW ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT

DRILLED BY GIEDLER/F-250
LOGGED BY MEK
CHECKED BY GAW

BORING NUMBER AE-1?
DATE STARTED 11/??/74
DATE COMPLETED 1?/??/74
JOB NUMBER SA-7??

COORDINATES N 129?.?
E 722.8

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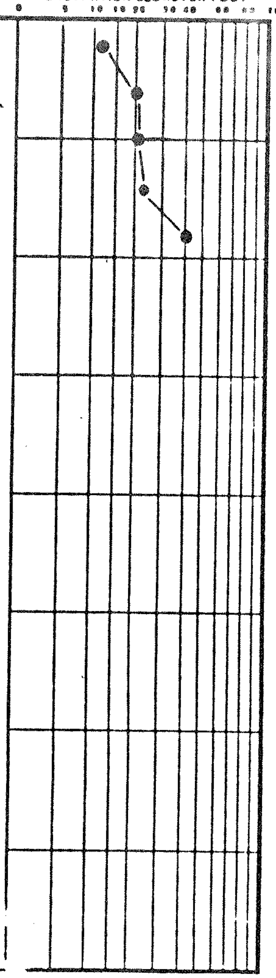
FIG 2G-A18

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
-34.9	43.	
-39.9		LIGHT GRAY TO GRAY-GREEN PARTIALLY CEMENTED SHELLY SLIGHTLY SILTY FINE SAND
	48.0	GRAY PARTIALLY CEMENTED SLIGHTLY SILTY FINE TO MEDIUM SAND
-44.9	49.5	BORING TERMINATED
-49.9		

PENETRATION-BLOWS PER FOOT



REMARKS:

DRILLED BY GIEDLER/F-250
LOGGED BY [illegible]
CHECKED BY GAW

BORING NUMBER AE-10
DATE STARTED 11/26/74
DATE COMPLETED 12/2/74
JOB NUMBER SA-737

DRILLED WITH AW ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT

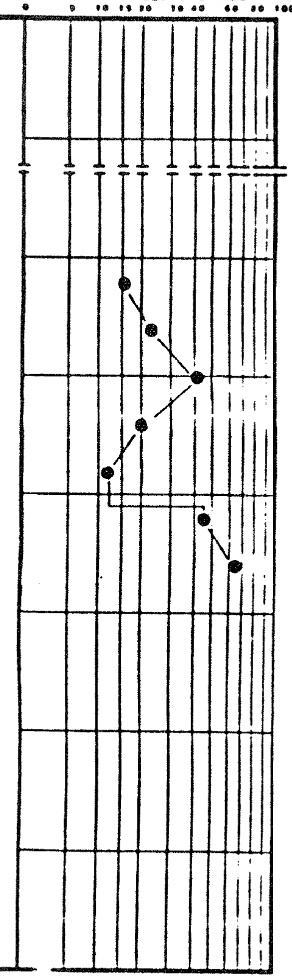
PAGE 2 OF 2 PAGES

FIG 2G-A18 Cont.

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
+ 5.0	0.0	
0.0		WASH DRILL TO 50.0 FEET
-45.0	50.0	
-50.0		GRAY SILTY SHELLY FINE SAND WITH SOME PARTIALLY CEMENTED SAND
-55.0	60.5	GRAY SILTY SHELLY FINE SAND
	63.5	
-60.0		BORING TERMINATED

PENETRATION-BLOWS PER FOOT



REMARKS:

DRILLED WITH AW ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT

DRILLED BY GIEDLER/F-250
LOGGED BY JAH
CHECKED BY GAW

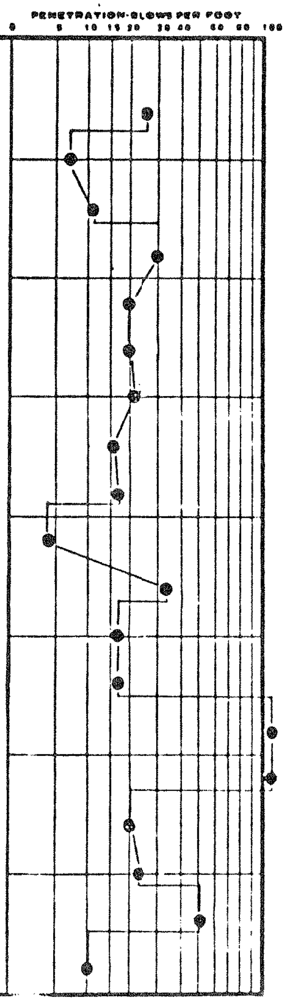
BORING NUMBER [illegible]
DATE STARTED 12/2/74
DATE COMPLETED 12/3/74
JOB NUMBER SA-737
COORDINATES N 1296.1
E 723.8

FIG 2G-A19

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
+ 6.0		GRAY SLIGHTLY SILTY FINE SAND
	3.5	GRAY SLIGHTLY SILTY FINE SAND WITH SOME ORGANIC MATTER
+ 1.0		
	7.5	GRAY SLIGHTLY SILTY SHELLY FINE SAND
- 4.0		
- 9.0		
-14.0	19.5	TAN TO GRAY COARSE TO FINE SAND AND SHELL FRAGMENTS, LITTLE SILT
	23.5	LIGHT GREEN PARTIALLY CEMENTED CLAYEY FINE SAND WITH CLAY SEAMS
-19.0		
	27.5	LIGHT GREEN PARTIALLY CEMENTED SILTY FINE SAND
-24.0		
	31.5	GRAY-GREEN SILTY FINE TO MEDIUM SAND
-29.0		
	35.5	BROWN-GREEN PARTIALLY CEMENTED SHELLY FINE SAND
	37.5	TAN COARSE TO FINE SAND AND SHELL FRAGMENTS
-34.0		

PENETRATION-BLOWS PER FOOT



REMARKS:

DRILLED BY CIESLER/F-250
LOGGED BY FR
CHECKED BY GAW

DRILLED WITH AW ROD AND 3-1/2" TOP DISCHARGE DRAG BIT

BORING NUMBER AE-11
DATE STARTED 11/25/71
DATE COMPLETED 11/25/74
JOB NUMBER SA-737
COORDINATES N 1193.2
E 693.4

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FIG 2G-A20

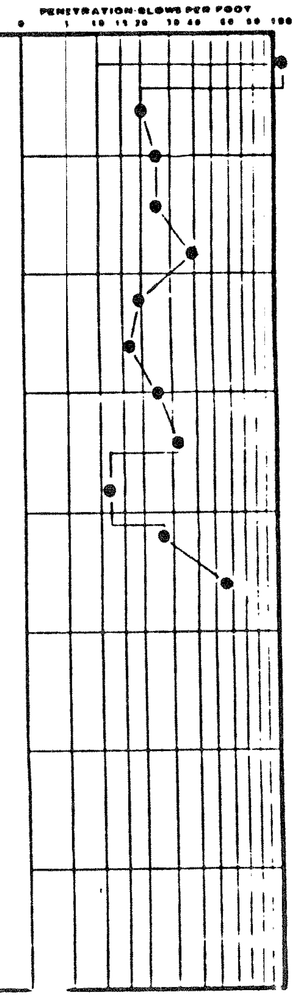
TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
-34.0	41.0	TAN TO GRAY COARSE TO FINE (1)
	42.0	GREEN PARTIALLY CEMENTED (2)
-39.0		GRAY-GREEN SHELLY FINE TO MEDIUM SAND
-44.0		
-49.0		
	57.5	TAN TO GRAY COARSE TO FINE SAND AND SHELL FRAGMENTS
-54.0		
	60.5	GRAY COARSE TO FINE SAND AND SHELL FRAGMENTS
	63.5	BORING TERMINATED
-59.0		

(1) SAND WITH SHELL FRAGMENTS

(2) FINE SAND

PENETRATION-BLOWS PER FOOT



REMARKS:

DRILLED BY CIESLER/F-250
LOGGED BY FR
CHECKED BY GAW

DRILLED WITH AW ROD AND 3-1/4" TOP DISCHARGE DRAG BIT

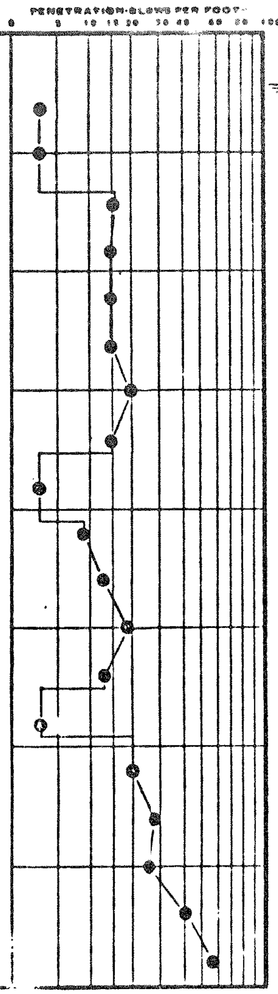
BORING NUMBER AE-11
DATE STARTED 11/25/74
DATE COMPLETED 11/25/74
JOB NUMBER SA-737

PAGE 2 OF 2 PAGES

FIG 2G-A20 Cont.

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
+ 5.2		
+ 0.2		TAN SILTY SHELLY FINE SAND WITH ROCK FRAGMENTS
	6.5	
- 4.8		GRAY SLIGHTLY SILTY SHELLY FINE SAND
- 9.8		
	17.5	GRAY COARSE TO FINE SAND AND SILT
-14.8	20.5	GRAY MEDIUM TO FINE SAND, AND SILT OR CLAY
-19.8		
	27.5	GRAY FINE SAND, SOME CLAY
-24.8	29.5	TAN MEDIUM TO FINE SAND AND SHELL FRAGMENTS
-29.8		
	36.0	LIGHT GREEN PARTIALLY CEMENTED FINE SAND
-34.8	40.0	



REMARKS:
WATER TABLE ON 11/22/74

DRILLED WITH AW ROD AND 3-1/2" TOP DISCHARGE DRAG BIT

DRILLED BY GIRDLER/F-250
LOGGED BY ER
CHECKED BY GAW

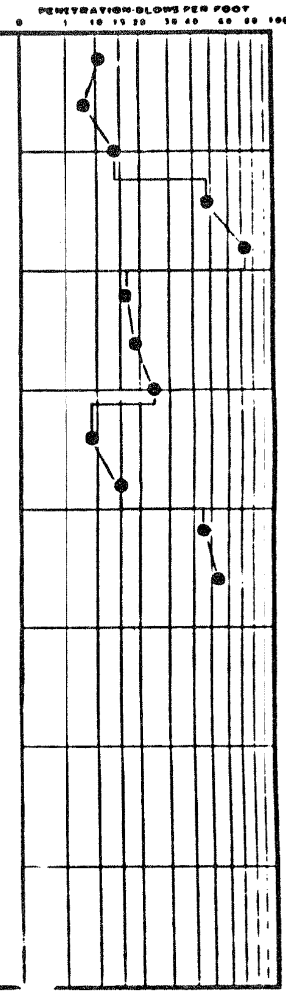
BORING NUMBER AE-12
DATE STARTED 11/20/74
DATE COMPLETED 11/22/74
JOB NUMBER SA-737
COORDINATES N 1223.2
E 616.3

PAGE 1 OF 2

FIG 2G-A21

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
-34.8	40.0	TAN COARSE TO FINE SAND WITH SHELL PARTICLES, LITTLE SILT
-39.8		
	46.0	TAN NODULES AND SHELL FRAGMENTS, SOME COARSE TO FINE SAND
-44.8	50.0	TAN-GRAY COARSE TO FINE SAND WITH SHELL PARTICLES
-49.8	55.5	GRAY COARSE TO FINE SAND WITH SHELL PARTICLES AND NODULES
-54.8	60.0	GRAY COARSE TO FINE SAND WITH SHELL FRAGMENTS
	63.5	
-59.8		BORING TERMINATED



REMARKS:

DRILLED WITH AW ROD AND 3-1/2" TOP DISCHARGE DRAG BIT

DRILLED BY GIRDLER/F-250
LOGGED BY ER
CHECKED BY GAW

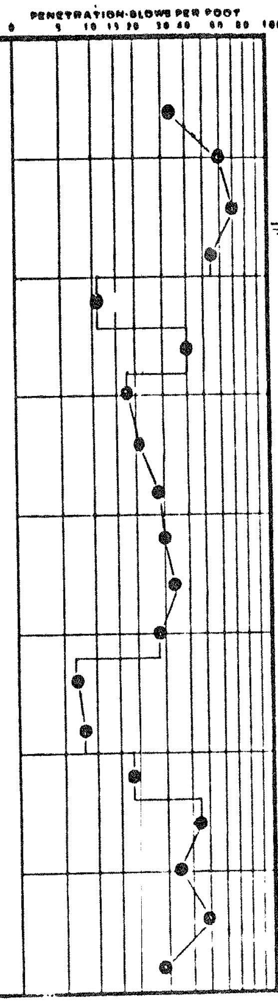
BORING NUMBER AE-12
DATE STARTED 11/20/74
DATE COMPLETED 11/22/74
JOB NUMBER SA-737

PAGE 2 OF 2 PAGES

FIG 2G-A21 Cont.

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
+10.7		GRAY FINE SLIGHTLY SHELLY SAND
+5.7		
	8.0	GRAY SHELLY MEDIUM SAND
+0.7	10.0	GRAY SHELLY MEDIUM TO COARSE SAND
	12.0	LIGHT BROWN SLIGHTLY SILTY FINE SAND
	14.0	GRAY-BLACK FINE TO COARSE SHELLY SAND
-4.3		
-9.3		
-14.3		
	26.0	GRAY COARSE TO FINE SAND AND SHELL FRAGMENTS, SOME SILT
-19.3	30.0	TAN COARSE TO FINE SAND AND SHELL PARTICLES, AND SILT AND CLAY
	32.0	TAN TO GRAY MEDIUM TO FINE SAND AND SHELLS, SOME SILT
-24.3		
-29.3		



REMARKS:
WATER TABLE ON 12/3/74

DRILLED WITH AW ROD
0-10.0' 3-7/8" TRICONE ROLLER BIT
10.0'-71.5' 2-15/16" SIDE DISCHARGE DRAG BIT

DRILLED BY GIRDLER/F-250
LOGGED BY MCH
CHECKED BY GAW

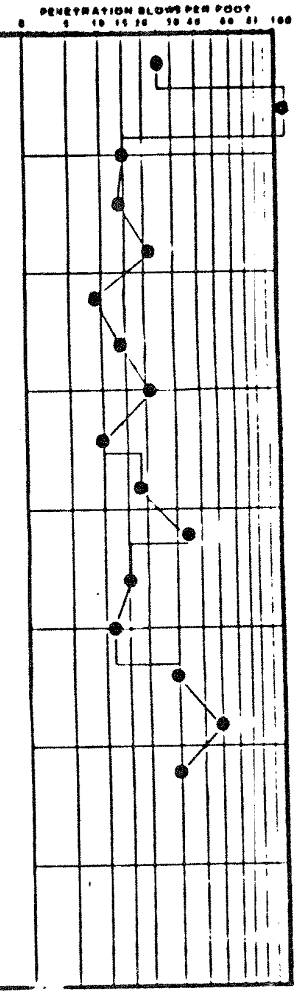
BORING NUMBER AE-13
DATE STARTED 11/22/74
DATE COMPLETED 11/25/74
JOB NUMBER SA-737
COORDINATES N 1131.6
E 716.5

PAGE 1 OF 2

FIG 2G-A22

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
-29.3		TAN PARTIALLY CEMENTED SILTY FINE SAND
	42.0	TAN PARTIALLY CEMENTED FINE TO MEDIUM SAND
-34.3	44.0	TAN FINE SAND AND PARTIALLY CEMENTED SHELLS AND SAND NODULES, SOME SILTY CLAY
-39.3		
	54.0	TAN COARSE TO FINE SAND AND SHELL PARTICLES, SOME SILT
-44.3		
	57.5	DARK GRAY SILTY SHELLY FINE TO COARSE SAND
-49.3		
	61.5	GRAY-BLACK SHELLY MEDIUM TO COARSE SAND
-54.3		
	66.5	SHELLY FINE TO COARSE SAND
-59.3		
	71.5	BORING TERMINATED
-64.3		



REMARKS:

DRILLED WITH AW ROD
0-10.0' 3-7/8" TRICONE ROLLER BIT
10.0'-71.5' 2-15/16" SIDE DISCHARGE DRAG BIT

DRILLED BY GIRDLER/F-250
LOGGED BY MCH
CHECKED BY GAW

BORING NUMBER AE-13
DATE STARTED 11/22/74
DATE COMPLETED 11/25/74
JOB NUMBER SA-737

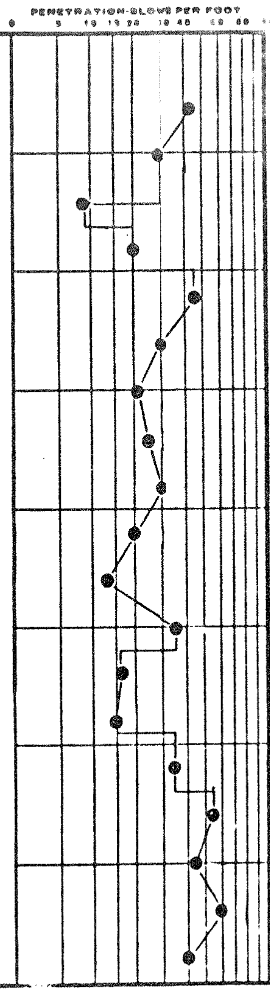
FIG 2G-A22 Cont.

PAGE 2 OF 2

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
+ 8.1		
+ 3.1		LIGHT BROWN PARTIALLY CEMENTED MEDIUM SHELLY FINE SAND
	7.0	NO RECOVERY (STUMP OR LOG *(1)
	8.0	GRAY BROWN SILTY FINE SAND
- 1.9	10.0	
- 6.9		GRAY TO BLACK COARSE TO FINE SAND AND SHELL FRAGMENTS, LITTLE SILT
-11.9		
-16.9	26.0	
		TAN COARSE TO FINE SAND AND SHELL FRAGMENTS, WITH NODULES
-21.9	29.5	TAN TO GRAY MEDIUM TO FINE SAND AND SHELL FRAGMENTS
	32.0	
-26.9		GREEN-GRAY TO GRAY SLIGHTLY SILTY FINE TO MEDIUM SAND WITH SOME PARTIALLY CEMENTED AND CEMENTED FRAGMENTS
-31.9		

PENETRATION-BLOWS PER FOOT



REMARKS: *(1) ENCOUNTERED DURING WASH)
WATER TABLE ON 11/25/74

DRILLED WITH AW ROD AND 2-15/16" SIDE DRAG BIT EXCEPT 0-10', 3-7/8" ROLLER CONE BIT USED THROUGH ROCKY FILL MATERIAL

DRILLED BY GIRDLER/F-250
LOGGED BY MCM
CHECKED BY MEK

BORING NUMBER AE-14
DATE STARTED 11/19/74
DATE COMPLETED 11/21/74
JOB NUMBER SA-737
COORDINATES N 1118.2
E 601.6

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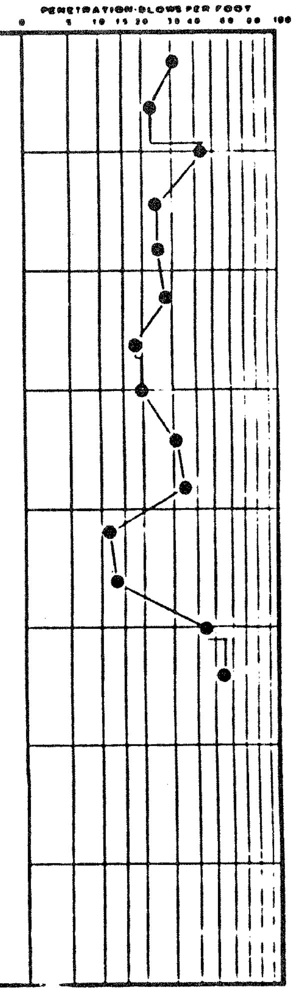
FIG 2G-A23

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
-31.9		GREEN-GRAY TO GRAY SLIGHTLY SILTY FINE TO MEDIUM SAND WITH SOME PARTIALLY CEMENTED AND CEMENTED FRAGMENTS
-36.9	45.5	
		GRAY SLIGHTLY SILTY TO SILTY FINE SAND
-41.9		
	53.5	
-46.9		
-51.9		GRAY AND BLACK COARSE TO FINE SAND AND SHELL FRAGMENTS, LITTLE SILT
-56.9	65.5	
		GRAY AND BLACK FINE SAND
	67.5	
-61.9		BORING TERMINATED

PENETRATION-BLOWS PER FOOT



REMARKS:

DRILLED WITH AW ROD AND 2-15/16" SIDE DISCHARGE DRAG BIT EXCEPT 0-10', 3-7/8" ROLLER CONE BIT USED THROUGH ROCKY FILL MATERIAL

DRILLED BY GIRDLER/F-250
LOGGED BY MCM
CHECKED BY MEK

BORING NUMBER AE-14
DATE STARTED 11/19/74
DATE COMPLETED 11/21/74
JOB NUMBER SA-737

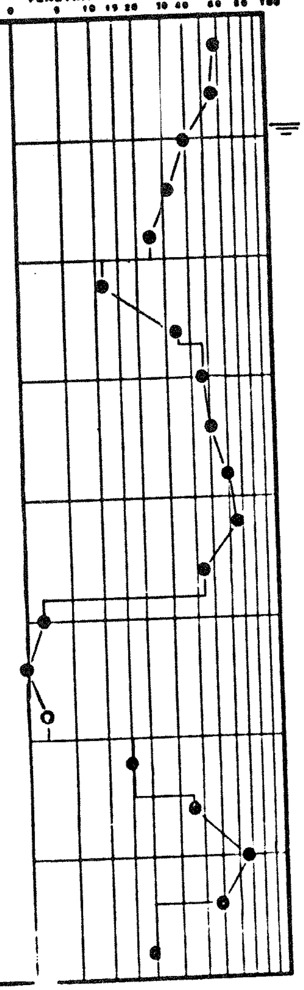
PAGE 2 OF 2

FIG 2G-A23 Cont.

TEST BORING RECORD

ELEV.	DEPTH, FEET	DESCRIPTION
+7.0		BROWN TO GRAY FINE SAND, WITH SOME CEMENTED SAND AND SHELL FRAGMENTS
+2.0		
-3.0	10.0	GRAY MEDIUM TO FINE SAND WITH SHELL FRAGMENTS, LITTLE SILT
	13.5	GRAY FINE SAND WITH SHELL FRAGMENTS
-8.0		
-13.0		
-18.0	24.0	GRAY MEDIUM TO FINE SAND, AND CLAY
-23.0	30.0	GRAY FINE SAND AND SHELL FRAGMENTS LITTLE CLAY
	32.5	GRAY FINE SAND WITH CEMENTED SAND AND SHELL FRAGMENTS
-28.6		
	37.5	LIGHT GRAY FINE SAND
-33.0	40.0	

PENETRATION-BLOWS PER FOOT



REMARKS:
WATER TABLE ON 11/14/74

W/H = WEIGHT OF HAMMER
DRILLED WITH AW ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT

DRILLED BY GIRDLER/F-250
LOGGED BY MEK
CHECKED BY GAN

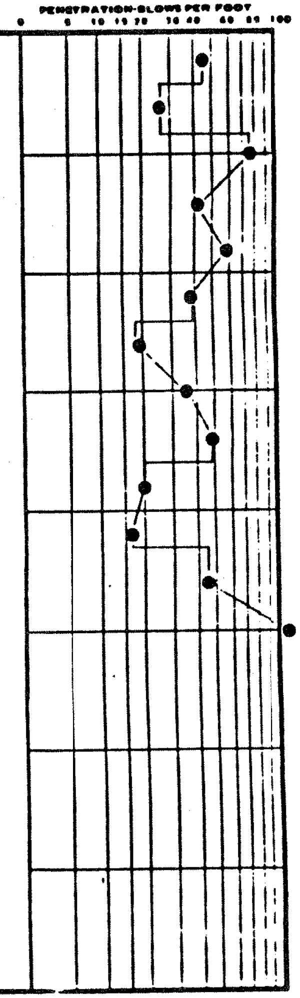
BORING NUMBER AE-15
DATE STARTED 11/11/74
DATE COMPLETED 11/13/74
JOB NUMBER SA-737
COORDINATES N 1046.1
E 555.2

FIG 2G-A24

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
-33.0	40.0	TAN SILTY PARTIALLY CEMENTED SHELLY FINE SAND
	42.0	TAN PARTIALLY CEMENTED SILTY FINE SAND
	44.0	GRAY-TAN PARTIALLY CEMENTED SLIGHTLY SILTY FINE SAND
-38.0		
-43.0		
	52.0	GRAY MEDIUM TO FINE SAND WITH SHELL FRAGMENTS
-48.0		
	58.0	GRAY FINE SAND WITH SHELL FRAGMENTS, LITTLE CLAY
-53.0		
	61.5	GRAY FINE SAND WITH MEDIUM SHELL FRAGMENTS, TRACE CLAY
-58.0	65.5	BORING TERMINATED

PENETRATION-BLOWS PER FOOT



REMARKS:
DRILLED WITH AW AND 3-7/8" SIDE DISCHARGE DRAG BIT

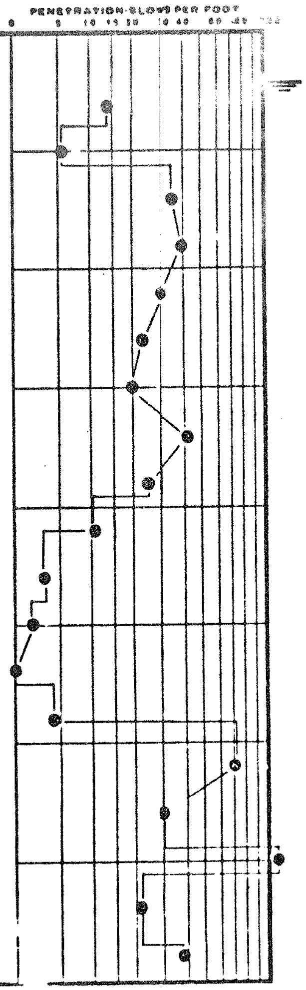
DRILLED BY GIRDLER/T-250
LOGGED BY [illegible]
CHECKED BY [illegible]

BORING NUMBER AE-15
DATE STARTED [illegible]
DATE COMPLETED [illegible]
JOB NUMBER [illegible]

FIG 2G-A24 Cont.

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
+ 3.0		LIGHT BROWN FINE SAND WITH FEW SHELL FRAGMENTS (FILL)
	2.0	GRAY MEDIUM TO FINE SAND WITH NUMEROUS SHELL FRAGMENTS AND *(1)
	3.8	BROWN PEAT WITH SAND AND ISOLATED CLAY SEAMS
+ 1.1	5.5	
- 6.1		GRAY FINE SAND WITH FEW TO NUMEROUS SHELL FRAGMENTS
-11.1		
	19.5	GRAY MEDIUM TO FINE SAND AND SHELL FRAGMENTS
-16.1	21.0	DARK GRAY-GREEN CLAYEY SILT
	24.0	GRAY CLAY, SOME MEDIUM TO FINE SAND AND SHELL FRAGMENTS
-21.1	27.5	GRAY FINE SAND AND SHELLS, LITTLE CLAY
	29.2	GRAY FINE SAND AND SHELL FRAGMENTS
-26.1		
	34.5	GRAY CEMENTED SAND AND *(2)
-31.1	35.5	GRAY FINE TO MEDIUM GRAVEL
	33.7	LIGHT GRAY FINE SAND WITH SOME ROCK FRAGMENTS



REMARKS: *(1) ISOLATED BROWN SILT SEAMS *(2) SHELL FRAGMENTS

WATER TABLE ON 11/25/74

WOH = WEIGHT OF HATTER

DRILLED WITH AW ROD AND 3-1/2" SIDE DISCHARGE DRAG BIT

DRILLED BY GIRDLER/F-250
LOGGED BY FFK
CHECKED BY MLK

BORING NUMBER AE-16
DATE STARTED 11/21/74
DATE COMPLETED 11/2/74
JOB NUMBER SA-737
COORDINATES N 1074.3
E 475.2

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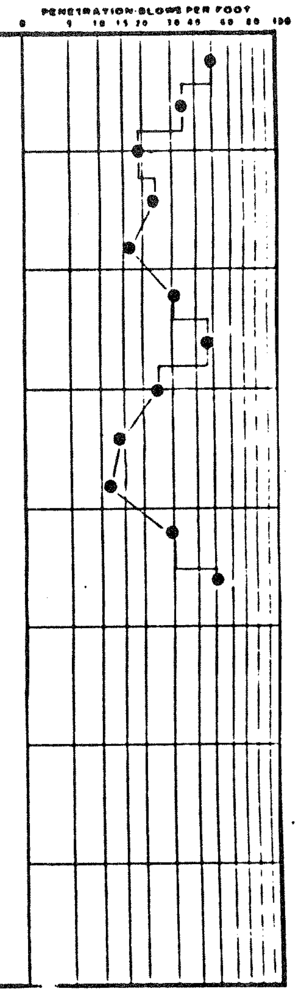
FIG 2G-A25

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
-36.1		LIGHT GRAY FINE SAND WITH SOME ROCK FRAGMENTS
	42.0	LIGHT GRAY TAN FINE SAND
	44.0	LIGHT GRAY SLIGHTLY SILTY TO SILTY FINE SAND
-41.1	46.0	GRAY MEDIUM TO FINE SAND WITH NUMEROUS SHELL FRAGMENTS
-46.1		
	52.0	GRAY-GREEN FINE SAND WITH FEW SHELL FRAGMENTS
	54.0	GRAY COARSE TO FINE SAND AND SHELL FRAGMENTS, LITTLE SILT
-51.1		
-56.1		
	62.5	GRAY FINE SAND WITH FEW (1)
	63.5	BORING TERMINATED
-61.1		
		(1) SHELL FRAGMENTS



REMARKS:

DRILLED WITH AW ROD AND 3-1/2" SIDE DISCHARGE DRAG BIT

DRILLED BY GIRDLER/F-250
LOGGED BY FFK
CHECKED BY MLK

BORING NUMBER AE-16
DATE STARTED 11/21/74
DATE COMPLETED 11/22/74
JOB NUMBER SA-737

PAGE 2 OF 2 PAGES

FIG 2G-A25 CONT.

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	S#	R
+ 4.3				
- 0.7				
- 5.7		WASH DRILL TO 20.0 FEET		
-10.7				
-15.7	20.0	GRAY-BLACK SHELLY MEDIUM SAND INTO DARK GRAY VERY CLAYEY SHELLY (1)	UD1	1.5
	22.0	NO RECOVERY	P	
	24.0			
-20.7	26.0	NO RECOVERY	P	
		GRAY-BROWN SAND INTO DARK GRAY SILTY CLAY	UD2	0.5
	28.0	GRAY CLAYEY FINE SAND	UD3	1.
	29.2	BORING TERMINATED		
-25.7				

PENETRATION-BLOWS PER FOOT: 0 5 10 15 20 30 40 60 80 100

(1) FINE SAND

REMARKS:
γ_t = BULK DENSITY (lb/ft^3)
S# = UNDISTURBED SAMPLE NUMBER
R = RECOVERY (FEET)
P = PISTON SAMPLE ATTEMPT
PISTON SAMPLE
DRILLED WITH AW ROD AND 3-7/8" TRICONE ROLLER BIT

DRILLED BY GIEDLER/F-250
LOGGED BY MCM
CHECKED BY GAW

BORING NUMBER AE-16A
DATE STARTED 12/2/74
DATE COMPLETED 12/2/74
JOB NUMBER SA-737
COORDINATES N 1068.0
E 475.1

FIG 2G-A26

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	S#	R
+ 4.5				
- 0.5				
- 5.5		WASH DRILL TO 20.0 FEET		
-10.5				
-15.5	20.0	LIGHT GRAY-BROWN FINE TO MEDIUM SHELLY SAND	UD1	1.4
	22.0	NO RECOVERY	P	
	24.0	DARK GRAY SLIGHTLY CLAYEY TO CLAYEY FINE SLIGHTLY SHELLY SAND	UD2	
-20.5	26.0	DARK GRAY SANDY CLAY	UD3	1.9
	28.0	DARK GRAY CLAYEY SAND INTO (1)	UD4	
	29.3	BORING TERMINATED		
-25.5				

PENETRATION-BLOWS PER FOOT: 0 5 10 15 20 30 40 60 80 100

(1) SILTY SAND

REMARKS:
γ_t = BULK DENSITY (lb/ft^3)
S# = UNDISTURBED SAMPLE NUMBER
R = RECOVERY (FEET)
P = PISTON SAMPLE ATTEMPT
PISTON SAMPLE
DRILLED WITH AW ROD AND 3-7/8" TRICONE ROLLER BIT

DRILLED BY GIEDLER/F250
LOGGED BY MCM
CHECKED BY GAW

BORING NUMBER AE-16B
DATE STARTED 12/2/74
DATE COMPLETED 12/2/74
COORDINATES N 1072.8
E 490.8

FIG 2G-A27

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	S#	R
+ 4.5				
- 0.5				
- 5.5		WASH DRILL TO 20.0 FEET		
-10.5				
-15.5	20.0			
	22.0	GRAY BLACK SHELLY MEDIUM SAND	UD1	
	22.5	WASH DRILL TO 22.5 FEET		
-20.5	24.5	GRAY-BLACK SHELLY FINE SAND INTO DARK GRAY SLIGHTLY SHELLY (1)	UD2	
	26.5	DARK GRAY SHELLY CLAYEY SILTY SAND	UD3	
	28.5	DARK GRAY SILTY CLAY	UD4	
-25.5	30.5	LIGHT GRAY SLIGHTLY SILTY SHELLY FINE SAND	UD5	
		BORING TERMINATED		
-30.5				
		(1) SILTY CLAY		

PENETRATION-BLOWS PER FOOT: 0 5 10 15 20 30 40 60 80 100

REMARKS:

γ_t = BULK DENSITY (lb/ft^3)

S# = UNDISTURBED SAMPLE NUMBER

R = RECOVERY (FEET)

PISTON SAMPLE

DRILLED WITH AW ROD AND 3-7/8" TRICONE ROLLER BIT

DRILLED BY GIRDLER/F-250

LOGGED BY MCM

CHECKED BY GAW

BORING NUMBER AE-16C

DATE STARTED 12/3/74

DATE COMPLETED 12/4/74

JOB NUMBER SA-737

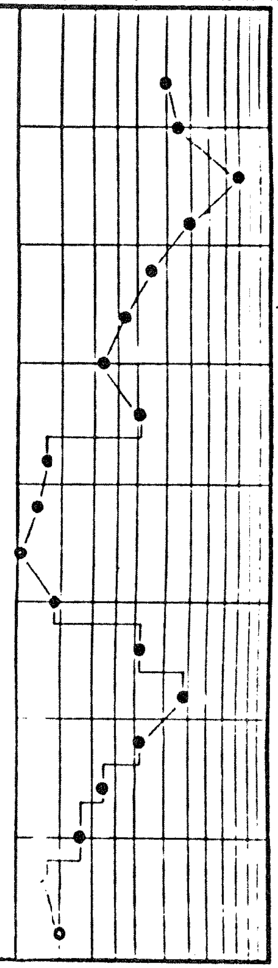
COORDINATES N 1072.5 E 495.6

FIG 2G-A28

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
+16.4		
+11.4		TAN PARTIALLY CEMENTED SHELLY SAND
+ 6.4	10.0	
+ 1.4		TAN PARTIALLY CEMENTED SLIGHTLY SHELLY TO SHELLY SAND
	18.0	
- 3.6		LIGHT, GRAY-TAN FINE SAND, SOME SILT
- 8.6	26.0	
	28.0	LIGHT BROWN MEDIUM TO FINE SAND AND SHELLS, SOME SILT
-13.6		GRAY-GREEN SLIGHTLY SILTY SAND
	32.0	GRAY FINE SAND, AND CLAY AND SILT
	33.5	TAN FINE SAND AND SHELL FRAGMENTS, LITTLE SILT
-18.6	36.0	
		GRAY CLAY, SOME FINE SAND AND SHELL FRAGMENTS
-23.6	10.0	

PENETRATION-BLOWS PER FOOT: 0 5 10 15 20 30 40 60 80 100



REMARKS:

WATER TABLE ON 12/3/74

WOH = WEIGHT OF HAMMER

WOR = WEIGHT OF ROD

DRILLED WITH AW ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT

PAGE 1 OF 2 PAGES

DRILLED BY GIRDLER/F-1500

LOGGED BY JLP

CHECKED BY GAW

BORING NUMBER AE-17

DATE STARTED 11/19/74

DATE COMPLETED 11/19/74

JOB NUMBER SA-737

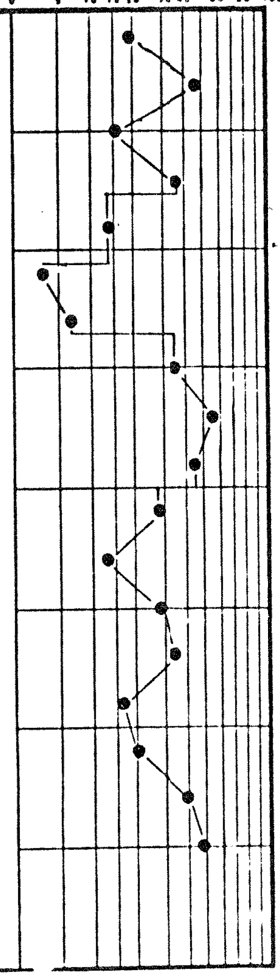
COORDINATES N 958.6 E 596.5

FIG 2G-A29

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
-23.6		
		GRAY FINE SAND AND SHELL FRAGMENTS, SOME CLAY AND SILT
-28.6		
	47.5	
		GRAY MEDIUM TO FINE SAND AND SHELL FRAGMENTS
-33.6	50.5	
		GRAY MEDIUM TO FINE SAND AND SHELLS, SOME CLAY
	53.5	
-38.6		
		LIGHT TAN FINE SAND AND SHELL FRAGMENTS, LITTLE CLAY WITH SOME ORGANICS
-43.6	60.0	
		GRAY SLIGHT SILTY SHELLY SAND
-48.6		
	66.0	
		DARK GRAY SANDY SHELLS
-53.6		
	72.0	
		DARK GRAY SHELLY FINE SAND
-58.6	75.5	
		BORING TERMINATED
-63.6		

PENETRATION-BLOWS PER FOOT



REMARKS:

DRILLED BY GIRGLER/F-1500
LOGGED BY JLP
CHECKED BY GAW

BORING NUMBER AF-17
DATE STARTED 11/19/74
DATE COMPLETED 11/19/74
JOB NUMBER SA-737
PAGE 2 OF 2 PAGES

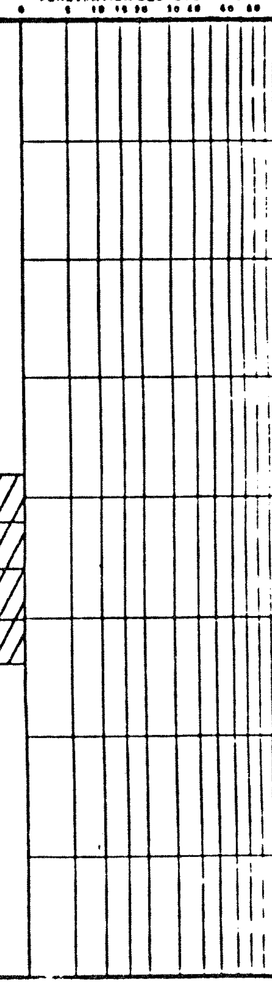
DRILLED WITH AW ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT

FIG 2G-A29 Cont.

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	S#	R
+15.5				
+11.5				
+ 6.5				
		WASH DRILL TO 19.0 FEET		
+ 1.5				
	19.0			
- 3.5		TAN SILTY FINE SAND	T-1	0.8
	21.0			
		TAN SILTY FINE SAND	T-2	0.8
	23.0			
		TAN SILTY FINE SAND	T-3	0.8
- 8.5	25.0			
		TAN SILTY FINE SAND	T-4	0.5
	27.0			
-13.5		BORING TERMINATED		

PENETRATION-BLOWS PER FOOT



REMARKS:

S# = UNDISTURBED SAMPLE NUMBER
R = RECOVERY (FEET)
DRILLED WITH N ROD AND 3-7/8" TRICONE ROLLER BIT

PISTON SAMPLE

DRILLED BY GIRGLER/F-250
LOGGED BY CJR
CHECKED BY GAW

BORING NUMBER AF-17A
DATE STARTED 11/ /74
DATE COMPLETED 11 / 74

COORDINATES N 954.8
E 5?4.8

FIG 2G-A30

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	S#	R		PENETRATION-BLOWS PER FOOT 0 5 10 15 20 30 40 60 80 100
+16.5						
+11.5						
+ 6.5						
+ 1.5		WASH DRILL TO 24.0 FEET				
- 3.5						
- 8.5	24.0	TAN SILTY FINE SAND	T-1	2.0		
	26.0	BORING TERMINATED				
-13.5						

REMARKS:

γ_t = BULK DENSITY (16/ft^3)
S# = UNDISTURBED SAMPLE NUMBER
R = RECOVERY (FEET)
DRILLED WITH N ROD AND 3-7/8" TRICONE ROLLER BIT

PISTON SAMPLE

DRILLED BY	GIRDLER/F-1500
LOGGED BY	CJR
CHECKED BY	GAW

BORING NUMBER	AE-17B
DATE STARTED	11/22/74
DATE COMPLETED	11/22/74
JOB NUMBER	SA-737
COORDINATES	N 947.0
	E 592.4

FIG 2G-A31

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	S#			PENETRATION-BLOWS PER FOOT 0 5 10 15 20 30 40 60 80 100
+16.3						
+11.3						
+ 6.3		WASH DRILL TO 19.5 FEET				
+ 1.3						
- 3.7	19.5	NO RECOVERY	P			
	21.5	LIGHT GRAY SILTY FINE SAND	P		JS	
	22.5	BORING TERMINATED				
- 8.7						

REMARKS:
S# = UNDISTURBED SAMPLE NUMBER
JS = JAR SAMPLE RETAINED
P = PISTON SAMPLE ATTEMPTED

DRILLED BY	GIRDLER/F-250
LOGGED BY	MCH
CHECKED BY	GAW

BORING NUMBER	AE-17C
DATE STARTED	11/22/74
DATE COMPLETED	11/25/74
JOB NUMBER	SA-737
COORDINATES	N 956.2
	E 599.7

DRILLED WITH AW ROD AND 0-19.5' WITH 3-7/8" TRICONE ROLLER B[illegible]
19.5-22.5' WITH 2 15/16" SIDE DISCHARGE
DRAG BIT

FIG 2G-A32

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	S#	R	
+16.3					
+11.3					
+ 6.3		WASH DRILL TO 19.0 FEET			
+ 1.3					
	19.0				
- 3.7		LIGHT GRAY SILTY FINE SAND	P		JS
	21.0	LIGHT GRAY-TAN SILTY FINE SAND	UD1	1.3	
	23.0	LIGHT GRAY-TAN SILTY FINE SAND	UD2	1.3	
- 8.7	25.0	BORING TERMINATED			

PENETRATION-BLOWS PER FOOT: 0 5 10 15 20 30 40 60 80 100

REMARKS:

γ_t = BULK DENSITY (lb/ft^3)
S# = UNDISTURBED SAMPLE NUMBER
R = RECOVERY (FEET) JS = JAR SAMPLE RETAINED
P = PISTON SAMPLE ATTEMPTED
DRILLED WITH AW ROD AND 3-7/8" TRICONE ROLLER BIT
▨ PISTON SAMPLE

DRILLED BY GIRDLER/F-250
LOGGED BY MCM
CHECKED BY GAW

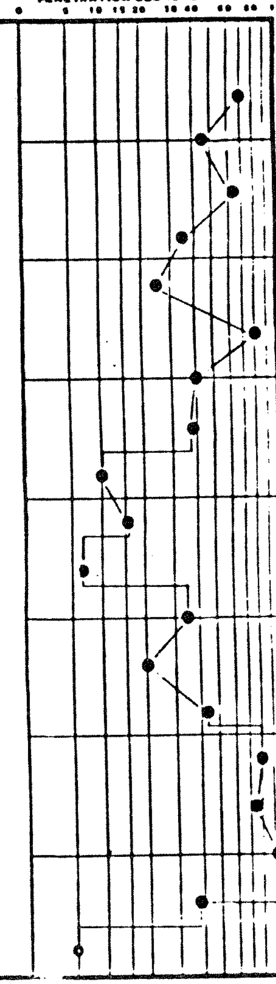
BORING NUMBER AE-17D
DATE STARTED 11/25/74
DATE COMPLETED 11/26/74
JOB NUMBER SA-737
COORDINATES N 960.1 E 600.5

FIG 2G-A33

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
+17.7		TAN SAND WITH SHELLS (FILL)
+12.7		
	7.5	TAN FINE SAND AND SILTY FINE SAND WITH SHELLS (FILL)
+ 7.7		
	12.5	TAN SHELLY SAND (FILL)
+ 2.7		
	18.0	TAN COARSE TO FINE SAND, SOME MEDIUM GRAVEL AND SHELL FRAGMENTS
- 2.3		
	21.5	GRAY MEDIUM TO FINE SAND WITH SHELLS, PARTIALLY CEMENTED
	23.5	GRAY MEDIUM TO FINE SAND WITH SHELL FRAGMENTS, LITTLE SILT
- 7.3		
	29.5	GRAY MEDIUM TO FINE SAND AND SHELLS, LITTLE SILT OR CLAY
-12.3		
-17.3		
	37.0	LIGHT GRAY SLIGHTLY SILTY FINE SAND
	38.0	DARK GRAY SLIGHTLY ORGANIC CLAY SOME FINE SAND WITH SHELL FRAGMENTS
-22.3		

PENETRATION-BLOWS PER FOOT: 0 5 10 15 20 30 40 60 80 100



REMARKS:

WATER TABLE ON 11/25/74
◀ LOSS OF DRILLING FLUID

DRILLED WITH AW ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT

DRILLED BY GIRDLER/F-1500
LOGGED BY JLP
CHECKED BY GAW

BORING NUMBER AE-18
DATE STARTED 11/25/74
DATE COMPLETED 11/25/74
COORDINATES N 927.1 S 522.0

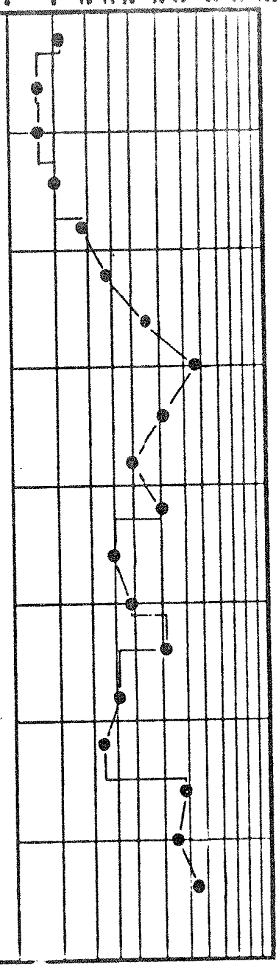
PAGE 1 OF 2

FIG 2G-A34

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
-22.3		DARK GRAY SLIGHTLY ORGANIC (1)
	41.5	GRAY SHELLY FINE SAND, AND CLAY AND SILT
-27.3	43.5	GRAY CLAY AND SILT, SOME FINE SAND AND SHELLS
	46.0	GRAY SILTY CLAY WITH ROOTS, LITTLE FINE SAND
-32.3	48.5	GRAY COARSE TO FINE SAND AND SHELLS, SOME SILT AND ORGANICS
-37.3	53.5	TAN TO GRAY COARSE TO FINE SAND WITH SHELL FRAGMENTS
	56.0	LIGHT GRAY SHELLY SILTY FINE SAND
-42.3	60.0	LIGHT GRAY SILTY SHELLY SAND
	61.5	LIGHT GRAY SILTY SHELLY SAND
-47.3	65.5	LIGHT GRAY SILTY·SHELLY SAND
-52.3	67.0	DARK GRAY SHELLY SAND
-57.3	72.5	GRAY SILTY SLIGHTLY SHELLY FINE SAND, PARTIALLY CEMENTED
-62.3	77.5	BORING TERMINATED

PENETRATION-BLOWS PER FOOT



REMARKS: (1) CLAY, SOME FINE SAND WITH SHELL FRAGMENTS

DRILLED BY GIRDLER/F-1500
LOGGED BY JLP
CHECKED BY GAU

BORING NUMBER AE-18
DATE STARTED 11/15/74
DATE COMPLETED 11/19/74
JOB NUMBER SA-737

DRILLED WITH AN ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT

FIG 2G-A34 Cont.

PAGE 2 of 2

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	Sa	R	
+17.7					
+12.7		WASH DRILL TO 18'			
+ 7.7					
+ 2.7					
	18.0	FRAGMENTED ROCKS AND SHELLS	O		JS
- 2.3	19.0	WASH DRILL TO 22'			
	22.0	FRAGMENTED ROCKS AND SHELLS INTO TAN SILTY FINE SAND	T-2		
	24.0	TAN SILTY FINE SAND	T-3		
- 7.3	25.0	WASH DRILL TO 26.0'			
	26.0	TAN AND GRAY SLIGHTLY SILTY FINE SAND WITH A TRACE OF SHELL FRAGMENTS	T-4	1.8	
	28.0				
-12.3		WASH DRILL TO 37'			
-17.3					
	37.0	DARK GRAY VERY FINE SANDY CLAY			
-22.3	39.0				

PENETRATION-BLOWS PER FOOT

REMARKS: OSTERBERG SAMPLE
γ_d=BULK DENSITY (LB/FT3)
O=OSTERBERG SAMPLE ATTEMPT
SE=UNDISTURBED SAMPLE NO.
R=RECOVERY (FT)
JS=JAR SAMPLE RETAINED

DRILLED BY GIRDLER/F-1500
LOGGED BY CJR
CHECKED BY

BORING NUMBER AE-18A
DATE STARTED 12/4/74
DATE COMPLETED 12/5/74
JOB NUMBER SA-737
COORDINATES N 91?.?
E 524.0

DRILLED WITH NW ROD ... TRICONE ROLLER BIT

FIG 2G-A35

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	S#	R	PENETRATION-BLOWS PER FOOT 0 5 10 15 20 30 40 60 80 100
-22.3	41.0	DARK GRAY VERY FINE SANDY CLAY (1)	T-6	2.0	
	43.0	DARK GRAY VERY FINE SANDY CLAY INTO GRAY SLIGHTLY CLAYEY FINE SAND	T-7	2.0	
-27.3	45.0	DARK GRAY VERY FINE SANDY CLAY	T-8	2.0	
	47.0	DARK GRAY VERY FINE SANDY CLAY WITH TRACE OF SHELL FRAGMENTS	T-9	2.0	
	49.0	DARK GRAY VERY FINE SANDY SILTY CLAY WITH SHELL FRAGMENTS INTO	T-10	2.0	
-32.3	51.0	TAN CLAYEY FINE SAND (PARTIALLY CEMENTED)	T-11	2.0	
	53.0	TAN FINE SAND (PARTIALLY CEMENTED)	T-12	2.0	
-37.3	55.0	TAN FINE SAND WITH SHELL FRAGMENTS INTO GRAY SILTY FINE SAND	T-13	2.0	
		BORING TERMINATED			
		(1) WITH POCKETS OF GRAY FINE SAND			

REMARKS: / OSTERBERG SAMPLE
γ_d=BULK DENSITY (LB/FT3)
ST-UNDISTURBED SAMPLE NO.
R=RECOVERY (FT.)
O=UNSUCCESSFUL SAMPLE ATTEMPT
JS=JAR SAMPLE RETAINED

DRILLED BY GIRDLER/T-1500
LOGGED BY CJR
CHECKED BY GAW

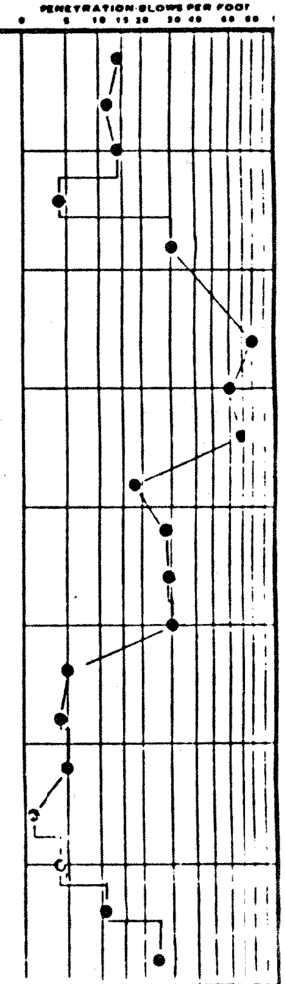
BORING NUMBER AE-19A
DATE STARTED 12/4/74
DATE COMPLETED 12/5/74
JOB NUMBER SA-737

DRILLED WITH AW ROD AND 3-7/8" TRICONE ROLLER BIT

FIG 2G-A35 Cont.

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
NA		TAN FINE TO MEDIUM SAND WITH SHELL AND SOME ROCK FRAGMENTS
	6.0	TAN FINE TO MEDIUM SAND
	7.7	GRAY SLIGHTLY SILTY FINE SAND
	18.0	GRAY FINE SAND
	26.0	GRAY CLAY AND SILT, FINE SAND AND SHELL PARTICLES
	29.5	GRAY FINE SAND AND SHELL PARTICLES AND CLAY
	34.0	GRAY COARSE TO FINE SAND AND SHELL PARTICLES, SOME CLAY
	36.0	GRAY FINE SAND, SOME ORGANICS (1)
	37.5	TAN TO GRAY COARSE TO FINE SAND AND SHELLS WITH CEMENTED NODULES



(1) AND CLAY

REMARKS:
GROUND WATER LEVEL 11/25/74

DRILLED BY GIRDLER/T-250
LOGGED BY ER
CHECKED BY MEK

BORING NUMBER AE-19
DATE STARTED 11/21/74
DATE COMPLETED 11/20/74
JOB NUMBER SA-737
COORDINATES NA

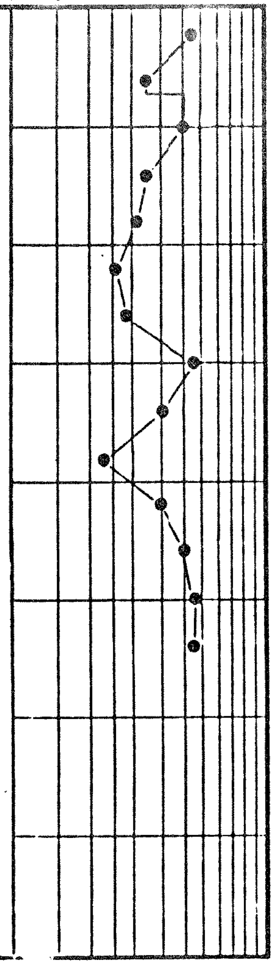
DRILLED WITH AW ROD AND 3-1/2" SIDE DISCHARGE BIT

FIG 2G-A36

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
NA		LIGHT GRAY SLIGHTLY SILTY FINE SAND WITH CEMENTED SAND FRAGMENTS
	43.5	GRAY TO GRAY-GREEN SLIGHTLY SILTY FINE SAND WITH NUMEROUS SHELL FRAGMENTS
	68.0	BORING TERMINATED

PENETRATION-BLOWS PER FOOT



REMARKS:

DRILLED BY GIRDLER/F-250
LOGGED BY ER
CHECKED BY MEK

DRILLED WITH AW ROD AND 3-1/2" SIDE DISCHARGE BIT

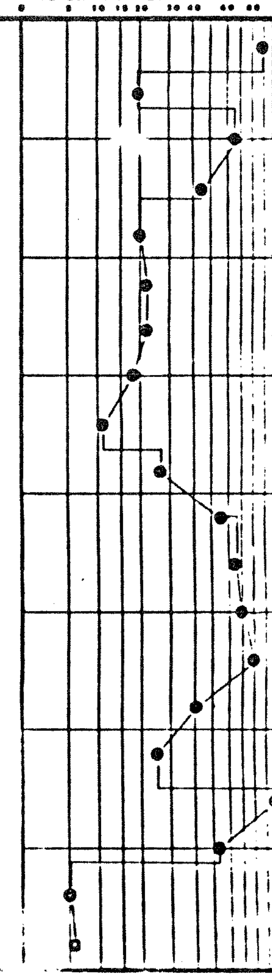
BORING NUMBER AE-19
DATE STARTED 11/19/74
DATE COMPLETED 11/20/74
JOB NUMBER SA-737

FIG 2G-A36 Cont.

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
NA		TAN FINE SAND (FILL)
	2.0	TAN FINE SAND (FILL)
	3.5	TAN SAND (FILL)
	7.5	TAN SAND (FILL)
	14.0	TAN COARSE TO FINE SAND WITH SHELL FRAGMENTS
	19.0	TAN AND LIGHT GRAY MEDIUM TO FINE SAND WITH SHELL FRAGMENTS
	21.0	TAN AND GRAY SAND WITH SHELL FRAGMENTS
	27.5	TAN SLIGHTLY SILTY FINE SAND
	32.5	DARK GRAY AND BLACK FINE SAND WITH SILT SEAMS
	35.5	GRAY MEDIUM TO F[illegible] SAND A[illegible] WITH SHELL FRAG[illegible]
	40.0	

PENETRATION-BLOWS PER FOOT



REMARKS:

WATER TABLE ON 11/25/[illegible]

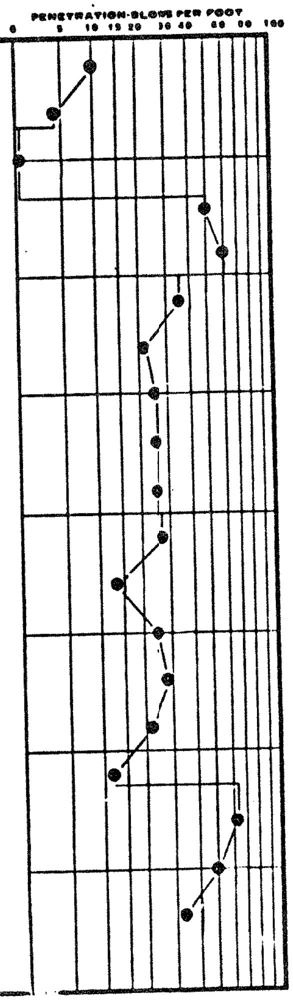
DRILLED BY [illegible] F-1500
LOGGED BY [illegible]
CHECKED BY [illegible]

DRILLED WITH AW [illegible] AND 3-1/[illegible] [illegible]DE DI[illegible]

BORING NUMBER AE-21
DATE STARTED 11/12/74
DATE COMPLETED 11/13/74
JOB NUMBER SA-737
COORDINATES NA

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
	40.0	GRAY MEDIUM TO FINE SAND WITH SOME CLAY AND SHELL FRAGMENTS
NA	43.5	GRAY FINE SAND WITH SHELL FRAGMENTS, TRACE CLAY AND SILT
	46.5	TAN AND GRAY COARSE TO FINE SAND AND PARTIALLY CEMENTED SHELL FRAGMENTS
	50.0	TAN AND GRAY CEMENTED SAND WITH SHELLS
	60.0	TAN AND GRAY COARSE TO FINE SAND WITH SHELL FRAGMENTS
	71.5	GRAY COARSE TO FINE SAND WITH SHELL FRAGMENTS
	77.5	BORING TERMINATED



REMARKS:
WOH-WEIGHT OF HAMMER

DRILLED WITH AW ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT

DRILLED BY GIRDLER/F-1500
LOGGED BY JLP
CHECKED BY GAW

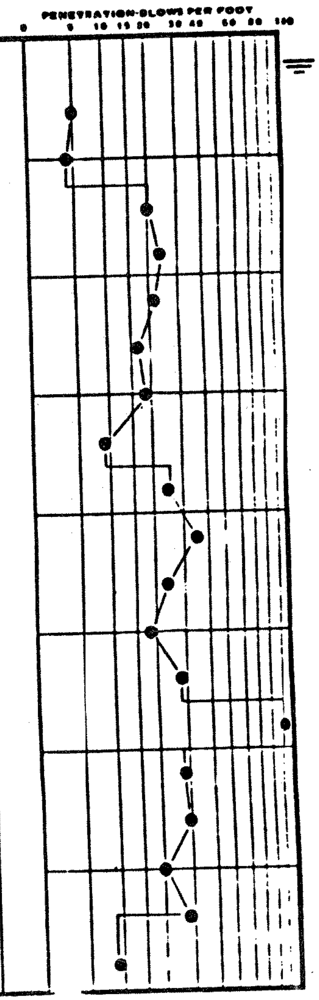
BORING NUMBER AE-21
DATE STARTED 11/12/74
DATE COMPLETED 11/13/74
JOB NUMBER SA-737

P. 2

FIG 2G-A37 Cont.

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
+ 4.4		GRAY BLACK ORGANIC SILTY SAND
- 0.6	6.0	GRAY SLIGHTLY SILTY SLIGHTLY SHELLY FINE SAND
- 5.6		
-10.6	15.5	GRAY COARSE TO FINE SAND AND SHELL FRAGMENTS
	18.0	TAN COARSE TO FINE SAND AND SHELL FRAGMENTS, LITTLE SILT
-15.6		
-20.6	28.0	LIGHT GREEN SILTY PARTIALLY CEMENTED FINE SAND
-25.6	30.0	LIGHT GREEN SILTY PARTIALLY CEMENTED FINE SAND
-30.6	37.0	TAN TO GRAY COARSE TO FINE SAND AND SHELL FRAGMENTS, LITTLE SILT
-35.6		



REMARKS:
WATER TABLE ON 11/22/74

DRILLED WITH AW ROD AND 3 1/2" TOP DISCHARGE DRAG BIT

PAGE 1 OF 2 PAGES

DRILLED BY GIRDLER/F-250
LOGGED BY ER
CHECKED BY GAW

BORING NUMBER AE-22
DATE STARTED 11/22/74
DATE COMPLETED 11/25/74

COORDINATES N 1281.3
E 655.0

FIG 2G-A38

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
-35.6		
		TAN COARSE TO FINE SAND AND SHELL FRAGMENTS, LITTLE SILT
-40.6		
	47.0	
-45.6		
		LIGHT GREEN SHELLY SILTY PARTIALLY CEMENTED FINE SAND
-50.6		
	55.5	
		TAN COARSE TO FINE SAND AND SHELL FRAGMENTS
-55.6		
	60.5	
		GRAY COARSE TO FINE SAND AND SHELL FRAGMENTS
	63.5	
		BORING TERMINATED
-60.6		

PENETRATION-BLOWS PER FOOT: 0 5 10 15 20 30 40 60 80 100

REMARKS:

DRILLED WITH AW ROD AND 3 1/2" TOP DISCHARGE DRAG BIT

DRILLED BY GIEDLER/F250
LOGGED BY ER
CHECKED BY GAW

BORING NUMBER AE-22
DATE STARTED 11/22/74
DATE COMPLETED 11/25/74
JOB NUMBER SA-737

PAGE 2 OF 2 PAGES

FIG 2G-A38 Cont.

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	S#	R
+ 4.5				
		WASH DRILL TO 38.0'		
-30.5				
	38.0	GRAY-BLACK SHELLY SAND	UD-1	
	39.0			
-35.5				
		WASH DRILL TO 41.0'		
	41.0	LIGHT GRAY-TAN SILTY FINE PARTIALLY CEMENTED SAND	UD-2	
	43.0			
	43.8	LIGHT GRAY-TAN SILTY FINE SAND	P	
-40.5				
		WASH DRILL TO 46.0'		
	46.0	BORING TERMINATED		
-45.5				

PENETRATION-BLOWS PER FOOT: 0 5 10 15 20 30 40 60 80 100

REMARKS:

γ_d =BULK DENSITY (LB/FT3)
S#=UNDISTURBED SAMPLE NO
R=RECOVERY (FT.)
P=PISTON SA[illegible] ATTEMPT

DRILLED WITH [illegible] AND [illegible] FT, [illegible] ROLLER [illegible] TRICONE [illegible]
[illegible] PISTON SAMPLE

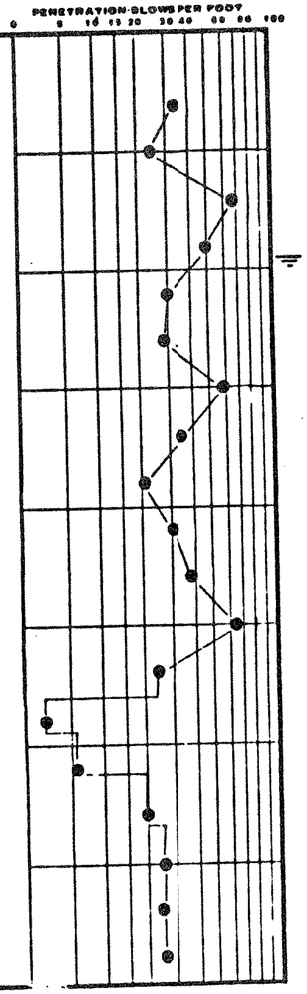
DRILLED BY [illegible]/F-250
LOGGED BY
CHECKED BY

BORING NUMBER [illegible]
DATE STARTED 11/26/74
DATE COMPLETED [illegible]
JOB NUMBER SA-737
COORDINATES N [illegible] E [illegible]

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
+ 4.5		
- 0.5		LIGHT BROWN SLIGHTLY SILTY TO SILTY FINE SAND WITH SOME ROCK FRAGMENTS
- 5.5	10.0	
-10.5		GRAY FINE SAND WITH SHELL FRAGMENTS
-15.5		
-20.5	24.5	GRAY FINE SAND
	28.0	GRAY MEDIUM TO FINE SAND WITH SHELL PARTICLES, SOME SILT AND CLAY
-25.5	29.5	TAN TO GRAY MEDIUM TO FINE SAND AND SHELL PARTICLES, SOME CLAY
	31.2	GRAY TO TAN FINE SAND, AND CLAY
	33.5	
-30.5		GRAY AND BROWN VERY CLAYEY FINE SAND
-35.5	40.0	

PENETRATION-BLOWS PER FOOT



REMARKS:
WATER TABLE ON 12/3/74

DRILLED WITH AW ROD AND 3-1/2" SIDE DISCHARGE DRAG BIT

PAGE 1 OF 2 PAGES

DRILLED BY GIRDLER/F-250
LOGGED BY MEK
CHECKED BY GAW

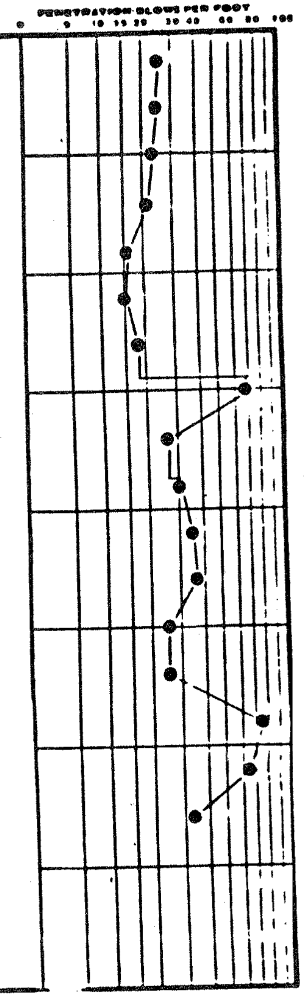
BORING NUMBER AE-23
DATE STARTED 11/22/74
DATE COMPLETED 11/25/74
JOB NUMBER SA-737
COORDINATES N 1178.6 E 776.3

FIG 2G-A40

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
-35.5	40.0	YELLOW-BROWN SLIGHTLY SILTY FINE SAND
-40.5	45.0	TAN TO GRAY-GREEN COARSE TO FINE SAND AND SHELL FRAGMENTS, LITTLE SILT
-45.5		
-50.5	54.5	TAN COARSE TO FINE SAND AND CEMENTED SAND AND SHELL FRAGMENTS
-55.5	58.7	GRAY FINE SAND WITH NUMEROUS SHELL FRAGMENTS
-60.5		
-65.5		
-70.5	73.5	BORING TERMINATED

PENETRATION-BLOWS PER FOOT



REMARKS:

DRILLED WITH AW ROD AND 3-1/2" SIDE DISCHARGE DRAG BIT

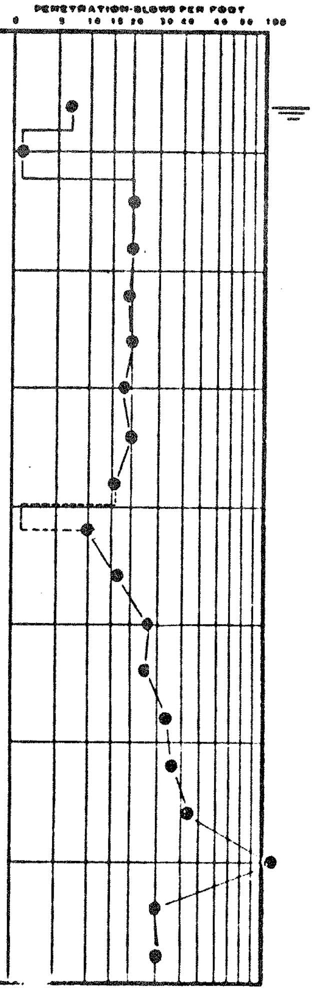
DRILLED BY GIRDLER/F-250
LOGGED BY MEK
CHECKED BY GAW

BORING NUMBER AE-23
DATE STARTED 11/22/74
DATE COMPLETED 11/25/74
JOB NUMBER SA-737
PAGE 2 OF 2 PAGES

FIG 2G-A40 Cont.

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
+ 5.8		GRAY SILTY FINE SAND WITH SHELL FRAGMENTS
+ 0.8	4.0	DARK BROWN PEAT
	6.0	
- 4.2		GRAY FINE SAND
- 9.2		
-14.2	20.0	DARK GRAY ORGANIC SANDY CLAY *(1)
	21.0	
-19.2		GRAY FINE SAND TO SILTY FINE SAND WITH PARTIALLY CEMENTED SAND FRAGMENTS
-24.2		
	34.0	
-29.2		GRAY FINE SAND
	35.5	GRAY SLIGHTLY SILTY FINE SAND
-34.2		*(1) WITH SHELL FRAGMENTS



REMARKS:
GROUND WATER LEVEL ON 11/25/74

DRILLED WITH AW ROD AND 3 7/8" SIDE DISCHARGE DRAG BIT

DRILLED BY GIRDLER/F-1500
LOGGED BY JLP
CHECKED BY EEK

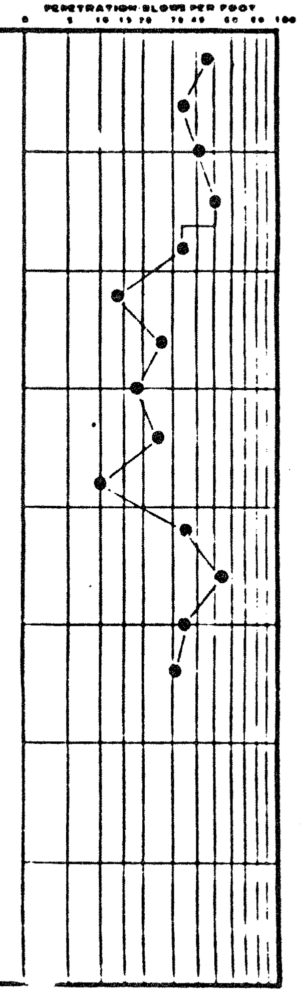
BORING NUMBER AE-24
DATE STARTED 11/20/74
DATE COMPLETED 11/21/74
JOB NUMBER SA-737
COORDINATES N 1124.1 E 532.3

PAGE 1 OF 2 PAGES

FIG 2G-A41

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
-34.2		
-39.2		GRAY SLIGHTLY SILTY FINE SAND
	48.0	
-44.2		
-49.2		
-54.2		GRAY COARSE TO FINE SAND AND SHELL PARTICLES, LITTLE SILT AND CLAY
-59.2		
	67.5	BORING TERMINATED
-64.2		



REMARKS:

DRILLED WITH AW ROD AND 3 7/8" SIDE DISCHARGE DRAG BIT

DRILLED BY GIRDLER/F-1500
LOGGED BY
CHECKED BY

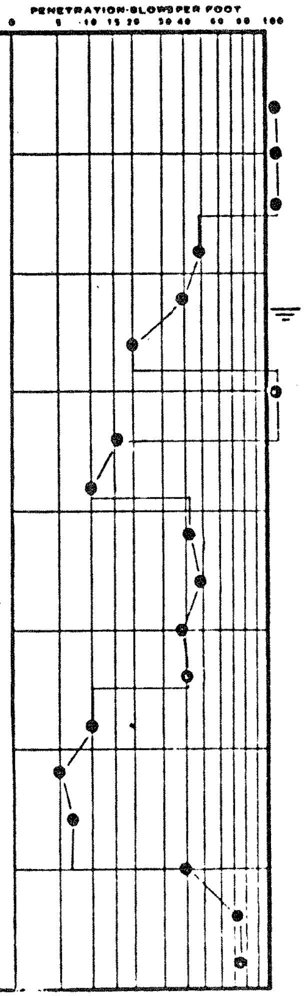
BORING NUMBER AE-24
DATE STARTED 11/20/74
DATE COMPLETED 11/21/74
JOB NUMBER SA-737

PAGE 2 OF 2 PAGES

FIG. 2G-A41 Cont

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
NA		TAN PARTIALLY CEMENTED SLIGHTLY SHELLY FINE SAND
	7.5	TAN FINE TO COARSE SAND
	14.0	GRAY-BLACK SILTY ORGANIC PARTIALLY CEMENTED SAND
	17.0	BROWN PEAT
	19.5	LIGHT GRAY SILTY SLIGHTLY SHELLY FINE SAND
	27.5	GRAY TO TAN COARSE TO FINE SAND AND SHELL FRAGMENTS, LITTLE SILT
	35.0	GRAY COARSE TO FINE SAND AND SHELL FRAGMENTS, PARTIALLY CEMENTED



REMARKS:
WATER TABLE ON 11/25/74

DRILLED BY GIRDLER/F-250
LOGGED BY CJR
CHECKED BY GAW

BORING NUMBER AB-25
DATE STARTED 11/19/74
DATE COMPLETED 11/20/74
JOB NUMBER SA-237
COORDINATES NA

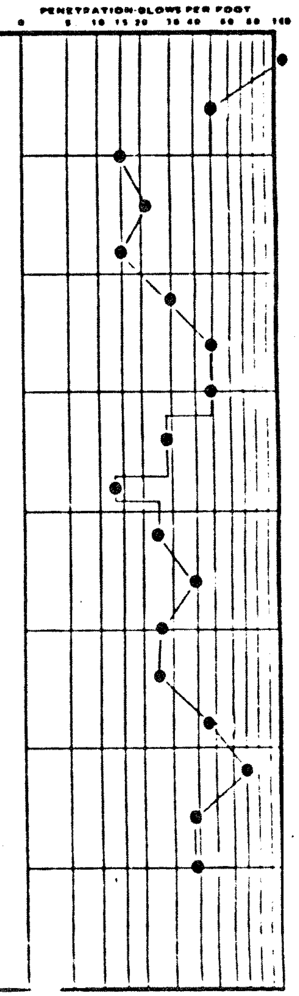
DRILLED W/AW ROD AND 3-1/2" TOP DISCHARGE DRAG BIT

FIG.2G-A42

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
NA		GRAY FINE SLIGHTLY SILTY TO SILTY PARTIALLY CEMENTED FINE SAND
	45.0	LIGHT GRAY SILTY FINE SAND WITH SOME CEMENTED ROCK FRAGMENTS
	56.0	GRAY SHELLY SAND
	58.5	GRAY SLIGHTLY CLAYEY SLIGHTLY (1)
	59.5	GRAY SHELLY SAND
	70.0	GRAY SLIGHTLY SHELLY FINE SAND
	72.0	GRAY SILTY FINE SAND
	75.5	BORING TERMINATED

(1) SHELLY FINE SAND



REMARKS:

DRILLED BY GIRDLER/F-250
LOGGED BY CJR
CHECKED BY GAW

BORING NUMBER AB-25
DATE STARTED 11/19/74
DATE COMPLETED 11/20/74
JOB NUMBER SA-237

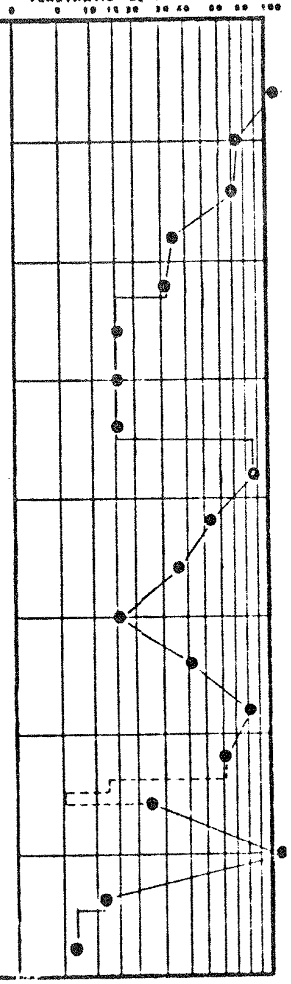
DRILLED W/AW ROD AND 3-1/2" TOP DISCHARGE DRAG BIT

FIG.2G-A42 Cont.

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
+18.5		
+13.5		TAN-GRAY FINE SAND WITH SHELL AND ROCK FRAGMENTS
+ 8.5		
	11.5	
+ 3.5		TAN FINE SAND WITH FEW SMALL SHELL FRAGMENTS
	17.5	
- 1.5		LIGHT GRAY-GREEN TAN FINE SAND
	24.0	
- 6.5		LIGHT GRAY-GREEN SILTY FINE SAND
	26.5	
		LIGHT GRAY-GREEN-BROWN SLIGHTLY SILTY FINE SAND
-11.5		
	32.0	*(1)
	32.5	*(2)
	33.0	
-16.5		GRAY COARSE TO FINE SAND AND SHELL FRAGMENTS
	37.5	
		GRAY MEDIUM TO FINE SAND AND SHELLS, SOME SILT AND CLAY
-21.5		

PENETRATION-BLOWS PER FOOT



REMARKS:

*(1) GRAY-GREEN VERY SILTY FINE SAND

*(2) LIGHT GRAY-GREEN VERY SILTY CLAY

DRILLED WITH AW ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT

PAGE 1 OF 2 PAGES

DRILLED BY GIRDLER/F-250
LOGGED BY MEK
CHECKED BY MEK

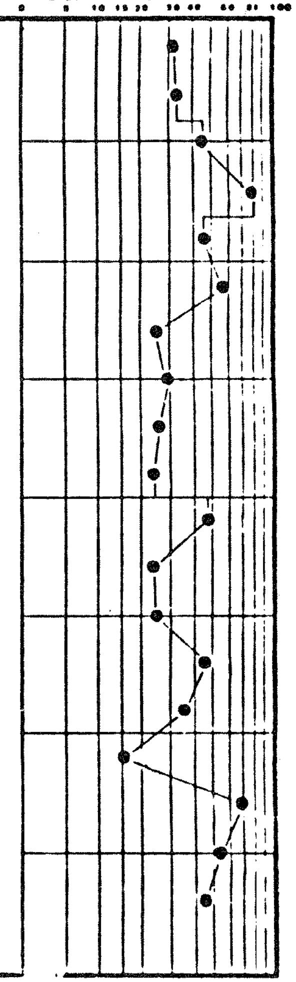
BORING NUMBER AE-26
DATE STARTED 11/14/74
DATE COMPLETED 11/18/74
JOB NUMBER SA-737
COORDINATES N 756.5 E 498.0

FIG.2G-A43

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
-21.5	41.0	
		GRAY MEDIUM TO FINE SAND AND SHELL FRAGMENTS, LITTLE SILT
	44.0	
-26.5		GRAY MEDIUM TO FINE SAND
	48.0	
-31.5		
		YELLOW-GREEN-TAN SLIGHTLY SILTY TO SILTY FINE SAND WITH SOME FINE SHELL FRAGMENTS
-36.5		
-41.5	60.0	
		GRAY MEDIUM TO FINE SAND AND SHELL FRAGMENTS, LITTLE SILT
-46.5		
-51.5	69.5	
		GRAY COARSE TO FINE SAND AND SHELL FRAGMENTS
	73.5	
-56.5		GRAY FINE SAND WITH NUMEROUS SHELL FRAGMENTS
	77.5	
		BORING TERMINATED
-61.5		

PENETRATION-BLOWS PER FOOT



REMARKS:

DRILLED WITH AW ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT

DRILLED BY GIRDLER/F-250
LOGGED BY MEK
CHECKED BY MEK

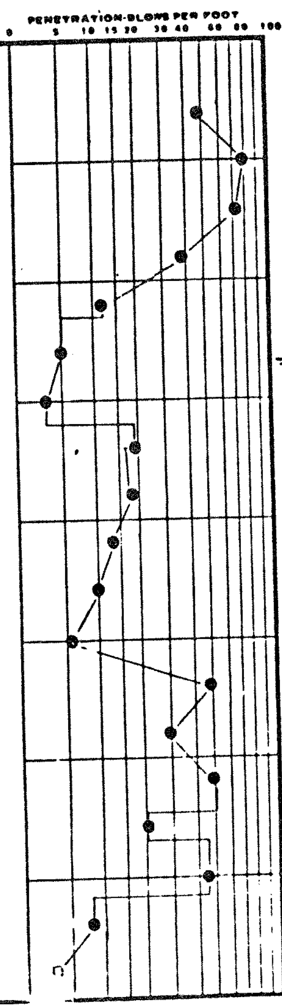
BORING NUMBER AE-26
DATE STARTED 11/14/74
DATE COMPLETED 11/18/74
SA-737

PAGE 2 OF 2 PAGES

FIG.2G-A43 Cont.

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
+18.5		TAN MEDIUM TO FINE SAND WITH SHELL FRAGMENTS AND NODULES. (FILL)
+13.5		
+ 8.5		
	11.5	TAN FINE SAND WITH SHELL FRAGMENTS, SOME SILT.
+ 3.5		
	16.0	DARK GRAY SILTY CLAY
	16.8	TAN FINE SAND WITH SHELL PARTICLES, SOME SILT.
- 1.5		
- 6.5		
-11.5		
	32.5	STIFF GRAY CLAY
	33.5	GRAY-BLACK MEDIUM TO FINE SANDY SHELL FRAGMENTS
-16.5		
	36.0	GRAY MEDIUM TO FINE SAND WITH SHELL FRAGMENTS, LITTLE SILT.
-21.5		



REMARKS:
GROUND WATER LEVEL ON 11/12/74

DRILLED BY GIRDLER/F-1500
LOGGED BY JLP
CHECKED BY MEK

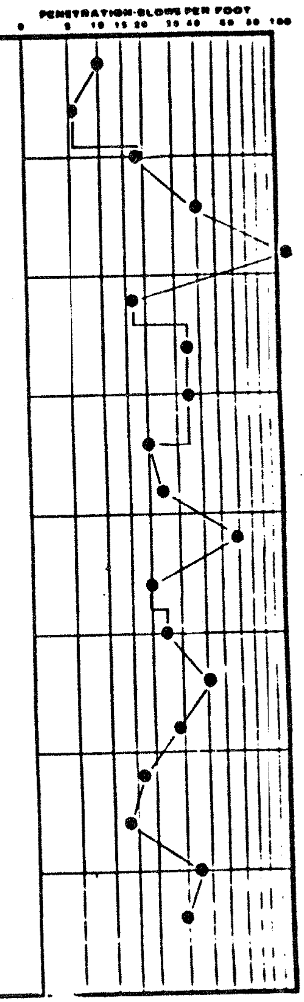
BORING NUMBER AS-27
DATE STARTED 11/11/74
DATE COMPLETED 11/15/74
JOB NUMBER SA-737
COORDINATES N 666.5 E 508.0

DRILLED WITH AW ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT

FIG.2G-A44

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
-21.5		GRAY MEDIUM TO FINE SAND AND SHELL PARTICLES, LITTLE SILT
-26.5	44.5	GRAY-TAN PARTIALLY CEMENTED FINE SAND
-31.5		
	52.0	TAN FINE SAND WITH SILT
-36.5		
	57.0	TAN PARTIALLY CEMENTED FINE SAND AND SHELL FRAGMENTS WITH CEMENTED SEAMS
-41.5		
	64.0	GRAY COARSE TO FINE SAND WITH SHELL FRAGMENTS
-46.5		
-51.5		
-56.5		
	77.5	BORING TERMINATED
61.5		



REMARKS:

DRILLED BY GIRDLER/F-1500
LOGGED BY JLP
CHECKED BY MEK

BORING NUMBER AS-27
DATE STARTED 11/11/74
DATE COMPLETED 11/15/74
JOB NUMBER SA-737

DRILLED WITH AW ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT

FIG.2G-A44 Cont.

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	S#	PENETRATION-BLOWS PER FOOT 0 5 10 15 20 30 40 60 80 100
+18.5		WASH DRILL TO 10.0'		
+13.5				
+ 8.5	10.0	NO RECOVERY	O	
	12.0	GRAY-TAN FINE SAND	T-2	
+ 3.5	14.0	GRAY SILTY FINE SAND		
	16.0	NO RECOVERY	O	
	18.0	TAN FINE SAND AND SHELL FRAGMENTS, LITTLE SILT		
- 1.5	20.0	TAN SLIGHTLY SILTY SAND	T-8	
	22.0	TAN SLIGHTLY SILTY SLIGHTLY SHELLY SAND		
- 6.5	24.0	TAN SLIGHTLY SILTY SAND	T-1	
	26.0	BROWN SILTY FINE SAND		
	27.5	BORING TERMINATED		
-11.5				

REMARKS: OSTERBERG SAMPLE
γ_t =BULK DENSITY (LB/FT3)
S#=UNDISTURBED SAMPLE NO.
O =OSTERBERG SAMPLE ATTEMPT

DRILLED BY GIRDLER/F-1500
LOGGED BY JLP
CHECKED BY GAW

BORING NUMBER AE-27A
DATE STARTED 11/22/74
DATE COMPLETED 11/22/74
JOB NUMBER SA-737
COORDINATES N 666.5
E 508.0

DRILLED W/N ROD AND 3-7/8" TRICONE ROLLER BIT BLOW COUNTS OBTAINED USING AW ROD

FIG.2G-A45

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	S#	PENETRATION-BLOWS PER FOOT 0 5 10 15 20 30 40 60 80 100
+18.5				
+13.5		WASH DRILL TO 11.5'		
+ 8.0				
	11.5	GRAY-TAN FINE SAND	T-1	
	13.5	GRAY-TAN SILTY SAND	JS	
+ 3.5	15.0	GRAY-TAN SILTY CLAY	T-3	
	17.0	SLIGHTLY SILTY FINE SAND WITH SHELLS	JS	
- 1.5	19.0	TAN FINE SAND	T-5	
	21.0	TAN SLIGHTLY SILTY FINE SAND	JS	
- 6.5	23.0	TAN SILTY FINE SAND	T-7	
	25.0	LIGHT BROWN SLIGHTLY SILTY FINE SAND	JS	
		BORING TERMINATED		

REMARKS:
γ_t =BULK DENSITY (LB/FT3)
S#=UNDISTURBED SAMPLE NO.
JS=JAR SAMPLE RETAINED
OSTERBERG SAMPLE

DRILLED BY GIRDLER/F-1500
LOGGED BY JLP
CHECKED BY GAW

BORING NUMBER AE-27B
DATE STARTED 11/22/74
DATE COMPLETED 11/22/74
JOB NUMBER SA-737
COORDINATES N 666.5
E 508.0

DRILLED W/AW ROD AND 3-7/8" SIDE DISCHARGE DRAG BIT, BLOW COUNTS OBTAINED USING AW ROD

FIG.2G-A46

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	S#
+18.5		WASH DRILL TO 10.0'	
+ 8.5	10.0	TAN FINE SAND WITH SHELL FRAGMENTS	T-1
	12.0	TAN FINE SAND	T-2
+ 3.5	14.0	TAN FINE SAND INTO GRAY SLIGHTLY SILTY CLAY	T-3
	16.0	NO RECOVERY	O
	18.0	NO RECOVERY	O
- 1.5	20.0	LIGHT GRAY SILTY FINE SAND	T-6
	22.0	LIGHT TAN SLIGHTLY SILTY FINE SAND	T-7
- 6.5	24.0	TAN SLIGHTLY SILTY FINE SAND	T-8
	26.0	GRAY SLIGHTLY SILTY FINE SAND	O
-11.5	28.0	WASH DRILL TO 38.0'	
	38.0	NO RECOVERY	O
-21.5	40.0	GRAY FINE SAND WITH CLAY SEAMS	T-11
	42.0	GRAY FINE SAND	T-12
-25.5	44.0	BORING TERMINATED	

PENETRATION-BLOWS PER FOOT 0 5 10 15 20 30 40 60 80 100

REMARKS:
γ_t=BULK DENSITY (LB/FT3)
S# = UNDISTURBED SAMPLE NO.
O = UNSUCCESSFUL SAMPLE ATTEMPT
J=JAR SAMPLE RETAINED
[illegible] SAMPLES
DRILLED W/N [illegible] AND 3-7/8" TRICONE ROLLER BIT

DRILLED BY GIEDLER/T-1500
LOGGED BY JLP
CHECKED BY GAW

BORING NUMBER AE-27C
DATE STARTED 11/15/74
DATE COMPLETED 11/26/74
JOB NUMBER SA-737
COORDINATES N 666.5 E 508.0

FIG. 1G-A47

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	S#	R
+18.5				
+13.5		WASH DRILL TO 10.0'		
+ 8.5	10.0	NO RECOVERY		
	12.0	TAN FINE SAND, SOME SILT	T-2	1.0
+ 3.5	14.0	TAN SILTY FINE SAND		
	16.0	GRAY MEDIUM TO FINE SAND WITH SHELL FRAGMENTS, TRACE CLAY	T-4	1.1
	18.0	TAN SILTY FINE SAND WITH SHELL FRAGMENTS		
- 1.5	20.0	GRAY FINE SAND, SOME SILT	T-6	1.0
	22.0	TAN FINE SAND, SOME SILT		
- 6.5	24.0	TAN FINE SAND, SOME SILT	T-8	2.0
	26.0	GRAY SLIGHTLY SILTY FINE SAND		
-11.5	27.5	BORING TERMINATED		

PENETRATION-BLOWS PER FOOT 0 5 10 15 20 30 40 60 80 100

REMARKS:
γ_t =BULK DENSITY (LB/FT3)
S#=UNDISTURBED SAMPLE NO.
R=RECOVERY (FT.)
[illegible] SAMPLES
DRILLED WITH N ROD AND 3-7/8" TRICONE ROLLER BIT
BLOW COUNTS OBTAINED USING AW ROD

DRILLED BY GIEDLER/T-1500
LOGGED BY CJR
CHECKED BY GAW

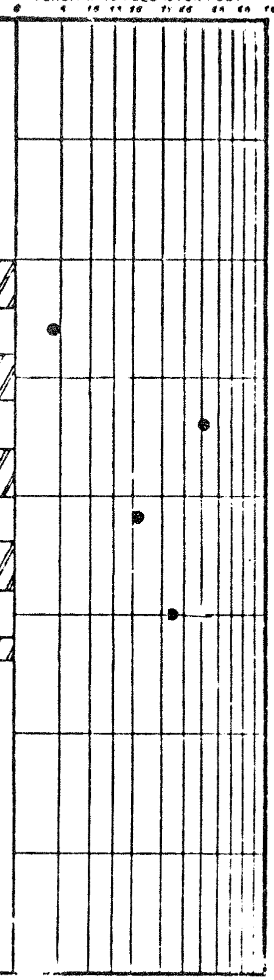
BORING NUMBER AE-27D
DATE STARTED 12/2/74
DATE COMPLETED [illegible]
JOB NUMBER SA-737
COORDINATES N [illegible] E [illegible]

FIG. 1G-A48

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION	SAMPLE
+18.5			
+13.5			
+ 8.5	10.0	TAN-GRAY FINE SAND WITH SHELL FRAGMENTS, SOME SILT	T-1
	12.0	TAN SILTY FINE SAND	
+ 3.5	14.0	GRAY SILT, SOME TAN FINE SAND	T-3
	16.0	GRAY-TAN SLIGHTLY SILTY FINE SAND	
- 1.5	18.0	GRAY TO TAN FINE SAND, SOME SILT	T-5
	20.0	TAN FINE SANDY SILT	
	22.0	TAN SILTY FINE SAND	T-6
- 6.5	24.0	TAN SLIGHTLY SHELLY SILTY FINE SAND WITH CLAY SEAMS	
	26.0	GRAY SLIGHTLY SHELLY FINE SAND	T-8
	27.0	BORING TERMINATED	
-11.5			

PENETRATION BLOWS PER FOOT



REMARKS: OSTERBERG SAMPLE

γ_t = BULK DENSITY (lb/ft^3)

S# = UNDISTURBED SAMPLE NO.

R = RECOVERY (ft)

DRILLED BY CIELLER/F-1500

LOGGED BY CJR

CHECKED BY GN

BORING NUMBER AE-27E

DATE STARTED 12/3/74

DATE COMPLETED 12/3/74

JOB NUMBER [illegible]

COORDINATES [illegible]

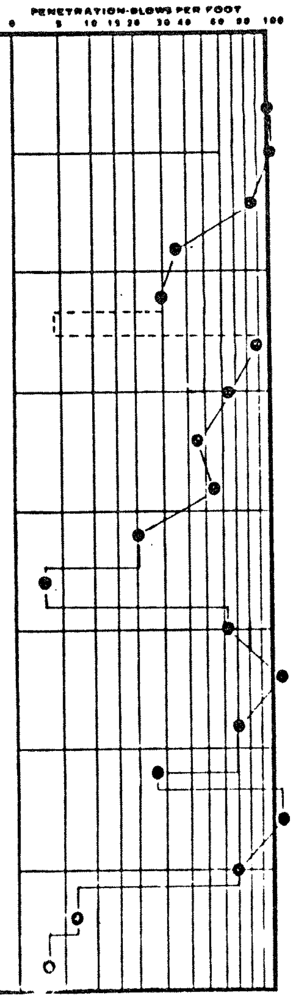
DRILLED WITH N ROD 3-7/8" TRICONE ROLLER BIT

FIG.2G-A49

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
+18.5		
+13.5		TAN FINE SAND WITH FEW SHELL FRAGMENTS
+ 8.5	11.6	
	12.5	BROWN SILTY CLAY
+ 3.5		LIGHT GRAY-TAN FINE SAND WITH SOME SHELL FRAGMENTS
- 1.5	22.3	
	23.8	TAN MEDIUM TO FINE SAND AND SHELL PARTICLES, SOME SILT
- 6.5		GRAY-TAN FINE SAND AND SHELLS, SOME SILT
-11.5	31.0	
	31.7	GRAY TAN SLIGHTLY CLAYEY FINE *(1)
-16.5	35.5	GRAY MEDIUM TO FINE SAND WITH NUMEROUS SHELL FRAGMENTS
	37.5	GRAY CLAY AND MEDIUM TO FINE SAND AND SHELL PARTICLES
-21.5		TAN TO GRAY FINE SAND, SOME CLAY

PENETRATION-BLOWS PER FOOT



REMARKS:
*(1) SANDY SILT

DRILLED WITH AW ROD AND 3 7/8" SIDE DISCHARGE DRAG BIT

DRILLED BY GIRDLER/F-250
LOGGED BY MEK
CHECKED BY MEK

BORING NUMBER AE-28
DATE STARTED 11/19/74
DATE COMPLETED 11/20/74
JOB NUMBER SA-737
COORDINATES N 596.5
E 495.0

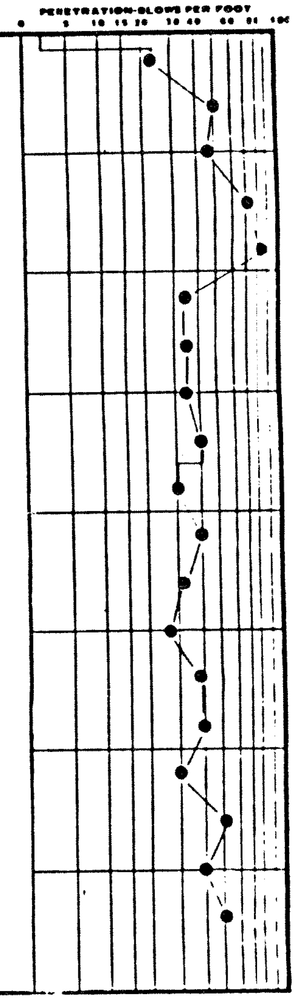
PAGE 1 OF 2 PAGES

FIG.2G-A50

TEST BORING RECORD

ELEV.	DEPTH FEET	DESCRIPTION
	40.5	[illegible] *(1)
-21.5		
-26.5		GRAY COARSE TO FINE SAND AND SHELL FRAGMENTS, TRACE SILT
-31.5		
-36.5	58.0	
-41.5		
-46.5		LIGHT GRAY TO GRAY SLIGHTLY SILTY FINE SAND WITH CEMENTED SAND AND SHELL AND LIMESTONE FRAGMENTS (to 66.0') AND NUMEROUS SHELL FRAGMENTS (66.0-74.0')
-51.5		
-56.5	77.5	
-61.5		BORING TERMINATED

PENETRATION-BLOWS PER FOOT



REMARKS:
*(1) TO VERY SILTY FINE SAND

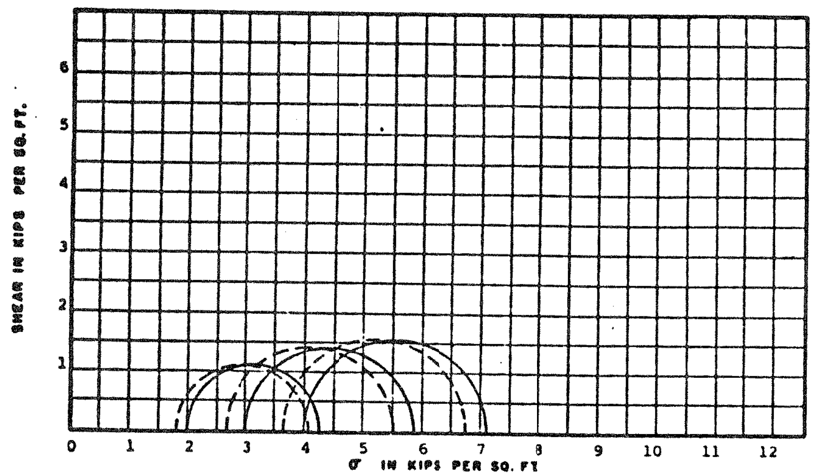
DRILLED WITH AW ROD AND 3 7/8" SIDE DISCHARGE DRAG BIT

DRILLED BY GIRDLER/F-250
LOGGED BY MEK
CHECKED BY MEK

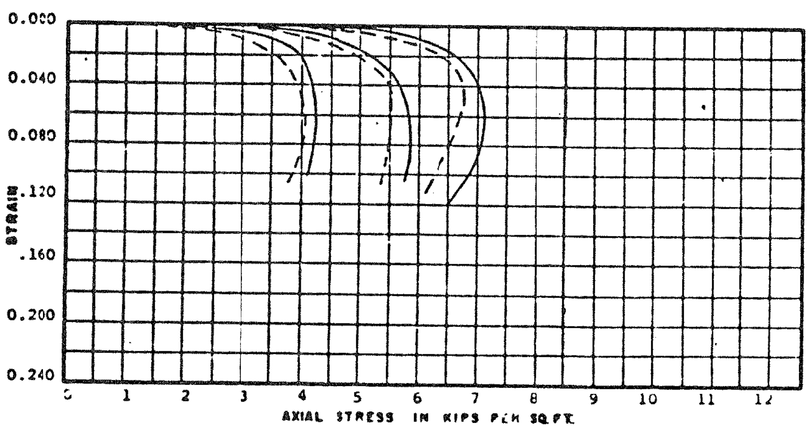
BORING NUMBER AE-28
DATE STARTED 11/18/74
DATE COMPLETED 11/20/74
JOB NUMBER SA-737

PAGE 2 OF 2 PAGES

FIG.2G-A50 Cont.



MOHR DIAGRAMS — φ



STRESS-STRAIN CURVES

"COHESION", C ______

ANGLE OF SHEAR RESISTANCE. φ ______

UNIT WEIGHT, γ_d 65.4, 74.5, 69.5

WATER CONTENT. w 61.0, 53.0, 65.5%

VOID RATIO. e 1.53, 1.22, 1.33

Fig 2G-B1

CONSOLIDATED UNDRAINED

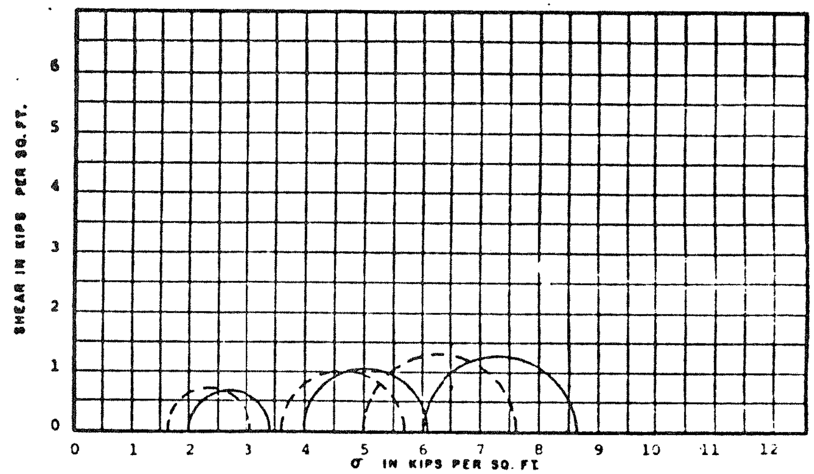
TRIAXIAL SHEAR TEST

BORING NO. AE-1A SAMPLE NO. UD-2

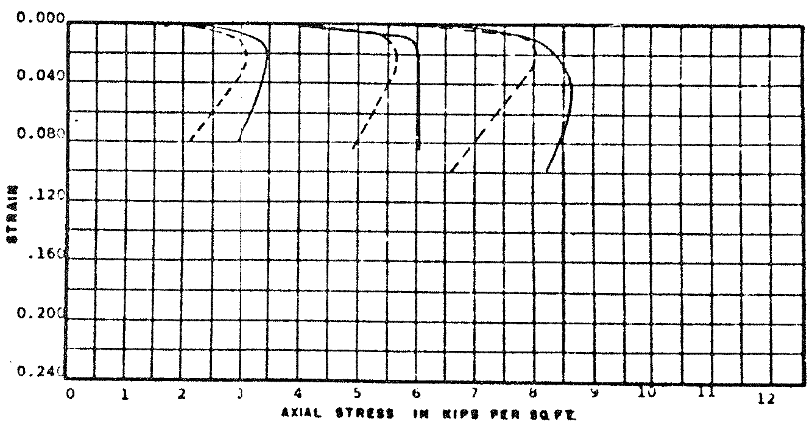
ELEV. OR DEPTH 34.0'-36.0' JOB NO. SA-737

LAW ENGINEERING TESTING CO.

JACKSONVILLE, FLORIDA



MOHR DIAGRAMS — φ



STRESS-STRAIN CURVES

______ Total σ_1
------ Effective σ_1

SATURATED, CONSOLIDATED UNDRAINED TRIAXIAL SHEAR TEST WITH PORE PRESSURE MEASUREMENTS

"COHESION", C ______

ANGLE OF SHEAR RESISTANCE. φ ______

UNIT WEIGHT, γ_1 62.0, 73.2, 60.6 PCF

WATER CONTENT. w 65.0, 46.6, 57.3%

VOID RATIO. e 1.72, 1.30, 1.78

Fig 2G-B2

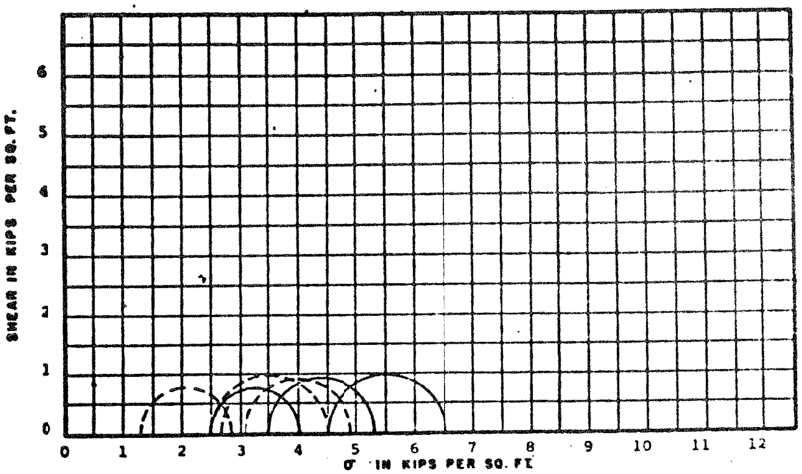
TRIAXIAL SHEAR TEST

BORING NO. AE-2A SAMPLE NO. UD-2

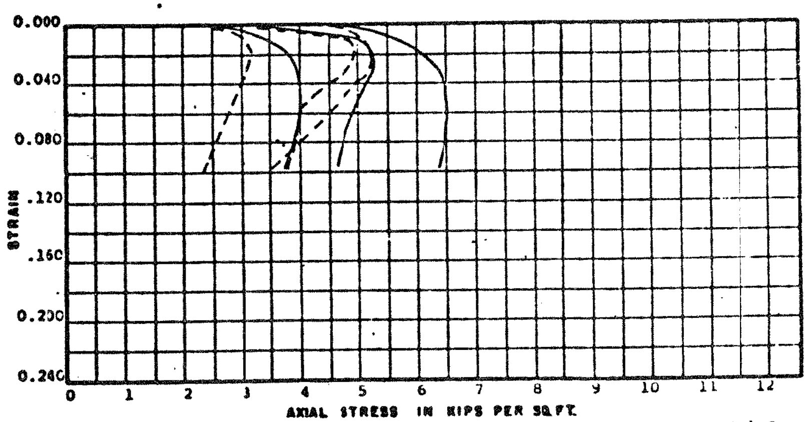
ELEV. OR DEPTH [illegible] JOB NO. [illegible]

LAW ENGINEERING TESTING CO.

BIRMINGHAM, ALABAMA



MOHR DIAGRAMS — φ



STRESS-STRAIN CURVES

Total σ_1

Effective σ_1

SATURATED, CONSOLIDATED UNDRAINED TRIAXIAL SHEAR TEST WITH PORE PRESSURE MEASUREMENTS

"COHESION", C

ANGLE OF SHEAR RESISTANCE, φ

UNIT WEIGHT, γ_d 62.4, 62.7, 71.4 PCF

WATER CONTENT, W 60.0, 55.0, 48.1%

VOID RATIO, e 1.70, 1.69, 1.36

TRIAXIAL SHEAR TEST

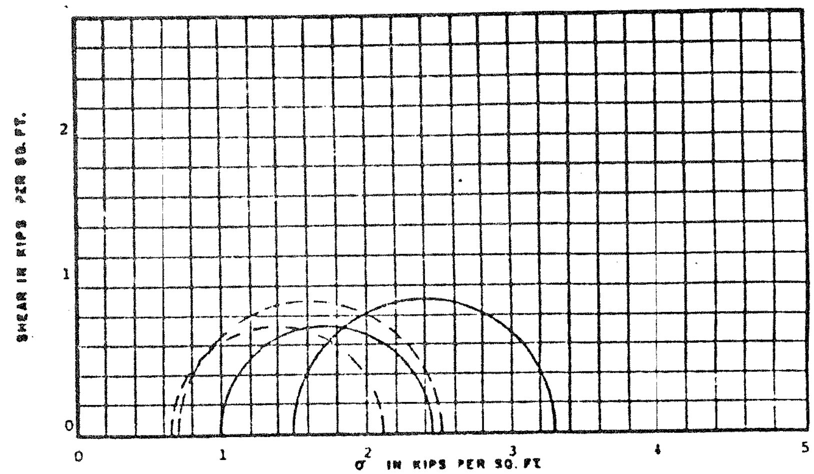
BORING NO. AE-2C SAMPLE NO. UD-4

ELEV. OR DEPTH 32.5'-34.5' JOB NO. SA-737

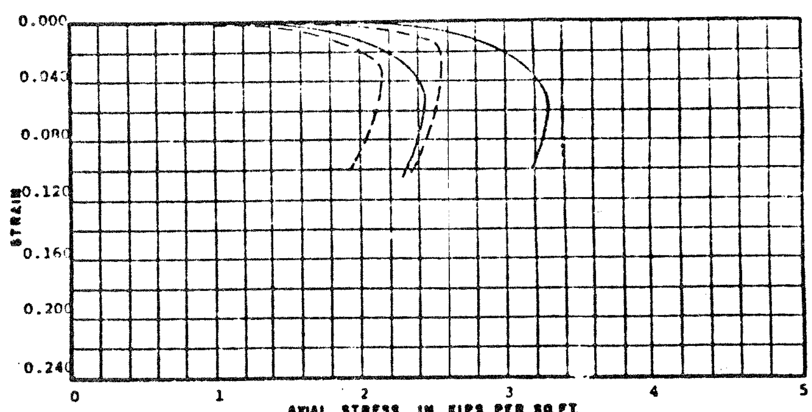
LAW ENGINEERING TESTING CO.

BIRMINGHAM, ALABAMA

Fig 2C-B3



MOHR DIAGRAMS — φ



STRESS-STRAIN CURVES

TOTAL σ_1

EFFECTIVE σ_1

SATURATED, CONSOLIDATED UNDRAINED WITH PORE PRESSURE MEASUREMENTS

"COHESION", C

ANGLE OF SHEAR RESISTANCE, φ

UNIT WEIGHT, γ_d 78.2, 75.7 PCF

WATER CONTENT, W 41.7, 41.0 %

VOID RATIO, e 1.11, 1.14

TRIAXIAL SHEAR TEST

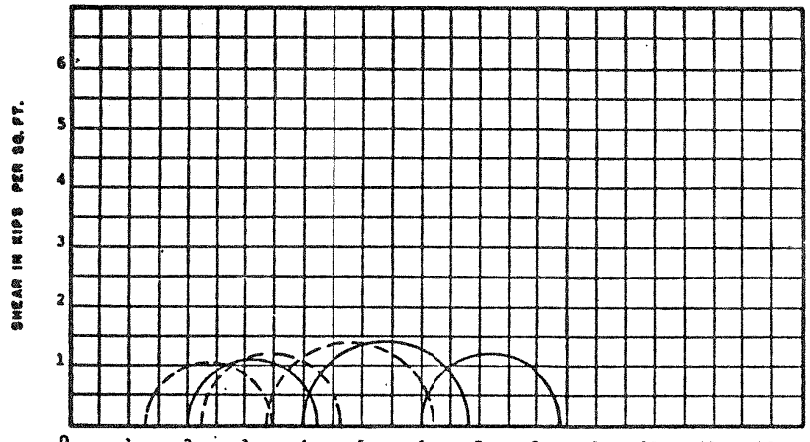
BORING NO. AE-2C SAMPLE NO. UD-5

ELEV. OR DEPTH 34.[illegible] JOB NO. [illegible]

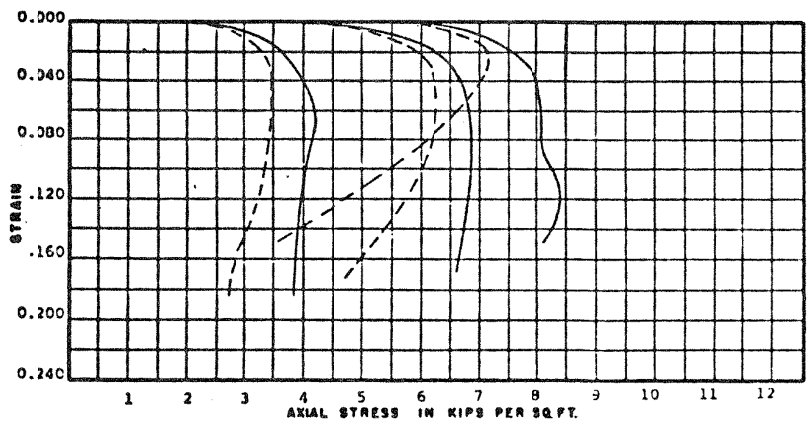
LAW ENGINEERING TESTING CO.

BIRMINGHAM, ALABAMA

Fig 2C-B4



MOHR DIAGRAMS — ϕ



STRESS-STRAIN CURVES

TOTAL σ_1
EFFECTIVE σ_1

"COHESION", C

ANGLE OF SHEAR RESISTANCE, ϕ

UNIT WEIGHT, γ_d 71.9, 63.9, 72.4

WATER CONTENT, W [illegible]

VOID RATIO, e [illegible]

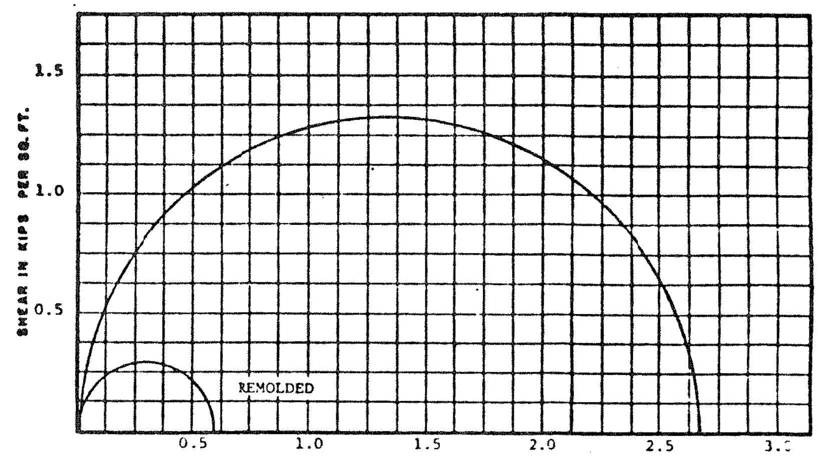
Fig 2G-B5

SATURATED CONSOLIDATED
UNDRAINED WITH PORE
PRESSURE MEASUREMENT

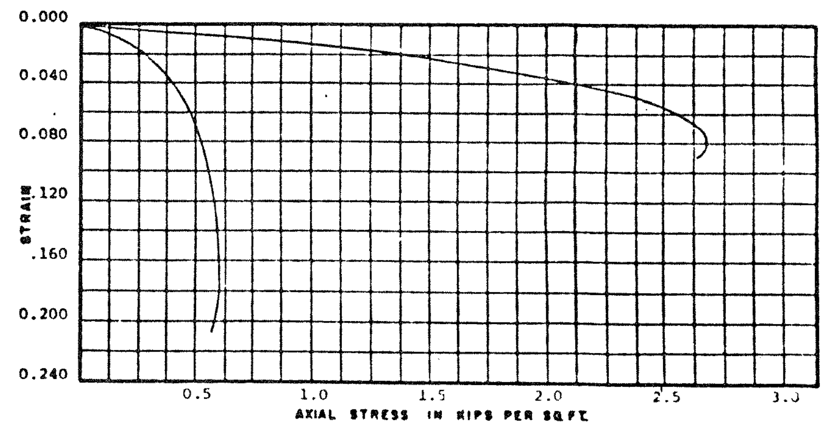
TRIAXIAL SHEAR TEST

BORING NO. AE-4A SAMPLE NO. T-6
ELEV. OR DEPTH [illegible]-40.0' JOB NO. SA-737

LAW ENGINEERING TESTING CO.
JACKSONVILLE, FLORIDA



MOHR DIAGRAMS — ϕ



STRESS-STRAIN CURVES

"COHESION", C

ANGLE OF SHEAR RESISTANCE, ϕ

UNIT WEIGHT, γ_d 61.0, 63.0 PCF

WATER CONTENT, W 60.7, 53.3%

VOID RATIO, e 1.6, 1.5

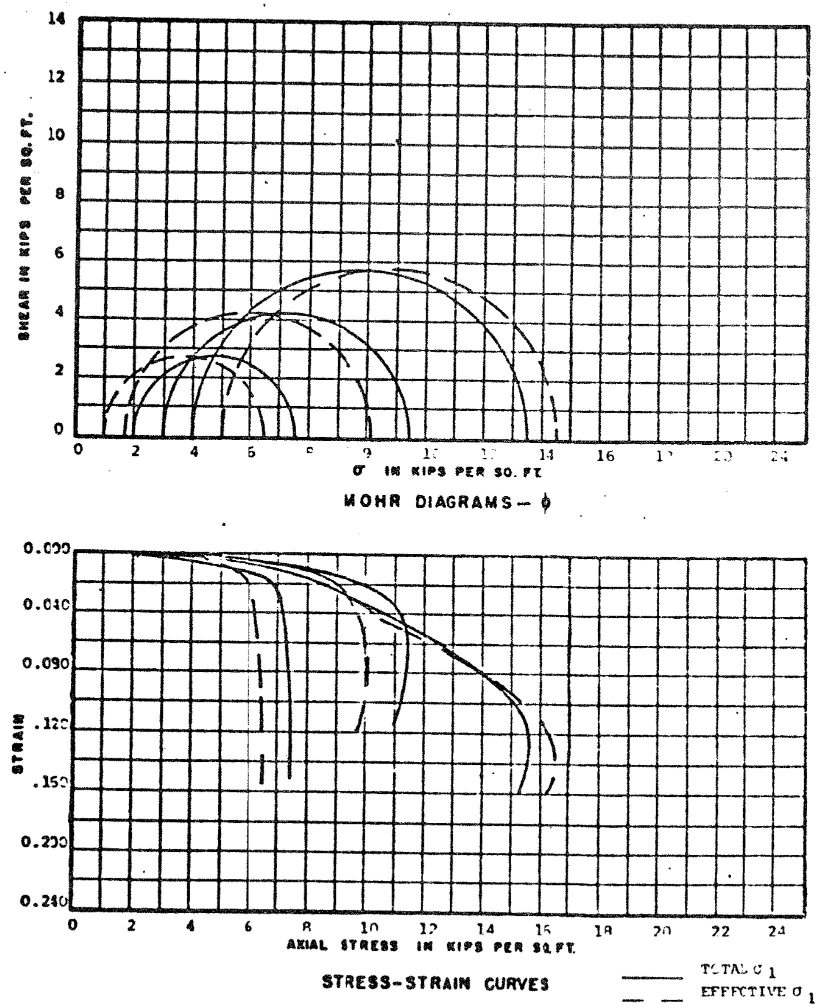
Fig 2G-B6

UNCONFINED COMPRESSION

TRIAXIAL SHEAR TEST

BORING NO. [illegible]-4A SAMPLE NO. T-6D
ELEV. OR DEPTH [illegible] JOB NO. [illegible]

LAW ENGINEERING TESTING CO.
JACKSONVILLE, FLORIDA



"COHESION", C ____________

ANGLE OF SHEAR RESISTANCE ϕ ____________

UNIT WEIGHT, γ_d 76.3, 75.0, 85.2 PCF

WATER CONTENT, w 32.6, 41.1, 32.8%

VOID RATIO, e 1.20, 1.25, 0.99

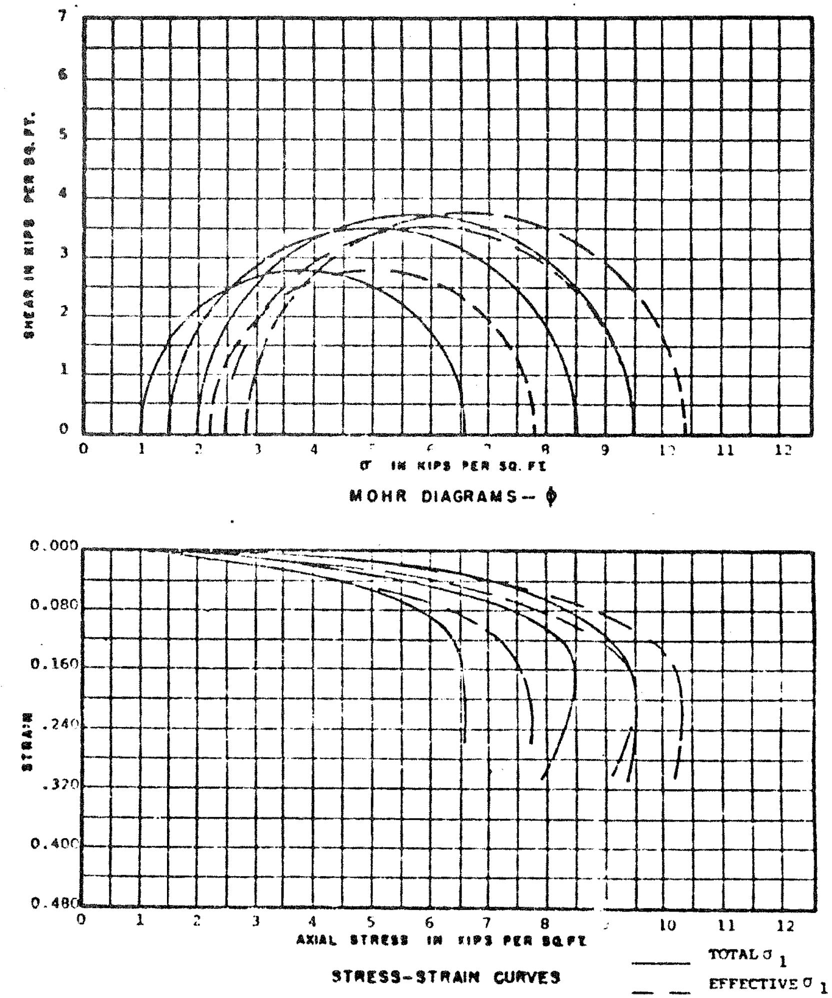
SATURATED, CONSOLIDATED UNDRAINED
WITH PORE PRESSURE MEASUREMENTS

TRIAXIAL SHEAR TEST

BORING NO. AE-1A SAMPLE NO. UD-1
ELEV. OR DEPTH 27.1-29.0 JOB NO. AA-732

LAW ENGINEERING TESTING CO.
BIRMINGHAM, ALABAMA

Fig 2G-B7



"COHESION", C ____________

ANGLE OF SHEAR RESISTANCE ϕ ____________

UNIT WEIGHT, γ_d 81.[illegible], 75.6, 77.1 PCF

WATER CONTENT, w 26.7, 26.8, 27.9%

VOID RATIO, e 1.06, 1.18, 1.10

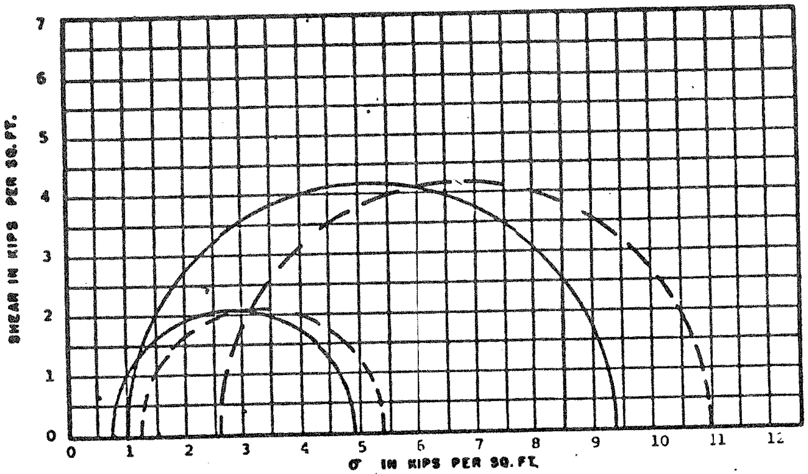
SATURATED, CONSOLIDATED UNDRAINED
WITH PORE PRESSURE MEASUREMENTS

TRIAXIAL SHEAR TEST

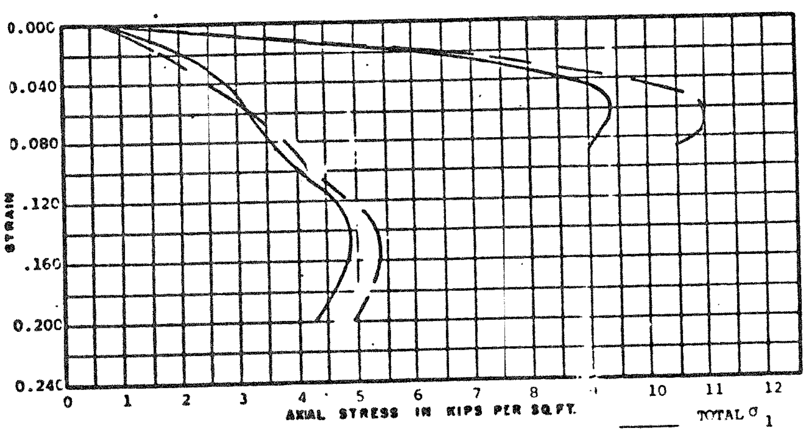
BORING NO. AE-5A SAMPLE NO. UD-2
ELEV. OR DEPTH 30.[illegible]-32.4 JOB NO. [illegible]

LAW ENGINEERING TESTING CO.
BIRMINGHAM, ALABAMA

Fig. 2G-B8



MOHR DIAGRAMS— φ



STRESS-STRAIN CURVES

——— TOTAL σ_1
— — EFFECTIVE σ_1

"COHESION", c ____

ANGLE OF SHEAR RESISTANCE, φ ____

UNIT WEIGHT, γ_d 71.0, 83.9 PCF

WATER CONTENT, w 43.2, 28.9%

VOID RATIO, e 1.30, 1.00

Fig. 2G-B9

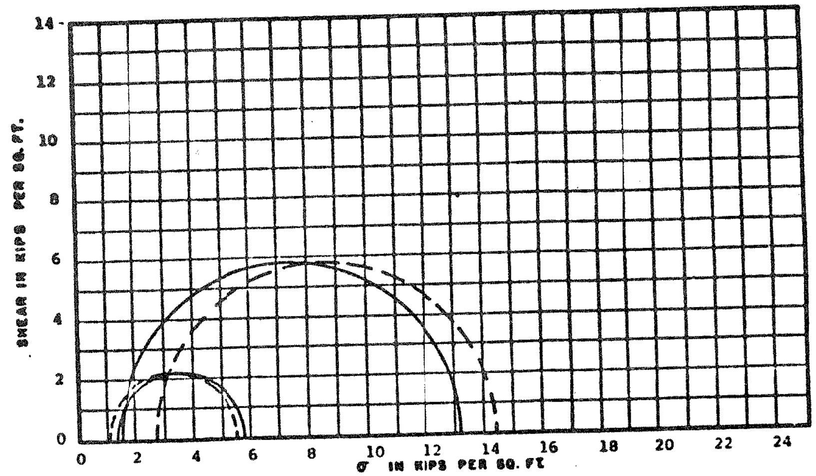
SATURATED, CONSOLIDATED UNDRAINED WITH PORE PRESSURE MEASUREMENTS

TRIAXIAL SHEAR TEST

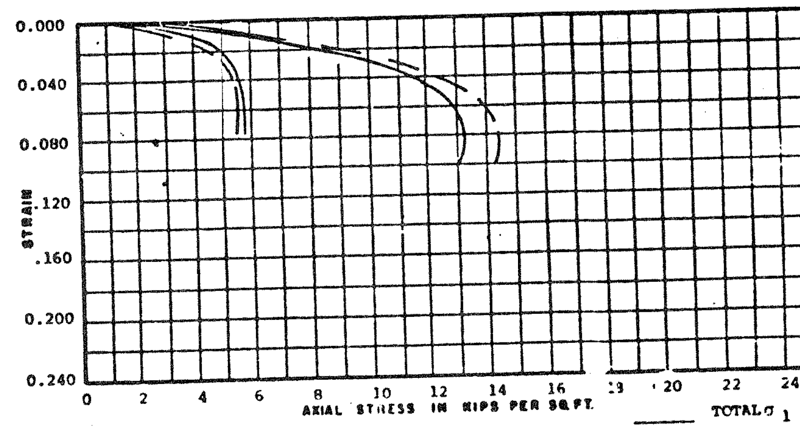
BORING NO. AE-5B SAMPLE NO. T-1

ELEV. OR DEPTH 10.0-12.0 JOB NO. SA-737

LAW ENGINEERING TESTING CO.
BIRMINGHAM, ALABAMA



MOHR DIAGRAMS— φ



STRESS-STRAIN CURVES

——— TOTAL σ_1
— — EFFECTIVE σ_1

"COHESION", c ____

ANGLE OF SHEAR RESISTANCE, φ ____

UNIT WEIGHT, γ_d 86.7, 82.8, PCF

WATER CONTENT, w 24.5, 33.2%

VOID RATIO, e 0.92, 1.00

Fig. 2G-B10

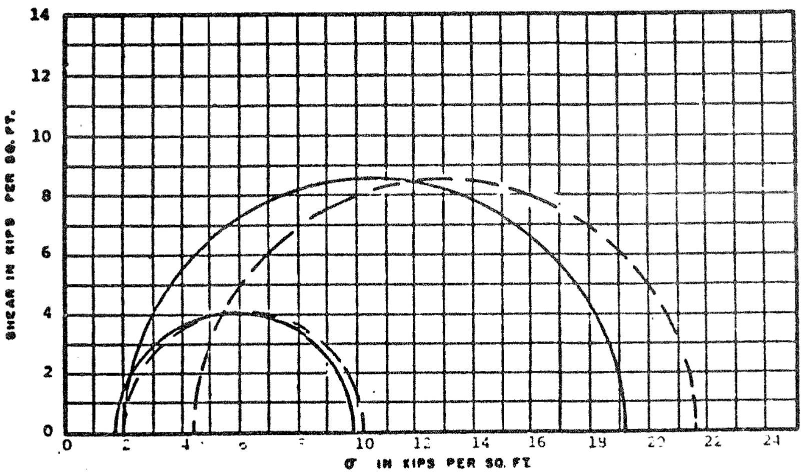
SATURATED, CONSOLIDATED UNDRAINED WITH PORE PRESSURE MEASUREMENTS

TRIAXIAL SHEAR TEST

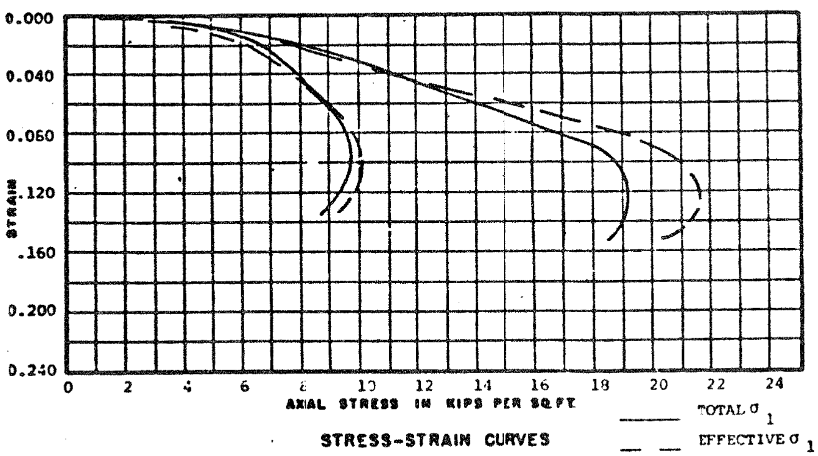
BORING NO. AE-5B SAMPLE NO. T-2

ELEV. OR DEPTH 12.0-1[illegible] JOB NO. [illegible]

LAW ENGINEERING TESTING CO.
BIRMINGHAM, ALABAMA



MOHR DIAGRAMS— φ



STRESS-STRAIN CURVES

"COHESION", C

ANGLE OF SHEAR RESISTANCE. φ

UNIT WEIGHT, γ_d 83.5, 85.4 PCF

WATER CONTENT. W 35.0, 33.5%

VOID RATIO. e 1.03, 0.98

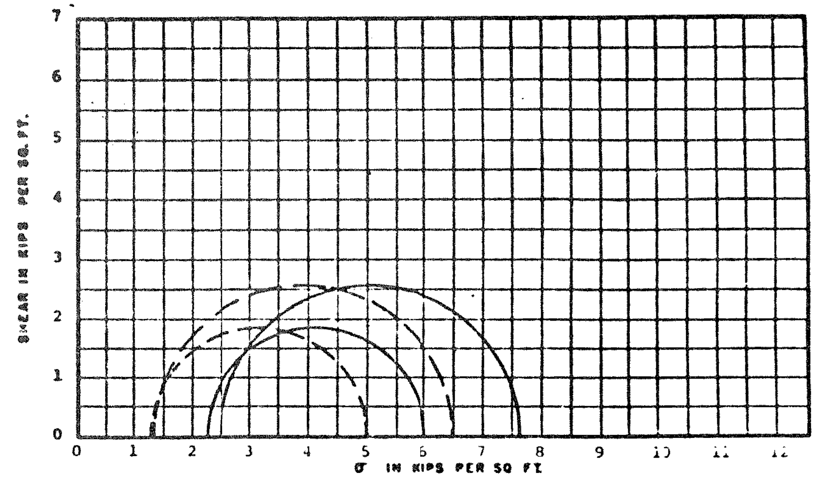
Fig 2G-B11

SATURATED, CONSOLIDATED UNDRAINED
WITH PORE PRESSURE MEASUREMENTS

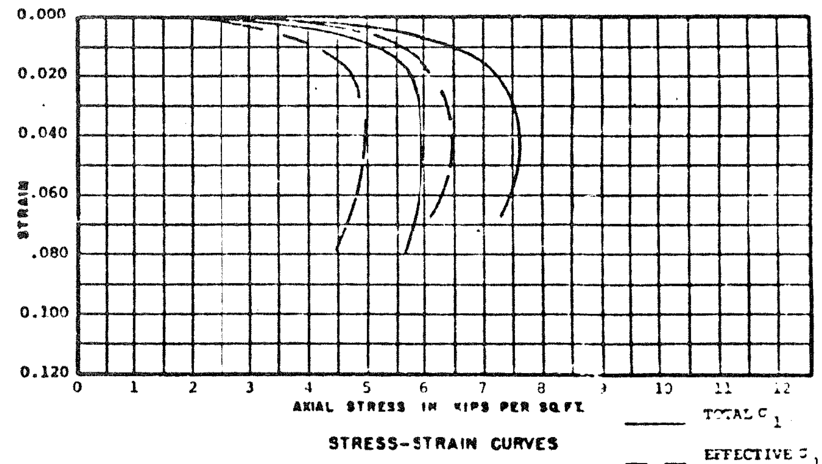
TRIAXIAL SHEAR TEST

BORING NO. AE-5B SAMPLE NO. T-3
ELEV. OR DEPTH 14.0-16.0 JOB NO. SA-737

LAW ENGINEERING TESTING CO.
BIRMINGHAM, ALABAMA



MOHR DIAGRAMS— φ



STRESS-STRAIN CURVES

"COHESION", C

ANGLE OF SHEAR RESISTANCE. φ

UNIT WEIGHT, γ_d 82.8, 80.9 PCF

WATER CONTENT. W 37.3, 37.6%

VOID RATIO. e 1.05, 1.03

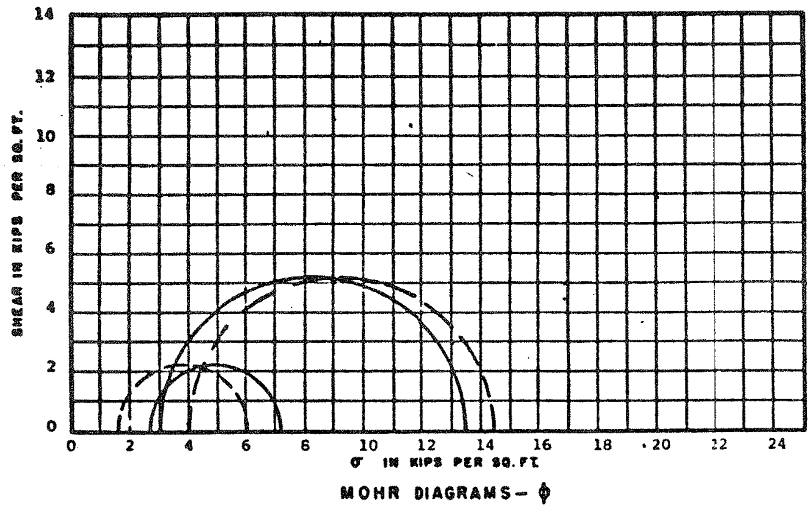
Fig. 2G-B12

SATURATED, CONSOLIDATED UNDRAINED
WITH PORE PRESSURE MEASUREMENTS

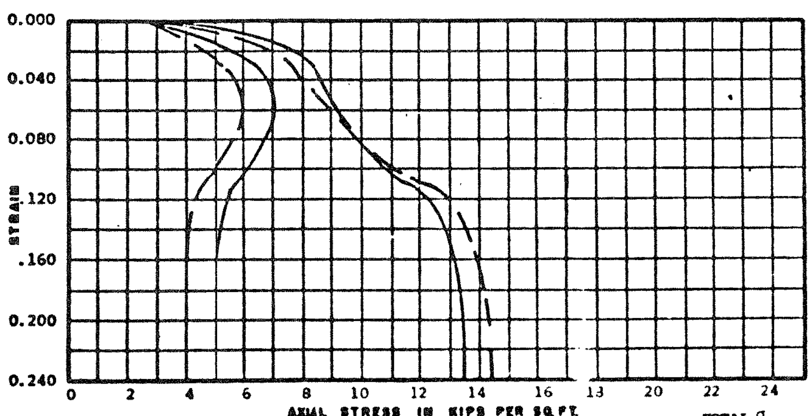
TRIAXIAL SHEAR TEST

BORING NO. AE-5B SAMPLE NO. T-[illegible]
ELEV. OR DEPTH 16.[illegible] JOB NO. SA-737

LAW ENGINEERING TESTING CO.
BIRMINGHAM, ALABAMA



MOHR DIAGRAMS— φ



STRESS-STRAIN CURVES

TOTAL σ_1

EFFECTIVE σ_1

"COHESION", C ____

ANGLE OF SHEAR RESISTANCE, φ ____

UNIT WEIGHT, γ_d 80.9, 87.8 PCF

WATER CONTENT, w 29.6, 29.8%

VOID RATIO, e 1.05, 1.11

Fig 2G-B13

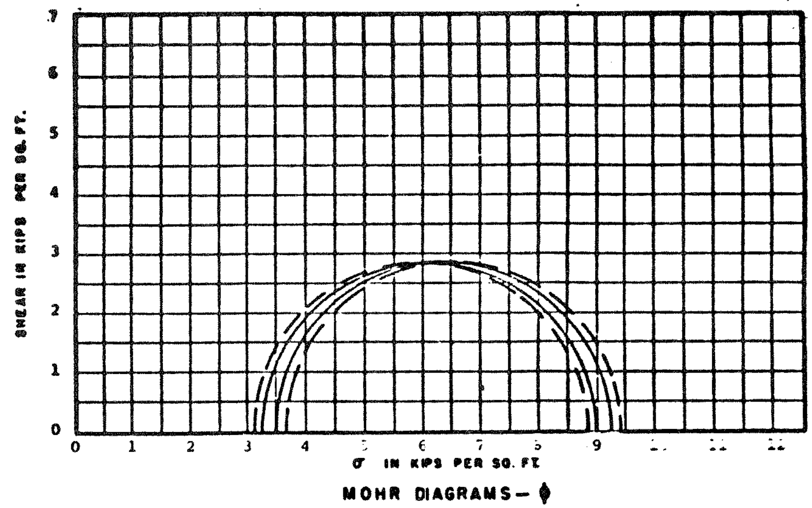
SATURATED, CONSOLIDATED UNDRAINED WITH PORE PRESSURE MEASUREMENTS

TRIAXIAL SHEAR TEST

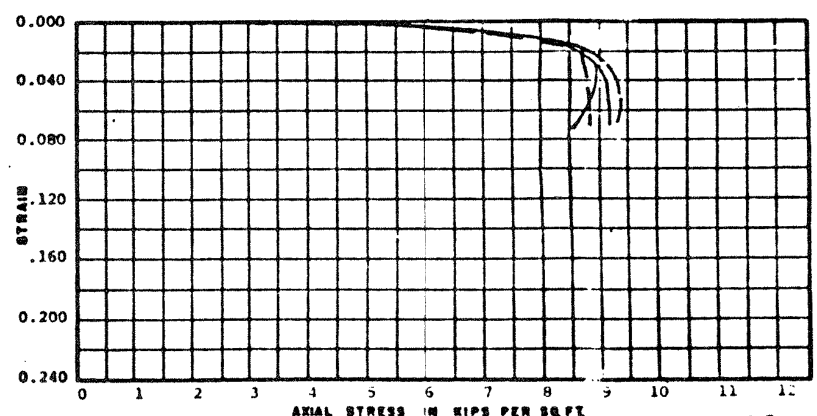
BORING NO. AE-5B SAMPLE NO. T-5

ELEV. OR DEPTH 28.0-30.0 JOB NO. SA-737

LAW ENGINEERING TESTING CO.
BIRMINGHAM, ALABAMA



MOHR DIAGRAMS— φ



STRESS-STRAIN CURVES

TOTAL σ_1

EFFECTIVE σ_1

"COHESION", C ____

ANGLE OF SHEAR RESISTANCE, φ ____

UNIT WEIGHT, γ_d 74.9, 77.4 PCF

WATER CONTENT, w 35.1, 29.7%

VOID RATIO, e 1.24, 1.19

Fig 2G-B14

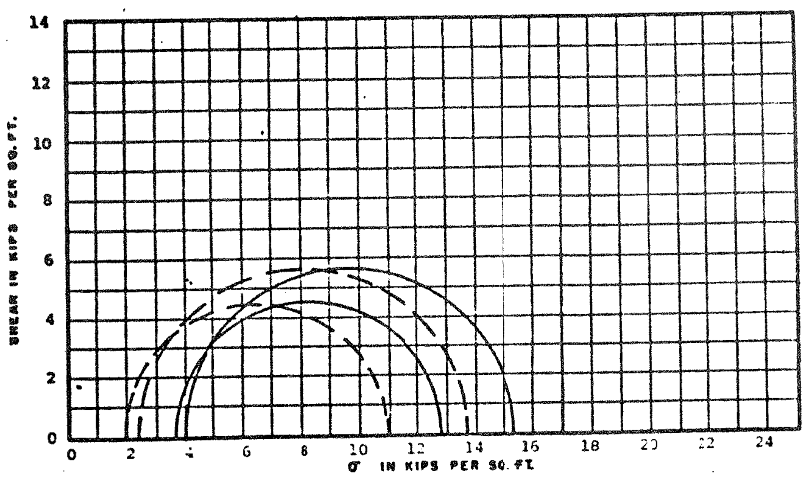
SATURATED, CONSOLIDATED UNDRAINED WITH PORE PRESSURE MEASUREMENTS

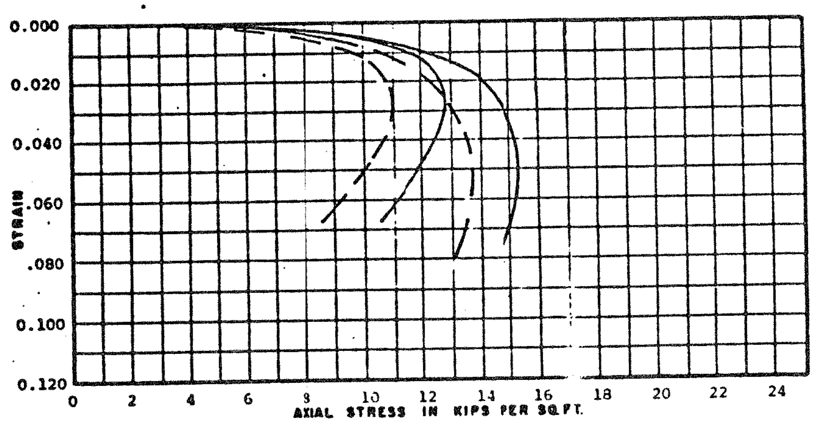
TRIAXIAL SHEAR TEST

BORING NO. AE-5B SAMPLE NO. T-6

ELEV. OR DEPTH 30.0-32.0 JOB NO. SA-737

LAW ENGINEERING TESTING CO.
BIRMINGHAM, ALABAMA





"COHESION", C ______

ANGLE OF SHEAR RESISTANCE, φ ______

UNIT WEIGHT, γ_d 87.5, 93.0 PCF

WATER CONTENT, w 28.4, 29.5%

VOID RATIO, e 0.94, 0.82

Fig 2G–B15

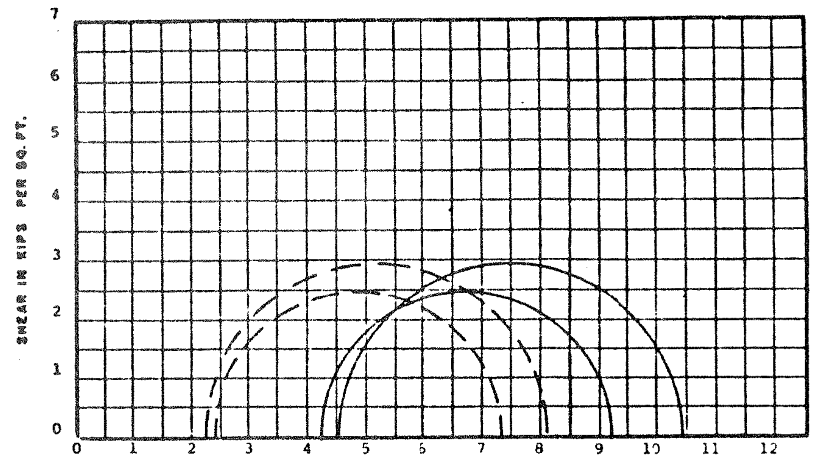
SATURATED, CONSOLIDATED UNDRAINED
WITH PORE PRESSURE MEASUREMENTS

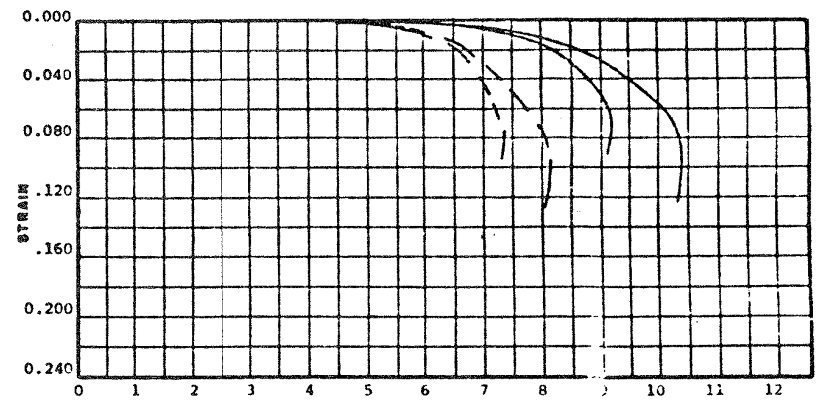
TRIAXIAL SHEAR TEST

BORING NO. AE-5B SAMPLE NO. T-7

ELEV. OR DEPTH 32.0-34.0 JOB NO. SA-737

LAW ENGINEERING TESTING CO.
BIRMINGHAM, ALABAMA





"COHESION", C ______

ANGLE OF SHEAR RESISTANCE, φ ______

UNIT WEIGHT, γ_d 100.0, 99.4 PCF

WATER CONTENT, w 22.9, 23.8%

VOID RATIO, e 0.69, 0.68

Fig 2G–B16

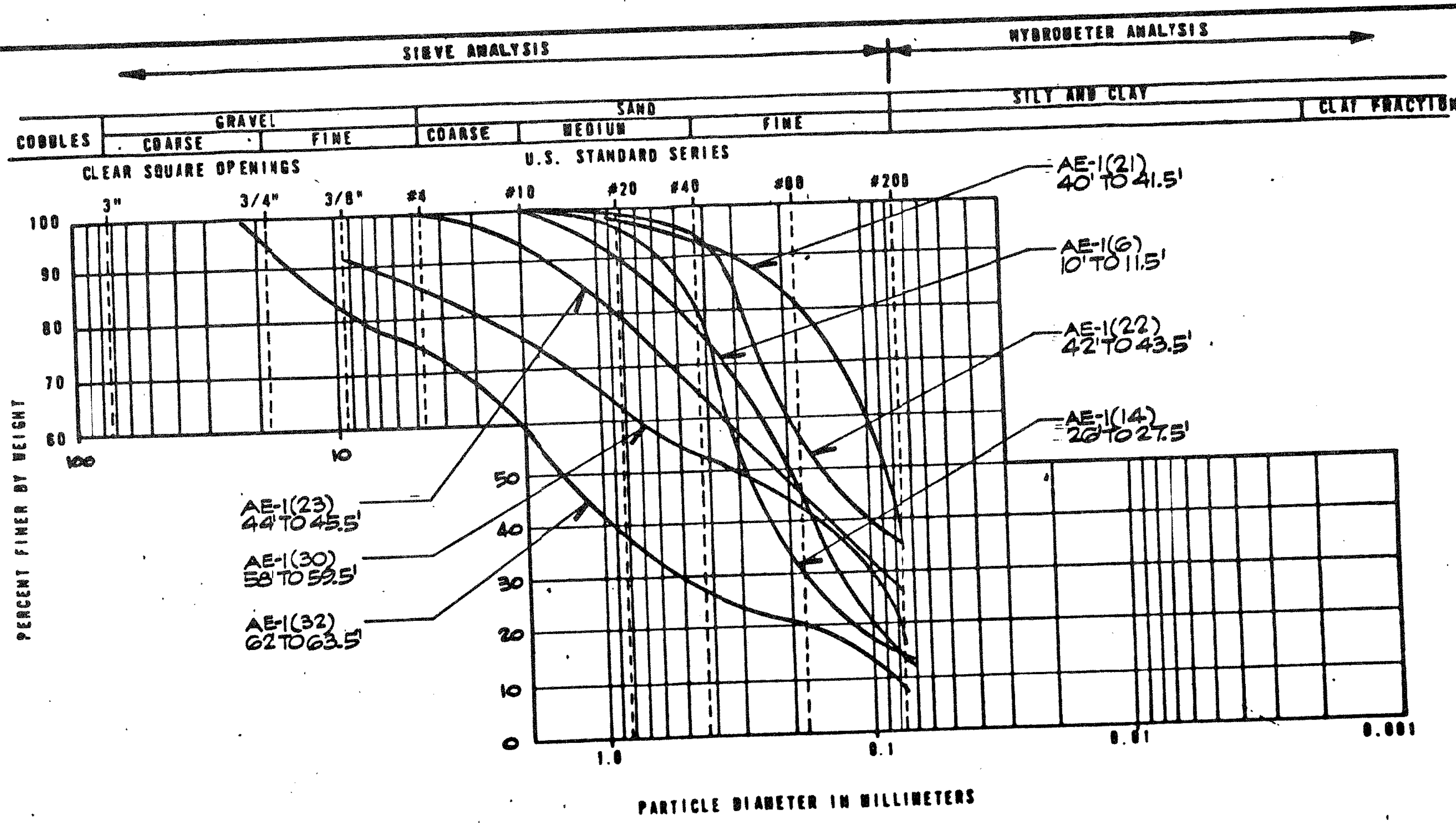
SATURATED, CONSOLIDATED UNDRAINED
WITH PORE PRESSURE MEASUREMENTS

TRIAXIAL SHEAR TEST

BORING NO. AE-5B SAMPLE NO. T-8

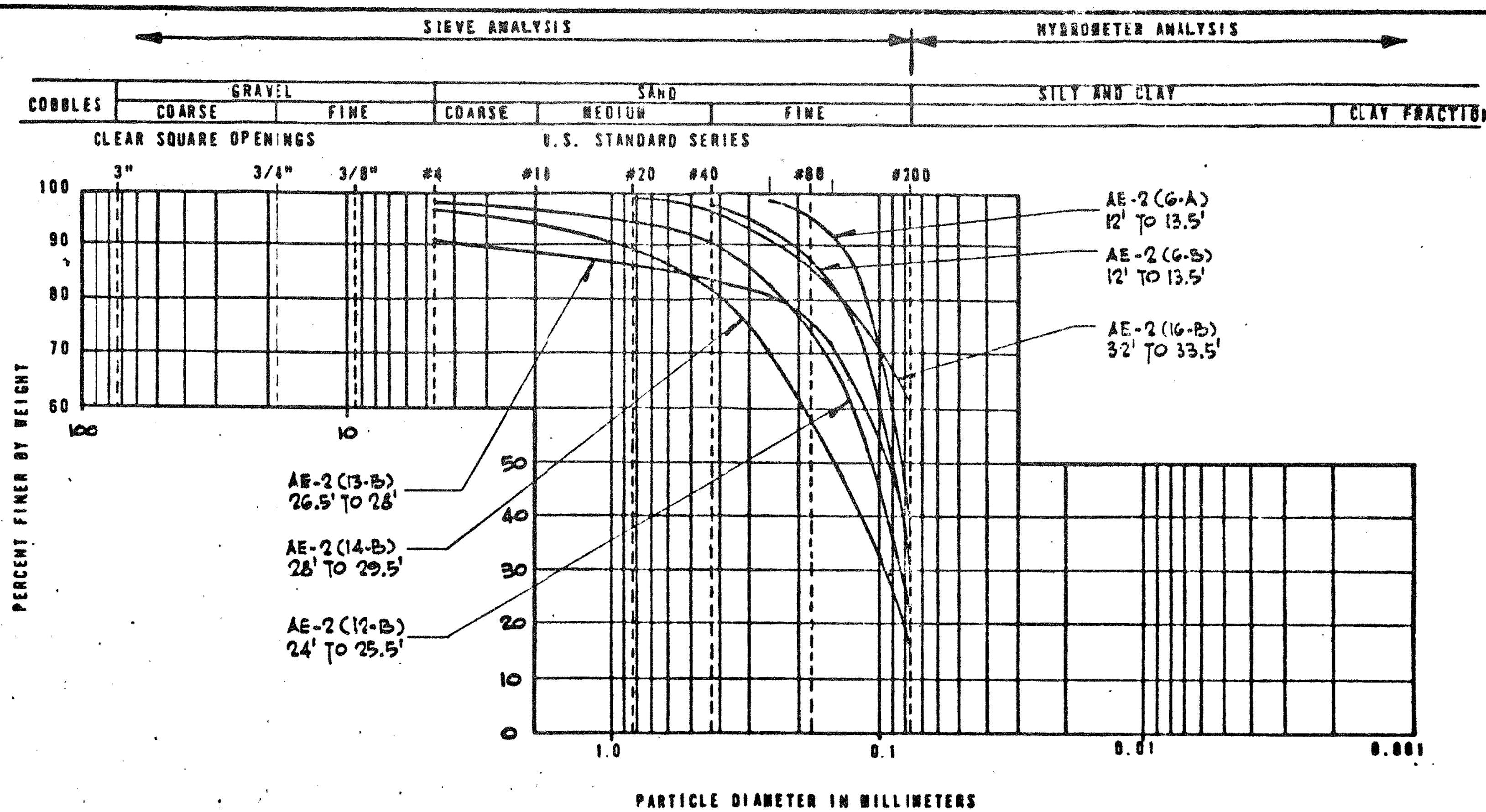
ELEV. OR DEPTH 34-36 JOB NO. [illegible]

LAW ENGINEERING TESTING CO.
BIRMINGHAM, ALABAMA



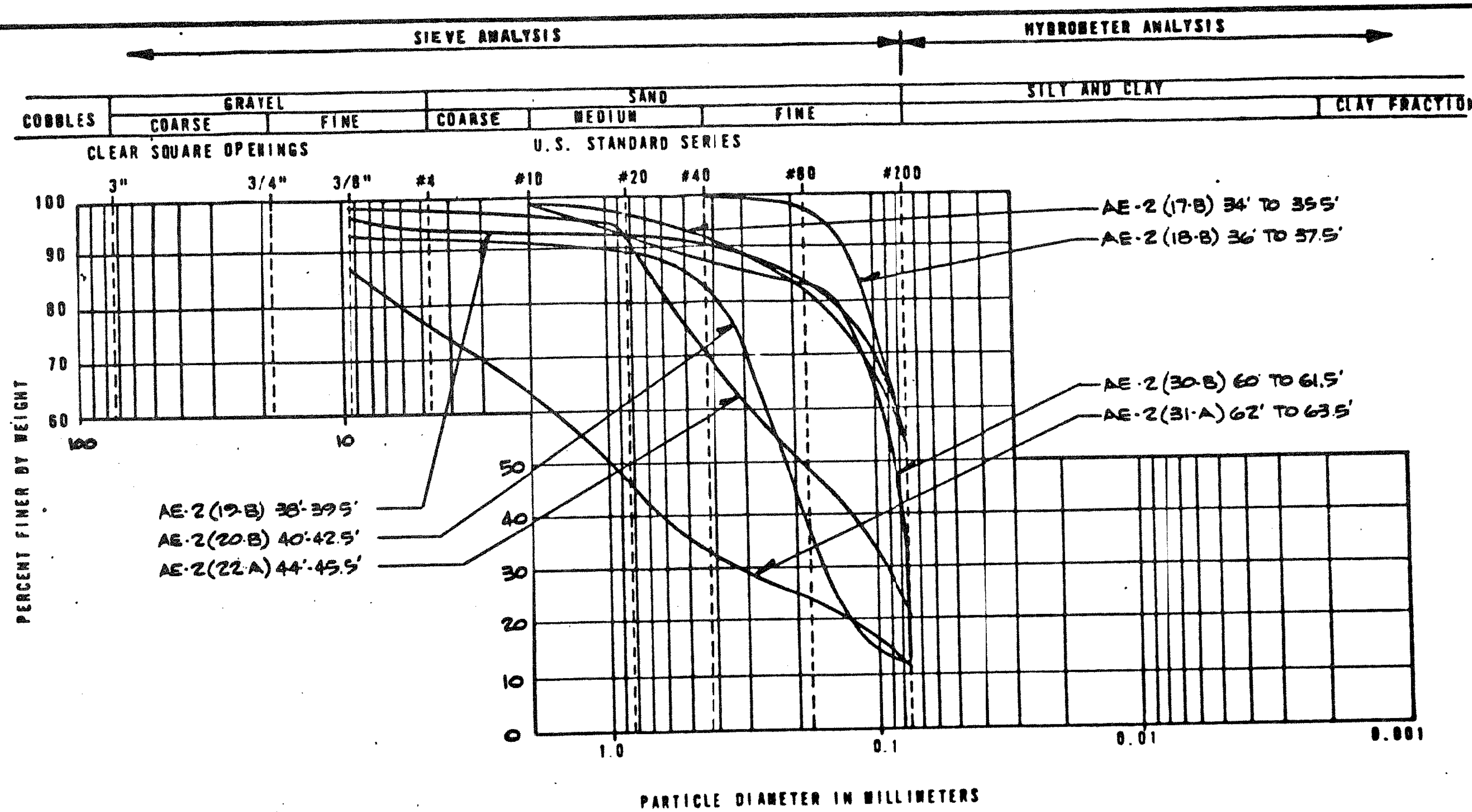
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-1
FIGURE 2G-C1



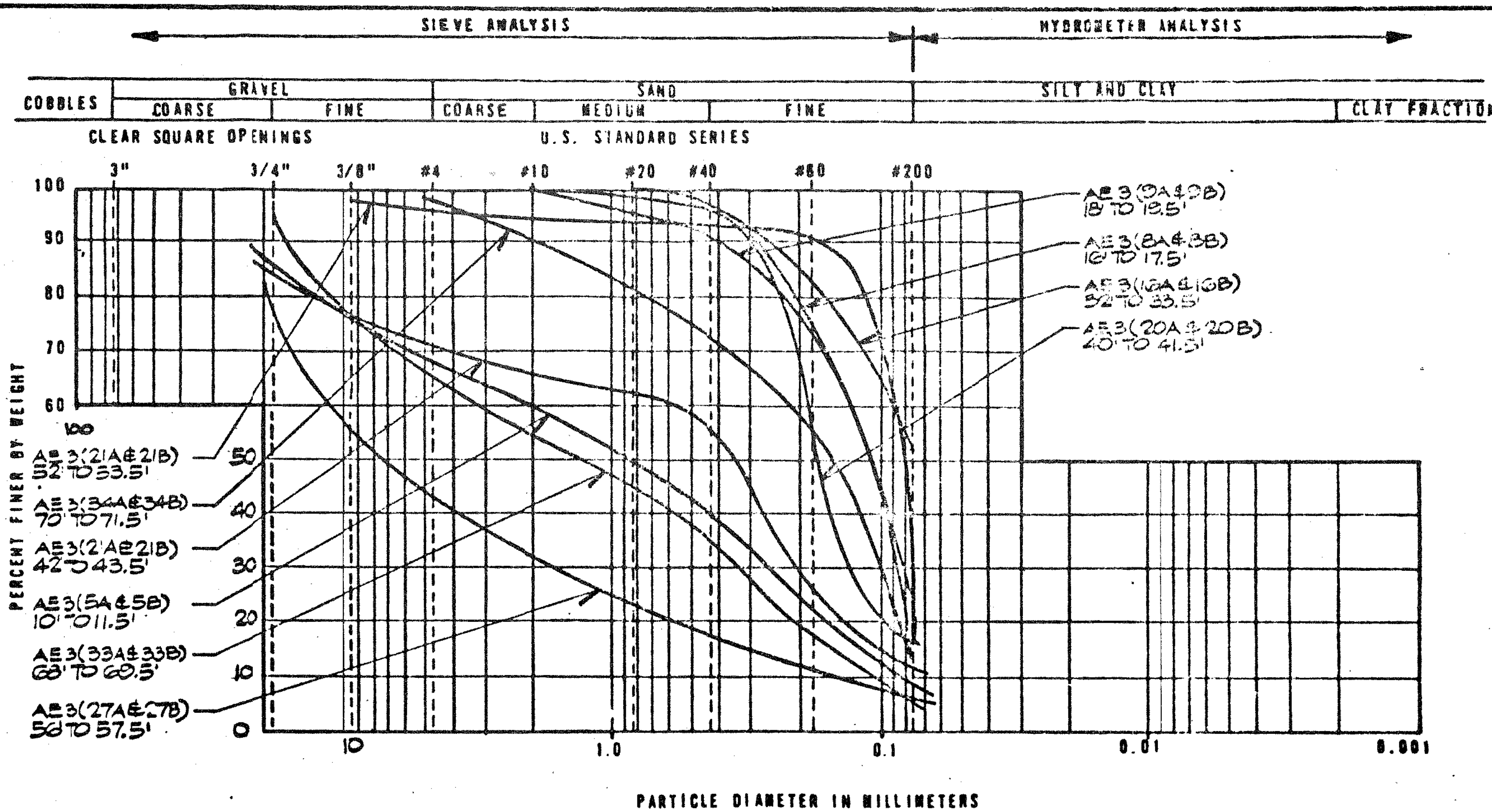
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-2
FIGURE 2G-C2



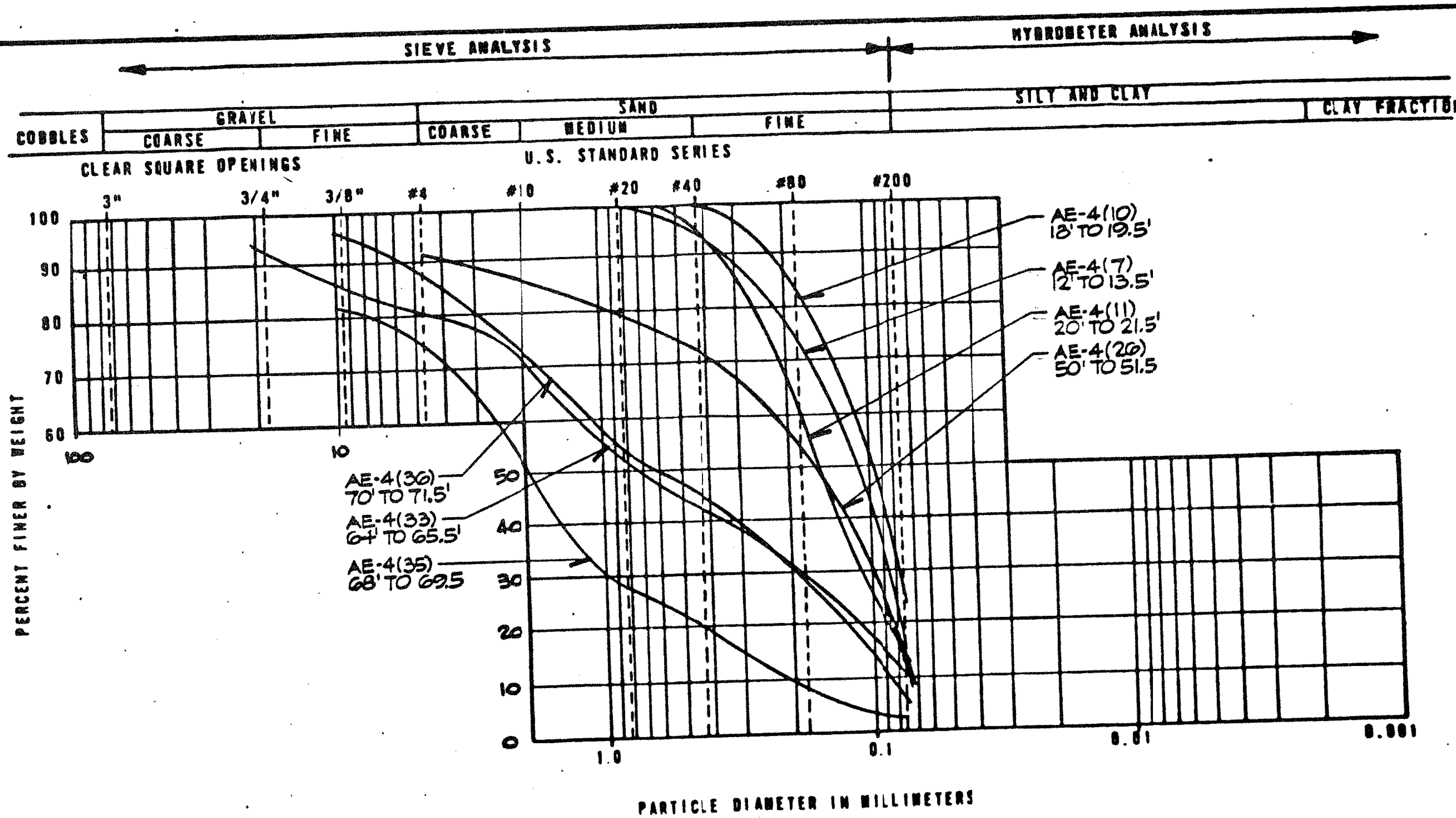
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-2
FIGURE 2G-C2 (CON'T)



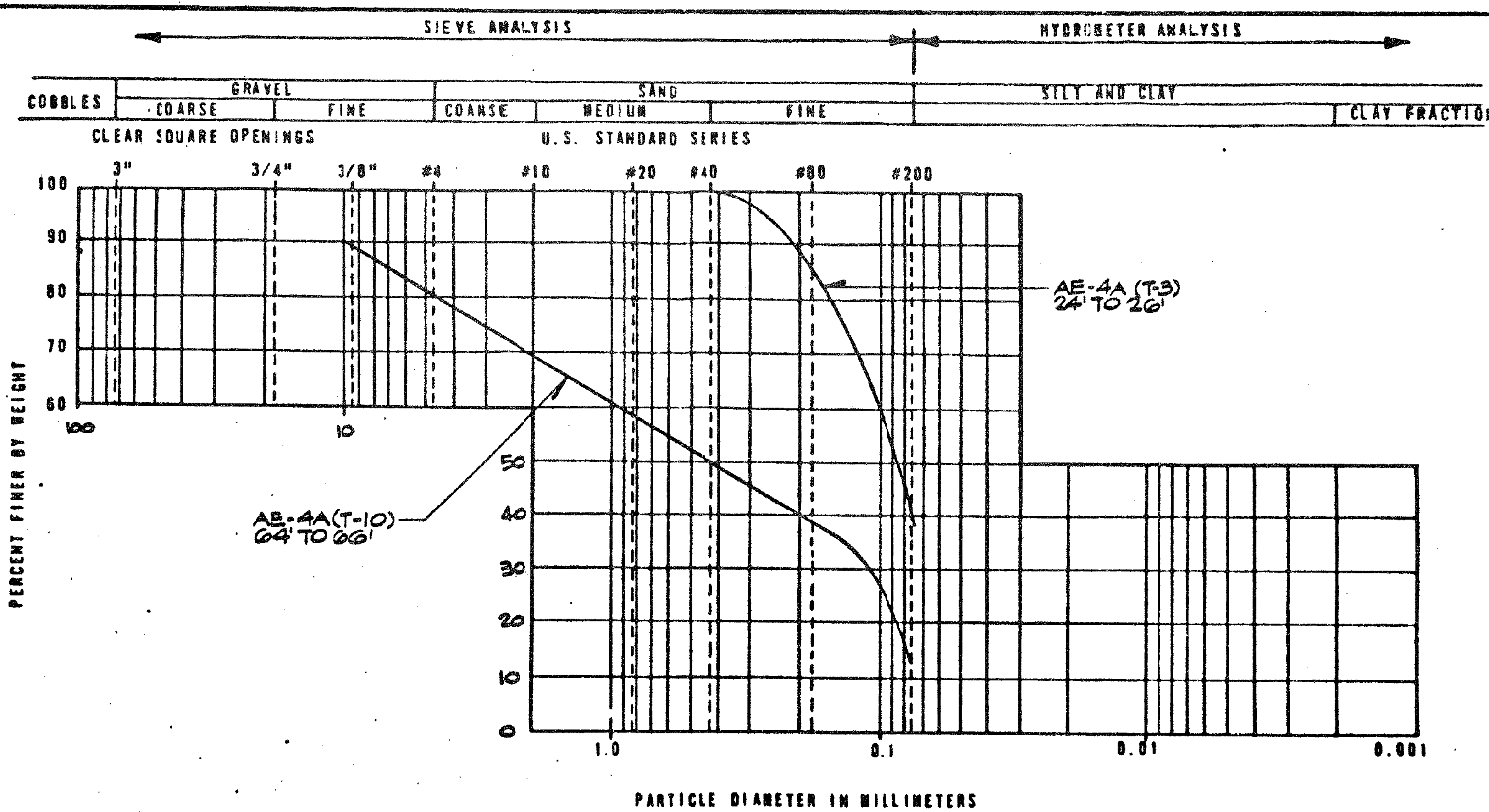
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-3
FIGURE 2G-C3



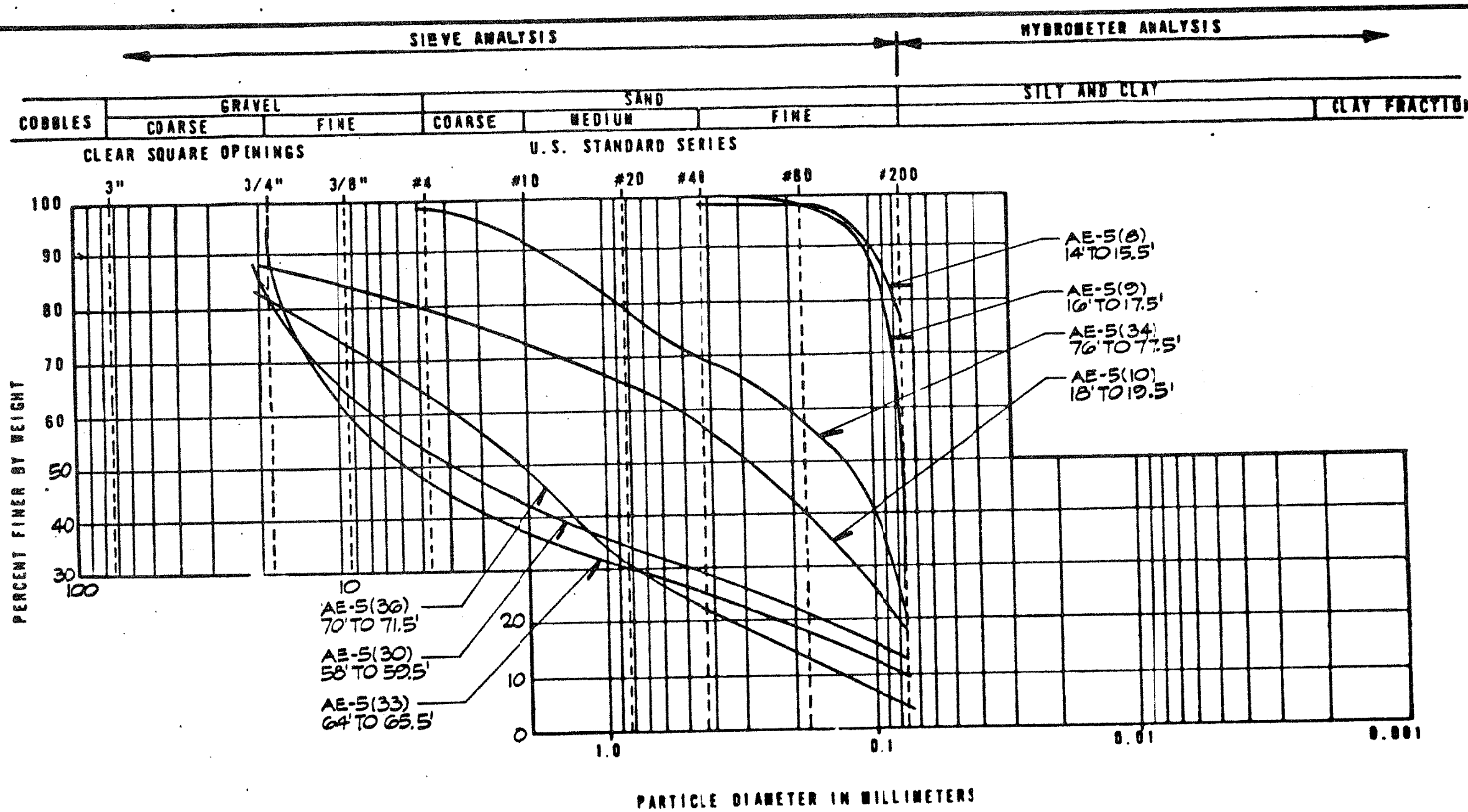
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-4
FIGURE 2G-C4



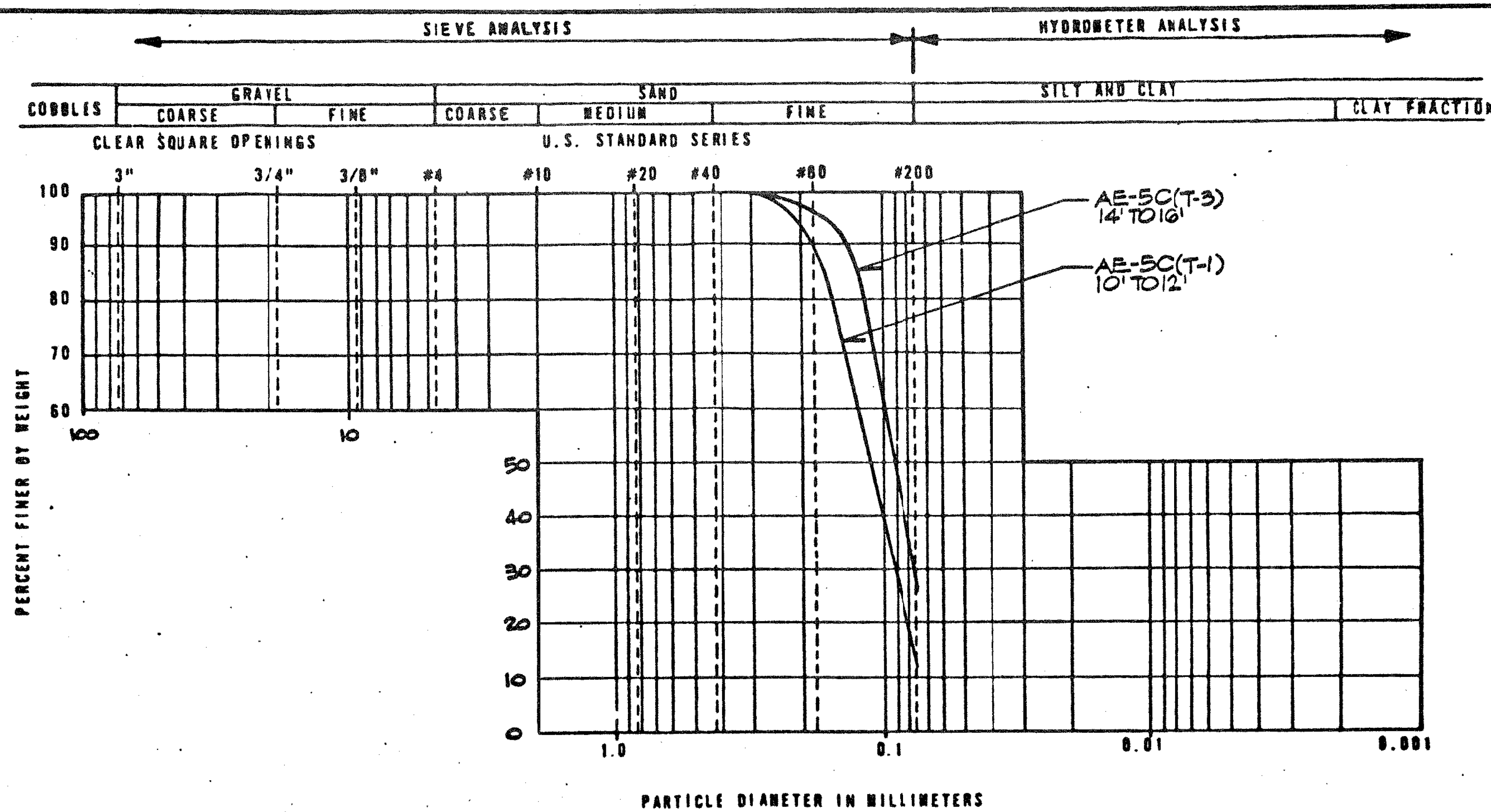
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-4A
FIGURE 2G-C5



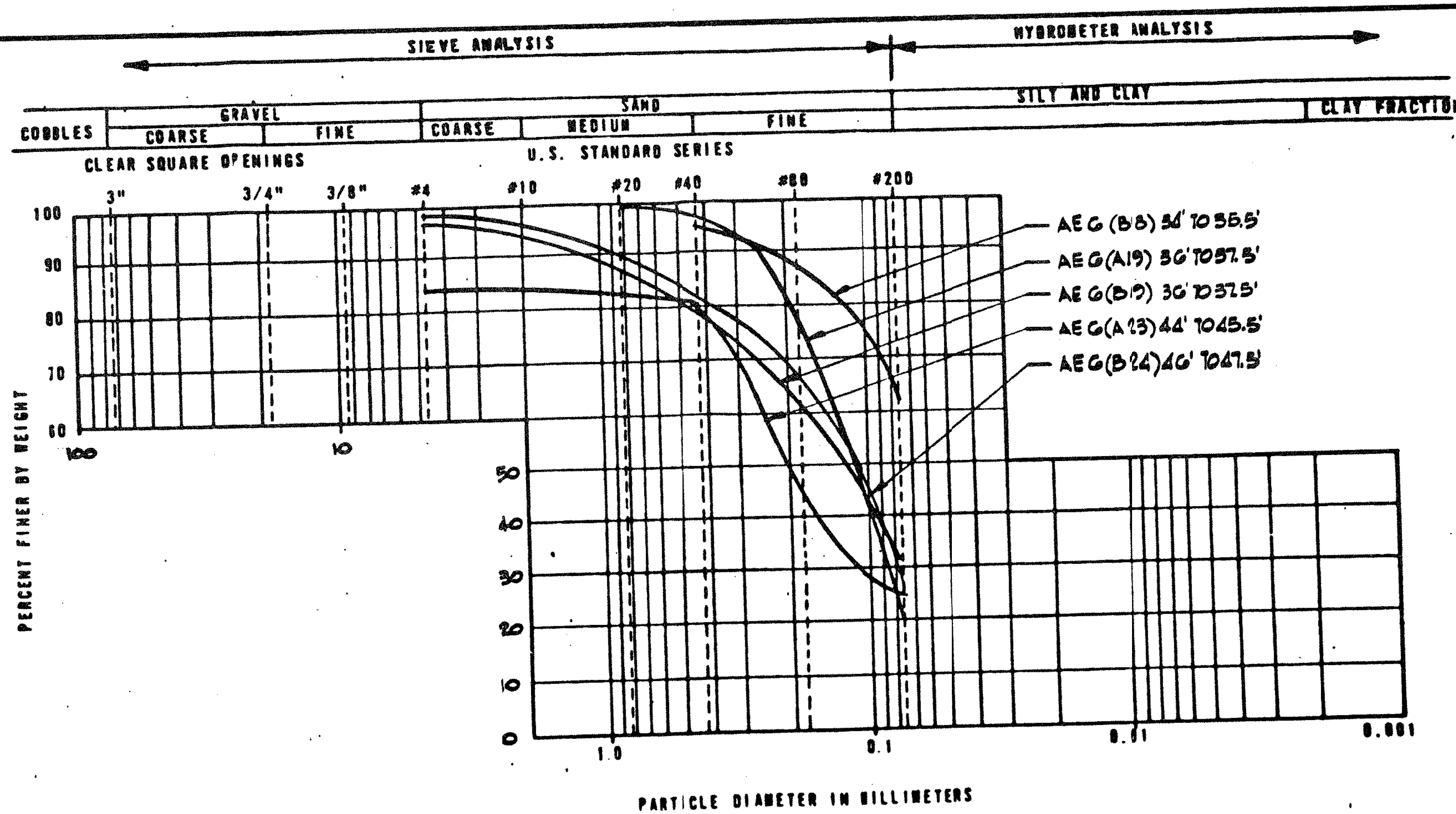
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-5
FIGURE 2G-C6



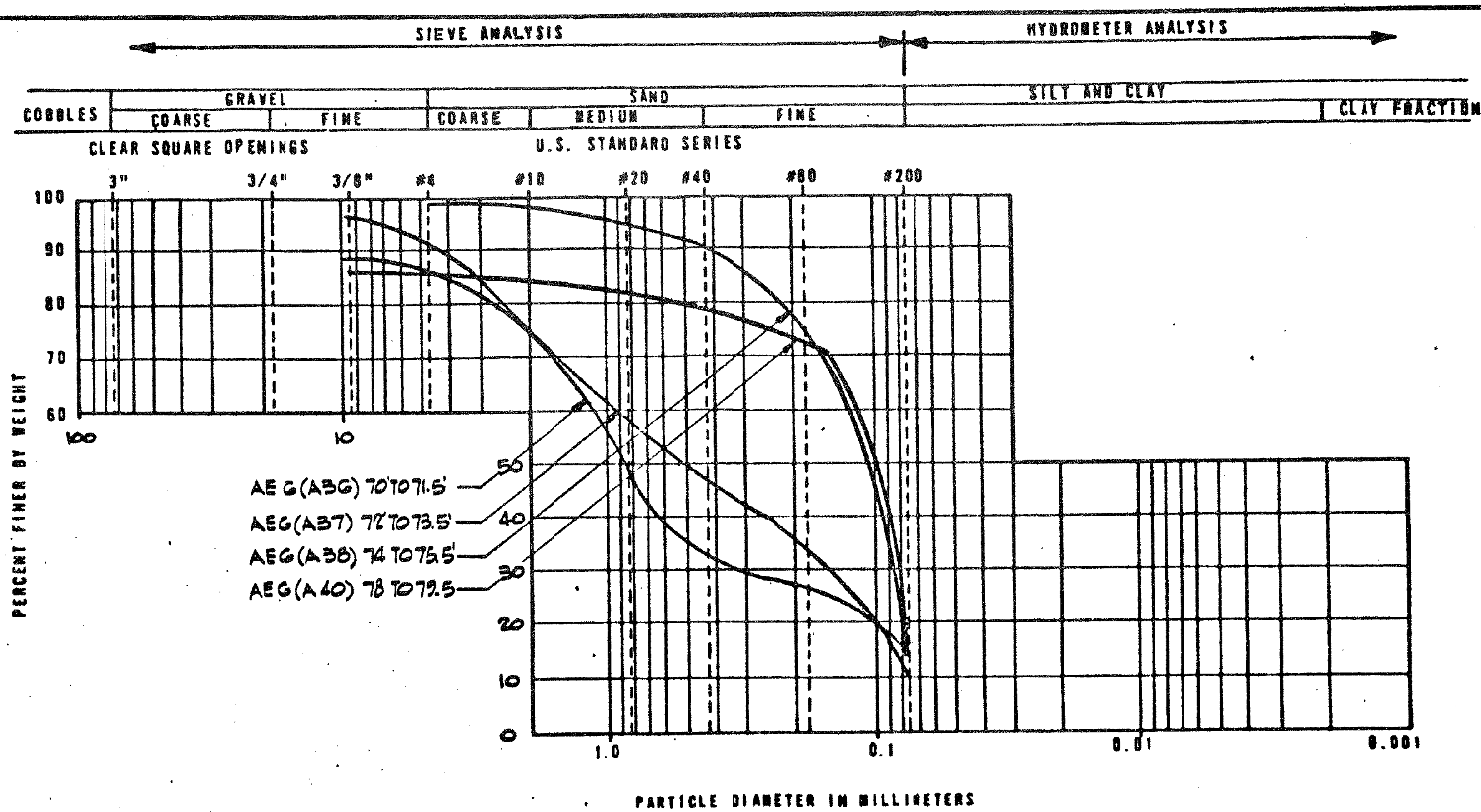
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-5C
FIGURE 2G-C7



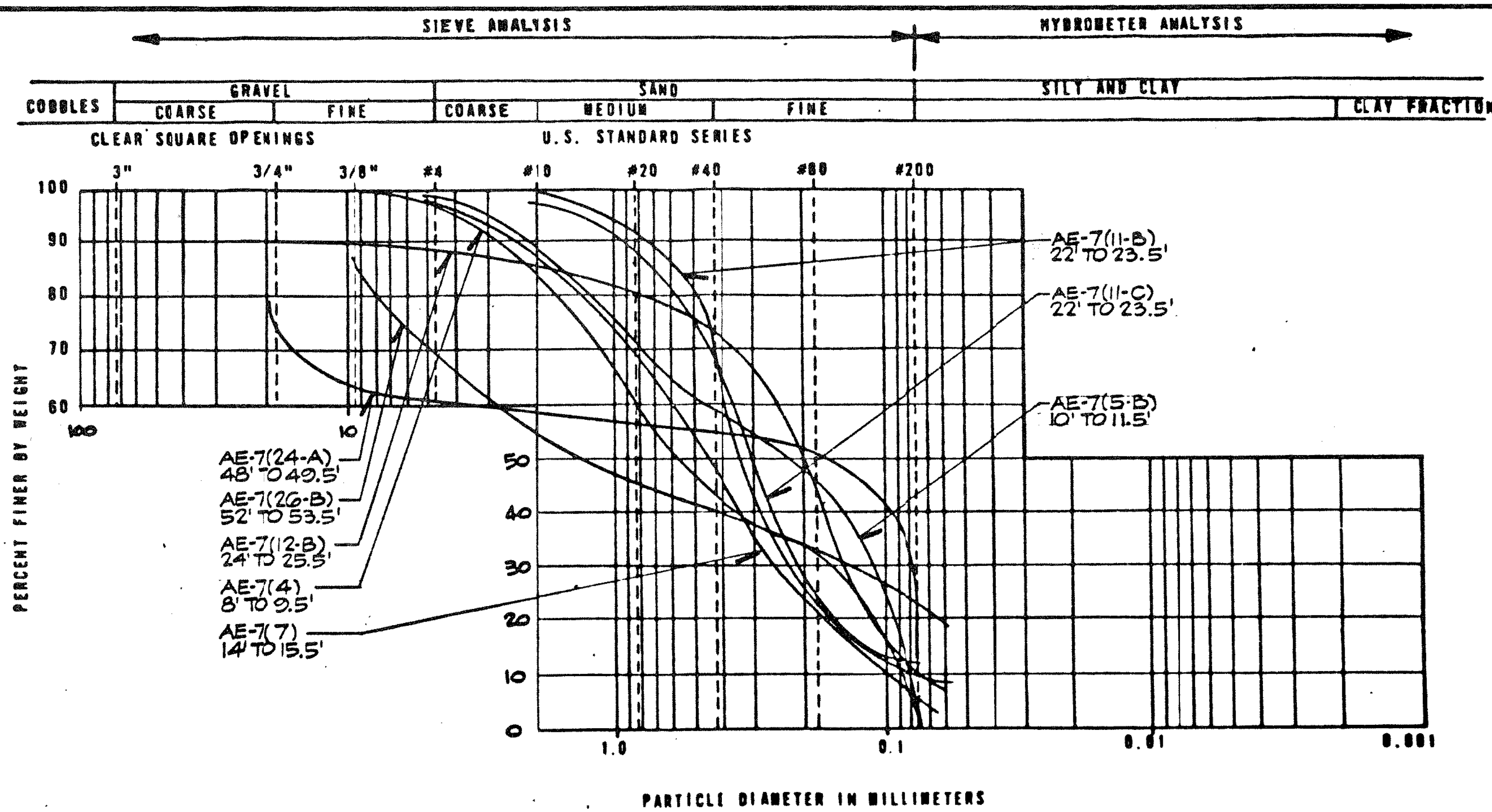
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-G
FIGURE 2G-C8



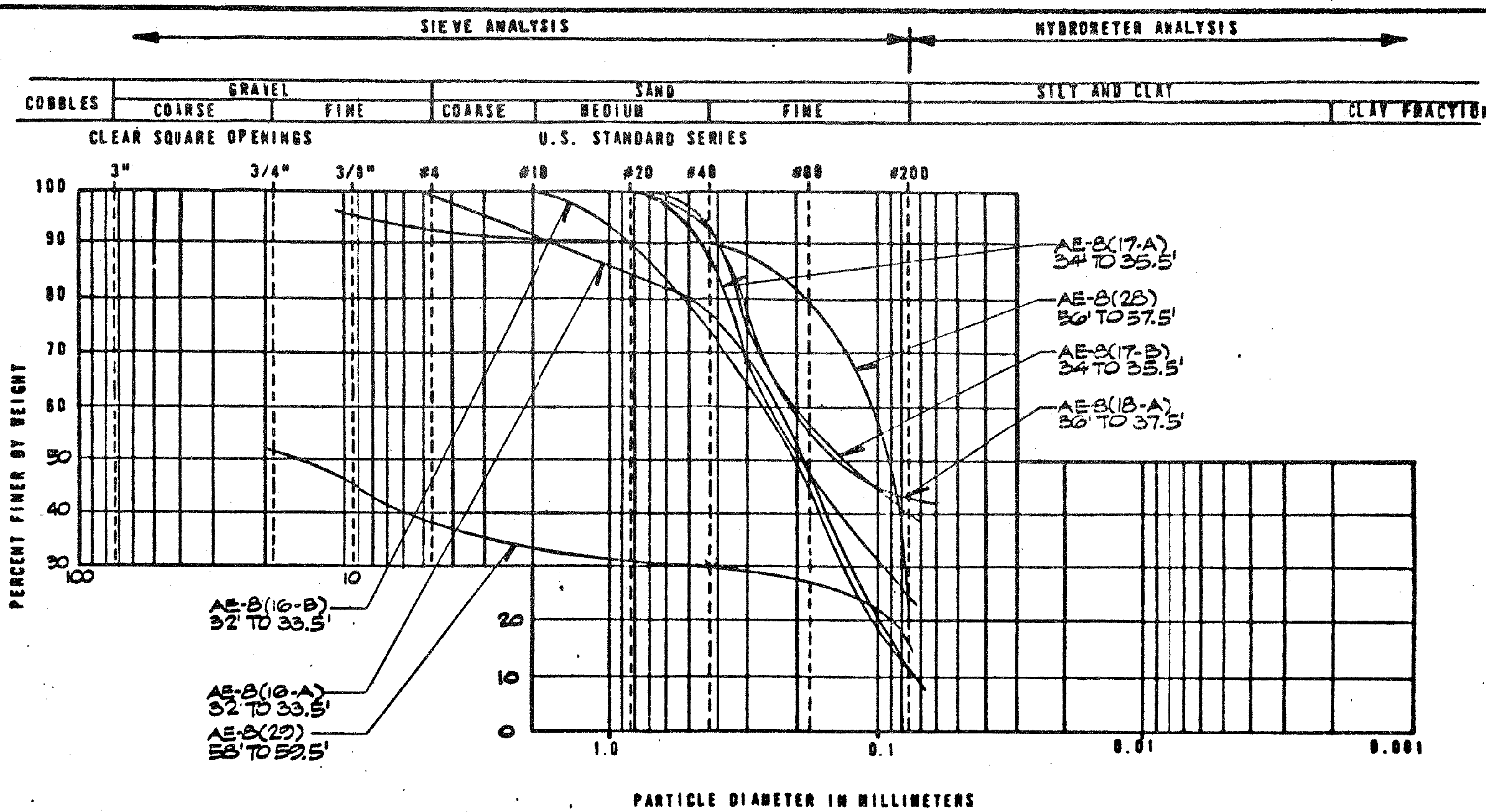
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-6
FIGURE 2G-C8 (CON'T)



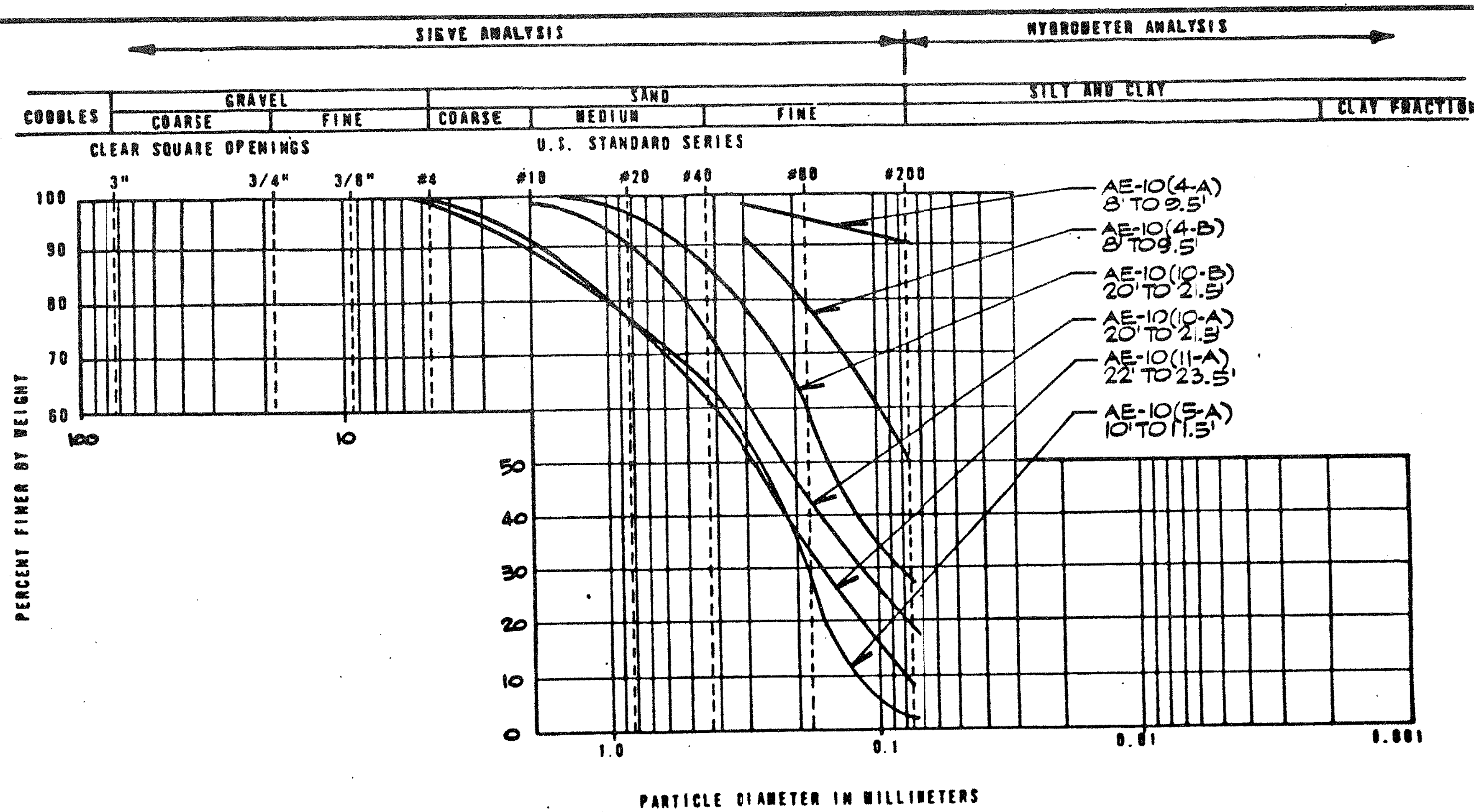
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-7
FIGURE 2G-C9



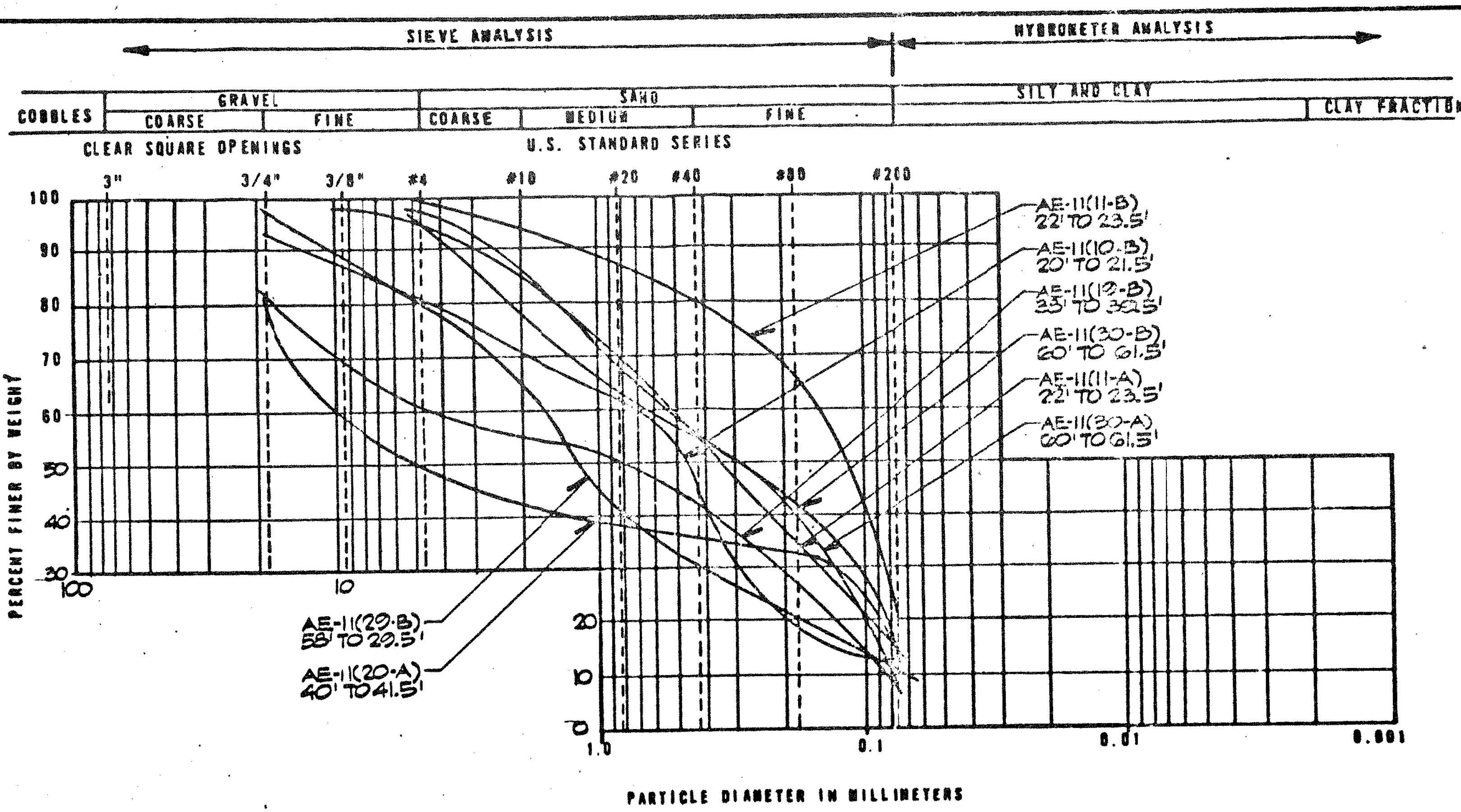
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-8
FIGURE 2G-C10



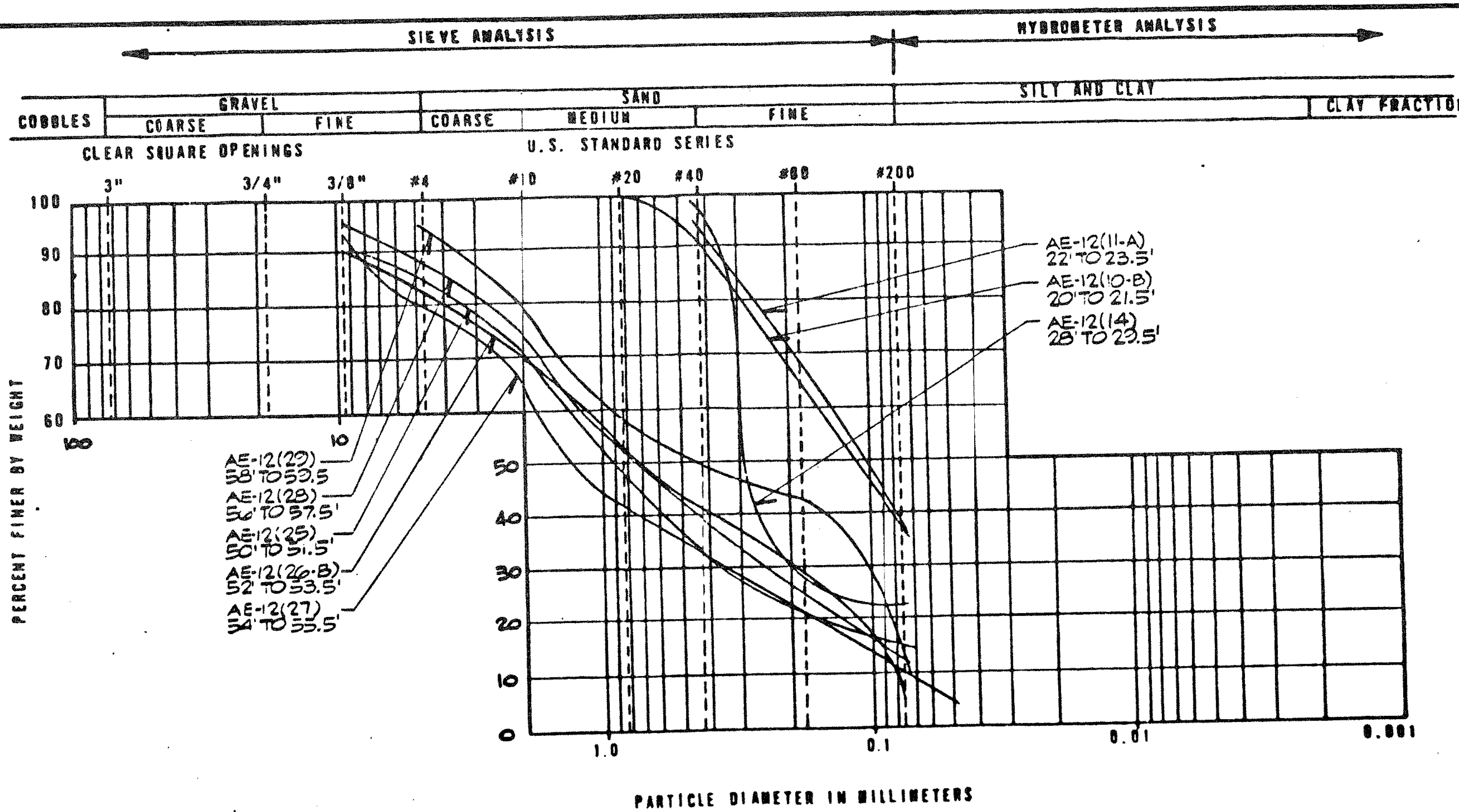
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-10
FIGURE 2G-C11



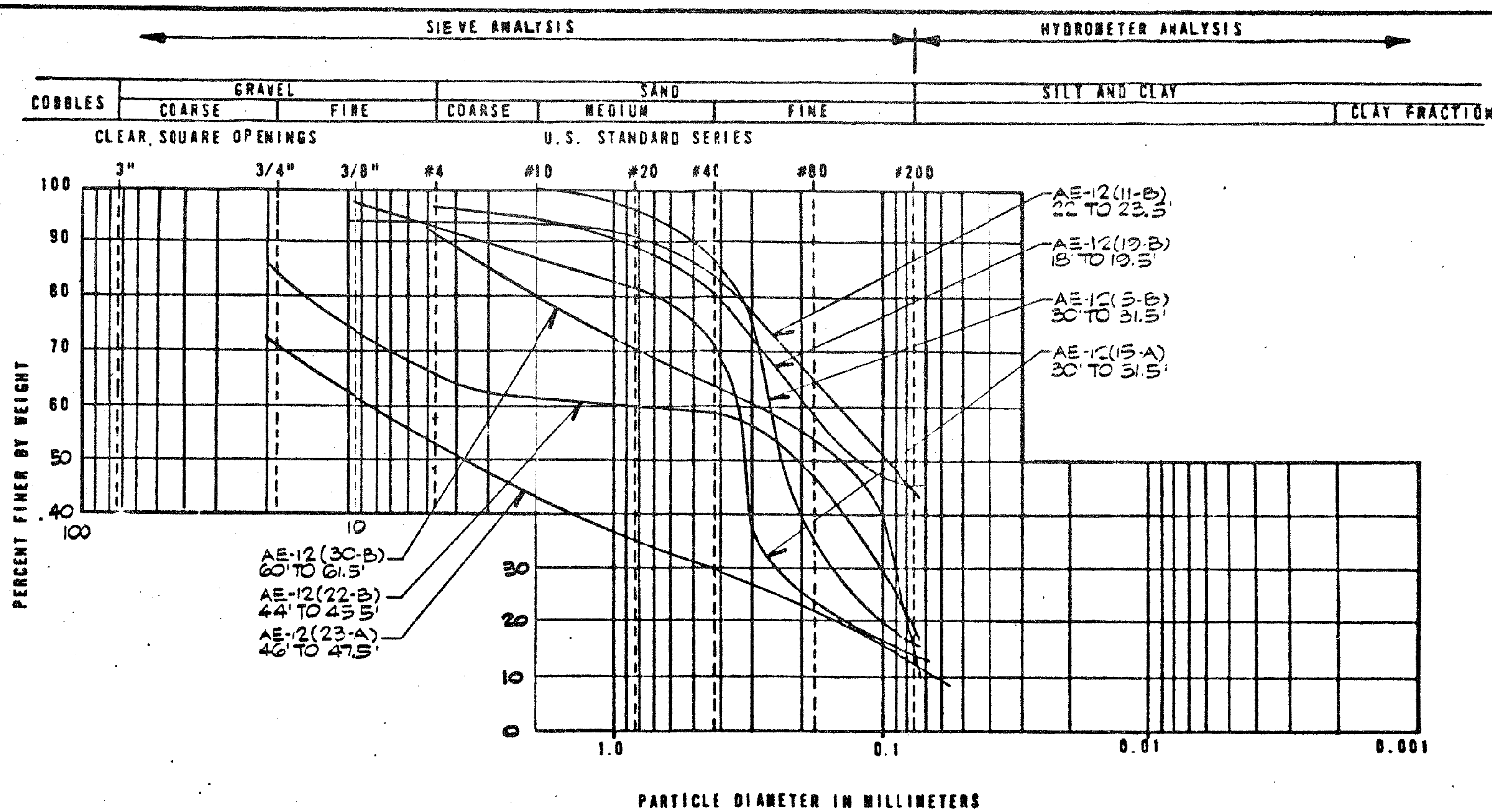
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-11
FIGURE 2G-C12



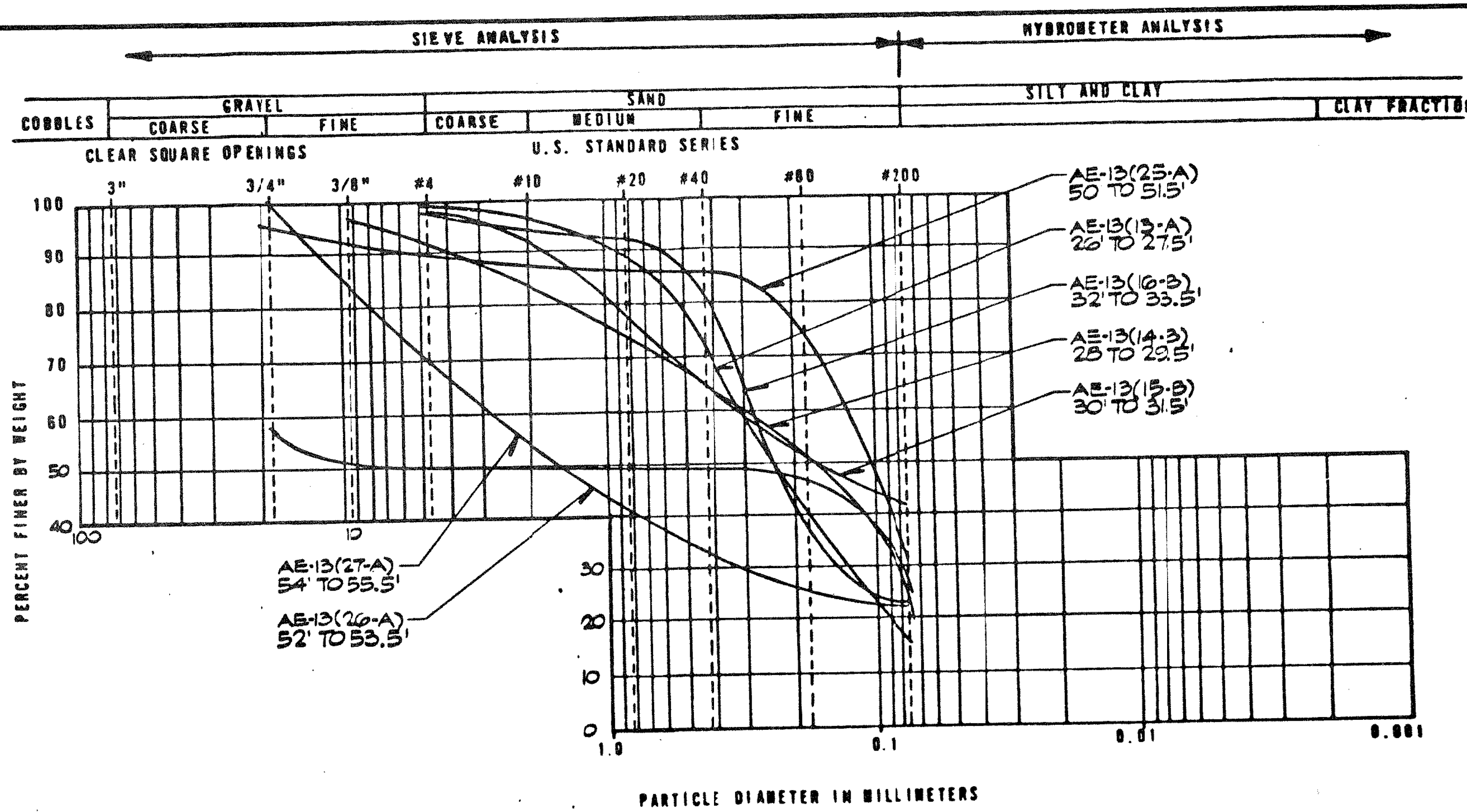
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-12
FIGURE 2G-C13



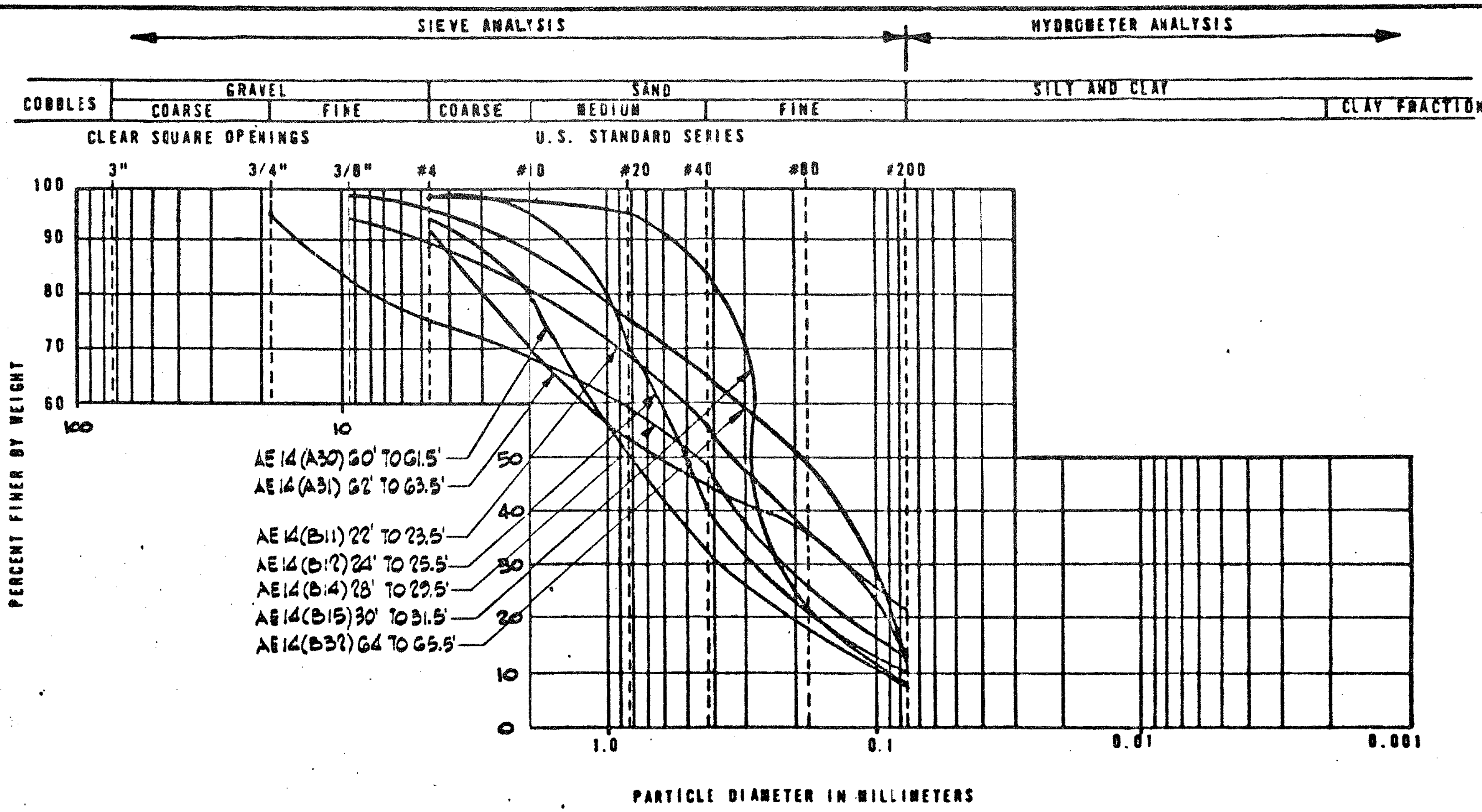
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-12
FIGURE 2G-C13 (CON'T)



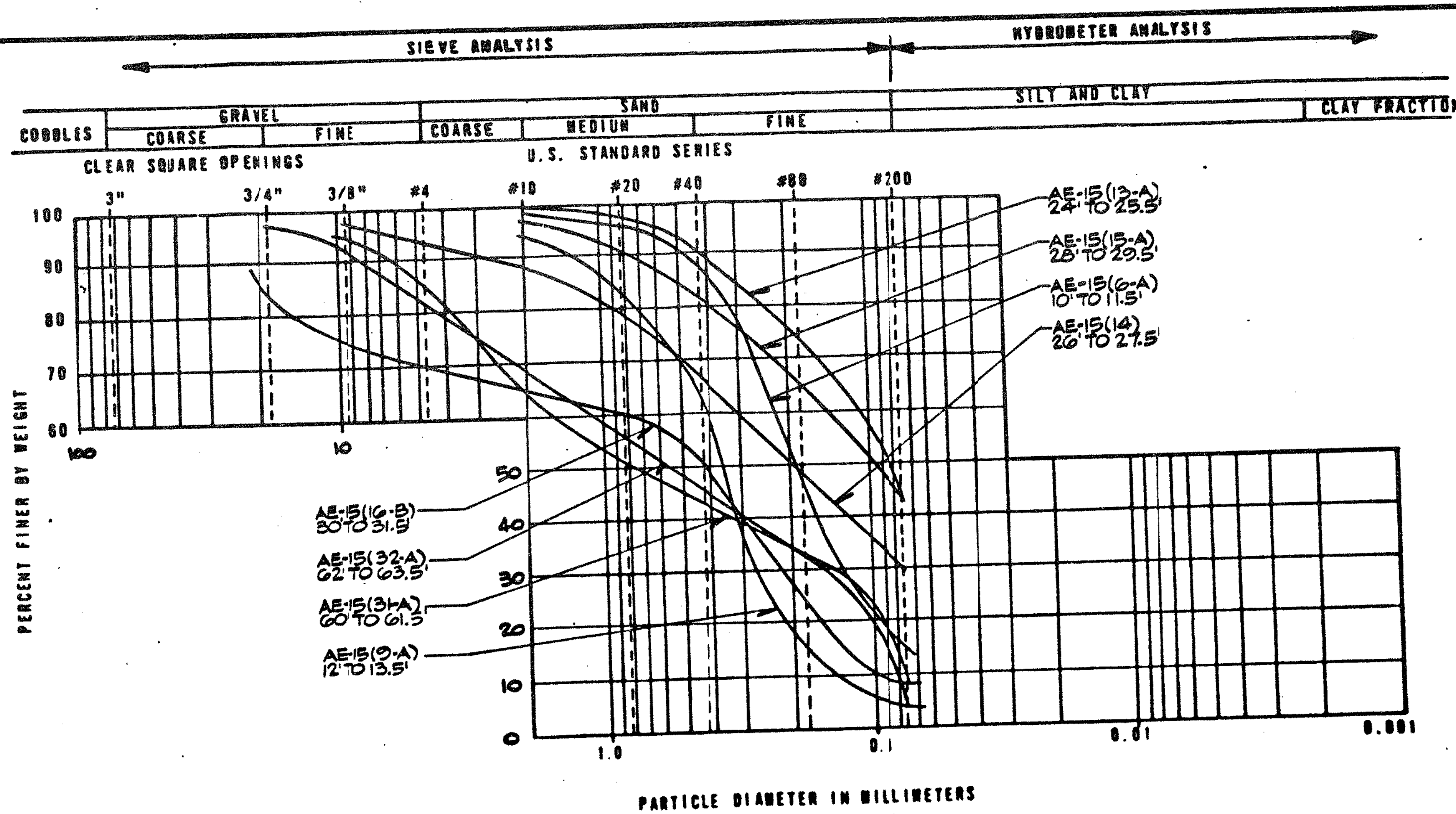
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-13
FIGURE 2G.C14



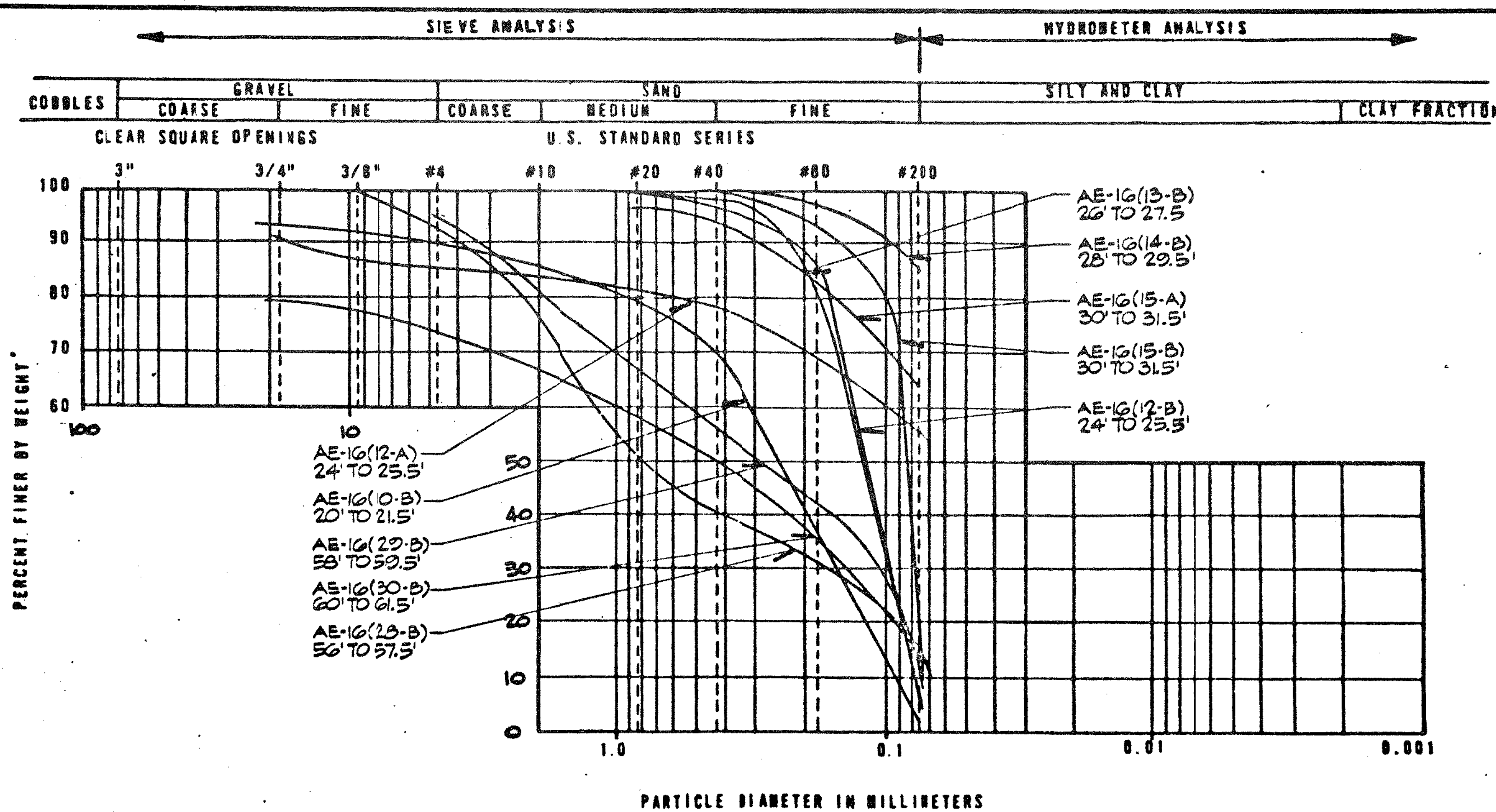
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-14
FIGURE 2G-C15



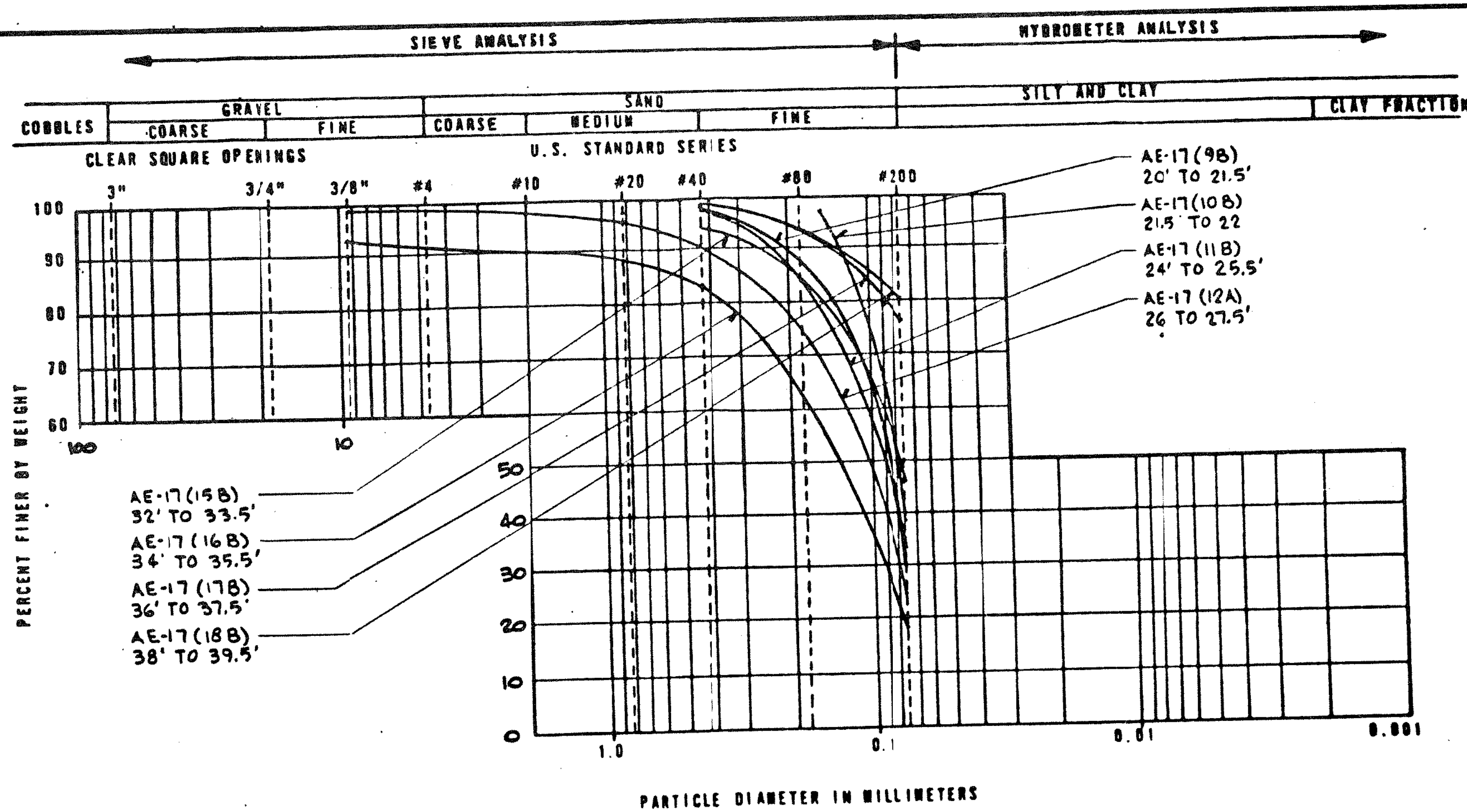
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-15
FIGURE 2G-C16



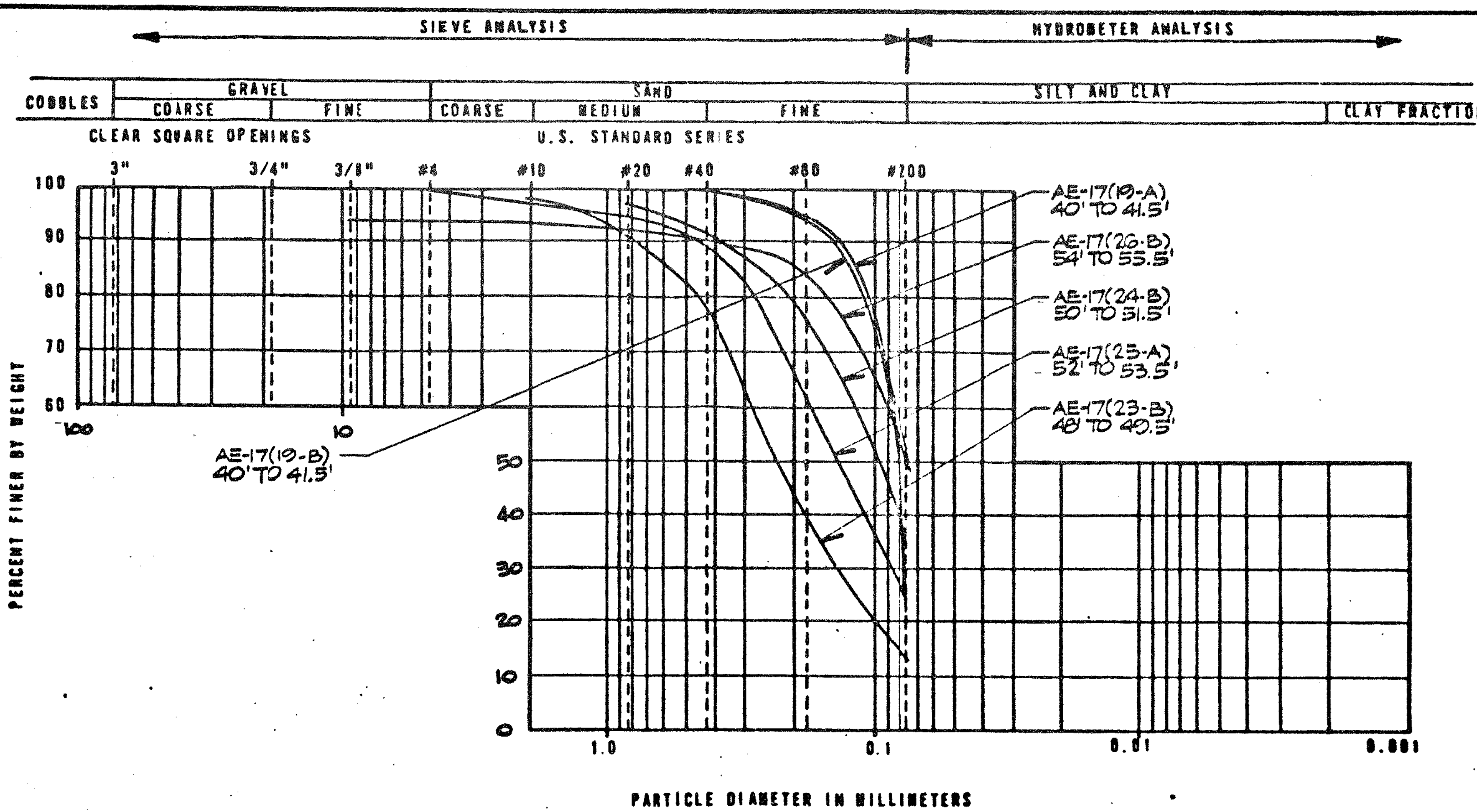
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-16
FIGURE 2G-C17



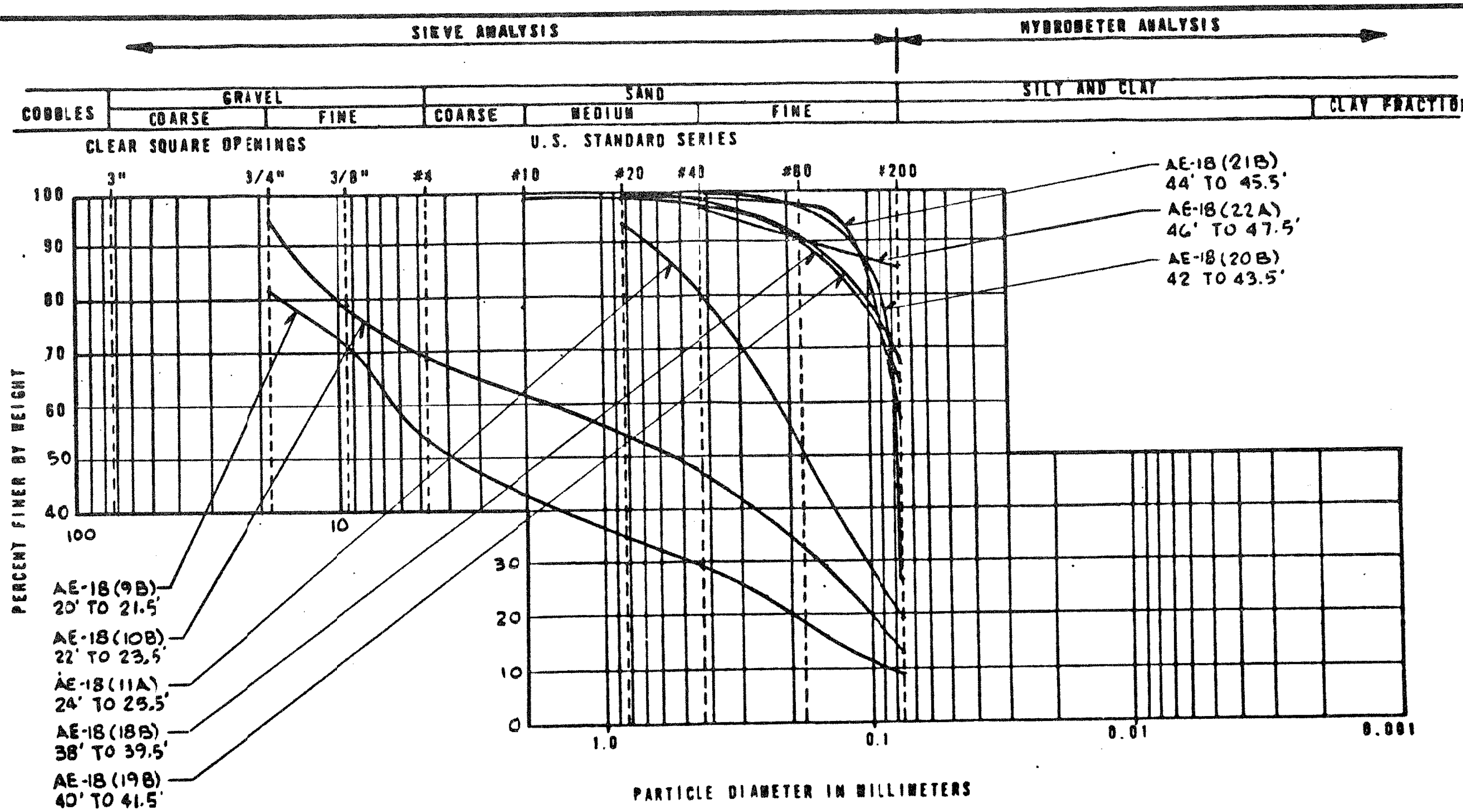
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-17
FIGURE 2G-C18



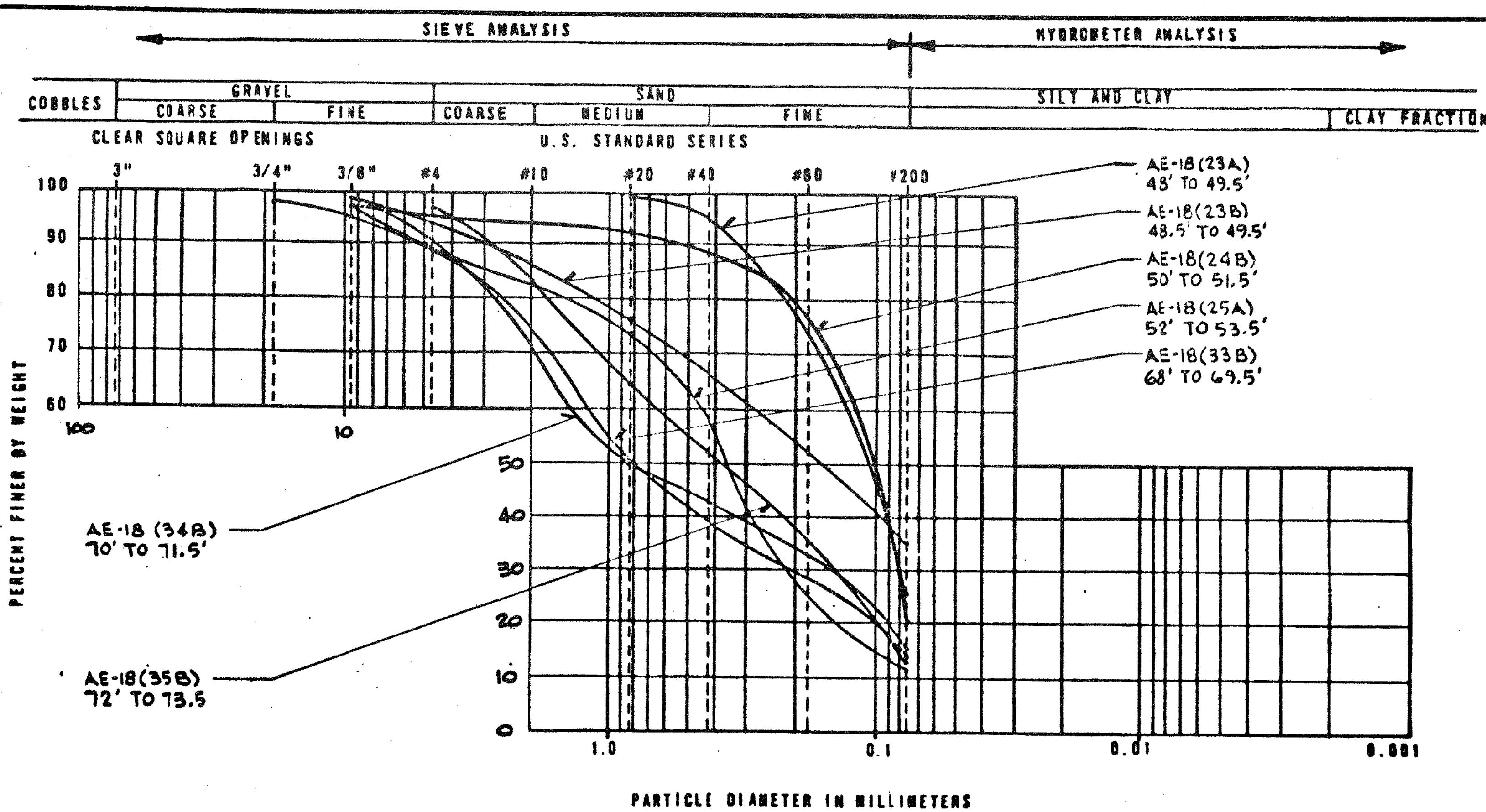
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-17
FIGURE 2G-C18 (CON'T)



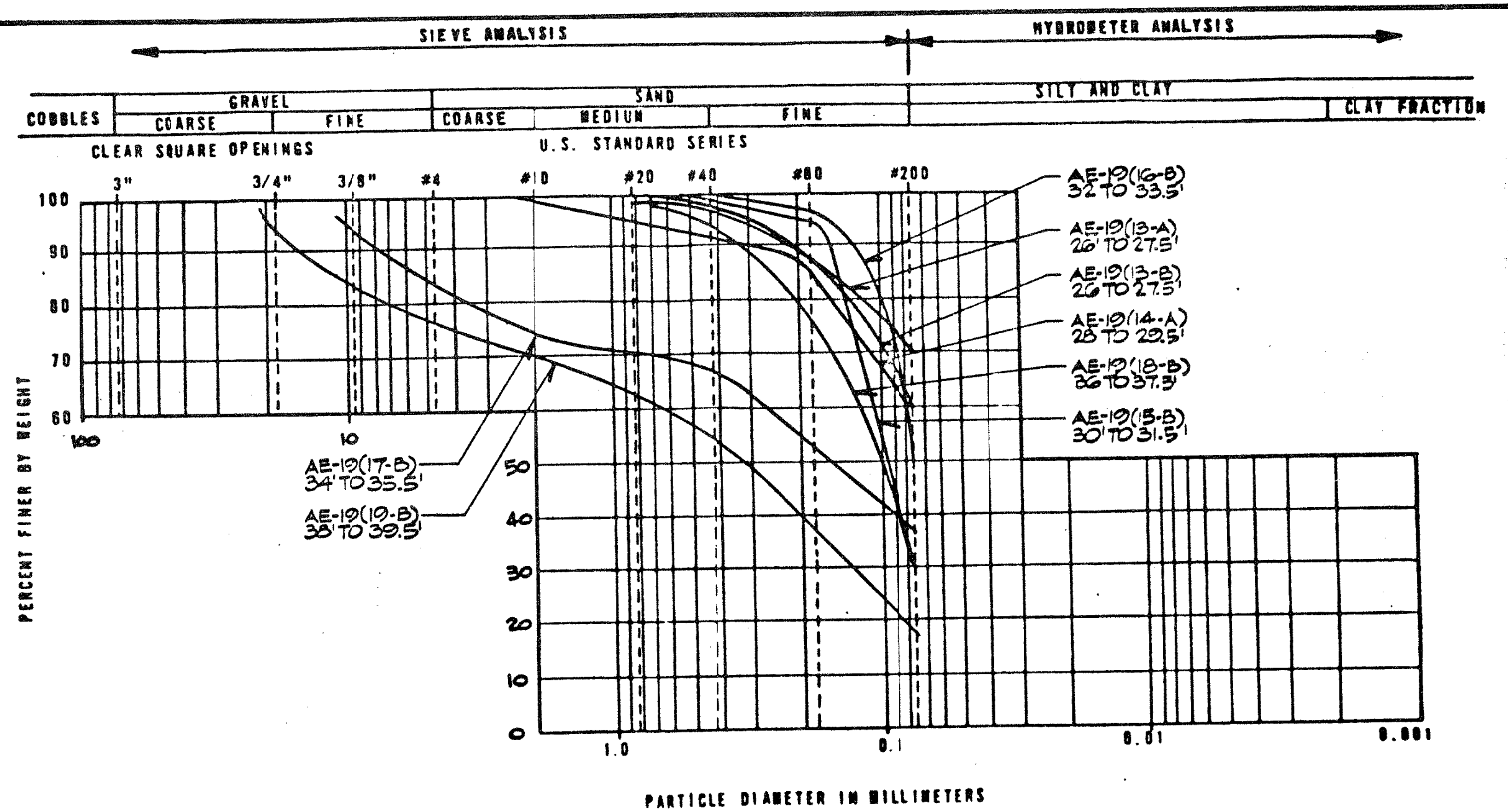
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-18
FIGURE 2GC19



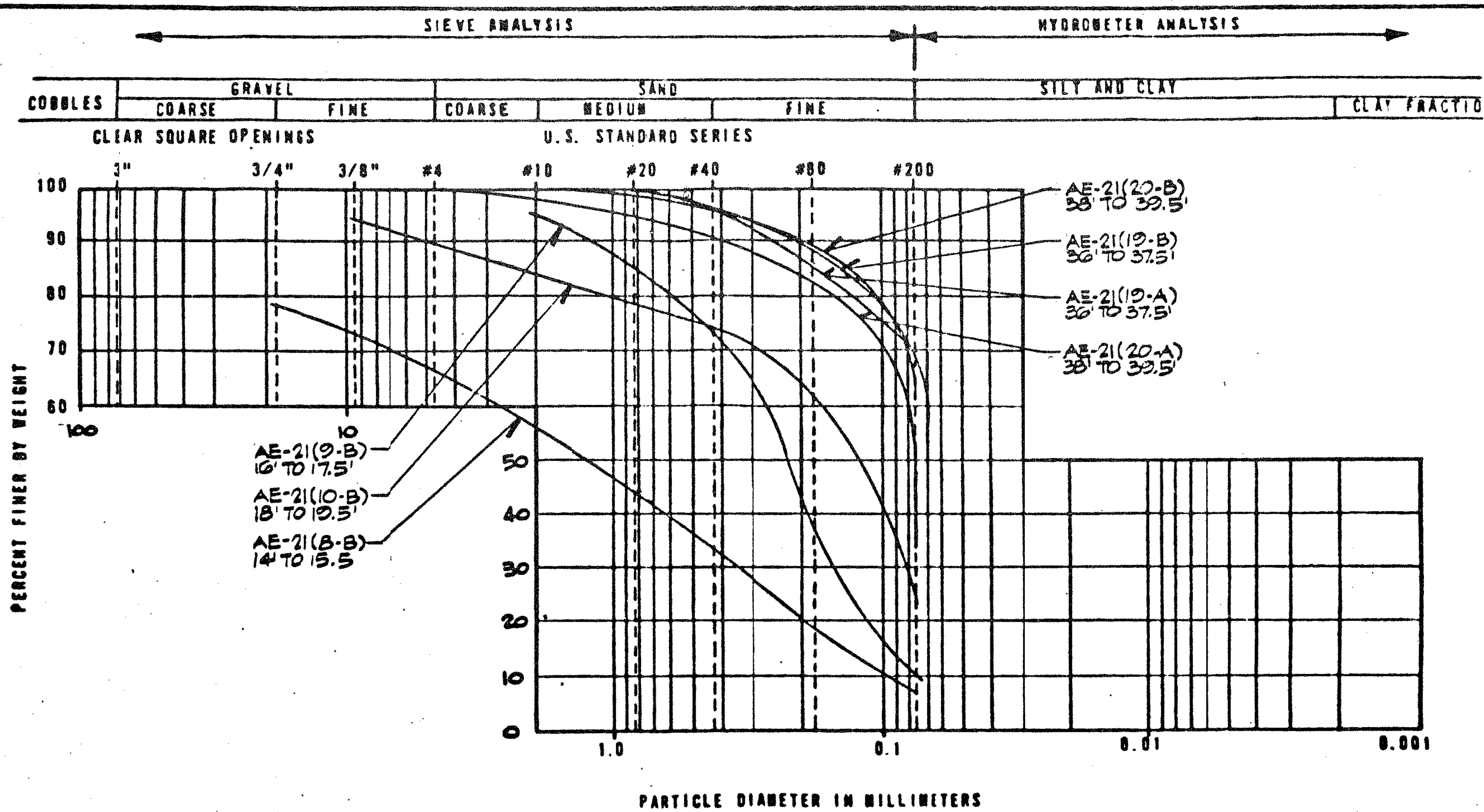
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-18
FIGURE 2G-C19 (CON'T)



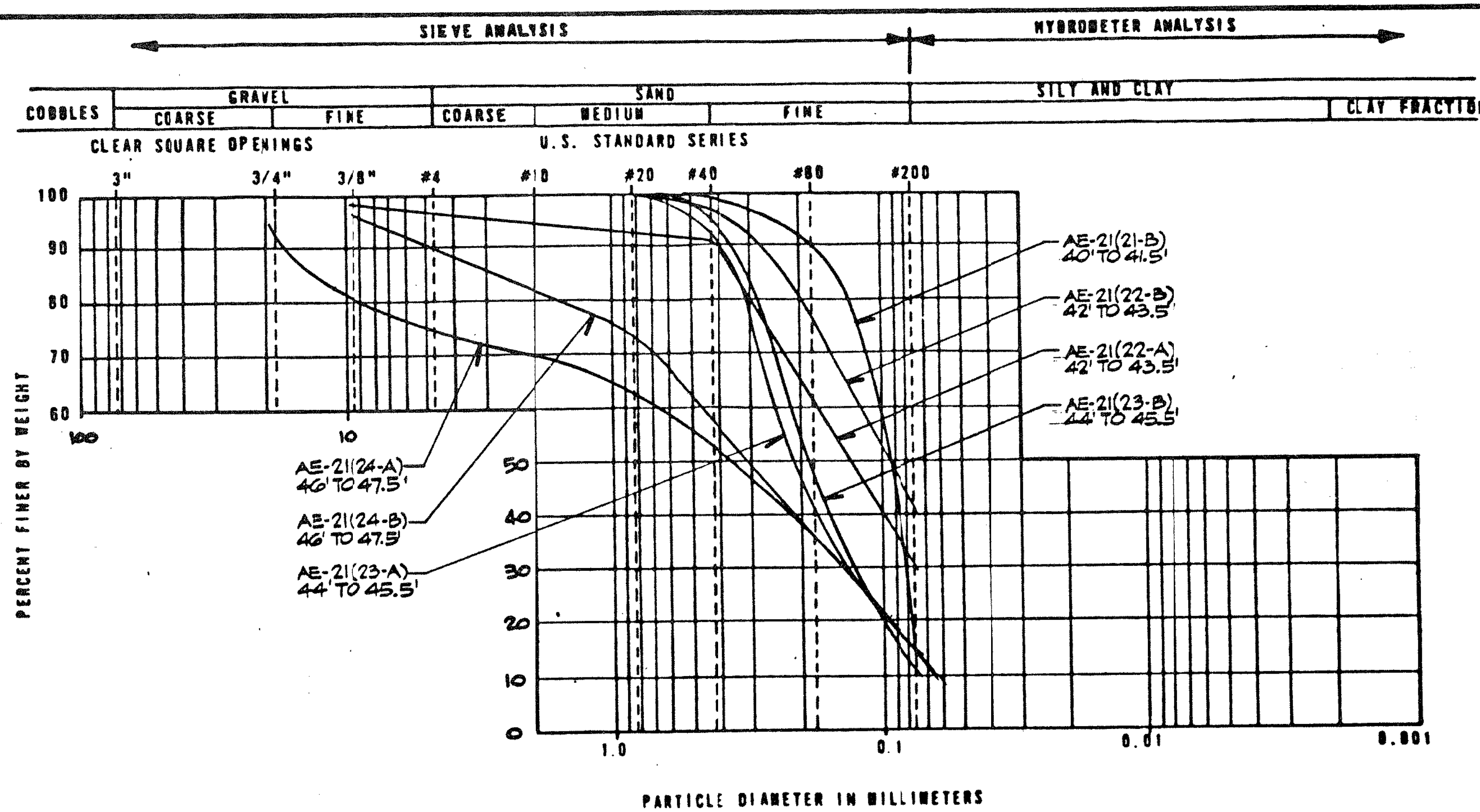
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-19
FIGURE 2G-C20



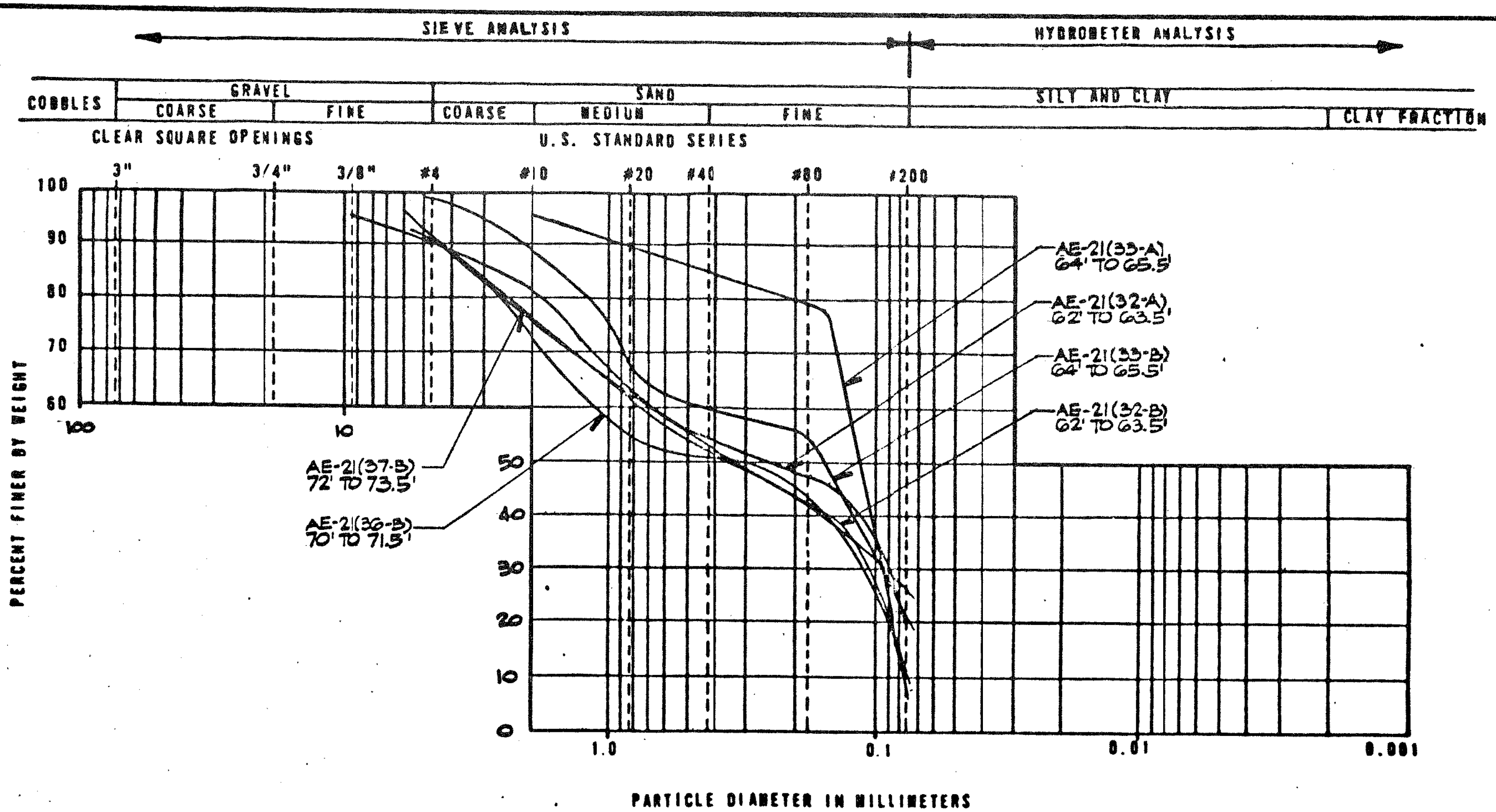
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-21
FIGURE 2G-C21



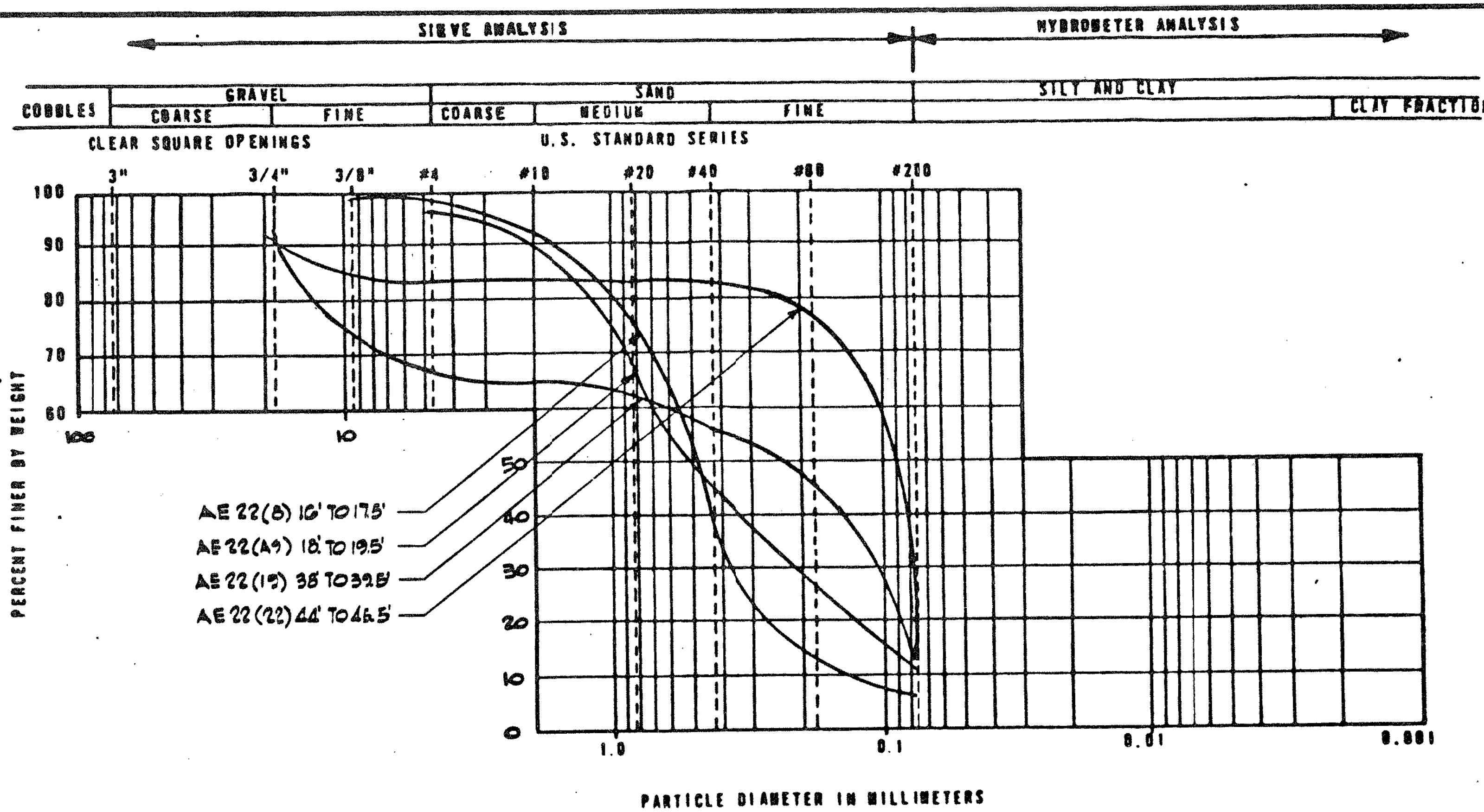
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-21
FIGURE 2G-C21 (CON'T)-1



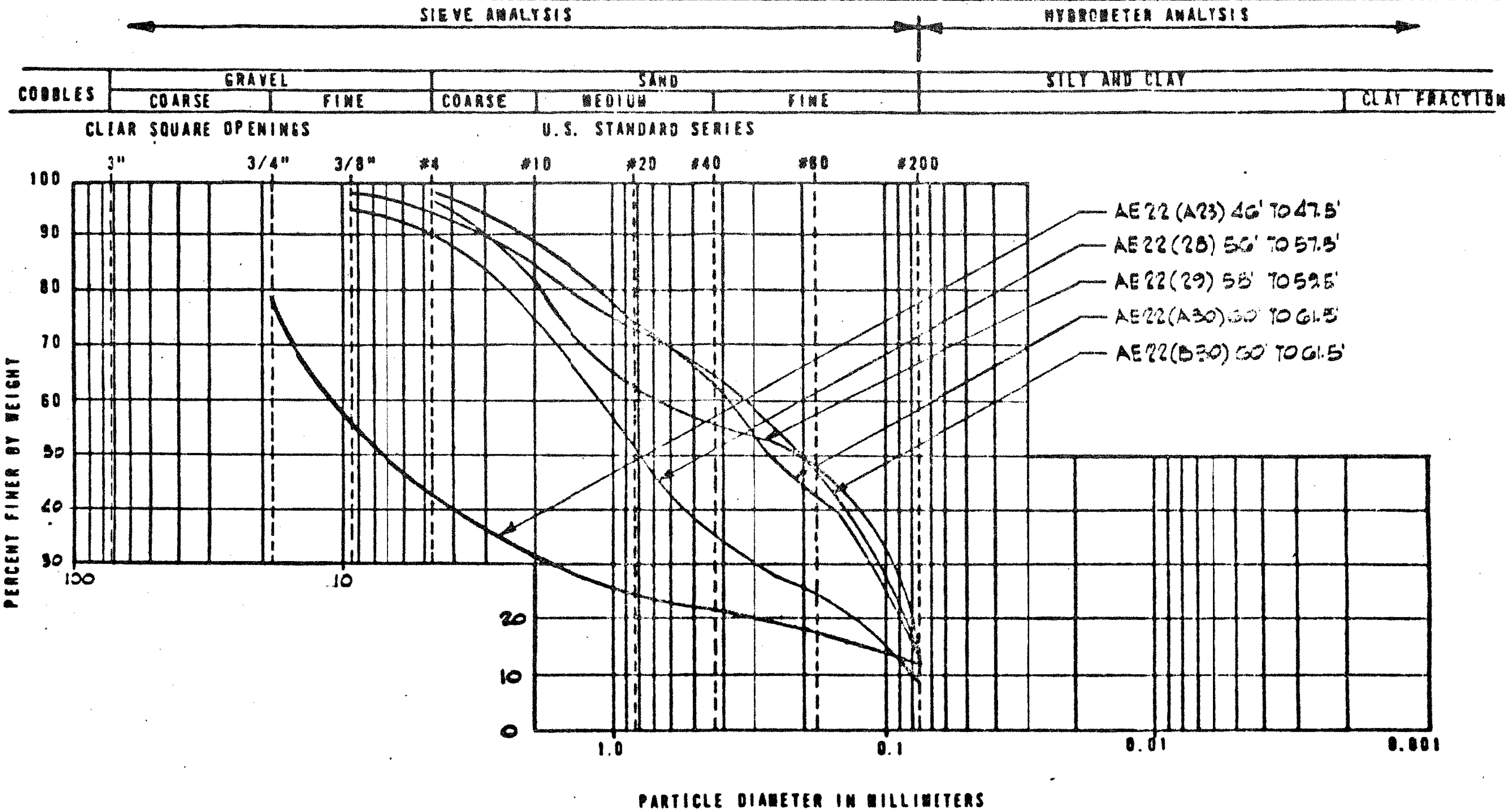
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-21
FIGURE 2G-C21 (CON'T)-2



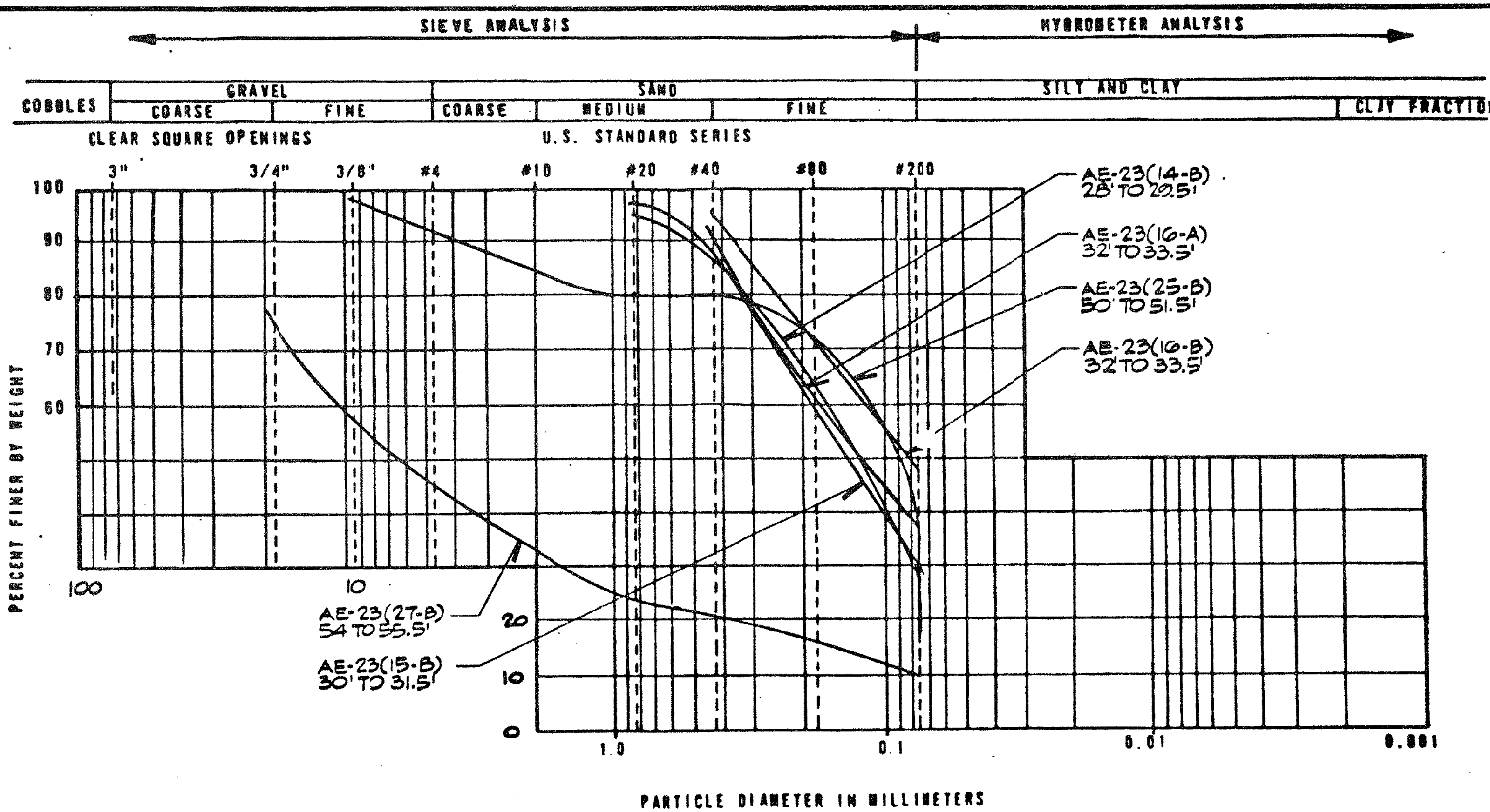
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-22
FIGURE 2G-C22



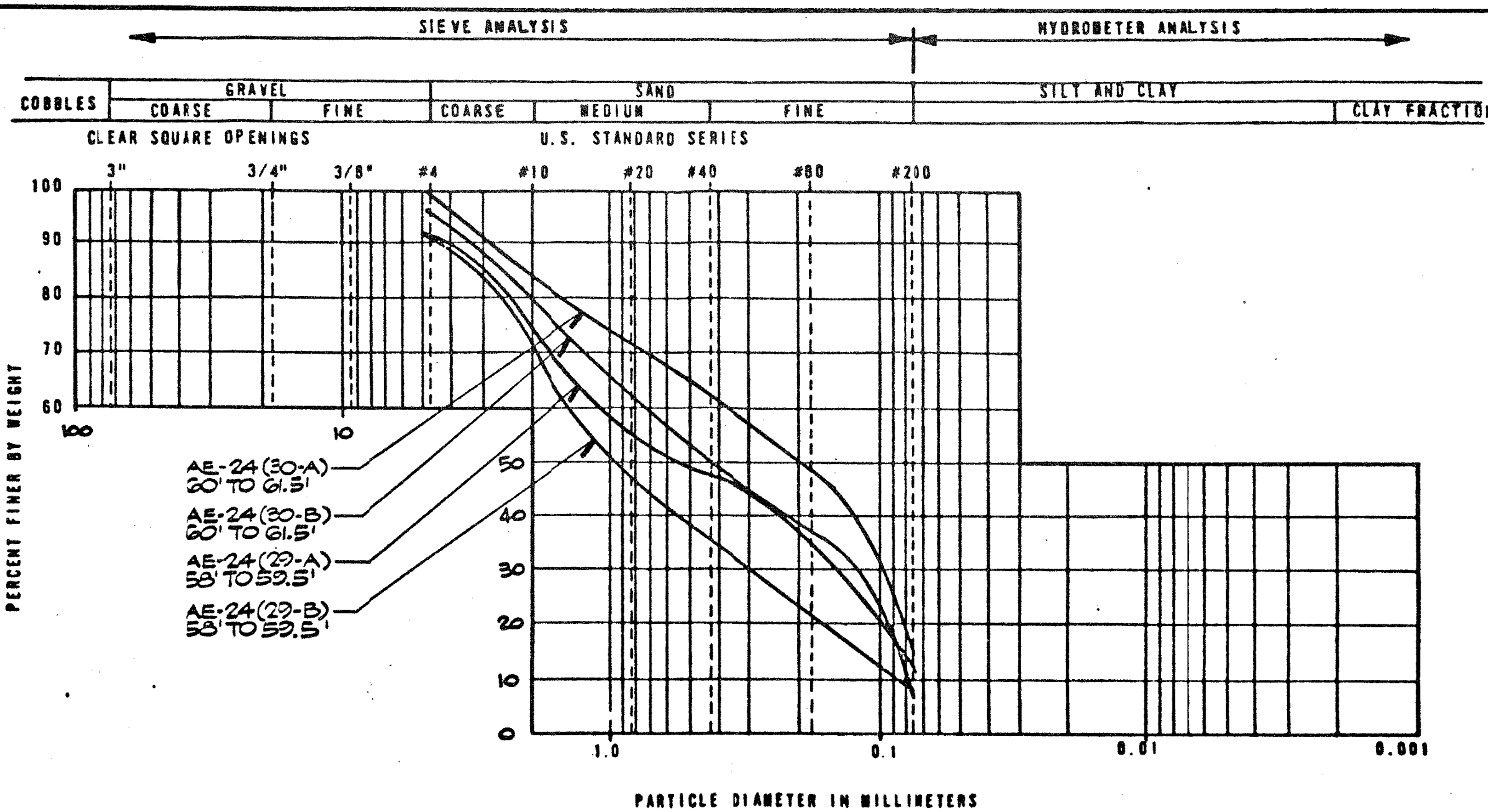
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-22
FIGURE 2G.C22 (CON'T)



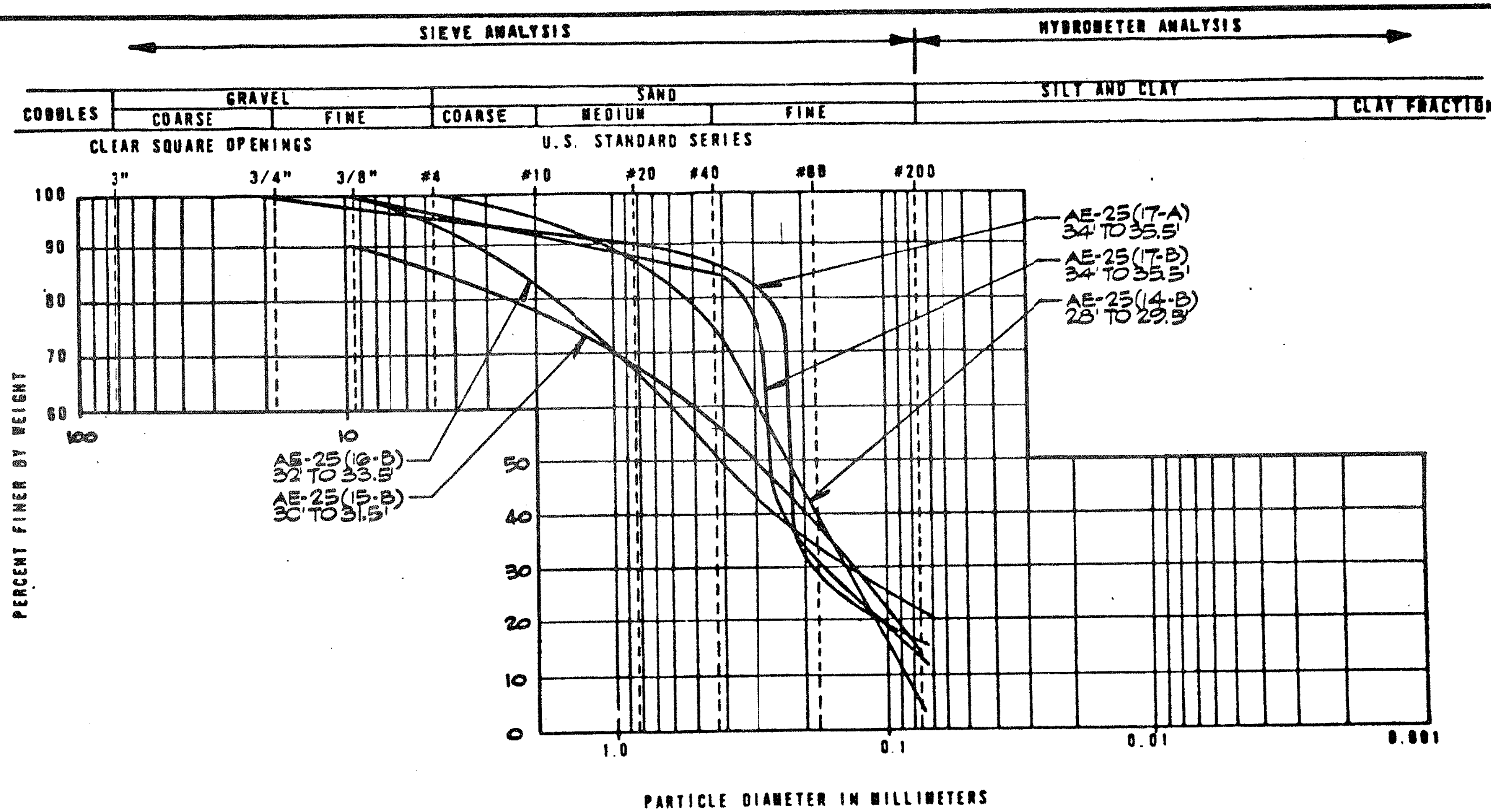
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 2

GRAIN SIZE DISTRIBUTIONS
BORING AE-23
FIGURE 2G-C23



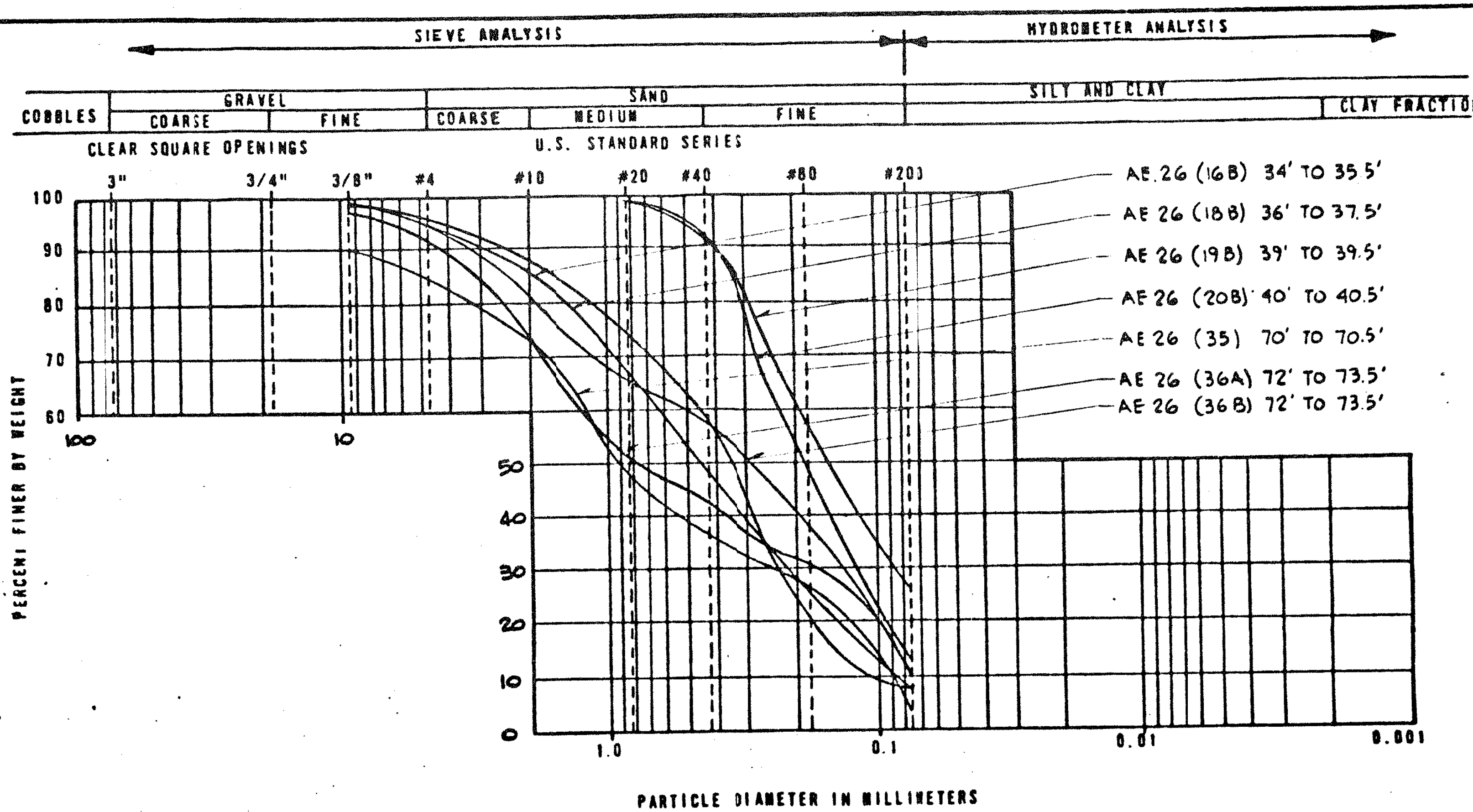
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-24
FIGURE 2G-C24



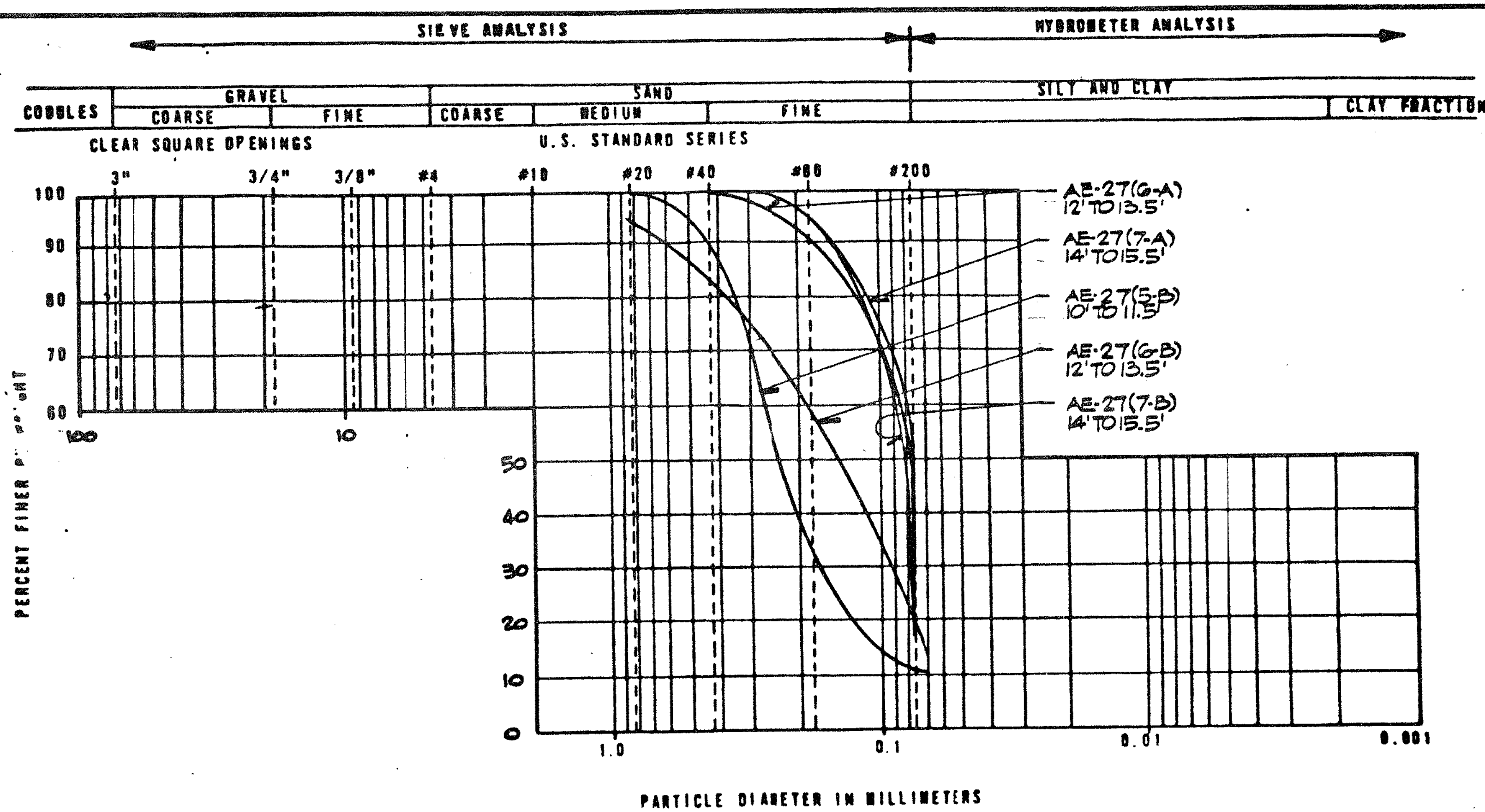
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-25
FIGURE 2G-C25



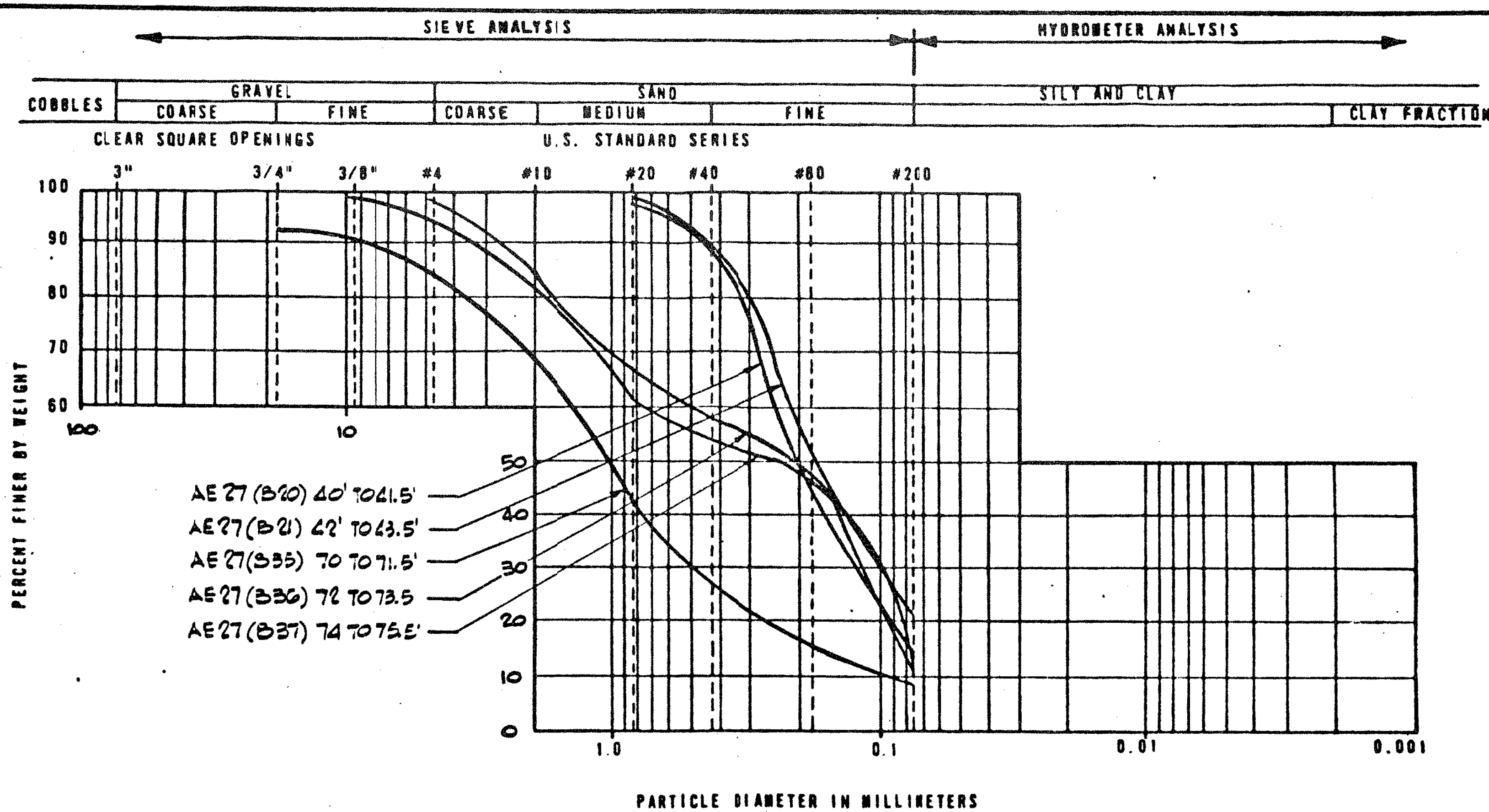
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-26
FIGURE 2G-C26



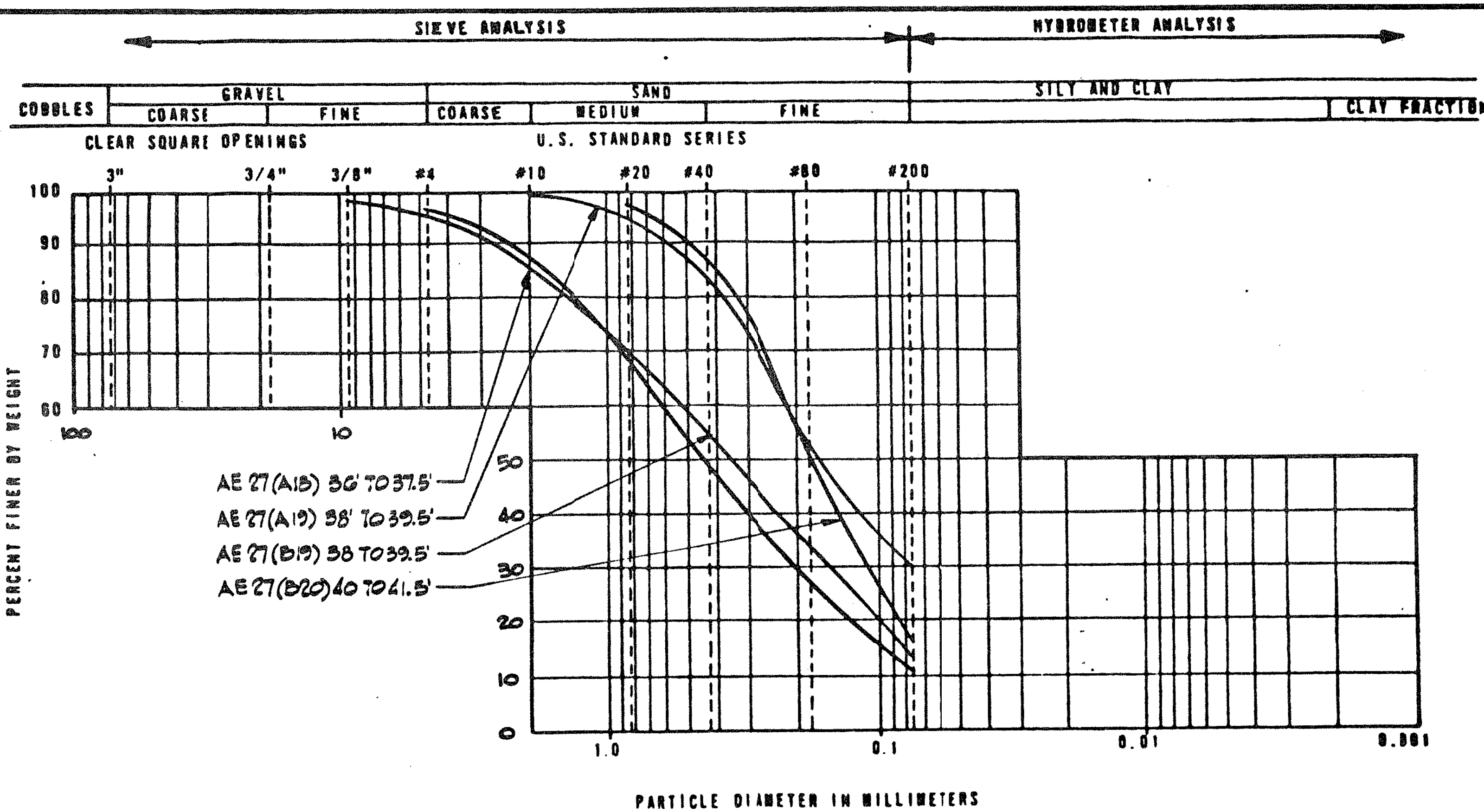
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-27
FIGURE 2G-C27



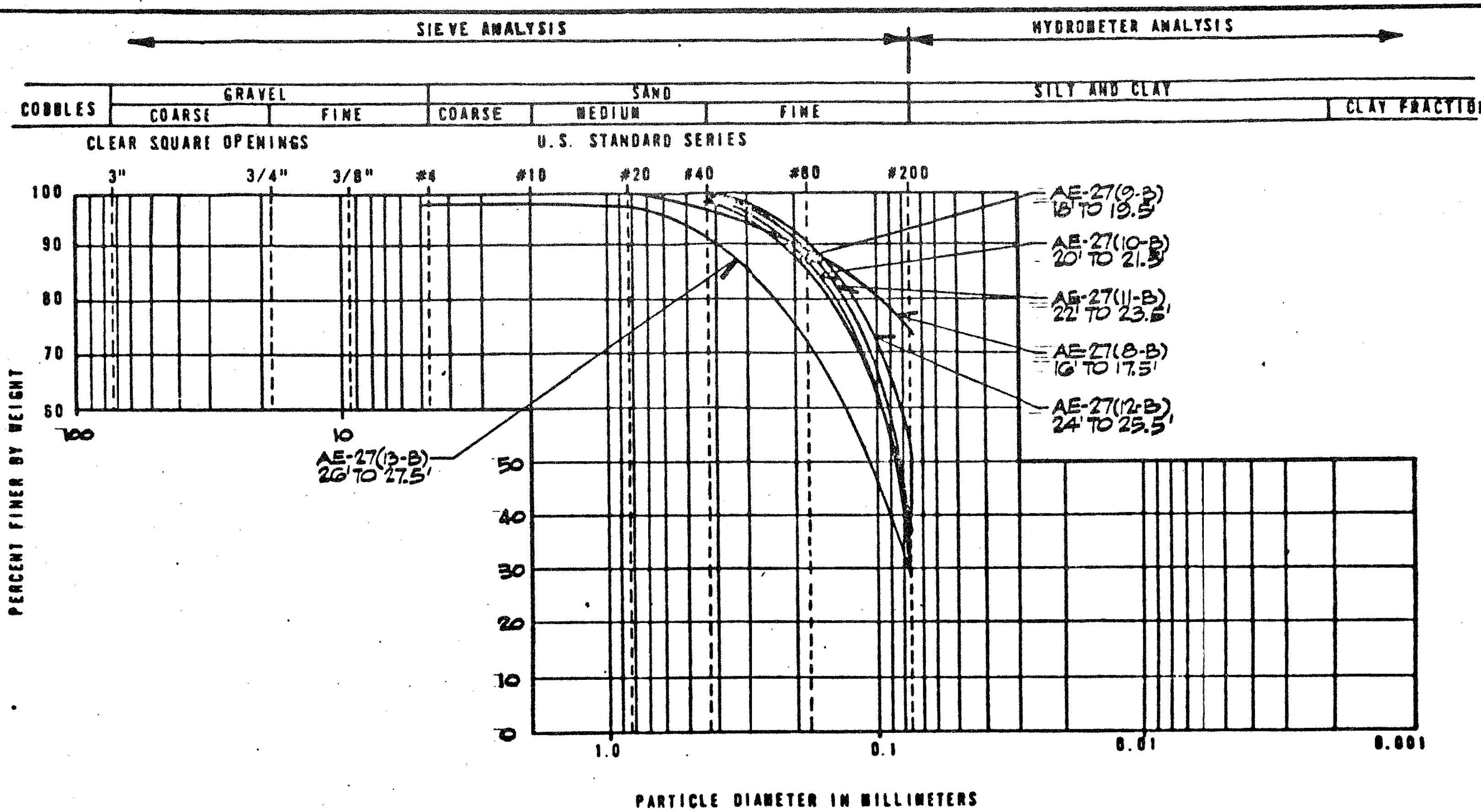
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-27
FIGURE 2G·C27 (CON'T) - 1



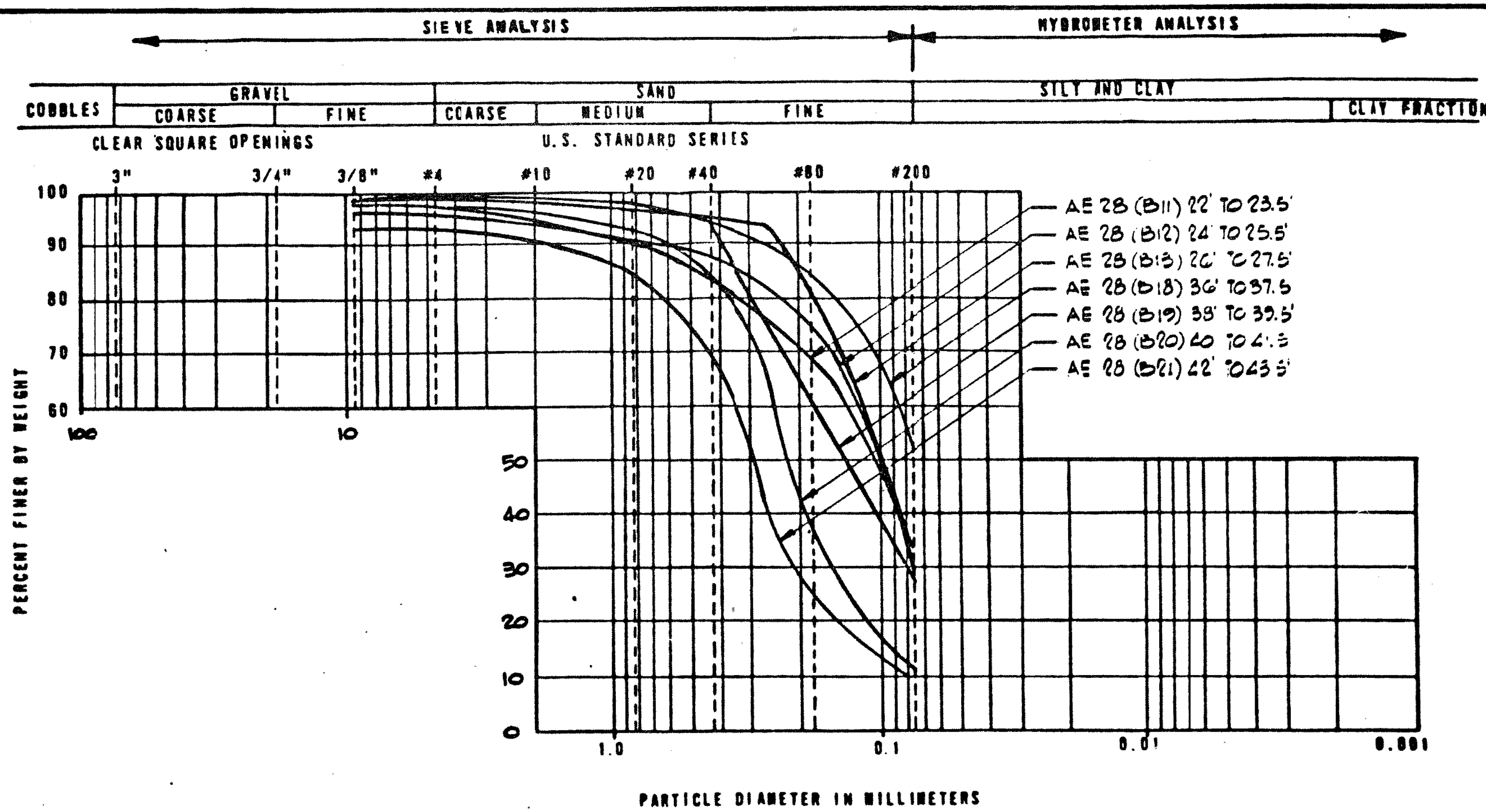
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-27
FIGURE 2G-C27 (CON'T)-2



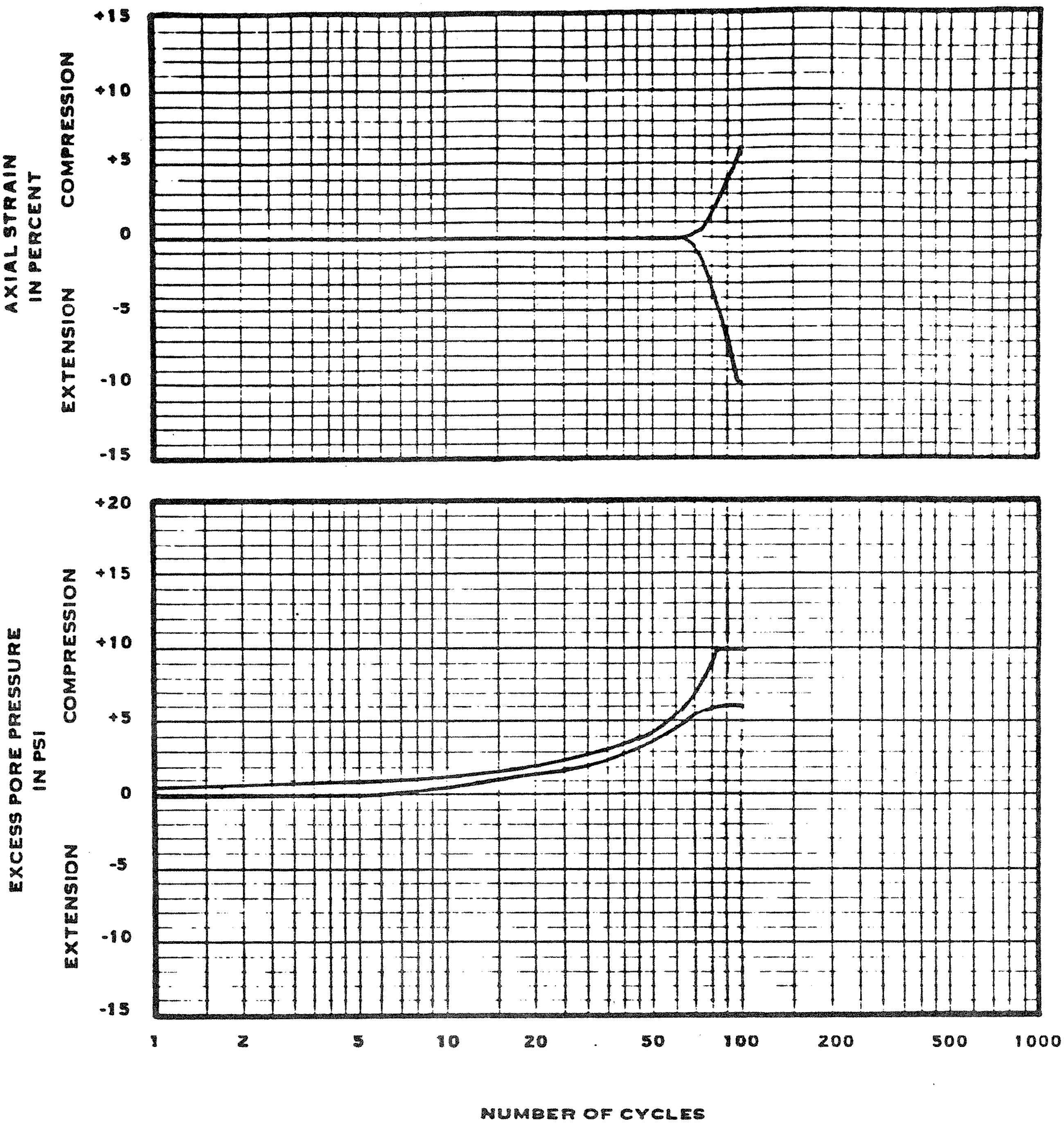
FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-27
FIGURE 2G-C27 (CON'T)-3



FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

GRAIN SIZE DISTRIBUTIONS
BORING AE-28
FIGURE 2G-C28



REMARKS: SPECIFIC GRAVITY = 2.73

Fig 2G-D1

CONFINING PRESSURE 17.4 PSI. BACK PRESSURE ______ PSI

CYCLIC DEVIATOR STRESS 1029 PSF. "B" VALUE ______

AXIAL STRAIN* (%)	NUMBER OF CYCLES
2	75
5	80
10	88
15	96

*DOUBLE AMPLITUDE STRAIN

CYCLIC TRIAXIAL TEST NO. 1

PROJECT: ST. LUCIE PLANT SA-737

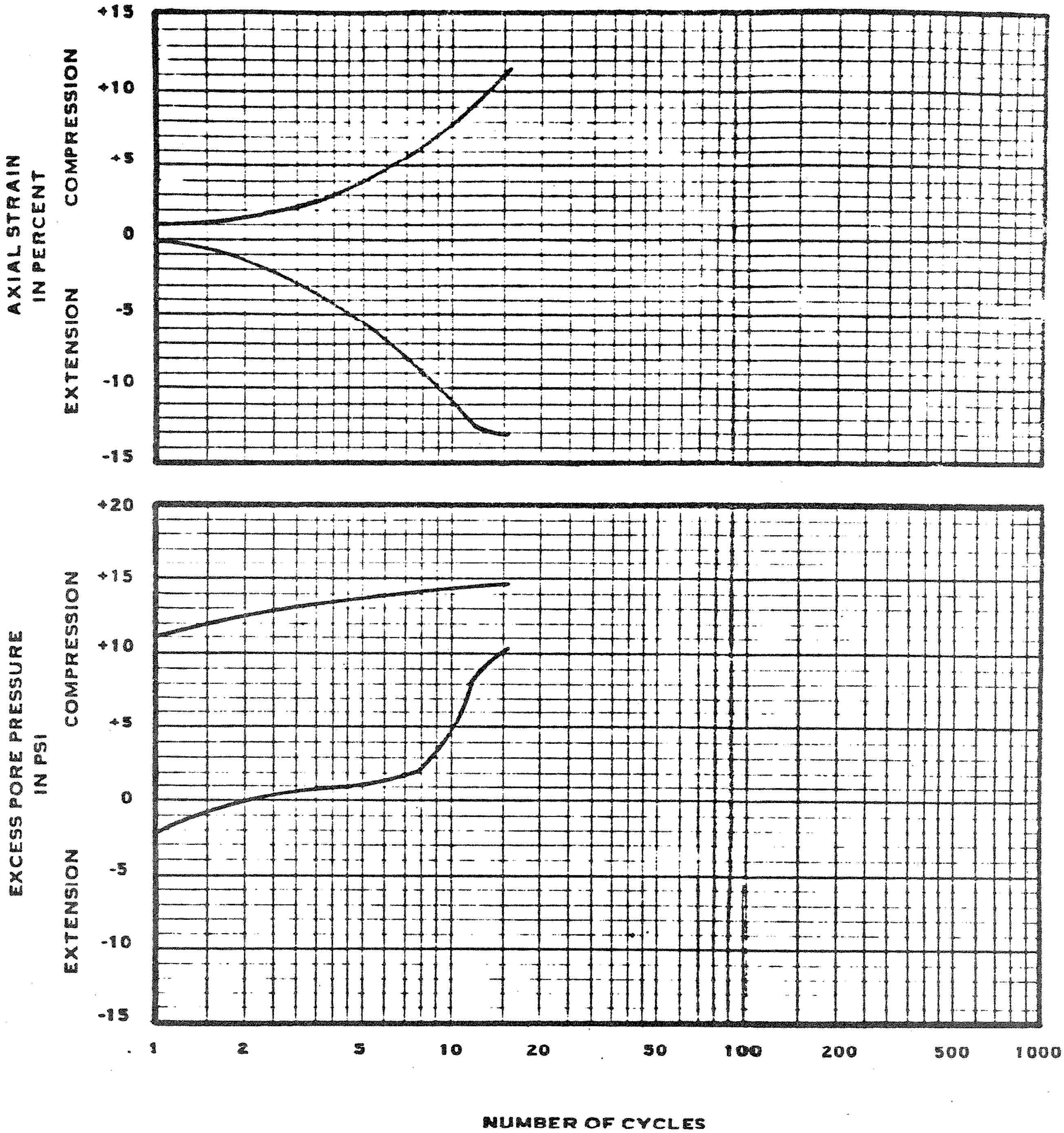
BORING NO. AE-27A SAMPLE NO. T-2 A

DEPTH 12-14 FT.

BORING LOCATION: UNIT 1 INTAKE

SAMPLE DESCRIPTION: TAN SHELLY SILTY FINE SAND

LAW ENGINEERING TESTING COMPANY
MARIETTA, GEORGIA



REMARKS: SPECIFIC GRAVITY = 2.73

Fig 2G-D2

CONFINING PRESSURE 17.4 PSI. BACK PRESSURE 60 PSI
CYCLIC DEVIATOR STRESS 1548 PSF "B" VALUE .99

AXIAL STRAIN* (%)	NUMBER OF CYCLES
2	2
5	3
10	5
15	8

*DOUBLE AMPLITUDE STRAIN

CYCLIC TRIAXIAL TEST NO. 2

PROJECT: ST. LUCIE PLANT SA-737

BORING NO. AE-27A SAMPLE NO. T-2 B

DEPTH 12-14 FT.

BORING LOCATION: UNIT 1 INTAKE

SAMPLE DESCRIPTION: TAN SHELLY SILTY FINE SAND

LAW ENGINEERING TESTING COMPANY
MARIETTA, GEORGIA

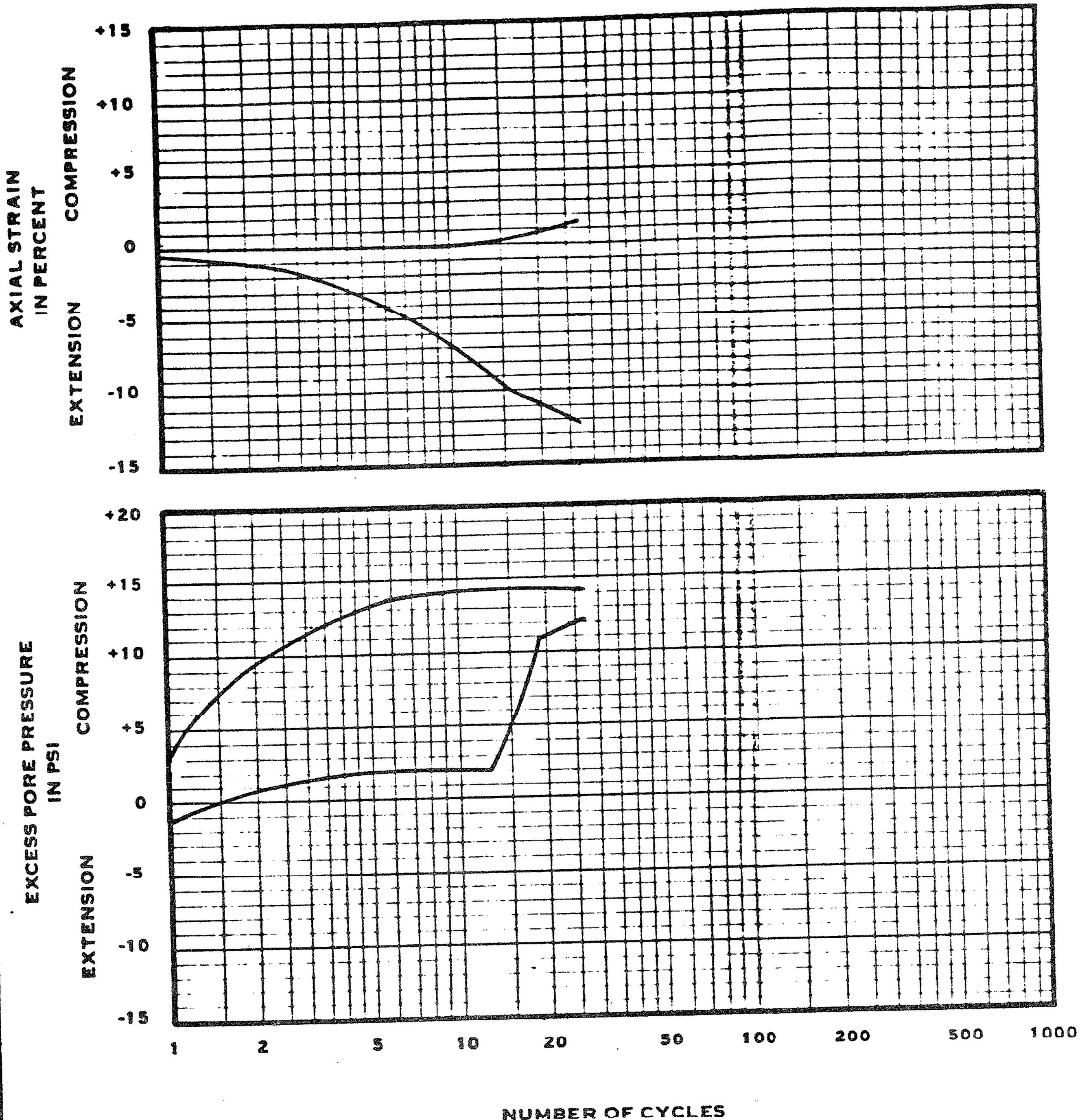


Fig 2G-D3

REMARKS: SPECIFIC GRAVITY = 2.73

CONFINING PRESSURE 17.4 PSI. BACK PRESSURE 60 PSI
CYCLIC DEVIATOR STRESS 1520 PSF. "B" VALUE .98

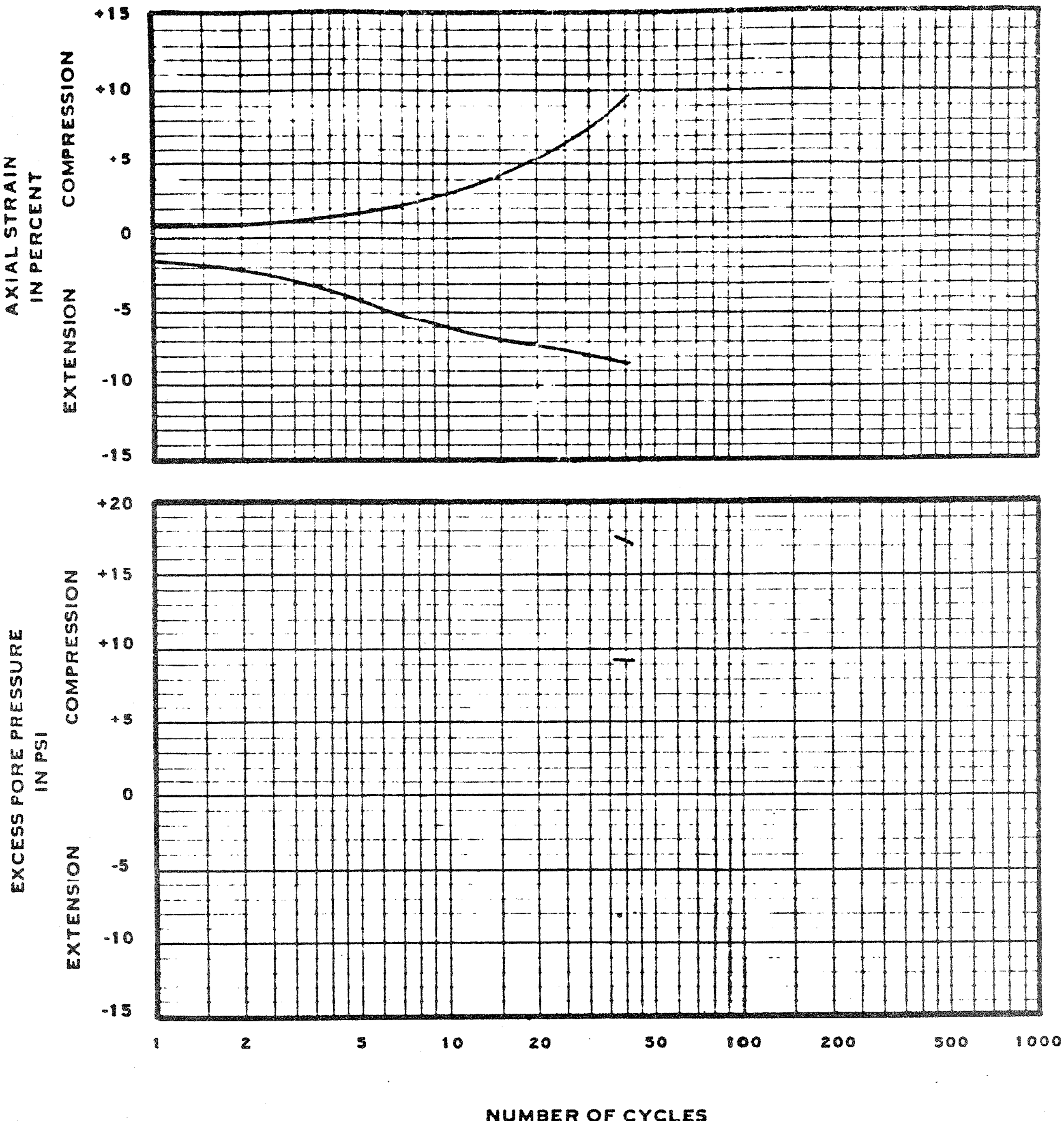
AXIAL STRAIN* (%)	NUMBER OF CYCLES
2	4
5	7
10	16
15	—

*DOUBLE AMPLITUDE STRAIN

CYCLIC TRIAXIAL TEST NO. 3

PROJECT: ST. LUCIE PLANT SA-737
BORING NO. AE-27A SAMPLE NO. T-8 A
DEPTH 20-22 FT.
BORING LOCATION: UNIT 1 INTAKE
SAMPLE DESCRIPTION: TAN SHELLY SILTY FINE SAND

LAW ENGINEERING TESTING COMPANY
MARIETTA, GEORGIA



REMARKS: SPECIFIC GRAVITY = 2.73

Fig. 2G-D4

CONFINING PRESSURE 17.4 PSI. BACK PRESSURE 60 PSI
CYCLIC DEVIATOR STRESS 1244 PSF. "B" VALUE .99

AXIAL STRAIN* (%)	NUMBER OF CYCLES
2	1
5	4
10	12
15	28

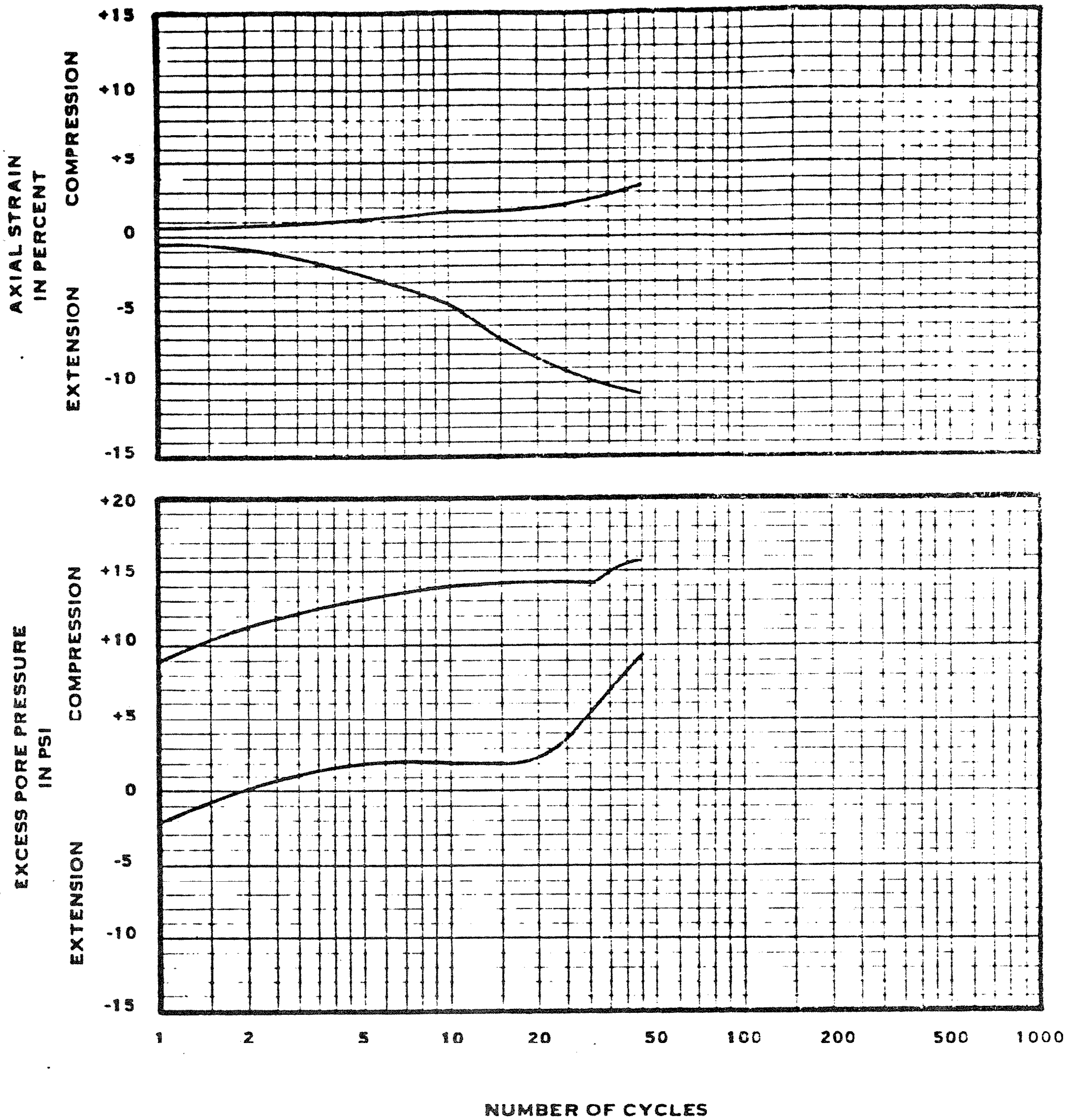
*DOUBLE AMPLITUDE STRAIN

CYCLIC TRIAXIAL TEST NO. 4

PROJECT: ST. LUCIE PLANT SA-737
BORING NO. AE-27A SAMPLE NO. T-11A
DEPTH 24-26 FT.
BORING LOCATION: UNIT 1 INTAKE

SAMPLE DESCRIPTION: TAN SILTY FINE SAND

LAW ENGINEERING TESTING COMPANY
MARIETTA, GEORGIA



REMARKS: SPECIFIC GRAVITY = 2.73

Fig 2G-D5

CONFINING PRESSURE 17.4 PSI. BACK PRESSURE 65 PSI
CYCLIC DEVIATOR STRESS 1714 PSF. "B" VALUE .99

AXIAL STRAIN* (%)	NUMBER OF CYCLES
2	3
5	8
10	18
15	—

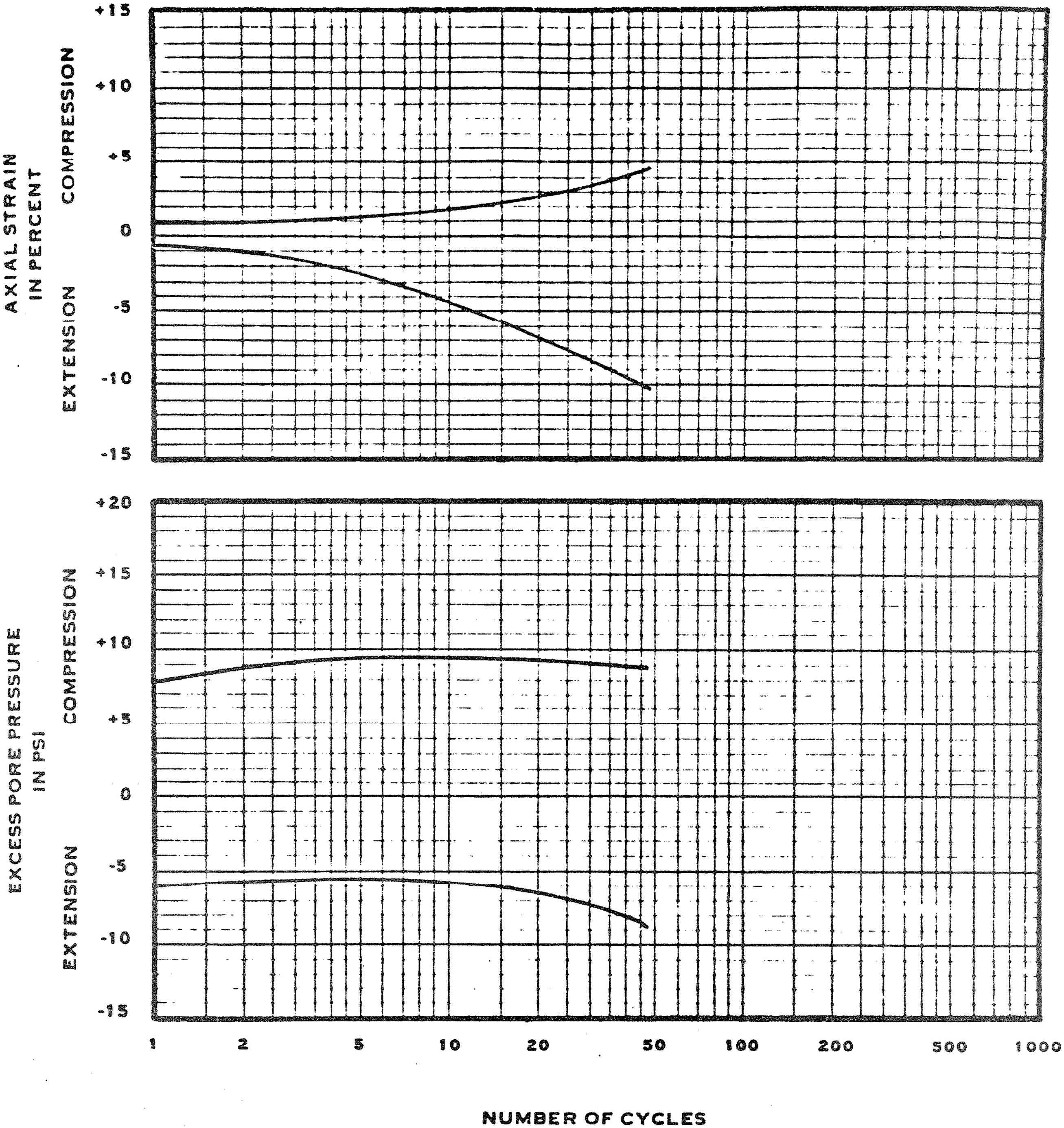
*DOUBLE AMPLITUDE STRAIN

CYCLIC TRIAXIAL TEST NO. 5

PROJECT: ST. LUCIE PLANT SA-737
BORING NO. AE-27A SAMPLE NO. T-8B
DEPTH 20-22 FT.
BORING LOCATION: UNIT 1 INTAKE

SAMPLE DESCRIPTION: TAN SILTY FINE SAND

LAW ENGINEERING TESTING COMPANY
MARIETTA, GEORGIA



REMARKS: SPECIFIC GRAVITY = 2.76

Fig 2C-D6

CONFINING PRESSURE 17.4 PSI, BACK PRESSURE 40 PSI

CYCLIC DEVIATOR STRESS 2046 PSF, "B" VALUE .98

AXIAL STRAIN* (%)	NUMBER OF CYCLES
2	2
5	7
10	20
15	—

*DOUBLE AMPLITUDE STRAIN

CYCLIC TRIAXIAL TEST NO. 6

PROJECT: ST. LUCIE PLANT SA-737

BORING NO. AE-27B SAMPLE NO. T-5A

DEPTH 19-21 FT.

BORING LOCATION: UNIT 1 INTAKE

SAMPLE DESCRIPTION: TAN SILTY FINE SAND

LAW ENGINEERING TESTING COMPANY
MARIETTA, GEORGIA

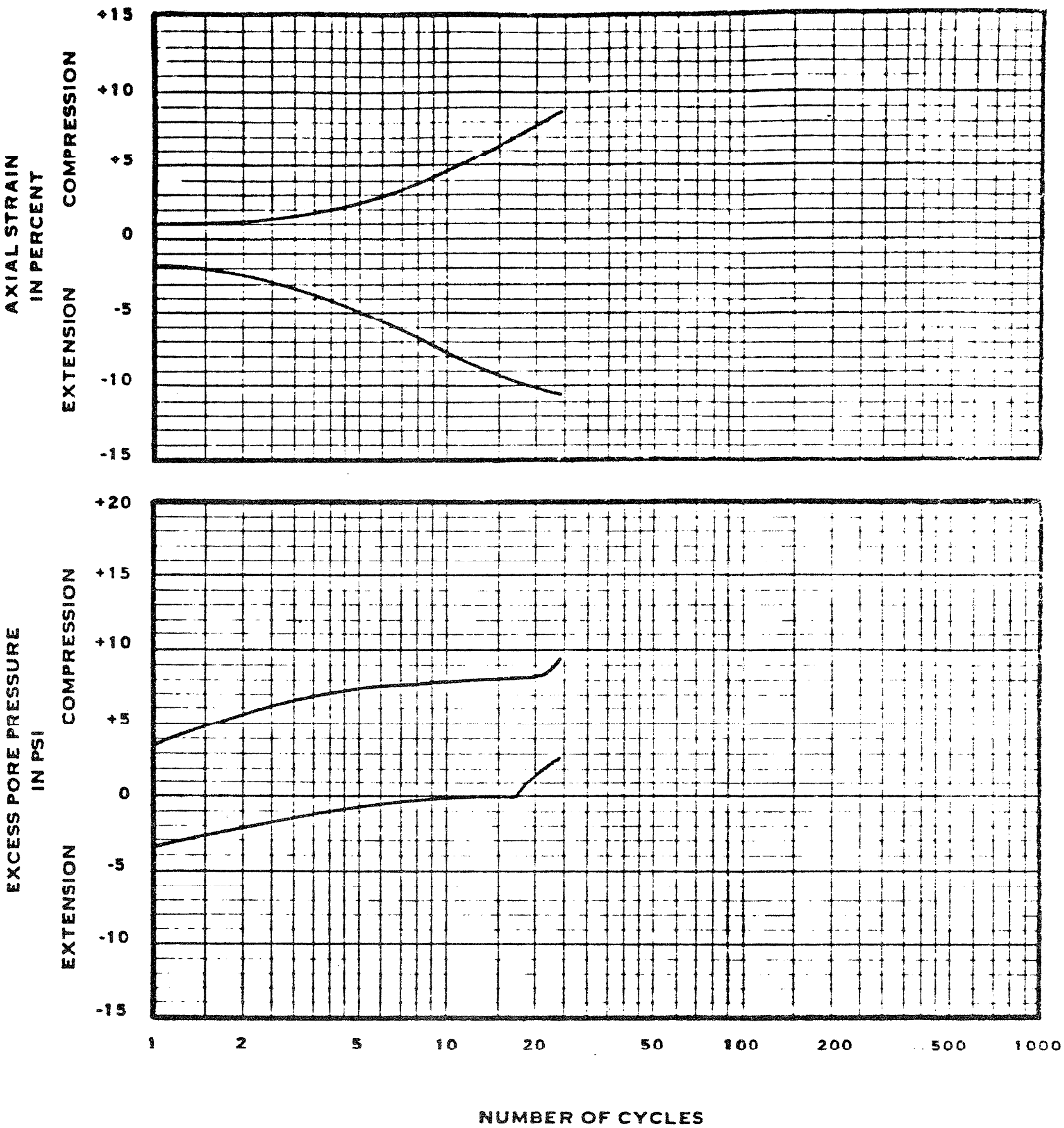


Fig 2G-D7

REMARKS: SPECIFIC GRAVITY = 2.75

CONFINING PRESSURE 17.4 PSI. BACK PRESSURE 40 PSI
CYCLIC DEVIATOR STRESS 1770 PSF. "B" VALUE .98

AXIAL STRAIN* (%)	NUMBER OF CYCLES
2	1
5	3
10	8
15	14

*DOUBLE AMPLITUDE STRAIN

CYCLIC TRIAXIAL TEST NO. 7

PROJECT: ST. LUCIE PLANT SA-737

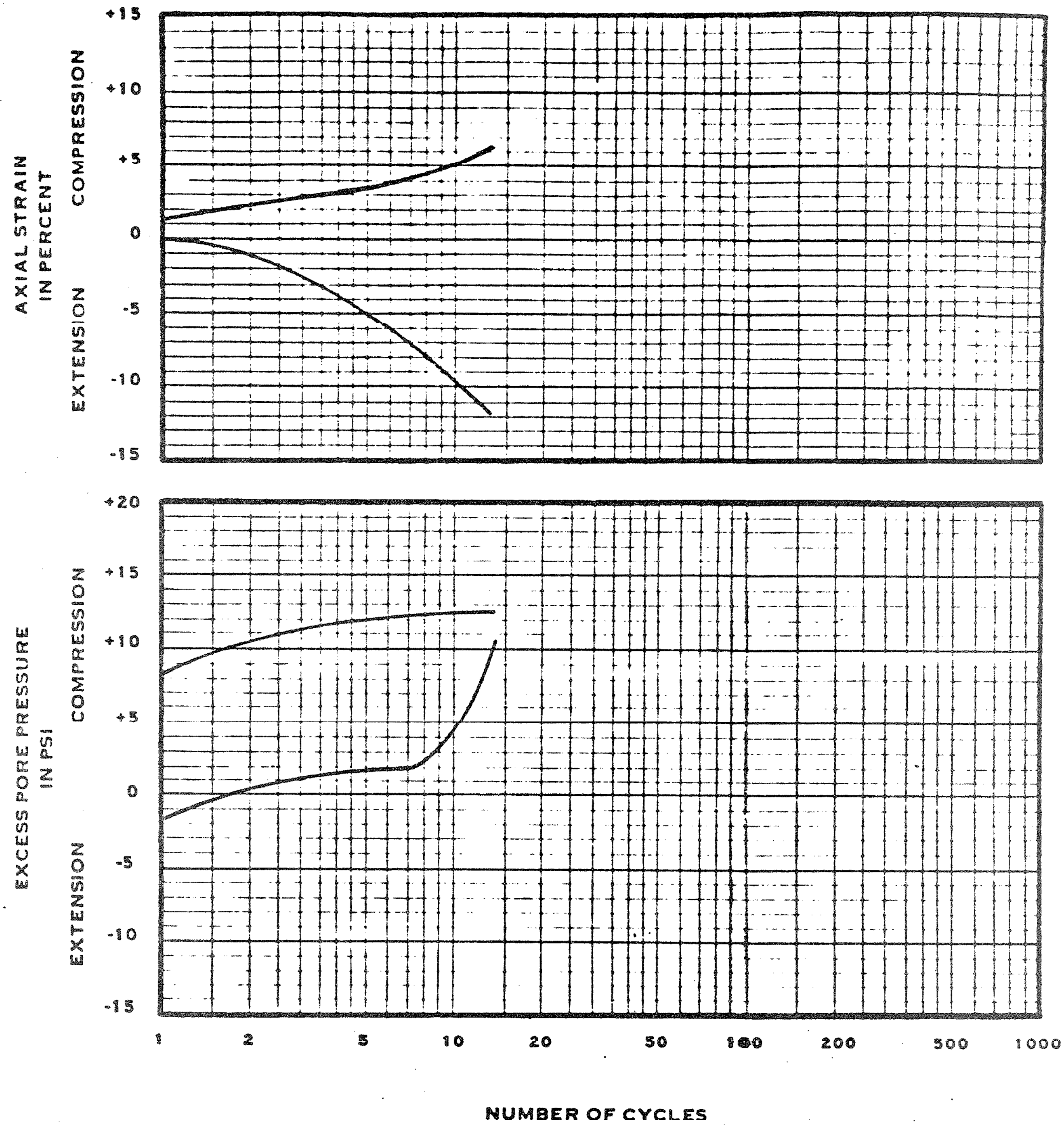
BORING NO. AE-27B SAMPLE NO. T-7

DEPTH 23-25 FT.

BORING LOCATION: UNIT 1 INTAKE

SAMPLE DESCRIPTION: TAN SILTY FINE SAND

LAW ENGINEERING TESTING COMPANY
MARIETTA, GEORGIA



REMARKS: SPECIFIC GRAVITY = 2.69

Fig. 2G-D8

CONFINING PRESSURE 17.4 PSI. BACK PRESSURE 60 PSI
CYCLIC DEVIATOR STRESS 1123 PSF. "B" VALUE .98

AXIAL STRAIN* (%)	NUMBER OF CYCLES
2	1
5	3
10	6
15	10

*DOUBLE AMPLITUDE STRAIN

CYCLIC TRIAXIAL TEST NO. 8

PROJECT: ST. LUCIE PLANT SA-737
BORING NO. AE-28 SAMPLE NO. UD-3
DEPTH 14.5-15.5 FT.
BORING LOCATION: UNIT 1 INTAKE
SAMPLE DESCRIPTION: LIGHT GREY SILTY FINE SAND

LAW ENGINEERING TESTING COMPANY
MARIETTA, GEORGIA

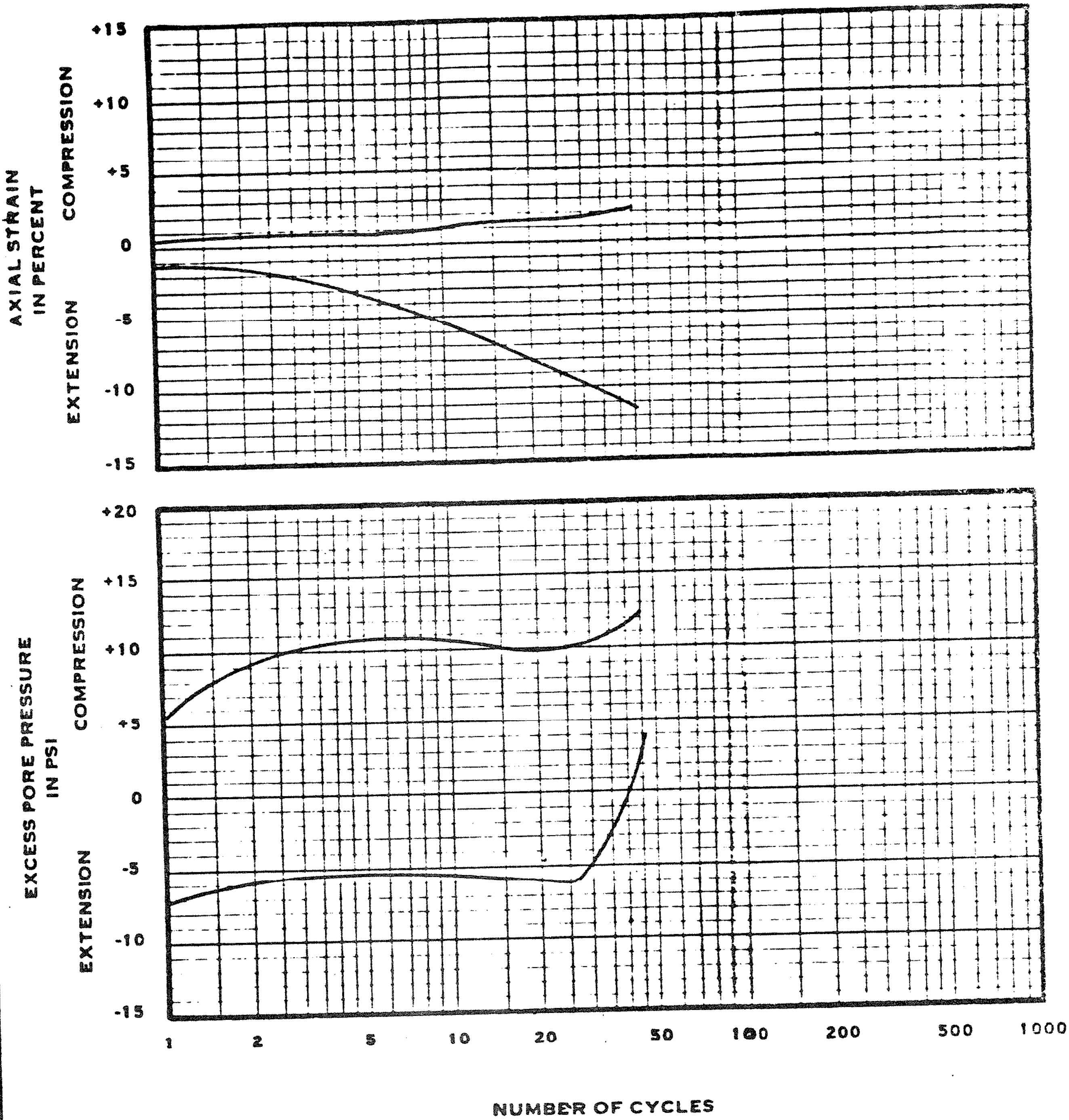


Fig 2G-D9

REMARKS: SPECIFIC GRAVITY = 2.71

CONFINING PRESSURE 17.4 PSI. BACK PRESSURE 30 PSI
CYCLIC DEVIATOR STRESS 2336 PSF. "B" VALUE .97

AXIAL STRAIN* (%)	NUMBER OF CYCLES
2	2
5	7
10	22
15	—

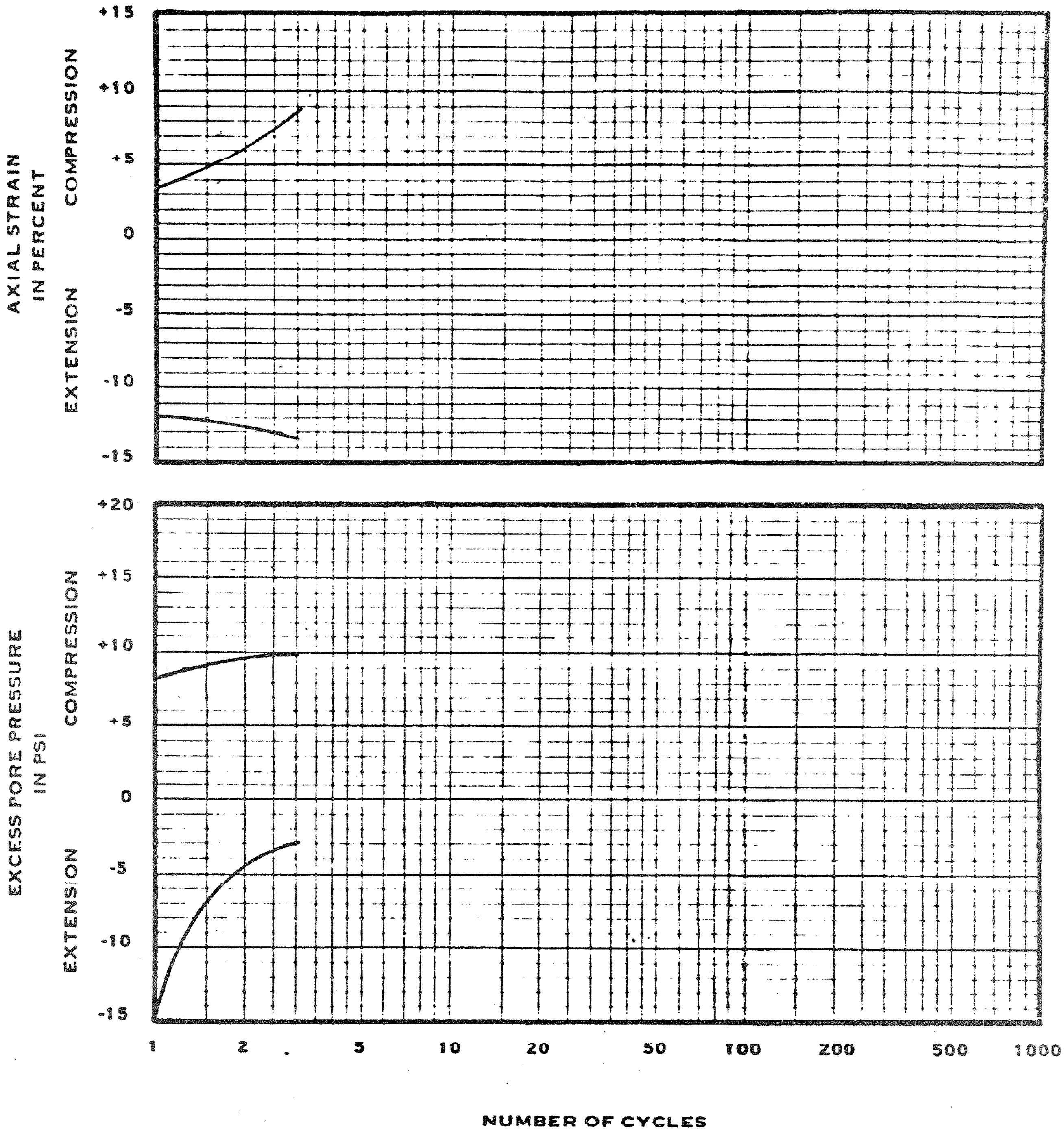
*DOUBLE AMPLITUDE STRAIN

CYCLIC TRIAXIAL TEST NO. 9

PROJECT: ST. LUCIE PLANT SA-737
BORING NO. AE-2C SAMPLE NO. UD-1
DEPTH 9.5-11.5 FT.
BORING LOCATION: UNIT 1 INTAKE

SAMPLE DESCRIPTION: LIGHT GREY SHELLY SILTY FINE SAND

LAW ENGINEERING TESTING COMPANY
MARIETTA, GEORGIA



REMARKS: SPECIFIC GRAVITY = 2.73

Fig 2G-D10

CONFINING PRESSURE 17.4 PSI. BACK PRESSURE 61 PSI
CYCLIC DEVIATOR STRESS 2295 PSF. "B" VALUE .98

AXIAL STRAIN* (%)	NUMBER OF CYCLES
2	
5	
10	
15	1

*DOUBLE AMPLITUDE STRAIN

CYCLIC TRIAXIAL TEST NO. 10

PROJECT: ST LUCIE PLANT SA-737
BORING NO. A 27C SAMPLE NO. T-8
DEPTH 24-26 FT.
BORING LOCATION: UNIT 1 INTAKE
SAMPLE DESCRIPTION: LIGHT TAN SILTY FINE SAND

LAW ENGINEERING TESTING COMPANY
MARIETTA, GEORGIA

SUPPLEMENT 1

TO

APPENDIX 2G

2G8.0 ADDITIONAL INFORMATION REQUESTED BY NRC STAFF

2G8.1 General

This supplement to Appendix 2G presents additional information and documentation requested by the NRC Staff at the April 1, 1975 meeting in Bethesda, Maryland. The Staff requested that five additional items be presented by April 9, 1975 so that they could conclude their review of the Switchyard and Canal Investigation and Analysis. These items are 1) a shear strength determination of the clay layer from known construction conditions, 2) a discussion of the applicability of using the laboratory data from the samples of borings AE-5A, B, and C to other areas, particularly North of the switchyard and West of the main plant area, 3) justification that the low blow count and high shear strength data do not present an unusual condition, 4) presentation, discussion and evaluation of the consolidated undrained triaxial shear tests (CIU tests), and 5) letters from Dr. G. Castro and Dr. E. D'Appolonia stating their conclusions based on the information originally presented in Appendix 2G. These five items will be discussed in the following sub-sections of this amendment.

Table 2G-3 has been revised and is resubmitted with corrected unit weights, based on field unit weights. The unit weights originally shown on this table were based on trimmed samples. Data obtained on Jan. 7, 1975 indicated the unit weights were too low and they have thus been revised.

2G8.2 Construction Condition Verification of Shear Strength of Clay

Figure 2G-S1 presents the results of the determination of the shear strength of the clay during actual construction conditions in the switchyard area for safety factors of 1.3 and 1.5 with the friction angle of the sand at 30° as proposed by the Staff. This analysis using the sliding wedge procedures recommended by the Corps of Engineers as discussed in Section 2G6.2 indicates a cohesion of at least 750 to 1000 psf in the clay layer. This is consistent with the laboratory test values as presented in Figure 2G6. Based on this, it is concluded that the analyses previously presented are satisfactory and that a slide cannot occur along the clay layer.

2G8.3 Applicability of Sample Data from AE5, A, B, and C to Other Areas

Investigation of the switchyard area indicated sufficient strength to preclude the possibility of a flow slide in the sandy zones. This is based on the fact that this area has been backfilled to raise the grade with approximately thirteen (13) feet of compacted fill and that this fill has increased the confining pressures on the underlying sandy materials to the point where they no longer present a flow slide problem. In order to be able to use the shear strength determined from samples obtained in this area, the NRC Staff requested that field and laboratory data be related. Table 2G-S1 presents the average blow count data from boring AE-5 with the averabe blow count data from two areas designated by the Staff as Area I (Borings) and Area II (Borings). The blow count data was arbitrarily divided into 10 foot zones for this comparison, with blow counts from clay samples omitted. The laboratory test samples were obtained from lower blow count zones as shown on Figure 2G-S1. All laboratory test samples were obtained from either Elevation +10 to Elevation 0 or from

Elevation -10 to Elevation -20. From a comparison of the data in Table 2G-S1, based on average blow counts, it would be very conservative to use undisturbed samples from zones of Elevation +10 to Elevation 0 and Elevation -10 to Elevation -20 at boring AE-5 to represent soil at any depth in borings AE-5 or Area I and Area II. This is based on the fact that the two zones sampled had the lowest average blow count (10 and 20) of any of the 10 foot zones and, in fact, was the reason for the concentrated sampling in those zones.

These facts represent conclusive evidence that all areas have sufficient shear strength to preclude the possibility of a flow slide, since the lowest blow count zone material which is acceptable for the switchyard is now shown to be very conservative for the entire site at every depth. Additional supportive data is presented in the following sections.

2G8.4 Comparison of Blow Counts and Shear Strength

The attached letter (see ATTACHMENT 1) from Dr. E. D'Appolonia, with resume included, presents a discussion of this seemingly anomalous condition. It is concluded from the discussion presented in this letter that it is not unusual to have low penetration test results and yet material with high shear strength.

2G8.5 Consolidated Undrained Triaxial Shear Tests

The attached letter (see ATTACHMENT 2) from Dr. G. Castro, with resume included, presents a discussion of this data and an analysis using this data as proposed by Dr. Castro in his paper to be published in June 1975 in the ASCE Journals. Based on this analysis, it is concluded that flow slides cannot occur in the sands as a result of cyclic mobility caused by the DBE.

2G8.6 Letters from Consultants

Attachments I and II present the letters and resumes of the consultants who have reviewed the original Appendix 2G and contain their conclusions. Based on their comments, it is concluded that a flow slide cannot occur.

2G8.7 Conclusions

The above sections present the additional information and documentation that has been requested. The detmination of the shear strength of the clay layer beneath the switchyard, calculated from the actual construction conditions in the switchyard, verifies the strength established by laboratory testing and indicates that failure cannot occur along the clay layer. The data presented in Table 2G-S1 proves conclusively that the laboratory test results are conservative and applicable to all areas and depths of the investigation.

The letter from Dr. D'Appolonia explains adequately the relationship between standard penetration test values and shear strength for this stie. The letter from Dr. Castro analyses the laboratory results with respect to his paper and indicates that the slopes are safe against a flow slide. In summary the conclusion of all of these sections is that flow slides cannot occur and that the proposed ultimate heat sink system is conservative and can function adequately.

TABLE 2G-S1

ST. LUCIE PROJECT

COMPARISON OF SPT BLOW COUNTS FOR BORING AE-5
AND AREAS I & II

ELEVATION	BORING AE-5	AREA I	AREA II	LAB. TEST SAMPLE LOCATION
+20 to +10	28	-	50	
+10 to 0	10	35	36	AE-5B T-1,2,3,4 AE-5C T-1,3
0 to -10	65	26	32	
-10 to -20	20	22	36	AE-5A UD-1,2,3 AE-5B T-5,6,7,8
-20 to -30	58	35	43	
-30 to -40	44	28	37	
-40 to -50	40	24	29	
-50 to -60	25	30	36	
-60 to -70	48	45	-	

Note: (1) SPT blow counts given are average for each 10 foot zone.
(2) Areas I & II are as selected by NRC staff in the canal area.

SUPPLEMENT TO APPENDIX 2G

ATTACHMENT 1

E. D'Appolonia Consulting Engineers, Inc.

April 7, 1975

Elio D'Appolonia
PRESIDENT

Project 75-654

Mr. Joseph Ehasz
Supervising Engineer
Ebasco Services Inc.
Two Rector Street
New York, New York 10006

Switchyard and Canal Foundation Investigation
St. Lucie Plant Unit No. 1
Florida Power & Light Company

Dear Mr. Ehasz:

I have reviewed the report "Switchyard and Canal Investigation and Analysis, Appendix 2G," Revision 27, dated January 31, 1975 of Docket No. 50-389 of the AEC.

My comments relate solely to the stability of the canal under static and seismic loadings.

The report describes the care and control exercised during drilling and securing of both disturbed and undisturbed soil samples. Dissipation of highly localized excess pore pressure resulting from drilling was allowed before soil samples were extruded from the hole. Care exercised in transporting the undisturbed samples to the laboratory and in cutting the tubes minimized their disturbance. This control of soil drilling and sampling assured a high degree of reliability and confidence in the results of laboratory tests and analysis.

Of particular note is the emphasis that was placed on securing undisturbed samples of the sand strata having the greatest potential for liquefaction as determined by Standard Penetration Tests. Samples at the desired depth were obtained by drilling a number of holes within a small radius from a boring used to identify the elevation at which the critical layers occurred. Both consolidated isotropically undrained (CIU) and cyclic triaxial tests were conducted on these "liquefiable" sands. This suggests for this site a higher level of conservatism in the results of analysis than normally obtained.

Unit weight determinations of undisturbed samples disclose that the sands and silty sands within the potential zones of "liquefaction" are in a medium-dense to dense state. The fine sands having approximately less

than 15 percent particle sizes passing the 200 mesh sieve are listed in Table 1, and their relative densities range from 72 to 82 percent. Tables 2 and 3 show relative densities for the fine sands with higher silt fractions, in all cases but two (Samples AE-27E-T2 and AE-27E-T1), have relative densities greater than 65 percent. The two samples mentioned have relative densities of 55 and 42 percent and silt fractions of 21.8 and 22.8 percent.

The CIU shear tests, in most cases, indicate that the sandy materials are dilative. This is indicative of medium-dense to dense granular deposits. Dilation implies an increase in shear strength as the soil tends to deform under seismic loadings, thereby lessening the danger of a stability failure or a flow slide during an earthquake.

The dynamic analyses of cyclic mobility disclose factors of safety greater than 1.31. The analyses are based on 10 cycles of loading and shear strains of 10 and 15 percent. The maximum excess pore water pressure under cyclic triaxial testing was 49 percent, again indicative of dilative soils whose shear strength tends to increase rather than decrease under straining from either static or dynamic loadings. The analyses of factors of safety of the site against cyclic mobility during an earthquake indicate adequate margins of safety against failure.

A consideration of the reserve of the residual shear strength as obtained from CIU shear tests to the combined induced dynamic shear stress and in-situ shear stress along potential surfaces of failure shows adequate margins of safety against flow slides.

A review of the Standard Penetration Tests indicates that there are wide fluctuations in blow counts with depth in the potentially "liquefiable" soils. These variations in blow count with depth can be attributed to a number of factors such as the temporary excess pore water pressure induced by the vibrations of the hammer during the conduct of the Standard Penetration Test, the presence of thin silt or organic partings interspersed through the sand deposits, the differences in in-situ densities of sand on the leeward and windward sides of dune deposits, influence of currents during deposition with rising and falling sea levels, etc. Nevertheless, Standard Penetration Tests conducted on silty sands should be used only as an indicator of the expected performance of the material to load and not to predict soil behavior.

At the St. Lucie site, there is an apparent disparity between low blow counts and high relative densities of the silty sands. Undisturbed samples of the potentially "liquefiable" sands were obtained to assess this factor relative to the stability of the canal under both static and dynamic loadings. As stated above, factors of safety against a stability failure under either dynamic or static loadings are adequate as

determined by acceptable state-of-the-art procedures for dynamic analysis. To explain the disparity between blow count and relative density of the silty sands, the following comments are offered.

Column 6 of Table 1 shows the blow count of Standard Penetration Tests of the sand with low silt content compared to values given by Gibbs and Holtz. They show a penetration resistance of practically zero for a saturated silty sand having a relative density of 60 percent. For higher relative densities and increasing effective stress, the resistance increases. Their work was conducted in the laboratory under carefully controlled conditions. One would expect to find the same trend in the field for carefully executed tests. Column 6 under the heading "N_f" shows the blow counts for fine, saturated sands with low silt content at the switchyard. The sands tested by Gibbs and Holtz are similar to the fine sands occurring at the site. It is interesting to note that in all but two cases the blow count in the field is nearly equal to or greater than the values given by Gibbs and Holtz. For test samples AE-5B, T-5A and T-5B taken at depths of 28 to 30 feet below the surface, the blow count in the field is less than that given by Gibbs and Holtz.

Further, Tables 2 and 3 show that with increasing silt content, the blow count generally increases; but there is no clear relationship between blow count and relative density. Equally well, with increasing silt content, the potential for "liquefaction" of the sand samples listed in Tables 2 and 3 decreases.

It is my professional opinion from the review of the data presented in Appendix 2G of Docket No. 50-389 and of the results of the dynamic analyses reported therein, as well as my interpretation of the data, that the risk of a failure of the slopes of the canal at the switchyard under either static or seismic loadings is remote.

Sincerely,

E. D'APPOLONIA CONSULTING ENGINEERS, INC.

E D'Appolonia

E. D'Appolonia

ED:asm

Encls.

TABLE 1

GROUP I
SAND SAMPLES
LOW SILT CONTENT
APPROXIMATELY LESS THAN 15 PERCENT
PASSING NO. 200 SIEVE

1		2	3	4	5	6		7
Sample No.		Depth Below Surface (ft)	Passing 200 Sieve (%)	Dry Unit Weight (pcf)	Relative Density (%)	Standard Penetration Test		CIU Shear Tests
						N_f	N_o^*	
AE-2C	UD-1	9.3/10.3	16.0	91.5	82	20	10	-
	UD-2A	11.3/12.1	13.0	92.7	75	5	5	-
	UD-2B	11.3/12.1	4.0	92.7	79	5	6	-
AE-5A	UD-2A	30.0/32.4	15.7	91.5	-	5	-	Dilative
AE-5B	T-5A	28.0/30.0	11.8	93.0	79	5	12	Dilative
	T-5B	28.0/30.0	11.0	93.0	79	5	12	Dilative
	T-6A	30.0/32.0	13.1	90.4	72	5	7	Neutral
	T-6B	30.0/32.0	15.4	90.4	72	5	7	Neutral
	T-7A	32.0/34.0	4.9	103.6	-	15	-	Contractive
	T-7B	32.0/34.0	4.6	103.6	-	15	-	Contractive
AE-5C	T-1	10.0/12.0	14.5	91.2	74	15	5	-

* Gibbs, H. J. and Holtz, W. G., "Research on Determining the Density of Sands by Spoon Penetration Testing, Proc.4th International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, 1957, p. 35, Figure 7.

TABLE 2

GROUP II
SAND SAMPLES
MEDIUM SILT CONTENT
APPROXIMATELY 15 TO 30 PERCENT
PASSING NO. 200 SIEVE

1	2	3	4	5	6	7
Sample No.	Depth Below Surface (ft)	Passing 200 Sieve (%)	Dry Unit Weight (pcf)	Relative Density (%)	Standard Penetration Test N_f	CIU Shear Tests
AE-5B T-8A	34.0/36.0	27.3	112.8	90	13	Contractive
AE-5C T-3	14.0/16.0	29.1	91.8	75	2	-
AE-27A T-2	12.0/14.0	20.0	87.7	65	15-5	-
T-8	20.0/22.0	22.0	95.7	77	30-15	-
T-8	20.0/22.0	23.0	99.1	85	30-15	-
AE-27B T-5	19.0/21.0	19.0	97.6	91	25-10	-
AE-27D T-2	12.0/13.0	21.8	88.2	55	10-2	-
T-6	20.0/21.0	25.8	102.7	87	35-5	-
AE-27E T-1	10.0/11.3	22.8	83.8	42	13-5	-
T-5	18.0/20.0	26.5	97.1	97	20-35	-

TABLE 3

GROUP III
SAND SAMPLES
HIGH SILT CONTENT
APPROXIMATELY GREATER THAN 30 PERCENT
PASSING NO. 200 SIEVE

1	2	3	4	5	6	7
Sample No.	Depth Below Surface (ft)	Passing 200 Sieve (%)	Dry Unit Weight (pcf)	Relative Density (%)	Standard Penetration Test N_f	CIU Shear Tests
AE-5B T-8B	34.0/36.0	34.1	100.0	90	13	-
AE-27A T-2	12.0/14.0	35.0	89.8	70	12-5	-
T-11	24.0/26.0	33.0	99.3	85	15-50	-
AE-27C T-8	24.0/26.0	36.0	90.1	70	10-50	-
AE-27E T-6	22.0/24.0	32.0	101.3	89	20-35	-

ELIO D'APPOLONIA

Chairman of the Board of Directors, President, Controller and Project Manager - E. D'Appolonia Consulting Engineers, Inc.

Education

Ph.D., Civil Engineering, University of Illinois
M.S., Civil Engineering, University of Alberta
B.S., Civil Engineering, University of Alberta

Registration

Professional Engineer: Alaska, Illinois, Indiana, Michigan, Pennsylvania, Rhode Island

Affiliations

American Arbitration Association, American Geophysical Union, American Institute of Consulting Engineers, American Society for Testing and Materials, American Society of Civil Engineers (Geotechnical Engineering Division, Underground Construction Research Council), American Water Resources Association, Association of Engineering Geologists, Association of Iron and Steel Engineers, Consulting Engineers Council, Engineering Institute of Canada, Highway Research Board, International Association for Bridge & Structural Engineers, International Commission on Large Dams, International Society for Rock Mechanics, National Society of Professional Engineers, Pennsylvania Society of Professional Engineers, Professional Engineers in Private Practice, Seismological Society of America, Sigma Xi Research Society

Experience and Background

While an instructor at the University of Alberta, Dr. D'Appolonia was retained by the United States Army Corps of Engineers on permafrost problems in northern Canada and Alaska, and was instrumental in the development of techniques for the construction of highways, airports and buildings in permanently frozen regions.

In 1948 he became associated with Carnegie Institute of Technology (now Carnegie-Mellon), teaching and undertaking research in applied mechanics and foundation engineering. For many years, he maintained an interest in research at Carnegie-Mellon University by advising and guiding graduate students in their theses.

In 1956 he formed his own firm and has since devoted his time wholly to the practice of civil engineering in the fields of soil and rock mechanics, foundation engineering and applied mechanics.

(Elio D'Appolonia)

His academic training, teaching and research work have qualified him as a consultant on broad and varied problems in the design and construction of foundations during the past twenty-five years. His experience relating to the earth sciences has been gained primarily as a consultant to heavy industry--steel, petrochemical, power, mining...--encompassing foundations for complex facilities subjected to heavy loadings and located in areas of difficult subsoils. His philosophy of continuity of engineering services, i.e., continuous service through the investigation, design and construction phases of a project, including observation of the behavior of completed foundations, has been applied to steel and concrete structures and earth and rock fill dams.

Dr. D'Appolonia has been a consultant on numerous problems involving failures of earthworks and similar facilities that have undergone distress; and has conducted research in applied mechanics, vibrations, fatigue of materials, soil mechanics, structural engineering and mathematical methods as they relate to stress analysis problems. Since 1967 he has served as a consultant to the Advisory Committee on Reactor Safeguards of the U. S. Atomic Energy Commission with regard to the hazards of proposed or existing reactor facilities and the adequacy of the proposed reactor safety standards, and related matters.

He was retained as a private consultant on the foundation design for the 1976 Olympic Sports Complex in Montreal, Canada. The Complex is comprised of three primary structures--the stadium, the tower supporting the cable structure for the removable stadium roof as well as housing offices and sports facilities, and the Velodrome. Owing to design differences and the varying quality of foundation rock at the site, each foundation posed different problems for design, construction and quality assurance. Utilizing seismicity and geophysical subsurface investigations coupled with analytical and finite element studies of the proposed foundation designs, Dr. D'Appolonia provided design parameters and acceptability criteria for the various deep foundations on rock. In addition, quality assurance field supervision and recommendations on construction were provided throughout the duration of the project.

He has a deep interest in underground engineering and construction and has been active in ASCE's Underground Construction Research Council since 1970, serving as Chairman of the Executive Committee from September 1973 to October 1974. Presently, he is Chairman of the Rapid Excavation and Tunneling Conference under the auspices of American Society of Civil Engineers and the American Institute of Mining Engineers. RETC's role is to effectively expedite the problems relating to urbanization and resource development by looking to the underground areas of urban centers and the mining

(Elio D'Appolonia)

of natural resources (including coal and oil shale) economically with improved underground mining technology.

Honors and Awards

1948, O'Keefer Medal, Engineering Institute of Canada, for the paper "Permanently Frozen Ground and Foundation Design."

1969, Thomas A. Middlebrooks Award, ASCE, for the paper "Settlement of Spread Footings on Sand."

January 1972, he was named "Civil Engineer of the Year" by the Pittsburgh Section of the American Society of Civil Engineers.

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"Contracting for Underground Systems," E. D'Appolonia, Proceedings, Workshop Seminar on "Cut-And-Cover Tunneling," The Federal Highway Administration First Annual FCP Research Progress Review, San Francisco, California, September 1973.

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SUPPLEMENT TO APPENDIX 2G

ATTACHMENT 2

SOIL AND ROCK ENGINEERING   CONSULTATION · DESIGN · TESTING

GEOTECHNICAL ENGINEERS INC.

1017 MAIN STREET · WINCHESTER · MASSACHUSETTS 01890   (617) 729-1625

RONALD C HIRSCHFELD
STEVE J POULOS
DANIEL P LA GATTA
RICHARD F MURDOCK
GONZALO CASTRO

April 8, 1975
Project 75230
Sent via telecopier 4/8/75

Mr. Joseph L. Ehasz
Supervising Soils Engineer
Ebasco Services Inc.
21 West Street
New York, New York 10006

Subject: St. Lucie Nuclear Plant

Dear Mr. Ehasz:

The purpose of this letter is to (1) present our review of Appendix 2G to the PSAR for the St. Lucie Nuclear Plant of Florida Power and Light Co. which deals with the subject of liquefaction and cyclic mobility potential in the area of the emergency cooling water intake and canal, and (2) document the liquefaction analysis which we have made based on the data in Appendix G and which we presented in the meeting held on April 1, 1975 with the staff of the NRC.

We also reviewed additional information provided verbally by Ebasco concerning their observations of the subsoils at the site during excavations and of the soil samples.

There are two phenomena which should be investigated when dealing with the behavior of saturated sands during earthquakes, namely cyclic mobility and liquefaction. Cyclic mobility consists of deformations induced by cyclic loading but does not entail loss of shear strength. Cyclic mobility can produce slumping of a slope but it cannot result in a flow slide. Liquefaction consists of a loss in shear strength, and can be induced by either static or cyclic loading. Flow slides are the most typical manifestation of liquefaction.

In the case of the slopes of the intake canals for the St. Lucie Plant, cyclic mobility would not constitute a serious problem. Liquefaction and a flow slide could, however, block the intake structures and prevent the access of the emergency cooling water. Thus, in what follows, I will comment only on the liquefaction potential for the sands at the site.

The standard penetration test can be used for clean sands to obtain preliminary information concerning the likelihood of cyclic mobility or liquefaction potential using published empirical information from occurrences of earthquakes (Refs. 2, 3, & 4). However, the standard penetration determinations are a very rough index of the compactness of the in situ soils and the empirical correlations are, at best, approximate since they are based on blowcounts and on assumed earthquake accelerations. Therefore, the use of such empirical correlations should be limited to preliminary assessments as to whether (1) there is a need to investigate liquefaction or cyclic mobility in more detail, or (2) the likelihood of a problem is so remote that such a study is not warranted.

The exploration reported in Appendix 2G consisted of 28 split-spoon sample borings and 25 undisturbed sample borings. The blowcounts in some zones of the sand strata are low and thus it was necessary to proceed to a more detailed investigation by means of tests on the undisturbed samples. The undisturbed samples were taken from those zones of the deposit that were representative of the loosest sands and of the softer clays as determined from the blowcounts obtained in previously drilled split-spoon sample borings.

The techniques used to obtain, handle and test the undisturbed samples as described in Appendix G correspond to the state of the art for such operations and should have ensured undisturbed samples of good quality.

Figure 2G-S2 shows a plot of the percentages of fines and the dry unit weight for all the undisturbed samples obtained. The liquefaction potential was investigated by means of consolidated-undrained triaxial ($\bar{R}$) tests performed on the undisturbed samples from borings AE5A and AE5B, both being in the vicinity of boring AE5 which was a split-spoon sample boring. The undisturbed samples used for the $\bar{R}$ tests, which are shown by means of square and octagonal symbols, correspond to the types of soil which are most susceptible to liquefaction, i.e., the cleaner sands with the lower unit weights. In addition, as explained above, the undisturbed samples themselves correspond to the lowest blowcounts in each boring location where they were taken. Undisturbed samples were taken in locations representative of the complete area of interest. In summary, it is felt that the $\bar{R}$ tests correspond to the most critical materials from the point of view of liquefaction for the complete area of the emergency water canal.

Eleven of the twenty $\bar{R}$ tests showed a dilative behavior and the other nine showed a contractive behavior. The maximum shear stress $(\sigma_1-\sigma_3)/2$ is plotted in Fig. 2G-S3 versus axial strain for all R tests on sands (less than 40% passing the No. 200 sieve). Some of these tests show a peak strength at small strains followed by a subsequent decrease in resistance until, in most cases, a more or less constant undrained residual strength was reached. In two cases, the samples were not strained sufficiently to define a value for the undrained residual strength.

The potential for the sand to liquefy and flow exists when the static shear stresses under the slope are larger than the undrained residual strength of the sands, Refs. 1 and 4.

The values of the average static shear stress for the two circles (AA & EE) shown to be the most critical for the slope shown at the bottom of Fig. 2G-30, are shown in Fig. 2G-S3 for a comparison with the undrained residual shear strength. The comparison indicates that the residual shear strength is higher by at least a factor of 2 and, in general, by a higher factor than the static shear stresses in the sand mass. Therefore, the exploratory program and the results of the $\bar{R}$ tests shows that a flow slide cannot develop in these sands. Only some minor slumping of the slopes could develop as a result of an earthquake. No attempt has been made by the writer to estimate the magnitude of the possible slumping.

I would be pleased to answer any questions you might have concerning this matter.

Very truly yours,
GEOTECHNICAL ENGINEERS INC.

G. Castro

Gonzalo Castro
Principal

GC:kmb

APPENDIX

REFERENCES

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January 1975

GONZALO CASTRO

Education

B. S. in Civil Engineering, Catholic University of Chile, 1961.

M. S. in Engineering, George Washington University, 1963.

Ph.D. in Engineering, Major in Soil Mechanics, Minors in Structural Dynamics and Applied Mechanics, Harvard University, 1969.

Professional Experience

1971-Present: Geotechnical Engineers Inc., Principal, 1974 to date; Director, 1973-74; Associate, 1972-73; Project Engineer, 1971-72.

1970: Associate Professor of Civil Engineering, Catholic University of Chile. Head of the Department of Soil Mechanics. Consultant on foundations for buildings and industrial installations. Member of a U. N. Mission to study the effects of the Peruvian Earthquake and to review the aseismic design of a U. N. experimental housing program.

1969: Research Fellow, Harvard University. Research on the phenomenon of Liquefaction of Sands. Consultant on the stability during earthquakes for several projects, including Tarbella Dam, West Pakistan; Arrow Dam, British Columbia, Canada; Burrard Inlet Crossing, British Columbia, Canada; San Juan River Project, Colombia.

1965-1968: Student and Research Assistant, Harvard University. Two years of formal study and two additional years of research for Ph.D. Thesis entitled "Liquefaction of Sands." Concurrently co-consultant on aseismic design for the foundations of a steel mill in Huachipato, Chile, and D. C. Cook Nuclear Power Plant in Michigan.

1963-1964: Associate Professor of Civil Engineering, Catholic University of Chile. Teaching courses in Theory of Elasticity and Soil Mechanics. In charge of Soil Mechanics Laboratory. Private consulting practice in Soil Mechanics.

Publications

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Professional Societies

American Society of Civil Engineers
Institute of Engineers, Chile

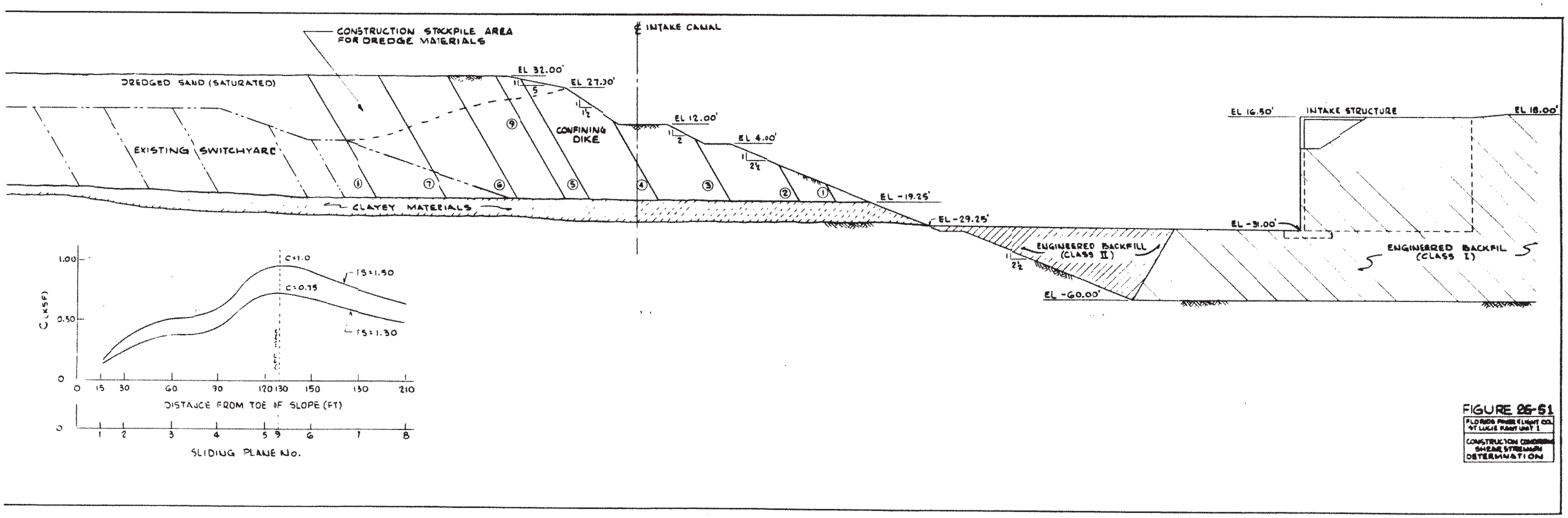
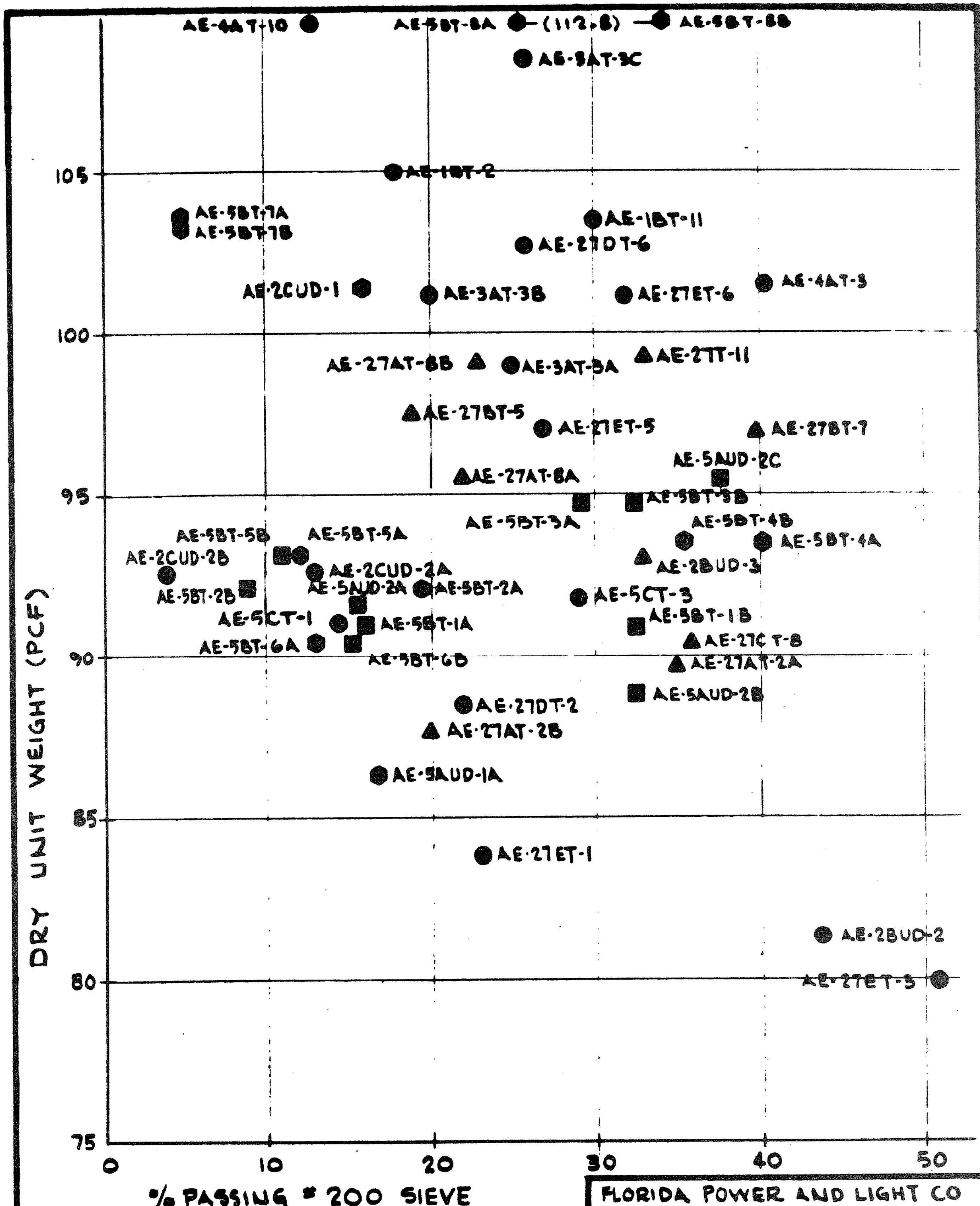


FIGURE 2G-51

ST LUCIE PLANT UNIT 1

DETERMINATION



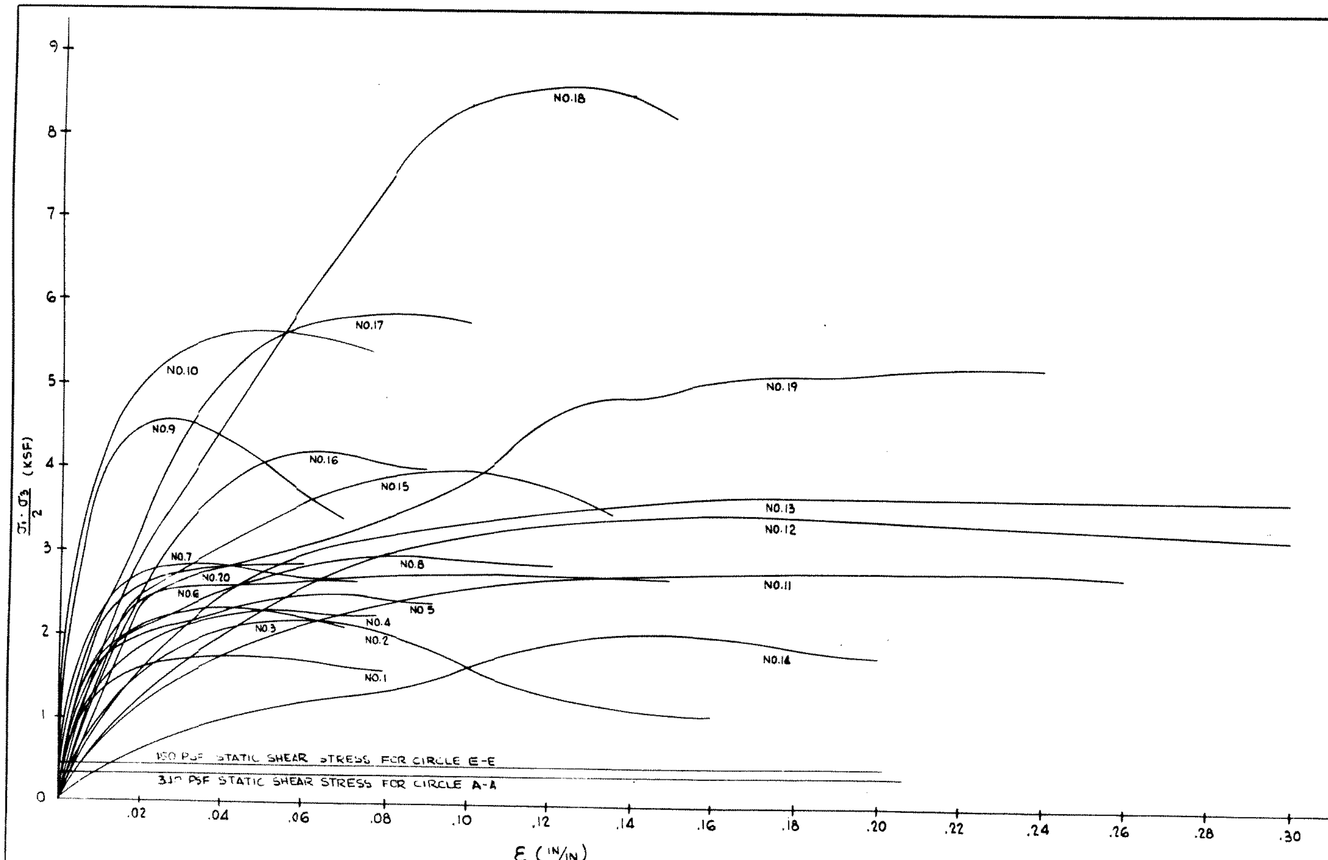
LEGEND

- ▲ CYCLIC TRIAXIAL TEST
- ■ STATIC CIU (DILATED)
- ● STATIC CIU (CONTRACTIVE)
- ● NOT TESTED

FLORIDA POWER AND LIGHT CO
ST LUCIE PLANT #1

CHARACTERISTICS OF SAMPLES TESTED

FIGURE 2G-S2



NO.	BORING NO.	SAMPLE NO.	DRY UNIT WEIGHT	% PASSING #200
1	AE-5B	T-4A	93.4 PCF	40.1 %
2	AE-5B	T-5A	93.0 PCF	11.8 %
3	AE-5B	T-4B	93.4 PCF	35.6 %
4	AE-5B	T-2A	92.8 PCF	19.5 %
5	AE-5B	T-8A	112.8 PCF	27.3 %
6	AE-5A	UD-1A	86.3 PCF	16.9 %
7	AE-5B	T-6A	90.4 PCF	13.1 %
8	AE-5B	T-8B	112.8 PCF	34.1 %
9	AE-5B	T-7A	103.6 PCF	4.9 %
10	AE-5B	T-7B	103.6 PCF	4.6 %
11	AE-5A*	UD-2A	91.5 PCF	15.7 %
12	AE-5A*	UD-2B	88.0 PCF	32.6 %
13	AE-5A*	UD-2C	95.2 PCF	37.6 %
14	AE-5B*	T-1A	90.8 PCF	15.8 %
15	AE-5B*	T-3A	94.6 PCF	28.6 %
16	AE-5B*	T-1B	90.8 PCF	32.2 %
17	AE-5B*	T-2B	92.8 PCF	9.0 %
18	AE-5B*	T-3B	94.6 PCF	32.5 %
19	AE-5B*	T-5B	93.0 PCF	11.0 %
20	AE-5B*	T-6B	90.4 PCF	15.4 %

* SAMPLE DILATED DURING TEST

FIGURE 2G-S3

FLORIDA POWER & LIGHT COMPANY
ST LUCIE PLANT UNIT 1

STRESS-STRAIN BEHAVIOR OF SANDY MATERIALS CONSOLIDATED UNDRAINED TRIAXIAL TESTS

Withheld Under 10 CFR 2.390

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

EXCAVATION AND BACKFILL
PLANS AND DETAILS
SHEET NO. 2

FIGURE 2G-S4

Withheld Under 10 CFR 2.390

FLORIDA POWER & LIGHT COMPANY
ST. LUCIE PLANT UNIT 1

EMERGENCY COOLING WATER SYSTEM
BARRIER WALL – PLAN AND SECTIONS
MASONRY

FIGURE 2G-S5

SUPPLEMENT 2

TO

APPENDIX 2G

2G 9.0 SOIL STABILIZATION REQUESTED BY NRC STAFF

2G 9.1 GENERAL

This second supplement to Appendix 2G presents information and documentation concerning the soil stabilization requested by the NRC Staff and agreed to by the Applicant at the June 20, 1975 meeting in Bethesda, Maryland. It should be noted that the Applicant's position is still that Appendix 2G and Supplement No. 1 have adequately addressed the liquefaction and slope stability of the soils in the vicinity of the ultimate heat sink (UHS) canal and the switchyard. Our conclusion from these detailed studies is that the soils in the area of the UHS canal barrier and the adjacent switchyard are not susceptable to flow slides for the site Design Basis Earthquake and that no remedial work is required. However, in order to satisfy NRC Staff concerns, soil stabilization will be done as discussed in the following sections.

2G 9.2 AREAS REQUIRING STABILIZATION

Figure 2G-S4 indicates the three areas of Staff concern where stabilization is to be performed. The areas are: 1) a rectangular area beneath the UHS barrier wall 2) a triangular area north of the Unit 1 intake structure area and 3) a triangular area south of the Unit 2 intake structure area. The slope of the soils south of the Unit 2 intake structure area has been cut back as suggested by the NRC to limit volume of soil in the area.

2G 9.3 METHOD OF SOIL STABILIZATION

Compaction piles were selected as a suitable method of densifying the soils. Compaction piles densify primarily by displacement, although some vibration does occur during driving. Prestressed concrete piles, 18 inches square, will be driven 15 feet on centers in those areas noted in Section 2G 9.2. Figure 2G-S5 shows the area and depth of the compaction piles. Sections A-A and B-B on Figure 2G-S5 show that the piles for the areas north and south of the intake will extend from the Class II fill to el - 60 ft. No pile will be less than 10 feet in length. The piles in the UHS barrier area will extend from el - 26 to - 60ft. The soil from - 20 to - 26 feet below the barrier wall will be excavated, backfilled and compacted in accordance with Class I soil requirements as noted in Appendix 2B.

The piles will be driven with a hammer to the specified depths. Heave plates will be set and monitored to ensure that the piles are displacing soil and densifying it.

The pile locations will be pre-augered in groups so that at least five (5) pre-augered holes are available adjacent to a pile being driven to allow for drainage and dissipation of pore pressures.

2G 9.4 CLASS II FILL

Figures 2G-S4 and S5 indicate Class II fill. Class II fill is an engineered backfill which contains no more than 40 percent silt, is free of large clay balls and rock fragments. Class II backfill is compacted to a minimum of 95 percent of the maximum density obtained in the Modified AASHO Compaction Test. Test locations and results are documented archived and available for inspection.

Class II material exhibits similar strength values to the Class I material. The difference in notation refers to the change in percent silt and type of document control. Since Class II material is a controlled, compacted backfill, it is only necessary to stabilize the soil beneath the Class II fill and above el - 60 ft.

2G 9.5 STABILITY ANALYSIS OF UHS BARRIER WALL

The stability analysis of the barrier walls is discussed in Section 3.8.1.7.5 of the SAR.

2G 9.6 AVAILABLE WATER SUPPLY WITHIN INTAKE CANAL

The quantity of water which is available within the intake canal under various flow slide conditions is discussed in Section 9.2.7 of the SAR.

ST. LUCIE UNIT I
FLORIDA POWER & LIGHT COMPANY
FSAR
METEOROLOGICAL
AMENDMENT

APPENDIX 2H
OVER WATER DISPERSION

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APPENDIX 2H

OVER WATER DISPERSION

Introduction

The purpose of this Appendix is to describe quantitatively the potential for atmospheric stabilization associated with over-water flow. Dispersion of gaseous plant releases from the St. Lucie facility may, under certain circumstances, be reduced by passage over the Indian River. An over-water dispersion model has been developed to conservatively quantify relative concentration values for the 5 percentile, 0-2 hour accident condition.

The area of specific interest is the west shoreline of the Indian River directly west and approximately two miles from the St. Lucie plant.

General

The plume characteristics associated with a gaseous release over land will not be significantly modified by a body of water until some critical distance downwind from the shoreline. Initially, the plume dimensions will correspond to standard overland Pasquill dispersion curves from the point of plant release to the shoreline. (The distance from the St. Lucie plant to the shoreline is approximately one-half mile.) The critical over-water distance the land released plume must travel before significant modification occurs varies from several hundred to several thousand meters. This variable distance is dependent on several physical parameters. As the plume transitions from an over-land to an over-water environment, several qualitative changes are noted, namely: wind speeds will increase because of the reduced friction drag associated with over-water flow, (resulting in enhanced atmospheric dispersion); lateral meander of the plume centerline will diminish (resulting in decreased atmospheric dispersion); and depending on the water-air temperature difference, the vertical growth rate of the plume will either decrease or increase. If a sufficient fetch (over-water distance) is available, the net change of atmospheric dispersion values will primarily depend on the water-air temperature differences. If the surface water body temperatures are colder than the ambient air temperature (normally during daytime unstable atmospheric conditions) the vertical growth of the plume will decrease or cease at the last limiting value associated with over-land atmospheric dispersion characteristics. It should be emphasized that the plume dimensions cannot diminish below the largest value attained prior to over-water modification, i.e., the plume may never reconstitute itself. If the surface water body temperatures are warmer than the ambient air temperatures (normally during nighttime stable atmospheric conditions), the vertical growth of the plume will increase. (Note: 84% of the St. Lucie atmospheric inversion conditions occurred during the nighttime for the period of record March 1, 1971 to February 29, 1972). Therefore, the majority of the worst atmoshperic conditions observed in an overland environment will improve with nighttime over-water passage.

Indian River Temperatures

Indian River temperatures were monitored for the period November 8, 1974 - March 10, 1975. The daily range of surface water temperature was approximately 2 to $3^{O}F$. During the four month period the River temperatures ranged from a minimum of $60^{O}F$ to a maximum of $78^{O}F$.

In contrast to the 10m air temperatures the Indian River was usually warmer at night and colder during the daytime by several degrees. The largest stabilizing temperature difference between the Indian River and the 10 meter level on the St. Lucie tower was $+14.45^{O}F$ which occured during the daytime.

Table 2H-1 presents a summary of the hourly frequency distribution of the temperature differential between the 10 meter level on the meteorological tower and the surface water temperature. Positive values indicate that the air is warmer than the Indian River while negative values indicate the opposite. Primarily nighttime hours have a negative temperature differential (air colder than water) and a positive temperature differential (air warmer than water) is most frequent and of greatest magnitude during the daytime. These observations are further substantiated in Table 2H-2 which provides average monthly daytime and nighttime temperature differentials. The months of November and March are not complete months as data was obtained for only a portion of them. Daytime was defined as the period between 9 a.m. and 6 p.m. inclusive. Based on Table 2H-2, a significant diurnal variation exists between the daytime and nighttime temperature differentials. During an easterly wind direction component, (approximately 52% of the time; based on 1972 on-site data) the daytime temperature differentials would provide a stabilizing influence on the air flow across the Indian River while nighttime temperature differentials would provide a destabilizing influence on the air flow across the Indian River.

In the analysis of overwater effects presented hereinafter the worst nighttime temperature differential (air - water), was used to calculate the critical overwater distance. Had a larger data base been available a value commensurate with a 5 percentile calculation would have been utilized. However, it must be noted that the atmospheric dilution factor is relatively insensitive to the critical distances determined for St. Lucie. The critical distance effect was found to reduce the atmospheric dilution factor by about 10 percent.

TABLE 2H-1

ST. LUCIE 10 METER LEVEL MINUS COINCIDENT INDIAN RIVER SURFACE WATER TEMPERATURE

Period of Record November 8, 1974 to March 10, 1975

Lower Class Interval	A.M. Hours of Day: Mid Night	1	2	3	4	5	6	7	8	9	10	11
+15												
+14												
+13												1
+12												
+11												1
+10											1	2
+9										1	3	4
+8										3	6	5
+7	2	1	1	1	1		2	3	4	5	5	12
+6	5	4	4	4	2	5	5	3	2	2	3	9
+5	2	3	3	2	2	3	3	3	5	3	13	6
+4	4	5	4	4	5	1	2	7	4	5	8	13
+3	5	5	3	5	7	5	4	3	3	6	6	11
+2	10	7	7	8	3	8	7	6	3	12	9	12
+1	10	11	10	9	11	8	7	7	8	7	7	6
+0	8	6	12	10	5	8	8	9	12	7	12	7
-1	17	11	8	7	7	7	7	4	2	7	7	9
-2	7	7	8	8	7	3	4	6	8	8	9	4
-3	9	10	10	6	8	8	6	5	7	11	5	2
-4	6	12	9	12	11	9	10	7	6	6	5	6
-5	6	7	4	7	9	14	8	8	10	4	3	2
-6	5	3	4	4	4	3	6	8	9	9	5	1
-7	2	6	9	7	8	6	7	4	3	4	1	2
-8	5	3	2	3	5	3	5	8	5	4	2	2
-9		2	3	3	3	5	5	4	6	2	1	
-10	1		2	2	2	3	2	3	3	5	2	2
-11	2	1	2	3	2	1	2	2	1	2	1	
-12	2	2	1	2	1	2	1	2	3	2	3	1
-13	2	4	5	2	4	4	2	3	2	2		
-14	2	3	2	3	2	3	4	1	4	1		
-15	2	1	1	2	4	2	4	6	1	1	1	
-16	1	2	1	1	4	4	2	3	5			
-17			2	3		1	1					
-18	1				1	1	2			1		
-19		1						3				
-20	1	1	1	1	1	1			1			
-21									1			
-22							1	1				
-23												

Lower Class Interval	P.M. Hours of Day: Noon	1	2	3	4	5	6	7	8	9	10	11
+15												
+14			1									
+13		2	2	2	1	1		1				
+12	1	1	1	2		1						
+11	4	1	2	2	2		1					
+10	2	4	4	3	2	2						
+9	4	4	2	5	4		2					
+8	14	7	8	7	6	1		2	1			
+7	4	8	13	7	7	8	2	1		1	1	1
+6	13	12	9	9	9	9	4	4	5	4	5	5
+5	12	10	7	7	6	8	7	6	5	7	4	4
+4	9	9	6	8	13	10	9	7	7	4	8	6
+3	9	8	9	12	9	8	7	9	8	9	7	7
+2	3	7	8	8	9	11	11	8	7	6	9	10
+1	10	7	10	13	12	15	7	4	7	10	10	11
+0	9	11	12	10	11	10	18	16	16	16	9	9
-1	9	7	8	4	6	8	8	15	15	15	17	13
-2	1	5	4	7	6	11	8	9	4	7	7	7
-3	4	3	5	3	5	4	12	9	14	11	13	9
-4	2	3	1	3	3	4	6	7	5	6	5	7
-5	1	2	1	2	2	2	4	7	4	6	6	4
-6	4	2	1		1	2	1	3	6	1	1	3
-7		2		1	1	1	1		1	1		6
-8	2		2	2	1	1	2	1	1	1	3	1
-9	1	2	1	2	2	1	2	2	1	3		
-10	2	1	2	1	1	2	2	2	3	1	1	
-11			1		1		1	2	1	3	1	
-12					1		1	3	4	4	9	5
-13						1				1	1	4
-14						1			2			2
-15							1			1		1
-16								1		1	1	
-17							1					1
-18											1	
-19												1
-20												
-21												
-22												
-23												

TABLE 2H-2

Average Monthly (10m Tower Minus Indian River) Temperature Difference

	November	December	January	February	March
Day	+3.60	+2.88	+2.75	+0.50	-0.85
Night	-0.19	-1.93	-0.94	-4.20	-6.86

Plume Trajectories

Plume trajectories, representing the most conservative dispersion pathways to the west bank of the Indian River were selected for further analysis. Trajectory A represents the shortest distance from the plant to the west bank of the Indian River. Trajectory B represents the shortest overland passage of the plume to the west bank of the Indian River. Plume trajectory A follows a straight line path directed west south west of the plant location. This trajectory represents a 0.64 mile overland passage, followed by a 1.03 mile overwater passage to the closest point of the Indian River west bank, 1.67 miles from the plant. Trajectory B follows an initial westnorthwest direction for an overland distance of 0.13 mile to Big Mud Creek. The plume is assumed to shift to a due west direction traversing 0.84 miles of Big Mud Creek to the eastern bank of the Indian River. A final direction shift to the west south west projects the plume 0.91 miles across the Indian River to the west bank. The circuitous distance is 1.88 miles. The direct distance from the plant to this point is 1.77 miles. The postulated dispersion trajectories represent the most conservative pathways available to reach the west bank of the Indian River.

Model Description

The following model was used to evaluate the five percentile worst 0-2 hour accident dilution factors for calculation distances on the west bank of the Indian River.

$$X/Q = \frac{1}{u(\pi \sigma_{ymod} \sigma_{zmod} + CA)}$$

$$\text{if } (\pi \sigma_{ymod} \sigma_{zmod} + CA) < 3\pi \sigma_{ymod} \sigma_{zmod}$$

$$\text{then } X/Q = \frac{1}{3u\pi \sigma_{ymod} \sigma_{zmod}}$$

where σ_{ymod} = represents the modified horizontal dispersion coefficient for over-water calculations. Defined as: $\sigma_{ymod} = 4/9\ (\sigma_y - \sigma_y{*}) + \sigma_y{*}$

σ_{zmod} = modified vertical dispersion coefficient. The vertical coefficient is held constant at the value attained overland. If this value is less than one half the height of the containment structure, the vertical dispersion coefficient will increase until it attains one half the height of the containment structure.

σ_y = standard horizontal overland dispersion coefficient at the calculation distances

$\sigma_y{*}$ = standard overland dispersion coefficient at the critical distance where overland diffusion characteristics are significantly modified by over-water passage

u = average windspeed for the analysis period

CA = the wake correction factor

1/2 V = one half the height of the containment structure

Shoreline Transition Results

The previously described model was used to compute the 0-2 hour worst five percentile relative concentration values assuming an instantaneous transition to over-water dispersion conditions at the shoreline. Relative concentration values were computed using a Pasquill F stability class with an average wind speed 1.2 meters per second for both plume trajectories previously described. A summary of the analysis follows:

Trajectory A - Instantaneous Shoreline Transition and Direct Distance to Nearest Indian River West Bank

Initial Parameters:

CA	= 1363.0 m^2
Over-land Distance	= 1030 m
Critical Distance	= 0.0 m
Calculation Distance	= 2682 m
σ_y^* (1030 m)	= 38.0 m
σ_y (2682 m)	= 91.6 m
σ_{zmod}(2682 m)	= 25.5 m
1/2 V	= 31.165 m

Results for 2682 meters (1.67 miles) $\chi/Q = 1.32 \times 10^{-4}$ sec/m^3

Trajectory B - Instantaneous Shoreline Transition and Circuitous Distance to Nearest Indian River West Bank

Initial Parameters:

CA	= 1363 m^2
Over-land Distance	= 215 m
Critical Distance	= 0.0 m
Calculation Distance	= 3025.6 m
σ_y^* (215 m)	= 8.6 m
σ_y (3025.6 m)	= 102.1 m
σ_{zmod}(3025.6 m)	= 27.3 m
1/2 V	= 31.165 m

Therefore, at a direct distance of 2848 meters (1.77 miles) from the plant

$$\chi/Q = 1.47 \times 10^{-4} \text{ sec/m}^3$$

Critical Distance

The plume characteristics associated with a gaseous release over-land will not be significantly modified by a body of water until some critical distance downwind from the shoreline. For purposes of the following analysis, the definition of significant modification is when the temperature lapse rate above the standard 10 meter air temperature monitoring level is initially affected. Several formulae have been reviewed to determine the appropriate critical distance values. A recent Brookhaven National Laboratory overwater dispersion study (Reference: BNL 18997, June 1974, Page 29, Equation 8) provides a conservative empirical relationship from which critical distance (fetch) values may be derived. The critical distance (fetch) equation utilized was:

$$D = H^2 \left(\frac{\bar{u}}{u_*}\right)^2 \left(\left| \frac{\Delta T/\Delta Z}{\theta_1 - \theta_2} \right| \right)$$

Reference BNL 18997, Page 28, Equation 8 and Page 20.

where D = critical distance in meters

H = standard 10 meter height

$\left(\frac{u}{u_*}\right)$ = reciprocal of drag coefficient (dimensionless) $0.654 \times 10^{+3}$

$\Delta T/\Delta Z$ = lapse rate above 10 meters in degrees Kalvin per meter or $.0275^{\circ}$ K/M for mid point of Pasquill F stability lapse rate

$\theta_1 - \theta_2$ = potential temperature difference between the water surface temperature and the 10 meter level (4.25°K).

D = 423 meters

The temperature lapse rate above 10 meters ($\Delta T/\Delta Z = 0.0275^{\circ}$ K/meter) represents the five percentile worst St. Lucie atmospheric condition associated with the 0-2 hour accident period.

The potential temperature is equivalent to temperature values at sea level. The potential difference value selected ($\theta_1 - \theta_2 = 4.25^{\circ}$ K) was the highest nighttime observed difference between the St. Lucie 10 meter tower level and Indian River surface water temperature during the four month on-site monitoring program.

Overwater Transition Results

The previously described model was used to compute the 0-2 hour worst five percentile relative concentration values. The transition to overwater dispersion characteristics occurs at the critical distance described previously. Relative concentration values were computed using a Pasquill F stability class with a windspeed of 1.2 meters per second for both previously described plume trajectories. A summary of the analysis follows:

Trajectory A - Over-Water Transition and Direct Distance to Nearest Indian River West Bank

Critical Distance Assumptions: Based on Brookhaven Study

Initial Parameters:

CA	=	1363.0 m^2
Over-land Distance	=	1030 m
Critical Distance	=	423 m
Calculation Distance	=	2682 m
σ_y* (1453m)	=	52.3 m
σ_y (2682m)	=	91.6 m
σ_{zmod} (2682m)	=	25.5 m
1/2 V	=	31.165 m

Therefore, at 2682 meters (1.67 miles) $X/Q = 1.20 \times 10^{-4}$ sec/m^3

Trajectory B - Over-Water Transition and Circuitous Distance to Nearest Indian River West Bank

Critical Distance Assumptions: Based on Brookhaven Study

Initial Parameters:

CA	=	1363.0 m^2
Over-land Distance	=	215 m
Critical Distance	=	423 m
Calculation Distance	=	3025.6 m
σ_y* (638m)	=	24.3 m
σ_y (3025.6m)	=	102.1 m
σ_{zmod} (3025.6m)	=	27.3 m
1/2 V	=	31.165 m

Therefore, at a direct distance of 2848 meters (1.77 miles) from the plant

$X/Q = 1.30 \times 10^{-4}$ sec/m^3

Model Evaluation

The above model may be considered conservative for the following reasons:

1. Wind speed values were not increased during overwater passage. Wind speed values would normally increase in excess of 30% during overwater stable atmpspheric, low wind speed conditions. Note: A study by Cormier (Reference - Estimating Wind Speed Changes At Land/ Sea Boundaries by Rene Cormier, Air Force Cambridge Research Laboratories, Bedford, Mass., June 1972, ENVPREDRSCHFAC Technical Note No. 4-72) indicated wind speeds may more than double during these conditions.

2. The overwater drag coefficient value used in the critical equation calculation is associated with wind speeds between 2 and 10 meters per second. During lower wind speed conditions the water roughness would decrease, therefore, the drag coefficient would decrease. However, definitive studies have not been conducted for the determination of low wind speed overwater drag coefficients.

3. The occurrence of a nighttime Pasquill F inversion associated with a extremely stable atmospheric layer between the water surface and the 10 meter level (+ 4.25° C/10 meters) is considered very conservative. The primary mechanism for the formation of a nighttime inversion is the rapid radiational cooling of a land surface. Water body temperatures do not respond rapidly to nocturnal cooling. Therefore, during the nighttime stable atmospheric periods when land temperatures are at a minimum, the higher water body temperatures will provide a heat source for the destabilization of land released gaseous plumes. However, the applicant's model assumes a stabilizing potential temperature difference of + 4.25°C between the 10 meter height and the water surface. The approximate sea level equivalent of a Pasquill F potential temperature difference is 0.15° C/10 meters to 0.4° C/10 meters. Therefore, the assumption that the temperature profile through the lowest 10 meters is an order of magnitude more stable than the upper level temperature profile is deemed extremely conservative.

4. The application of the over-water dispersion model, as provided in this study, is associated with the five percentile worst atmospheric conditions. However, a rigorous application of the over-water dispersion model should be restricted to the following coincident conditions:

   a. Only seven wind directions are associated with plume trajectories from the St. Lucie plant across the Indian River.

   b. Specific initial overland distances, from the St. Lucie plant to the east bank of the Indian River per each of the seven wind directions, vary from several hundred to several thousand meters.

   c. During the worst atmospheric conditions only the Indian River surface water temperatures colder than the St. Lucie 10 meter tower temperatures would provide a significant stabilizing influence on overwater plume passage.

Summary

Two sets of atmospheric dispersion factor values associated with an over-water dispersion model have been provided. One set of values represents a straight line distance calculation from the St. Lucie plant directly to the nearest west bank of the Indian River (Trajectory A). The other set of values represents a circuitous distance calculation from the St. Lucie plant to the nearest shoreline, a meandering trajectory over Big Mud Creek and then a straight line distance to the nearest west bank of the Indian River (Trajectory B). For all the initial overland distances, the atmospheric dispersion characteristics are based on the standard conservative ground release equation. Two assumptions regarding the over-water modification distance have been incorporated in the over-water dispersion model namely, an immediate transition to over-water dispersion characteristics at the shoreline and a critical distance transition prior to over-water modification. The resulting atmospheric dispersion factor values (sec/m^3) are:

	Direct Distance	Shoreline Transition	Over-Water Transition
Trajectory A	1.67 miles	1.32×10^{-4}	1.20×10^{-4}
Trajectory B	1.77 miles	1.47×10^{-4}	1.30×10^{-4}

The over-water transition χ/Qs are representative of what would be expected, whereas, the shore line transition values should be considered a bounding upper limit calculation. It should also be noted that the method utilized for determination of the critical distance is considered conservative. Larger values are likely to be realized.

APPENDIX 2I

SHORT-TERM (ACCIDENT) ATMOSPHERIC DISPERSION FACTORS FOR THE EXCLUSION AREA BOUNDARY AND LOW POPULATION ZONE FOR AST

Short-Term (Accident) Atmospheric Dispersion Factors for the Exclusion Area Boundary and Low Population Zone

Objective

Conservative values of atmospheric dispersion factors at the exclusion area boundary (EAB) and the low population zone (LPZ) were calculated for appropriate time periods using meteorological data collected onsite during the time period including the years 1997, 1998, 1999, 2002 and 2003 which were deemed to represent the most reliable data sets.

Methodology

The methodology used for this calculation is consistent with Regulatory Guide 1.145 as implemented by the PAVAN computer code (Reference 2). Using joint frequency distributions of wind direction and wind speed by atmospheric stability, the PAVAN computer code provides relative air concentration (χ/Q) values as functions of direction for various time periods at the EAB and LPZ. Three procedures for calculation of χ/Qs are utilized for the site boundary and LPZ; a direction-dependent approach, a direction-independent approach, and an overall site χ/Q approach. The χ/Q calculations are based on the theory that material released to the atmosphere will be normally distributed (Gaussian) about the plume centerline. A straight-line trajectory is assumed between the point of release and all distances for which χ/Q values are calculated.

The theory and implementing equations employed by the PAVAN computer code are documented in Reference 2.

Calculations PAVAN Computer Code Input Data

The minimum EAB distance assumed for all directions is 0.97 miles from the center of the Unit 1 containment building. The LPZ distance is taken as 1 mile from the center of the Unit 1 containment building in all directions.

All of the releases were considered ground level releases because the highest possible release elevation is from the plant stack at 184 ft. From Section 1.3.2 of Reference 1, a release is only considered a stack release if the release point is at a level higher than two and one-half times the height of adjacent solid structures. For the St. Lucie plant, the elevation of the top of the Unit 1 containment is 225.5 ft. Therefore, the highest possible release point is not 2.5 times higher than the adjacent containment buildings, and thus all releases were considered ground level releases. As such, the release height was set equal to 10.0 meters as required by Table 3.1 of Reference 2. The building cross-sectional area used for the building wake term was 1,565 m^2. This area was calculated to be conservatively small in that the height used in the area calculation was from the highest roof elevation of a nearby building to the elevation of the bottom of the containment dome.

The tower height at which the wind speeds were measured is 10 m and 57.9 m above plant grade. The wind speed units are given in miles per hour, therefore, the PAVAN variable UCOR was set equal to 101 to convert the wind speeds to meters per second as described in Table 3.1 of Reference 2. The maximum wind speed in each wind speed category was chosen to match the raw joint frequency distribution data, which conforms to the wind speed bins in Table 1 of Reference 3. The maximum wind speed values are 1, 3, 7, 12, 18, 24, and 30 mph. The maximum windspeed in each windspeed category was chosen to match the recommendation of RIS-2006-4. (Reference 4)

Results

PAVAN computer runs for the EAB and LPZ boundary distances were performed using the data discussed previously. Per Section 4 of Reference 1, the maximum χ/Q for each distance was determined and compared to the 5% overall site value for the boundary under consideration. The maximum EAB and LPZ χ/Qs that resulted from this comparison are provided in the table below:

Exclusion Area Boundary and Low Population Zone χ/Qs

Time Period	EAB χ/Q (sec/m^3)	LPZ χ/Q (sec/m^3)
0-2 hours	9.84E-05	9.56E-05
0-8 hours	5.53E-05	5.34E-05
8-24 hours	4.15E-05	3.99E-05
1-4 days	2.22E-05	2.12E-05
4-30 days	9.06E-06	8.55E-06

<u>References</u>

1. USNRC Regulatory Guide 1.145, “Atmospheric Dispersion Models for Potential Accident Consequence Assessments at Nuclear Power Plants,” Revision 1, November 1982. (Reissued February 1983 to correct page 1.145-7).

2. NUREG/CR-2858, “PAVAN: An Atmospheric Dispersion Program for Evaluating Design Basis Accidental Releases of Radioactive Materials for Nuclear Power Stations,” November 1982.

3. Safety Guide 23, “Onsite Meteorological Programs,” February 17, 1972.

4. USNRC Regulatory Issue Summary RIS-2006-4, “Experience with Implementation of Alternative Source Terms,” March 7, 2006.

APPENDIX 2J

SHORT-TERM (ACCIDENT) ATMOSPHERIC DISPERSION FACTORS FOR THE CONTROL ROOM FOR AST

Short-Term (Accident) Atmospheric Dispersion Factors for the Control Room

Objective

Conservative values of atmospheric dispersion factors for the control room were calculated for appropriate time periods using meteorological data collected onsite during the time period including the years 1997, 1998, 1999, 2002 and 2003 which were deemed to represent the most reliable data sets.

Methodology

The ARCON96 computer code is used by the USNRC staff to review licensee submittals relating to control room habitability (Reference 1). Therefore, the ARCON96 computer code was used to determine the relative concentrations (χ/Qs) for the control room air intakes and inleakage locations.

The ARCON96 computer code uses hourly meteorological data for estimating dispersion in the vicinity of buildings to calculate relative concentrations at control room air intakes that would be exceeded no more than five percent of the time. These concentrations are calculated for averaging periods ranging from one hour to 30 days in duration.

The theory and implementing equations employed by the ACRCON96 computer code are documented in Reference 1.

Calculations/ARCON Computer Code Input Data

Five years of meteorological data were used for the ARCON96 computer code runs.

A number of various release-receptor combinations were considered for the control room χ/Qs. These different cases were considered to determine the limiting release-receptor combinations for the various events. The case matrix for these combinations is provided in Table 2J-2.

The distance and direction inputs for the ARCON96 runs may be found in Table 2J-1. The distances were converted from feet to meters with a factor of 0.3048 m/ft. The distances in meters were then rounded down to the nearest tenth for conservatism. The elevation difference term was set equal to zero for each case since all elevation points are taken with respect to the same datum. The intake heights were determined as the intake elevations less the plant grade elevation of 19 ft.

The lower and upper measurement heights for the meteorological data were entered as 10 m and 57.9 m, respectively, for each case. The mph option was selected for the wind speed units.

A ground level release was chosen for each scenario since none of the release points are 2.5 times taller than the closet solid structure as called out in Section 3.2.2 of Reference 3 for stack releases. The top of the containment structures is at an elevation of 225.5 ft. The highest release point is from the top of the plant stack at an elevation of 184 ft., which is not 2.5 times higher than the nearby containment structure. The vertical velocity, stack flow, and stack radius terms were all set equal to zero since each case is a ground level release. The vent release option was not selected for any of the scenarios.

The actual release height was used in the cases. No credit was taken for effective release height due to plume rise; therefore, for the releases from the stacks, the release elevations were set equal to the stack top elevation. The release heights were taken as the release elevations less the plant grade elevation of 19 ft.

The only cases in this analysis that take credit for the building wake effect are the scenarios where the release is from the containment building. Some of the other scenarios have buildings between the release and receptor points, but for these cases the building wake was not credited for the sake of conservatism. Not crediting wakes was accomplished by setting the building area term equal to 0.01 m^2 as stated in Table A-2 of Reference 3. The building area used is a conservatively determined containment cross sectional area. The width used is equal to the inside diameter of the containment building plus the thickness of the wall, while the height is taken as the distance between the top of the cylinder potion of the containment structure and the highest auxiliary building roof elevation. This building cross-sectional area is equal to 1,565 m^2.

All of the default values in the ARCON96 code were unchanged from the code default values with the following exceptions as recommended in Table A-2 of Reference 3:

- A value of 0.2 is used for the surface roughness length, m, in lieu of the default value of 0.1, and
- A value of 4.3 is used for the averaging sector width constant, in lieu of the default value of 4.0.

The minimum wind speed was left at 0.5 m/s per the guidance instruction in Table A-2 of Reference 3.

Results

ARCON96 computer runs for the various release points and control room intake locations were performed using the data discussed previously. Per Reference 3, the 95th percentile χ/Q values were determined. The resulting χ/Qs are listed in Table 2J-2.

References

1. NUREG/CR-6331 PNL-10521, “Atmospheric Relative Concentrations in Building Wakes,” May 1995, with Errata dated July 1997.
2. Safety Guide 23, “Onsite Meteorological Programs,” February 17, 1972.
3. USNRC Regulatory Guide 1.194, “Atmospheric Relative Concentrations for Control Room Radiological Habitability Assessments at Nuclear Power Plants,” June 2003.

TABLE 2J-1
Direction and Distance Data

Release Point	Receptor Point	Release Height (ft)	Release Height (m)	Receptor Height (ft)	Receptor Height (m)	Distance (ft)	Distance (m)	Direction With Respect to True North
Stack/Plant Vent	N CR intake	184	56.1	59.75	18.2	48.08	14.6	58
Stack/Plant Vent	S CR intake	184	56.1	59.75	18.2	126.69	38.6	354
RWT	N CR intake	48.22	14.6	59.75	18.2	245.31	74.7	65
RWT	S CR intake	48.22	14.6	59.75	18.2	263.64	80.3	39
FHB Closest Point	N CR intake	43.25	13.2	59.75	18.2	120.6	36.7	48
FHB Closest Point	S CR intake	43.25	13.2	59.75	18.2	184.26	56.1	11
Aux. Bldg. Louver L-7B	N CR intake	38.17	11.6	59.75	18.2	123.77	37.7	72
Aux. Bldg. Louver L-7A	S CR intake	38.17	11.6	59.75	18.2	136.97	41.7	34
Condenser	N CR intake	5.25	1.6	59.75	18.2	153.24	46.7	245
Closest ADV	N CR intake	53	16.1	59.75	18.2	105.68	32.2	306
Closest ADV	S CR intake	53	16.1	59.75	18.2	214.82	65.4	319
Closest Feedwater Line Point	N CR intake	17	5.2	59.75	18.2	83.29	25.3	306
Closest Feedwater Line Point	S CR intake	17	5.2	59.75	18.2	193.15	58.8	321

TABLE 2J-1
Direction and Distance Data

Release Point	Receptor Point	Release Height (ft)	Release Height (m)	Receptor Height (ft)	Receptor Height (m)	Distance (ft)	Distance (m)	Direction With Respect to True North
Containment Maintenance Hatch	N CR Intake	16	4.9	59.75	18.2	172.4	52.5	359
Containment Maintenance Hatch	S CR Intake	16	4.9	59.75	18.2	279.09	85.0	348
FHB Closest Point	Midpoint Between Intakes	43.25	13.2	59.75	18.2	142.19	43.3	25
Stack/Plant Vent	Midpoint Between Intakes	184	56.1	59.75	18.2	74.85	22.8	8
RWT	Midpoint Between Intakes	48.22	14.6	59.75	18.2	244.91	74.6	52
Aux. Bldg. Louver L-7A	Midpoint Between Intakes	38.17	11.6	59.75	18.2	118.59	36.1	59
Closest ADV	Midpoint Between Intakes	53	16.1	59.75	18.2	160.26	48.8	314
Closest Feedwater Line Point	Midpoint Between Intakes	17	5.2	59.75	18.2	138.15	42.1	315
Containment Maintenance Hatch	Midpoint Between Intakes	16	4.9	59.75	18.2	223.66	68.1	351

TABLE 2J-1
Direction and Distance Data

Notes:

1. Release heights are calculated as 19 feet less than the reference elevations to account for the plant grade elevation.

2. The FHB closest point release elevation is taken as the roof elevation since the SW corner of the roof is the closest building point to the intakes.

3. Release and receptor points are considered to be at the centerpoint or centerline of all openings.

4. The only release/receptor combination that does not have the intakes in the same wind direction window from the release point is for the releases from the plant stack. All other release points analyzed result in both control room intakes being in the same wind direction window. Therefore, credit may be taken for intake dilution only for releases from the plant stack.

5. The receptor point for the "midpoint between intakes" is taken as being on the outside of the control room (and H&V room) east wall. The receptor elevation is taken as the average of the receptor elevations for the two outside air intakes.

6. Atmospheric dispersion factors for the releases to the midpoint between the control room intakes are required for the limiting cases to be used during the time period when the control room intakes are isolated. This midpoint receptor location is used to calculate the χ/Q value to be used for the unfiltered control room inleakage dose.

7. The closest containment/shield building penetration to the intakes that is directly exposed to the atmosphere is the closest feedwater line penetration.

TABLE 2J-2

Control Room χ/Qs

This table summarizes the results for χ/Q factors for the control room intakes for the various accident scenarios. Values are presented for the unfavorable intake prior to intake isolation, the midpoint between the intakes for during isolation, as well as values for the favorable intake due to the manual selection of the favorable control room intake after unisolation and initiation of filtered air make-up. These values are not corrected for Control Room Occupancy Factors but do include taking credit for dilution where allowed. Section 3.3.2.3 of Reg. guide 1.194 provides the following guidance for dual intake ventilation systems which allow the operator to manually select the least contaminated outside air intake as a source of outside air makeup and close the other intake:

"The χ/Q value for the limiting intake should be used for the time interval prior to intake isolation. This χ/Q value may be reduced by a factor of 2 to account for dilution by the flow from the other intake. The χ/Q values for the favorable intake are used for the subsequent time intervals. The χ/Q values may be reduced by a factor of 4 to account for the duel inlet and the expectation that the operator will make the proper intake selection. This protocol should be used only if the dual intakes are in different wind direction windows."

Based on the layout of the site, the only cases that may take credit for dilution are when the releases are from the plant vent stack. Therefore, the Plant Stack values shown below include a reduction by a factor of 2 prior to isolation and a reduction by a factor of 4 after the control room is aligned to the favorable intake for filtered make-up. However, dilution is not credited during the time period when the control room intakes are isolated for these cases.

∗ Indicates credit for dilution taken for this case.

The atmospheric dispersion factors corresponding to ADVs were determined to be more limiting than those from the MSSVs for all time periods. Therefore, the more limiting ADV values have been used throughout the analyses for all secondary releases. No distinction is made between automatic steam relief from the MSSVs and controlled releases from the ADVs for radiological purposes.

Release-Receptor Pair	Release Point	Receptor Point	0-2 hour χ/Q (sec/m^3)	2-8 hour χ/Q (sec/m^3)	8-24 hour χ/Q (sec/m^3)	1-4 days χ/Q (sec/m^3)	4-30 days χ/Q (sec/m^3)
A∗	Stack/Plant Vent∗	N CR Intake∗	2.39E-03				
B∗	Stack/Plant Vent∗	S CR Intake∗	6.93E-04	4.88E-04	2.19E-04	1.46E-04	1.28E-04
C	RWT	N CR Intake	1.37E-03				
D	RWT	S CR Intake	1.12E-03	9.10E-04	3.84E-04	2.93E-04	2.37E-04
E	FHB Closest Point	N CR Intake	4.99E-03				
F	FHB Closest Point	S CR Intake	2.01E-03	1.44E-03	6.25E-04	4.34E-04	3.33E-04
G	Aux. Bldg. Louver L-7B	N CR Intake	4.80E-03				
H	Aux. Bldg. Louver L-7A	S CR Intake	3.61E-03	2.87E-03	1.20E-03	9.07E-04	7.13E-04
I	Condenser SJAE	N CR Intake	3.02E-03				
J	Closest ADV#	N CR Intake	6.30E-03				

TABLE 2J-2

Release-Receptor Pair	Release Point	Receptor Point	0-2 hour χ/Q (sec/m^3)	2-8 hour χ/Q (sec/m^3)	8-24 hour χ/Q (sec/m^3)	1-4 days χ/Q (sec/m^3)	4-30 days χ/Q (sec/m^3)
K	Closest ADV#	S CR Intake	1.62E-03	1.32E-03	5.06E-04	3.88E-04	3.30E-04
L	Closest Feedwater Line Point	N CR Intake	7.29E-03				
M	Closest Feedwater Line Point	S CR Intake	1.76E-03	1.41E-03	5.72E-04	4.29E-04	3.57E-04
N	Stack/Plant Vent	Midpoint Between Intakes	3.91E-03				
O	RWT	Midpoint Between Intakes	1.34E-03				
P	Aux. Bldg. Louver L-7A	Midpoint Between Intakes	5.03E-03				
Q	Closest ADV#	Midpoint Between Intakes	2.84E-03				
R	Closest Feedwater Line Point	Midpoint Between Intakes	3.17E-03				
S	Containment Maintenance Hatch	N CR Intake	1.90E-03				
T	Containment Maintenance Hatch	S CR Intake	8.22E-04	6.57E-04	2.87E-04	1.92E-04	1.74E-04
U	Containment Maintenance Hatch	Midpoint Between Intakes	1.21E-03				
V	FHB Closest Point	Midpoint Between Intakes	3.27E-03				